UNIVERSITY OF NAIROBI

School of Engineering

PROJECTED FLEXIBLE ROAD PAVEMENT PERFORMANCE OF NAIROBI - THIKA SUPERHIGHWAY IN KENYA

A thesis submitted in partial fulfilment for the Degree of Master of Science in Civil Engineering (Transportation Engineering)

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DECLARATION

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

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This thesis has been submitted with my approval as the University Supervisor

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ABSTRACT

The Nairobi – Thika Superhighway has been open to traffic for five years and yet sections of the road pavement have already deteriorated substantially and require reconstruction. That was established for the section between Kenyatta University and Thika Town despite the fact that the road was designed to last twenty years with periodic and routine maintenance. The research was based on the following objectives: (i) to establish the pavement surface condition by conducting roughness survey, (ii) to evaluate the structural performance of the pavement based on the deflection measurements under the prevailing and projected traffic loading conditions (iii) to evaluate the remaining life of the pavement.

Traffic survey was conducted at the Nairobi – Thika Superhighway at the Eastern Bypass from Tuesday 7 May 2013 to Monday 13 May and the Average Daily Traffic (ADT) computed. The as-built records of the flexible pavement were reviewed and the properties of the pavement layers obtained from the design records. The road condition survey and the deflection measurements were conducted by the Kenya Ministry of Roads in the year 2012.

Pavement analysis was done to establish the performance of each lane. The back-calculated values of the Residual Modulus of Subgrade Reaction and the Equivalent Pavement Moduli of the pavement layers were compared with the as-built records of each lane. The remaining or residual life of the pavement in years and the corresponding overlay requirements were also established for each homogeneous section as an indicator of the performance.

Analysis of the roughness survey established that the road performance corresponded to that of a new pavement. However the deflection analysis revealed the road had started deteriorating early and the sections that were most affected were those where speed bumps had been placed.

The last one kilometer of the road from Kenyatta University to Thika Town on all lanes had failed and reconstruction was recommended after 5years of opening the road to traffic. The rest of the sections also deteriorated substantially and an overlay of 55mm on all lanes was recommended after 5years of projected traffic use. Similarly, the first one kilometer from Thika Town to Kenyatta University had also failed and required reconstruction after 5years of projected traffic use. The deterioration of the rest of the sections also indicated that the road required an overlay of 80mm after 10years of projected traffic.

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I am grateful to The Chief Engineer, Ministry of Roads (Materials Brach) and Engineer Wanyiri for conducting deflection and roughness measurements. I also appreciate Esther Amimo who helped with data analysis. I acknowledge Eng Wilfred Oginga of Kenya Urban Roads Authority (KURA) who helped with additional data on Nairobi – Thika Superhighway. I also appreciate Eng Kosgey of Kenya Roads Board (KRB) who helped with data and reports on Axle Load Surveys on Nairobi – Thika Superhighway.

I acknowledge my wife and children for their emotional support during the research period. I am also grateful to the data enumerators who helped in traffic data collections.

DEDICATION

This work is dedicated to my loving mother Christabel Adoyo Odongo for having faith in me and encouraging me to pursue education despite her not having gone to school. I also dedicate this research to all those children from humble beginnings who have struggled all through their lives to pursue education.

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List of Abbreviations

AASTHO	American Association of State and Highway Officials
ADF	African Development Fund
ADT	Average Daily Traffic
CBR	California Bearing Ratio
FWD	Falling Weight Deflectometer
IRI	International Roughness Index
KeNHA	Kenya National Highways Authority
KU	Kenyatta University
LHS	Left Hand Side (from Kenyatta University to Thika)
MOR&PW	Ministry of Roads and Public Works, Kenya
M.O.T	Ministry of Transport, Kenya
NEPAD	New Partnership for Africa’s Development
NDT	Non Destructive Testing
RHS	Right Hand Side (from Thika to Kenyatta University)
PSI	Present Serviceability Index
PSR	Present Serviceability Rating
ROC	Radius of Curvature
SSI	Structural Strength Index
SSIF	Statistical Structural Strength Index
SCI	Surface Curvature Index
TTI	Texas Transportation Institute
SN	Structural Number

1 INTRODUCTION

1.1 Background Introduction

The Kenya Vision 2030 aspires for a country firmly interconnected through a network of roads, railways, ports, airports, waterways and telecommunications as well as adequate energy. Road Transport Infrastructure was identified as a key driver to the realization of the vision. Road transport remains the predominant mode of transport and carries about 93% of all cargo and passenger traffic in the country. Kenya's road network was established to be 178,000 km long by the year 2009. About 63,290 kilometres of those roads were classified while the rest were not classified. Before improvement, the Nairobi-Thika superhighway was then a dual-carriageway road of about 45 km and was part of the classified international trunk road A2 which originated in downtown Nairobi and extended to Moyale at the Ethiopian border. The Nairobi to Thika section of highway was constructed to bitumen standard in the early 1970's (Kenya Ministry of Transport, 2009).

The Greater Nairobi (Metropolitan Area) was considered the most dynamic engine of growth and employment creation in Kenya accounting for more than 30% of the National GDP. However, economic growth and access to job opportunities were constrained because of rapid urbanization coupled with the explosive growth in motorization leading to constraints on the transportation system. These included the urban arterials and the major corridors linking the Central Business District (CBD) to the suburbs and satellite towns, and among them was the Nairobi-Thika Superhighway (A2). (African Development Fund, ADF – 2007)

The initial planning and diagnostic studies of the inadequacy of Nairobi-Thika Superhighway were done within the context of the Nairobi Metropolitan Area Urban Transport Master Plan (JICA, 2006). The findings among others highlighted the generally inadequate urban transportation infrastructure and urban public transportation system. The study particularly mentioned the extremely poor levels of service and shortage of capacity along the Nairobi-Thika corridor with low operating speeds, long delays, accidents and high operating costs (ADF, 2007).

It is because of these findings that the Government of Republic of Kenya through the Development Budget, and support from African Development Bank Group, spent substantial amounts of money in rehabilitating and upgrading the Nairobi-Thika road to a superhighway by constructing additional lanes on each carriageway and rehabilitating the existing carriageways. (ADF, 2007)

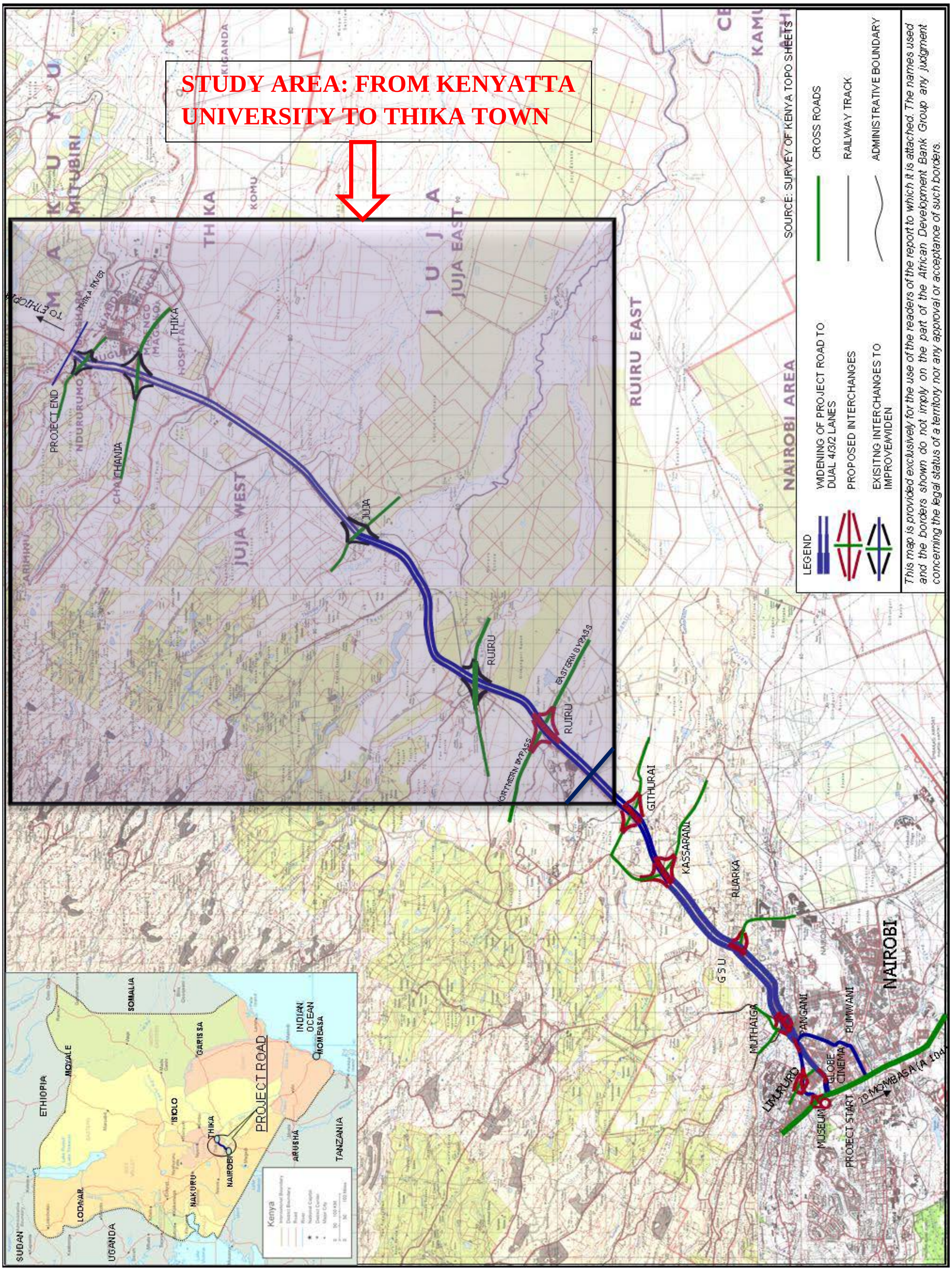
1.2 Description of Study Area

Nairobi-Thika superhighway project passes through Nairobi and Kiambu counties in the Republic of Kenya. It starts from downtown of Nairobi Central Business District and ends in Thika near the bridge across Thika River after the overpass leading to Thika town. The Government of Kenya divided the project into three contracts for ease of execution and the three contracts were named Lot 1, Lot 2 and Lot 3 denoting three sections of the network as summarized in Table 1.1 (MoRPW, 2007). This research was done under Lot 3 from Kenyatta University to Thika Town, and the reference point (km 00+000 from Kenyatta University to Thika Town) in this report corresponds to km 18+000 of the overall network. Figure 1.1 is the location map of the study area.

Table 1.1: Description of the Study Area

Section	**Contract No.1 (Lot 1)**	**Contract No.2 (Lot 2)**	**Contract No.3 (Lot 3) (STUDY AREA)**
Contract Location Description	Three 3No. Arterial Connectors dispersed to Nairobi City at Pangani Round About upto Muthaiga Round About. A total of 12.4km	Muthaiga Round About to Kenyatta University. Total length 14.1km from Km 3+900 to Km 18+000.	Kenyatta University to Thika (from Km 18+000 to Km 42.1). A total of 24.1km

Source: MORPW (2007)



Not to Scale

Figure 1.1 The location plan of Nairobi – Thika Superhighway (ADF, 2007)

1.3 Problem Statement

The design of the flexible pavement for Nairobi – Thika Superhighway considered the design life to be 20 years with routine and periodic maintenance only (MoRPW, 2007). However, Roads in Kenya were found to deteriorate faster than their design lives. These were attributed to many factors including poor construction materials, poor construction quality controls, inadequate designs, poor maintenance strategies, and overloading of the heavy goods vehicles. According to the Kenya Self-Regulatory Vehicle Load Control Charter (KRB 2014), overloaded axles were by far the dominant factor in reducing pavement life in Kenya.

Both the Kenya National Highways Authority (KeNHA) and Kenya Roads Board reported that there were overloading practices on the Kenyan Roads which included Nairobi Thika Superhighway (Quarterly Reports - KRB, 2014).

It was observed that sections of Nairobi – Thika Superhighway had started deteriorating early, predominantly through rutting. Further, the presence of speed bumps seemed to have changed the functional classification of the road from movement to access. It was observed that there was severe rutting just before the speed bumps as shown in Figure 1.2 and Figure 1.3. The presence of the speed bumps reduced the speeds of heavy commercial vehicles thus substantially increasing the transient loads, and so the use of speed bumps on a highway needed to be checked. However, Figure 1.4 and 1.5 also show that rutting occurred at other locations, indicating that either the road was failing due to overloading, or because the pavement structure was weak. Because of the accelerated deterioration and the need for major rehabilitation of sections of the carriageway barely six years after of projected traffic, this research became very necessary so that appropriate intervention could be taken.

Figure 1.2: Rutting of LHS pavement (near the speed bumps at km 11+000 from Kenyatta University to Thika) (Source: Author, 2016)

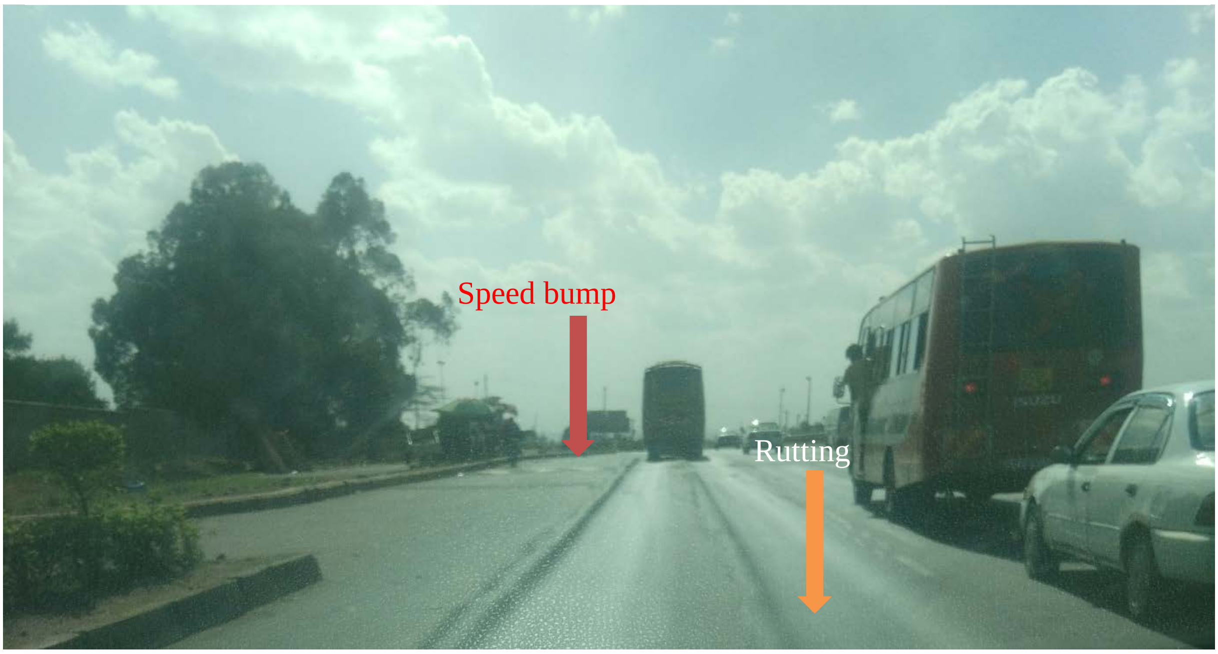


Figure 1.3 Severe rutting of RHS pavement (near the speed bumps at km 14+000 from Thika to Kenyatta University) (Source: Author, 2016)

Figure 1.4 Rutting LHS pavement (at km 20+000 from Kenyatta University to Thika) (Source: Author, 2016)

Figure 1.5: Rutting of RHS pavement (at km 13+000 from Thika to Kenyatta University) (Source: Author, 2016)

1.4 Objectives

1.4.1 Hypothesis

The research was expected to provide baseline data and information on the performance of the pavement of the Nairobi-Thika Superhighway by answering the following questions.

i. What was the roughness rating of the pavement under the prevailing traffic conditions?
ii. What was the structural performance of the pavement based on the deflection measurements under the current and projected traffic loading conditions?
iii. What was the remaining life of the pavement based on the deflection measurements?

1.4.2 Objectives

This research will be based on the following objectives.

i. To establish the pavement surface condition by conducting roughness survey;
ii. To evaluate the structural performance of the pavement based on the deflection measurements under the prevailing and projected traffic loading conditions.
iii. To evaluate the remaining life of the pavement.

1.5 Justification of the Study

Before improvement, the Nairobi Thika superhighway operated beyond capacity, carrying more than 60,000 vehicles per day. By the year 2007, the road was characterized by heavy traffic that was increasing with time as a result of the rising urban population along the route. Traffic demand was almost twice the existing capacity. In addition, main centres along the road, namely Kasarani, Githurai, Ruiru, and Juja, were burgeoning industrial and commercial centres, further adding traffic loads onto the highway. (African Development Fund, ADF 2007)

The poor level of service resulted in long traffic delays and travel times and high level of accident rates. This, together with the poor physical condition of the road and its limited capacity, resulted in significant travel delays, high fuel consumption, high vehicle emissions as well as social inconveniences. The upgrading of the highway was expected to provide adequate capacity and considerably decrease the accident rate by minimizing vehicle conflicts with traffic by providing interchanges, construction of additional lanes and by providing separate service roads for local and Non-motorized traffic (African Development Fund, ADF, 2007).

The total project cost in the year 2007 was estimated at 350 million U.S dollars which was an equivalent of 24.5billion Kenya Shillings in the year 2007. (African Development Fund, ADF, 2007) Nairobi-Thika superhighway was Therefore, one of the most expensive investments by the Government of Kenya in the road transport sub-sector.

Therefore, consistent, cost-effective, and accurate monitoring of pavement was necessary for improving the performance and serviceability of Nairobi Thika Superhighway pavement, and to schedule proactive repair and maintenance activities. This research thesis provided baseline pavement performance indicators for Nairobi – Thika Superhighway upon which future studies will be based.

1.6 Scope and Limitation of Study

i. Deflection survey using the Falling Weight Deflectometer (FWD) from Kenyatta University (km 18+000) to Thika Town on each lane;
ii. Determination of Pavement condition by taking roughness measurements using the Bump Integrator to compute the International Roughness Index (IRI) for each lane;
iii. Determination of traffic loading by conducting 7-day traffic survey;

The deflection study was limited to the initial (baseline) investigations upon which further research will be based. The axle load surveys conducted during design in the year 2007 had established that there was overloading on the road (MoRPW, 2007). Subsequent Axle load monitoring by both the Kenya National Highway Authority and Kenya Roads Board (KRB, Quarterly Reports 2012-2014) also established that there were overloading practices on the road. Therefore, the Vehicle Equivalence Factors that were used at the design of Nairobi Thika superhighway were also adopted for evaluation of traffic loading in this research.

2 LITERATURE REVIEW

2.1 Nairobi – Thika Superhighway Pavement

Literature on the Design and Construction of the Nairobi – Thika Superhighway pavement was reviewed to establish the as-built pavement layers structure. Literature on axle load monitoring on the road was also reviewed to establish the traffic loading used in this research. A summary of literature specific to Nairobi – Thika Superhighway adopted in this research is summarized in Table 2.1.

Table 2.1 Summary of Publications on Nairobi –Thika Superhighway

No.	Title of Publication	Author	Date of publication
1	Feasibility Study, Detailed Design, Tender Administration and Construction Supervision of Nairobi – Thika Road (A2)	Kenya Ministry of Roads and Public Works (MoRPW)	2007
2	Completion Report (As Built Drawings), Rehabilitation and Upgrading of Nairobi – Thika Road, Lot 3 – Contract No:0532	Kenya National Highways Authority (KeNHA)	2013
3	Consultancy Services For Axle Load Monitoring (Eastern Package) Contract No. KRB/594A/2011-2014 Quarterly Reports	Kenya Roads Board (KRB)	2012-2014
4	Kenya Self-Regulatory Vehicle Load Control Charter	Kenya Roads Board (KRB)	2014

2.2 Classification of Roads in Kenya

By the year 2012, Kenya had a road network of about 177,800 km out of which only 63,575 km was classified. The classified road network had increased from 41,800 km at independence in 1963 to 63,575 km by the year 2012, a development rate of less than 600 km per annum. During the same period, the paved road length grew from 1,811 km to 9,273 km. It was estimated that about 70% (44,100 km) of the classified road network was in good condition and was maintainable while the remaining 30% (18,900 km) required rehabilitation or reconstruction.

Table 2.2 is a summary of the classified road network in Kenya which included the Nairobi – Thika Superhighway falling under Class A (KeNHA (2014)), at the time of study.

Table 2.2 Classified Road Network in Kenya

Road class	**Premix**	**Length by Surface Type (km)**			**Total**
		Surface dressing	**Gravel**	**Earth**	
International Trunk Roads (A)	1,244.91	1,563.81	715.11	94.48	3,618.31
National Roads (B)	350.21	1,166.26	819.29	346.14	2,681.90
Primary Roads (C)	642.89	2,198.16	3,601.64	1,552.90	7,995.59
Secondary Roads (D)	76.63	1,183.10	5,701.93	4,087.73	11,049.39
Minor Roads (E)	165.81	542.04	8,215.89	17,982.57	26,906.31
Special Purpose Roads	24.88	114.63	4,929.69	6,253.78	11,322.98
All classes	**2,505.33**	**6,768**	**23,983.55**	**30,317.60**	**63,574.4**

Source: KeNHA (2014).

Table 2.3 International Trunk Roads

Road	**Link**
A1	Tanzania border (Isebania-Kisumu-Kitale-Sudan Border (Lokichoggio) 886 km
A2	Nairobi-Thika-Isiolo-Moyale (Ethiopia border) (833 km)
A3	Thika-Garissa-Somalia border (Liboi) 556 km
A104	Uganda border (Malaba)-Nakuru-Nairobi-Athi River-Tanzania border (Namanga) 648 km
A109	Athi River Mombasa 473 km
A14	Mombasa-Tanzania border (Lunga Lunga) 106 km
A23	Voi-Tanzania border (Taveta) 114 km

Source: KeNHA (2014).

There were seven Class A defined roads comprising 3,755 km of which 2,886 km were paved and 869 km unpaved as shown in Table 2.3. Class A roads comprise international trunk roads linking centres of international importance and crossing international boundaries or terminating at international ports such as Mombasa International Harbour.

2.3 Pavement Design of the Nairobi Thika Superhighway

2.3.1 Traffic Loading

- **Vehicle Equivalence Factors**

The first systematic attempt to quantify the relationship between the axle load and the damage caused to the road was made as part of a comprehensive road experiment known as the AASHO Road Test. The experiment involved allowing vehicles of various axle loads to travel along different sections of a road built and then subjected to traffic, in Illinois, USA between 1956 and 1960, and comparing the number of load repetitions applied to the road before a defined level of distress in the pavement was reached (Department of Transport, RSA, 1997).

This work resulted in Equation 2.1

$$A_{EQ} = \left(\frac{AL}{8.16}\right)^n \qquad \text{Equation 2.1}$$

Where,

A_{EQ}= Axle load equivalence factor for individual axle

AL = Measured Axle Load in Tons

This formula compares the damaging effect on the road structure of any axle load, with that of a standard single-axle load of 8.16 tons (80kN). An average value of n = 4.2 was determined in this AASHO Road Test. The application of the Equivalency Factor makes it convenient to convert all axle loads and vehicle configurations into an equivalent number of standard axles. This relationship indicates that the increase in the damaging effect is exponential. Figure 2.1 shows that pavement life can be reduced by approximately 50% if axles are overloaded by 35%.

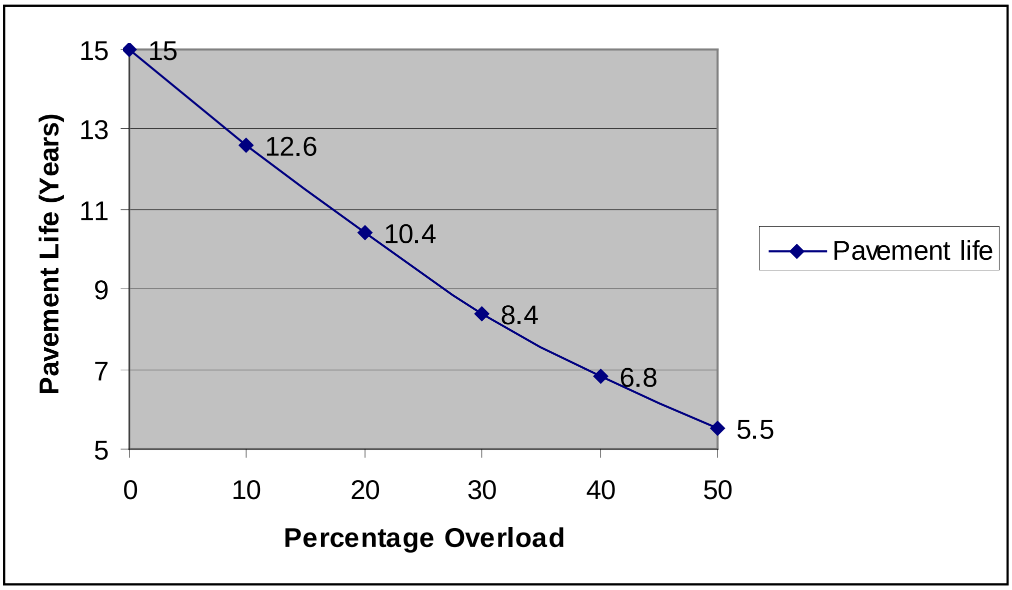


Figure 2.1 Pavement Life Reductions due to Overloading

Source: Kenya Roads Board (2014)

For Nairobi Thika Superhighway, the actual Axle load surveys conducted between 12 May 2007 and 17 May 2007 revealed that there was overloading of heavy goods vehicles on the network. These axle loads were converted into equivalence standard axles using Equation 2.1 and the value of n=4.5 (MoRPW, 2007).

Therefore, the Axle Load Equivalence Factor is given by Equation 2.2,

$$A_{EQ} = \left(\frac{AL}{8160}\right)^{4.5} \qquad \text{Equation 2.2}$$

Vehicle equivalence factors VEF for all vehicle types were computed by summing up the individual axle load equivalence factors for all axles (N) such that:

$$V_{EF} = \sum_{i=1}^{N} A_{EQ\,(i)} \qquad \text{Equation 2.3}$$

A summary of the computed vehicle equivalence factors was compared with the factors adopted by the Kenya Road Design manual Part III (RDM Part III, 1987). A comparison of these set of factors was summarised in Table 2.4

Table 2.4 Vehicle Equivalence Factors

Vehicle Type	V_{EF} Required by Kenya Road Design Manual Part III (Legal limits) (RDM, Part III, 1987)	Actual (Computed) V_{EF} for Nairobi - Thika Superhighway (MoRPW, 2007)
Large Bus	3.0	2.12
Light Truck	-	0.04
Medium Truck	3.4	4.27
Heavy Truck	3.4	7.78
Articulated	6.8	15.16

Adopted (MoRPW, 2007 and RDM Part III, 1987)

The Kenya Roads Board also conducted Axle Load Monitoring between 2012 and 2014. Table 2.5 summarised the overloading on Nairobi – Thika Road from October 2012 to September 2014.

Table 2.5 Percentage Overload per Axle on Nairobi – Thika Superhighway

REPORTING PERIOD	**QUARTERLY REPORT No.**	**% OVERLOAD PER AXLE**	
		Monitoring by Kenya National Highways Authority (KeNHA)	**Monitoring by Kenya Roads Board (KRB)**
October 2012 - December 2012	Q1	42.4%	60.5%
January 2013 - March 2013	Q2	54.7%	56.96%
April 2013 – June 2013	Q3	51.04%	64.39%
July 2013 - September 2013	Q4	40.02%	59.14%
January 2014 – March 2014	Q6	35.62%	35.62%
April 2014 – June 2014	G7	16.95%	16.95%
July 2014 – September 2014	Q8	26.83%	39.51%
Average		**38.22%**	**47.58%**

Source: Kenya Roads Board, (Quarterly Reports 2012, 2013, 2014)

Table 2.6 gives a summary of the comparison of the actual Vehicle Equivalence Factors obtained at the design stage with the recommended values by Kenya Road Design Manual Part III and the overloading values obtained by both the Kenya National Highways Authority the Kenya Roads Board independent axle load monitoring.

Table 2.6: Comparison of Vehicle Equivalence Factors on Nairobi–Thika Superhighway

Vehicle Type	V_{EF} Recommended by Kenya Road Design Manual Part III (Legal Limits)	Computed V_{EF} for Nairobi - Thika Superhighway (MoRPW)	Variance (%) (computed from (MoRPW)	Variance (%) (KeNHA)	Variance (%) (KRB)
Large Bus	3.0	2.12	-29%	-	-
Light Truck	-	0.04	-	-	-
Medium Truck	3.4	4.27	26%	38.22%	38.22%
Heavy Truck	3.4	7.78	129%	38.22	47.58%
Articulated	6.8	15.16	123%	38.22	47.58%

Adopted RDM III 1987, MoRPW (2007), Kenya Roads Board (Quarterly Reports 2012, 2013, 2014)

From Table 2.6, it was observed that the values adopted at design were higher than those recommended by the Ministry of Roads and Public Works (RDM III, 1987), and since overloading was still rampant on Kenyan roads, the Vehicle Equivalent Factors adopted in design stage were considered reasonable (MoRPW, 2007).

2.3.2 Traffic Forecast

The Kenya Road Design Manual Part III (RDM III, 1987) recommends that when more precise information is not available, the growth rate in traffic could be estimated from the growth rate of the Gross National Product (GNP) or the Gross Domestic Product (GDP). The Annual Average Gross Domestic Product for the previous decade (from 2003 to 2012) was summarised in Table 2.7. On the other hand, the Ministry of Roads and Public Works generated expected future growth scenarios for Nairobi-Thika Road corridor as summarised in Table 2.8, and these were the values used to generate the cumulative equivalent standard axles for the design of Nairobi – Thika Superhighway. For this research, the Annual Average GDP growth rate of 4.62% was used for the computation of the Cumulative Equivalent Standard Axles.

Table 2.7 Kenya Annual Average GDP growth rates from 2003 to 2012

Year	**Annual GDP Growth Rate (%)**
2003	2.9%
2004	5.1%
2005	5.9%
2006	6.3%
2007	7.0%
2008	1.5%
2009	2.7%
2010	5.8%
2011	4.4%
2012	4.6%
Average	**4.62%**

Source: Kenya National Bureau of Statistics, 2012

Table 2.8 Average Traffic Forecast growth rates

Period	Motor Cars	Utilities, Pick-ups, Parcel Vans	Lorries, Trucks, Heavy Vans	Matatu	Buses and Minibuses	Two Wheeler	Trailers	**Total Average for Period**
2005-2010	4.80%	4.88%	4.48%	3.53%	3.00%	6.00%	4.88%	**4.51 %**
2011-2015	4.62%	4.23%	4.23%	2.97%	2.53%	5.88%	4.23%	**4.10 %**
2016-2020	4.40%	3.25%	3.25%	2.48%	2.31%	5.72%	3.25%	**3.52 %**
2021-2025	4.32%	3.25%	3.25%	1.62%	1.70%	5.76%	3.25%	**3.31 %**
2026-2030	4.41%	3.25%	3.25%	1.30%	1.60%	5.39%	3.25%	**3.21 %**
Total Average for Vehicle Type	**4.51%**	**3.77%**	**3.69%**	**2.38%**	**2.23%**	**5.75%**	**3.77%**	**3.73 %**

Adopted: MoRPW, 2007

2.3.3 Calculating the Cumulative Equivalent Standard Axles

According to Kenya Road Design Manual Part III (RDM III – 1987), the cumulative number of standard axles, **T** over the chosen design period **N** (in years) is calculated by Equation 2.4

$$T = 365\, t_1 \frac{(1+i)^N - 1}{i} \qquad \text{Equation 2.4}$$

Where t_1 = the average daily number of standard axles in the first year of opening,

i = the annual growth rate expressed as a decimal fraction.

2.3.4 Design of flexible pavement thickness based on AASHTO method

The AASHTO Guide for Design of Pavement Structures (AASHTO, 1993) further gives a full description of the structural number (SN) method. The method can be used for new and rehabilitation pavement design. The method is based on the results of the AASHO road test done in Ottawa, Illinois during the late 1950s to early 1960s.

The structural number method is based on a reduction in the functional level of service of the pavement. The structural capacity estimation is based on a reduction in the Present Serviceability Index (PSI), a measure of riding quality. The basic formula to estimate the structural capacity of a pavement is given by Equation 2.5 (AASTHO, 1993)

$$\log_{10} W_{18} = Z_R * S_0 + 9.36*\log_{10}(SN+1) - 0.2 + \frac{\log_{10}\frac{(\Delta PSI)}{(4.2-1.5)}}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32*\log_{10} M_R - 8.07$$

Equation 2.5

Where

W_{18}	= Structural capacity of the pavement (Standard Axles) or the predicted number of 8.16 ton equivalent single axle load application;
ZR	= Standard normal deviate
S0	= Combined standard error of the traffic and performance predictions
SN	= Structural number of the total pavement thickness
∆PSI	= Difference between the initial (PSI_O) and terminal (PSI_t) serviceability indices

M_R = Effective roadbed (subgrade) resilient modulus adjusted for seasonal variation (psi)

The design procedures for both highways and low volume roads are all based on cumulative expected 18-kip (or 8.16 ton) equivalent single axle loads (ESAL) during the analysis period ($\widehat{w}_{8.16}$). Equation 2.6 was used to determine the traffic ($w_{8.16}$) (AASTHO, 1993)

$$w_{8.16} = D_D x D_L x \widehat{w}_{8.16} \qquad \text{Equation 2.6}$$

Where

$\boldsymbol{D_D}$ = a directional distribution factor. Since traffic counts were taken for each direction independently, a value 1.0 was adopted.

$\boldsymbol{D_L}$ = lane distribution factor. The analysis of the deflection was based on three lanes and the corresponding value of 0.6 of DESA was also adopted.

$\boldsymbol{\widehat{w}_{8.16}}$ = the cumulative two-directional 8.16ton ESAL units predicted for a specific section of highway during the analysis period

2.3.5 Reliability and Serviceability

The reliability design factor accounts for chance variations in both traffic prediction ($w_{8.16}$) and the performance prediction ($W_{8.16}$), and Therefore, provides a predetermined level of assurance (R) that pavement sections will survive the period for which they were designed. Application of the reliability concept requires the following steps:

- Defining the functional classification of the facility and determining whether a rural or urban condition exists as shown in Table 2.9
- Selecting a reliability level from the range given in Table 2.10. The greater the value of reliability, the more pavement structure required. It indicates the terminal serviceability indices for the road categories recommended by AASTHO (1993).
- A standard deviation ***(So)*** should be selected that is representative of local conditions.

The performance prediction error developed at the Road Test was 0.25 for rigid and 0.35 for flexible pavements. This corresponds to a total standard deviation for traffic of 0.35 and 0.45 for rigid and flexible pavements, respectively.

Considering the suggested levels of Reliability and Functional Classification in Table 2.9, Nairobi-Thika Superhighway was considered as classified under Interstate or Freeway and an average reliability factor $\boldsymbol{F_R}$ **of 95%** was selected and used to factor the design period traffic prediction ($w_{8.16}$) to produce design applications ($W_{8.16}$) (AASTHO, 1993).

For a given reliability level (R), the reliability factor is a function of the overall standard deviation $\boldsymbol{(S_0)}$ that accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given $W_{8.16}$. This was adopted for this research and the corresponding **Z value of 1.645** obtained from the Standard Normal Tables (AASTHO, 1993).

Table 2.9 Suggested Levels of Reliability for Various Functional Classifications

Functional Classification	Recommended Levels of Reliability	
	Urban	Rural
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

Source: AASTHO 1993

Table 2.10: Serviceability Indices for Road Categories

Terminal Serviceability Level	Percent of People Stating Unacceptable
3.0	12
2.5	55
2.0	85

Source: AASTHO, 1993

AASTHO (1993) also suggested a Terminal Serviceability P_t index of 2.5 or higher for the design of major highways, and an Initial Serviceability Index P_0 of 4.2. Therefore, the serviceability loss is given by equation 2.7

$$\Delta \boldsymbol{PSI} = (\boldsymbol{P_0} - \boldsymbol{P_t}) \qquad \text{Equation 2.7}$$

2.3.6 Traffic Classes

The Kenya Road Design Manual Part III (RDM PART III, 1987) recommends that the predicted number of equivalent standard axles be divided into the classes summarized in Table 2.11. These were deemed to account for all traffic categories likely to be carried by the bitumen roads in Kenya. The Cumulative Equivalent Standard Axles established for Nairobi – Thika Superhighway during design were 177 million on the left carriageway (Kenyatta University to Thika) and 155million on the right carriageway (Thika to Kenyatta University) for 20 years design life (MoRPW, 2007). These figures were in excess of 60 million which is the maximum limit for designing the pavement following the Kenyan Road Design Manual Part III. Subsequently the design guidelines of AASHTO were adopted in this research. However, the design of Nairobi-Thika Superhighway was based on the AASTHO Method and Therefore, the huge loading was considered in design.

Table 2.11: Traffic Classes in Kenya

Class	Cumulative number of standard axles (8.16 tons)
T1	25 – 60 million
T2	10 – 25 million
T3	3 – 10 million
T4	1 – 3 million
T5	0.25 – 1 million

Source: Kenya Road Design Manual Part III, 1987

2.3.7 The Structural Number (SN)

The Structural Number (SN) is an index providing an indication of the strength of the pavement layers and of the total pavement structure. It is an empirical approach derived by taking the layer material type specific coefficient multiplied by the layer thickness and the sum of these are then the pavement Structural Number (SN). In essence, it is the sum of the strengths of all the layers in the pavement and it is used as an indicator to determine the strength of a total pavement structure. The SN determines the total number of ESALs (Equivalent Single Axle Loads) that a particular pavement can support. The most commonly used equivalent load is 8160 kg (80kN) or 18 kips (Kilo pounds) (Schnoor, Horak, 2012).

Structural layer coefficient (ai) is the parameter for representing the relative strength of individual pavement layer materials. The strength of an individual layer (i) of thickness (Di) is assessed as the product of ai, Di and mi. Each layer in the pavement contributes to the structural number according to a layer coefficient depending on material type, the thickness of the layer and a drainage coefficient for the layer, calculated using Equation 2.8.

$$SN = a_1D_1 + a_2D_2m_2 + a_3D,\ m_3 \qquad \text{Equation 2.8}$$

SN= (for all xai Di mi)

Where,

SN=Structural Number of pavement

ai =structural layer Co-efficient of i-th layer

Di=thickness of i-th pavement layer in inches

mi=drainage coefficient for the layer.

The first step in the process of estimating the structural capacity of a pavement with the SN method is to determine the effective roadbed resilient modulus, which is an average subgrade resilient modulus adjusted for seasonal changes. Each layer is then assigned a layer coefficient, representing the strength of the material. The value of the layer coefficients increases with increasing material quality.

The layer coefficients ai, can be determined by the following equations that are based on the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide (Noureldin et al., 2005):

Surface layer coefficient,

$$a_1 = \left(\frac{Temperature\ corrected\ Surface\ Modulus\ in\ Ksi}{11x10^3}\right)^{\frac{1}{3}} \qquad \text{Equation 2.9}$$

Support layer coefficient,

$$a = \left(\frac{Supprt\ Modulus\ in\ Ksi}{11x10^3}\right)^{\frac{1}{3}} \qquad \text{Equation 2.10}$$

Conversion of Ksi to MPa in SI units

$1\text{MPa} = \frac{Ksi}{0.1450377}$

Where Ksi = kilo-pound per square inch

Table 2.12 is a summary of the structural layer coefficients and Equivalent Pavement Layer Moduli for the design of Nairobi-Thika Superhighway pavement.

Table 2.12: Structural layer coefficients for Nairobi – Thika Superhighway

Layer	Equivalent Modulus	Structural coefficient
AC	E_{AC}=40,000 kg/cm^2	0.40
DBM	E_{DBM}=50,000kg/cm^2	0.35
GCS	E_{GCS}=30,000kg/cm^2	0.17
SUB-BASE	$E_{SUBBASE}$ =20,000kg/cm^2	0.12

Source: MoRPW, 2007

The drainage coefficients were also considered as 1.0 for m1 (value for conditions at the AASHO Road Test is 1.0, regardless of the type of material) and 1.1 for m2 and m3 (for good quality drainage material such as GCS and Cement Improved Gravel exposed to moisture levels approaching saturation by between 5% - 25% of the time) (AASTHO, 1993)

Consequently, the detailed process of thickness calculations is outlined in the following steps:

i. Calculate SN3 (SN sub-grade)based upon MR value of sub-grade;

ii. Calculate SN2(sub-base) based upon E-Equivalent value of sub-base;

iii. CalculateSN1 (base) based upon E-Equivalent value of base course

iv. Calculate thickness of bituminous material

$D1=SN_1/a_1$ Equation 2.11

a1=structural layer coefficient of bituminous concrete

Provide bituminous thickness D1 as per constructability criteria

$SN_1=D_1a_1$ Equation 2.12

v. Calculate thickness of bituminous material

$D_2=SN_2-SN_1/a_2$ Equation 2.13

a_2=structural layer coefficient for base material

Provide D_2 as per constructability requirement

$SN_2=D_2xa_2$ Equation 2.14

vi. Calculate thickness of sub-base material

$D_3=SN_3-SN_2/a_3$ Equation 2.15

a_3=structural layer coefficient for sub-base material

Provide D_3 as per constructability requirement

vii. Calculate overall structural number provided:

$SN_{provided}= (_{for\ all}\ x\ ai\ Di\ mi)$ Equation 2.16

viii. Compare $SN_{provided}>SN3$

The design Structural Number for Nairobi Thika Superhighway was established as SN = 5.8 (MoRPW, 2007)

2.3.8 Pavement Materials

- **<u>Resilient Modulus of Subgrade Reaction (M_R)</u>**

For a rational approach to pavement design, the most important characteristic of the subgrade is its elastic modulus. However, it has been established that there is relationship between the California Bearing Ratio (CBR) and the Resilient Modulus of Subgrade Reaction (M_R). The modulus of subgrade reaction, (M_R), (also referred to as Coefficient of Elastic Uniform Compression, Cu) is a relationship between soil pressure and deflection which is proportional to its vertical displacement as idealized in Winkler's soil model (Hetenyi, 1946; Jones, 1997).

It can also be defined as the ratio of uniform pressure imposed on the soil to the elastic part of the settlement (Kameswara, 2000). Some work that has been reported for the correlation between modulus of subgrade reaction (M_R) and CBR test though the mechanism of deformation is similar. Heukelom and Klomp (1962) studied the correlation of CBR with E and proposed on empirical relationship in Equation 2.17 as,

E = 1500 CBR (Psi) Equation 2.17

However, this correlation is only for fine grained non expansive soils with a soaked CBR < 100% (AASHTO, 1993). The constant of proportionality of 1500 can also vary quite considerably from 0.5 to 2 times that amount. Heukelom and Klomp (1962) obtained field measurements of the resilient modulus based on vibratory loading.

Moreover, Powell et.al (1984) proposed a correlation of the CBR with E in Equation 2.18 as,

E = 17.6 $CBR^{0.64}$ (MPa) Equation 2.18

The correlation between E and CBR developed by NAASRA (1950) has been divided into two parts.

For CBR less than 5%,

$$E = 16.2\ CBR^{0.7}\ \text{(MPa)} \qquad \text{Equation 2.19}$$

Then, for CBR more than 5%,

$$E = 22.4\ CBR^{0.5}\ \text{(MPa)} \qquad \text{Equation 2.20}$$

A survey of the Kenyan subgrade soils by the Kenya Ministry of Roads and Public Works (MoRPW, 1978, Report No. 345) showed that they can be grouped into six bearing strength groups. The E-Modulus for the corresponding classes were computed and summarized in Table 2.13.

Table 2.13: Subgrade Bearing Strength Ranges and E-Modulus in Kenya

Soil class	CBR Range	Median	Heukelom and Klomp (1962)		E-Modulus (Powell et.al (1984))	E-Modulus (NAASRA (1950))	
			E = 1500CBR (Psi)	E = 15CBR (MPa)	E = 17.6 $CBR^{0.64}$ (MPa)	E = 16.2 $CBR^{0.7}$ (MPa) for CBR < 5%	E = 22.4 $CBR^{0.5}$ (MPa) for CBR > 5%
S1	2 - 5	3.5	5250	52.5	39.24	38.94	-
S2	5 - 10	7.5	11250	112.5	63.91	-	61.34
S3	7 - 13	10	15000	150	76.83	-	70.84
S4	10 - 18	14	21000	210	95.29	-	83.81
S5	15 - 30	22.5	33750	337.5	129.10	-	106.25
S6	30	30	45000	450	155.19	-	122.69

Source: Adopted (MoRPW,1987, Heukelom and Klomp (1962), (Powell et.al (1984)) (NAASRA (1950))

The design of Nairobi to Thika Superhighway considered the structural strength of the existing subgrade in terms of the Resilient Modulus of Subgrade Reaction, MR. These values were established and summarized in Tables 2.14 and 2.15 between Kenyatta University and Thika Town (MoRPW, 2007). From table 2.14, it was observed that the subgrade soils for the various homogeneous sections on the Left Carriageway were between S4 and S6 based on the classification in Table 2.13. Similarly for the Right Carriageway (Table 2.15), the subgrade soils were between S3 and S4.

Table 2.14: Residual Modulus of Subgrade Reactions, from Kenyatta University to Thika

From	To	Residual Modulus, M_R (MPa)
00+000	02+900	171
02+900	04+300	276
04+300	12+200	292
12+200	13+200	478
13+200	16+200	408
16+200	18+400	198
18+400	19+900	185
19+900	21+600	270
21+600	24+200	322

Source: Adopted from MoRPW (2007)

Table 2.15: Residual Modulus of Subgrade Reactions from Thika to Kenyatta University

From	To	Residual Modulus, M_R (MPa)
00+000	01+800	185
01+800	04+200	203
04+200	05+100	190
05+100	06+500	237
06+500	09+900	142
09+900	10+300	115
10+300	12+500	126
12+500	14+000	116
14+000	15+600	147
15+600	21+800	201
21+800	24+100	279

Source: Adopted from MoRPW (2007)

- **Pavement layers**

The following moduli have been attributed to the various pavement materials in Kenya (RDM PART III, 1987).

a) Subbase Materials:
   - Natural Material — $2x10^5$ kN/m^2 (2,000 kg/cm^2)
   - Cement or Lime Improved Material — $3x10^5$ kN/m^2 (3,000 kg/cm^2)
   - Graded Crushed Stone — $3x10^5$ kN/m^2 (3,000 kg/cm2)

b) Base Materials:
   - Natural gravel — $3x10^5$ kN/m^2 (3,000 kg/cm^2)
   - Cement or Lime Improved Material — 10^6 kN/m^2 (10,000 kg/cm^2)
   - Cement Stabilized Gravel — $4x10^6$ kN/m^2 (40,000 kg/cm^2)
   - Graded Crushed Stone — $4x10^5$ kN/m^2 (4,000 kg/cm^2)
   - Sand Bitumen Mix — 10^6 kN/m^2 (10,000 kg/cm^2)
   - Dense Bitumen Macadam — $5x10^6$ kN/m^2 (50,000 kg/cm^2)
   - Lean Concrete — 10^7 kN/m^2 (100,000 kg/cm^2)

c) Surfacing Materials:
   - Asphalt Concrete
     Type I (High Stability) — $4x10^6$ kN/m^2 (40,000 kg/cm^2)
   - Asphalt Concrete
     Type II (Flexibilty),
     Sand and Gap Graded Asphalt — $2.5x10^6$ kN/m^2 (25,000 kg/cm^2)

The Nairobi Thika Superhighway was designed based on the AASTHO method of design described in section 2.3.1. Considering the material properties in section 2.3.8, the thickness required for the section under study (from Kenyatta University to Thika Town) was summarized in Table 2.16.

Table 2.16 Nairobi – Thika Superhighway As-built Pavement layers

Layer	Thickness	E-Modulus
Asphalt Concrete (AC) Surfacing Type I	50mm	$5x10^6$ kN/m^2 (50,000 kg/cm^2)
Dense Bitumen Macadam (DBM) Base course	150mm	$5x10^6$ kN/m^2 (50,000 kg/cm^2)
Graded Crushed Stone (GCS) Road Base	250mm	$4x10^5$ kN/m^2 (4,000 kg/cm^2)
Cement Improved Gravel Sub-base	250mm	$3x10^5$ kN/m^2 (3,000 kg/cm^2)
Total Pavement Thickness	**700mm**	

Source: MOR&PW, 2007

2.4 Pavement Evaluation

The structural adequacy of a pavement is defined as its ability to carry traffic without developing appreciable structural deterioration. It is dependent upon proper construction with suitable materials and of sufficient thicknesses to prevent traffic from overstressing the subgrade or any other pavement layers. An evaluation of the existing pavement is necessary to determine its adequacy and to decide on the maintenance or rehabilitation measures, which may be needed to meet future demands. Pavement evaluation includes both surface condition ratings and structural adequacy ratings. In the late 1950s, systems of objective measurement (such as roughness meters, deflection and skid test equipment) began to appear that could quantify a pavement's condition and performance (Muench, Mahoney and Pierce, 2003). These systems, along with visual distress surveys, were used to aid in making maintenance and rehabilitation decisions, which, over the years have been refined and upgraded to provide rapid, objective means to:

i. **Establish maintenance priorities -** Condition data such as roughness, distress, and deflection are used to establish the projects most in need of maintenance and rehabilitation. Once identified, the projects in the poorest condition (low rating) will be more closely evaluated to determine repair strategies;

ii. Determine **maintenance and rehabilitation strategies** - Data from visual distress surveys are used to develop an action plan on a year-to-year basis. The most appropriate strategy is then adopted for a given pavement condition;
iii. Predict **pavement performance** - Data, such as ride, skid resistance, distress, or a combined rating, are projected into the future to assist in preparing long-range budgets or to estimate the condition of the pavements in a network given a fixed budget.

2.5 Surface condition

Surface condition ratings give an indication of how well the road is serving the travelling public. According to (Molenar, 2009), the thickness design and the material selection should be such that some major defect types are under control during the design life. This means that they should not appear too early, and that they can be repaired easily if they appear. Major defect types that can be observed on flexible pavements are cracking, deformations, disintegration and wear.

The surface condition surveys provide valuable and necessary information, but are not sufficient to judge the structural adequacy of the pavement. The results of the pavement condition surveys are mainly used to:-

i. assess the effects on the road user,
ii. establish the probable causes of surface distress,
iii. determine the need for, and establish priorities for, maintenance operations and surface rehabilitation,
iv. determine the need for structural evaluation and
v. Assess the rate of pavements deterioration, so the approximate time for planning future work or for carrying out another condition survey can be predicted.

A condition assessment can be based on one or a combination of the following: -

i. Measurements of surface distress, showing locations and extent of each defect observed.
ii. Measurements of surface roughness.
iii. Subjective rating of the pavement riding quality and surface condition.

2.5.1 Surface Roughness

Pavement roughness is defined as an expression of irregularities in the pavement surface that adversely affect the ride quality of a vehicle and the user comfort. There are a number of established techniques for measuring surface roughness. Roughness is an important pavement characteristic because it affects not only ride quality but also vehicle operating costs, fuel consumption and maintenance costs. Roughness is typically quantified using present serviceability rating (PSR) and international roughness index (IRI), with IRI being the most prevalent. The Present Serviceability concept was developed in connection with the AASHO Road Test, and presents serviceability as the ability of a specific section of road to provide a smooth, safe and comfortable ride at that particular time. A present day serviceability value may be obtained by either subjectively rating the pavement through visual observations (present serviceability rating) or by quantitative measurement of surface characteristics (present serviceability index) (University of Michigan, 2002).

The international roughness index (IRI) was developed by the World Bank in the 1980s and is used to define a characteristic of the longitudinal profile of a travelled wheeltrack and constitutes a standardized roughness measurement, usually reported in meters per kilometre (m/km) or millimetres per meter (mm/m). The IRI is based on the average rectified slope (ARS), which is a filtered ratio of a standard vehicle's accumulated suspension motion (in mm, inches, etc.) divided by the distance travelled by the vehicle during the measurement (km, mi, etc.). IRI is then equal to ARS multiplied by 1,000. The open-ended IRI scale is shown in Figure 2.6 (University of Michigan, 2002).

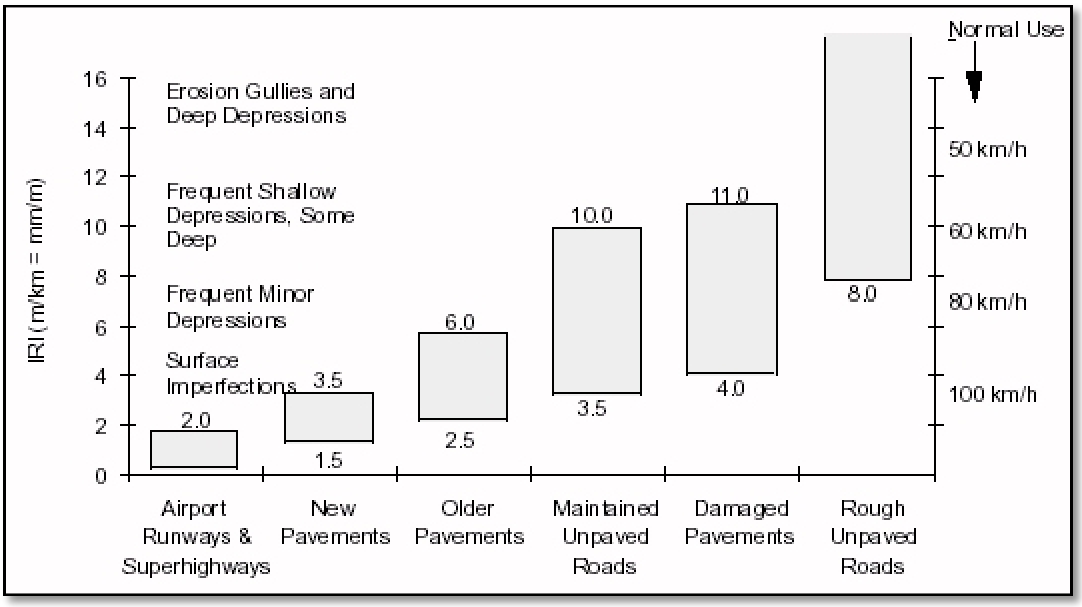


Figure 2.2: IRI Roughness Scale (University of Michigan, 2002)

2.5.2 Present Serviceability Index (PSI)

The Present Serviceability Index is an equation that can be used, together with results of measured surface defects and roughness, to quantify a road section's ride-ability. The PSI for flexible pavements may be computed from Equation 2.21 (Kenya Road Design Manual Part V, 1988).

$$PSI = AO + A1 \cdot \log[1 + SV] + A2 \cdot RD^2 + B1[C + P]^{1/X} \qquad \text{Equation 2.21}$$

Where:

SV = the mean of the slope variance in the two wheel paths

RD = the mean wheel path rut depth

C = the percentage of pavement surface with Class 2 or 3 cracking and crazing.

P = percentage of pavement surface patched and pot-holed A_0, A_1, A_2 and B_1, are coefficients depending on the equipment used for measuring slope variance.

The PSI is mainly dependent upon the roughness of the pavement surface and consequently a simplified PSI may be determined from the following equation: -

$$PSI = 5.00 - a \cdot R - b \cdot \log R \qquad \text{Equation 2.22}$$

Where " R " is roughness and "a" and "b" are coefficients subject to the following

i. The above coefficients depend on the country, types of pavements analyzed and the equipment used for measuring longitudinal profile variations or roughness.
ii. A single PSI value is not itself a measure of absolute pavement performance but it is representative of the trend of serviceability that gives indications about the performance of the pavement.

2.5.3 Present Serviceability Rating (PSR)

According to the Kenya Road Design Manual Part V (RDM V, 1988), PSR involves the use of a group of raters who ride the pavement section, observe its riding quality, assess its condition and record their impressions on a standard form. Ratings vary from "0" (very poor) to "5" (very good). Low ratings indicate poor surface condition and suggest that a more detailed examination of the pavement is required. The PSR may be used as a first step in evaluating the adequacy of a pavement.

2.6 Pavement Deflections

2.6.1 Introduction

Pavements deteriorate due to the combined influences of traffic and environmental loads. This means that at a given moment, maintenance activities should be scheduled in order to restore the level of service that the pavement should give to the road user. Deflection-based measurements and Dynamic Cone Penetrometer (DCP) tests are used to help identify the cause of differential performance between sub-sections and to provide information for the maintenance or rehabilitation of the section. (Overseas Road Note 18,)

2.6.2 Deflection measurement tools

According to the United States Department of Transport, (US, Advisory Circular 150/5370-11B), deflection measuring equipment for nondestructive testing of pavements can be broadly classified as static or dynamic loading devices. Dynamic loading equipment can be further classified according to the type of forcing function used, whether vibratory or impulse devices. Non-deflection measuring equipment that can supplement deflection testing includes ground-penetrating radar, infrared thermography, dynamic cone penetrometer, and devices that measure surface friction, roughness, and surface waves. There are several categories of deflection measuring equipment: static, steady state (for example, vibratory), and impulse load devices. A static device measures deflection at one point under a non-moving load. Static tests are slow and labor intensive compared to the other devices. Examples of static devices include the Benkleman Beam and other types of plate bearing tests.

Vibratory devices induce a steady-state vibration to the pavement with a dynamic force generator and there is a small static load that seats the load plate on the pavement. The dynamic force is then generated at a pre-computed frequency that causes the pavement to respond (deflect). The pavement deflections are typically measured with velocity transducers. There are several types of steady-state vibratory devices, including Dynaflect and Road Rater. Impulse load devices, such as the Falling Weight Deflectometer (FWD), impart an impulse load to the pavement with free-falling weight that impacts a set of rubber springs. The magnitude of the dynamic load depends on the mass of the weight and the height from which the weight is dropped (US, Advisory Circular 150/5370-11B).

Whereas the deflection device that currently receives the highest popularity is the Falling Weight Deflectometer (FWD); other deflection measuring devices like the Benkelman Beam (BB) and the Lacroix Deflectograph (LD) are still used in different places of the world. Table 2.17 lists the more common deflection devices, their loading regimes and output. The FWD equipment was used in the deflection survey used in this research.

Table 2.17: Deflection Measurement Devices

Device	Type of applied load	Output
Deflection Beam	Moving Wheel	Maximum deflection
Deflectograph	Moving Wheel	Deflection Bowl
Road Rater	Vibratory Load	Deflection Bowl
Dynaflect	Oscillatory Load	Deflection Bowl
Falling Weight Deflectometer (FWD)	Impact Load	Deflection Bowl

Source: *Advisory Circular 150/5370-11B*

2.6.3 Falling Weight Deflectometer, FWD

Falling Weight Deflectiometer (FWD) testing allows the structural condition of a road pavement to be rapidly assessed non-destructively and they are effective and economical for the collection of information on the structural condition of road and airport pavements. FWD are key to precise performance assessment when carrying out ongoing maintenance work or when rehabilitating and reinforcing existing pavements. FWD machines are either van mounted as shown in Figure 2.3, or trailer mounted as shown in Figure 2.4. The machines follow the modular principle, meaning that there is no need for replacement with new equipment if the demands on the equipment change.

The Falling Weight Deflectometer (FWD) has the advantage of being able to apply impact loads. These impact loads more accurately simulate the effect on pavements of heavy vehicles moving at normal traffic speeds than the slowly moving load applications associated with the Deflectograph or the deflection beam.

Figure 2.3: Van Mounted FWD Equipment

Source: Grontmij, Carl Bro A/S, 2010

Figure 2.4: Trailer mounted FWD machine

Source: Grontmij, Carl Bro A/S, 2010

2.7 Application of FWD test results

2.7.1 Elastic Layer Theories

Analysis of pavement deflections is based on Elastic layer Theories. Elastic-Layered theory assumes the soil material to be an elastic, isotropic, semi-infinite body. In this theory, the term “elastic " means that the stiffness of the layer is independent of the rate at which the load is applied and is constant throughout a range of load magnitude. In a layered linear elastic model of a pavement, each layer is characterized by its Young's modulus of elasticity, E, and Poisson's ratio, μ. Reasonable values of Poisson's ratio are assumed for different pavement materials (Mehta, 1990).

In 1885, Boussinesq did the first work assuming the characterization of supporting layers in pavement system as elastic solids through the assumption of the soil to be linearly elastic, isotropic, homogeneous solid of infinite extent in both horizontal directions. Boussinesq considered the case of an elastic, isotropic, homogenous and infinite half pace with the assumption that elastic properties are identical in every direction under uniform circular loading (Gichaga & Parker, 1988).

$$\delta z = P\ \{1 - z^3 / (a^2 + z^2)^{3/2}\} \quad \text{……………………Equation 17}$$

$$\delta x = \delta y = P/2\ \{(1 + 2\mu) - 2(1 + \mu)/ (a^2 + z^2)^{1/2} + z^3 / (a^2 + z^2)^{3/2}\} \qquad \text{Equation 2.23}$$

Where;

P = Applied surface pressure

$\delta x = \delta y$ = Horizontal stress on vertical axis of loading

δz = Vertical stress along the vertical axis of loading

a = Radius of applied circle of loading

z = Distance of the point from the surface

μ = Poisson’s ratio

In the mid-1940's Burmister applied the elastic solids concept of Boussinesq to two and three layer systems for the analysis of stresses and deflections in flexible pavements. In his work, Burmister found that stiff upper layers reduce stresses and deflections in the subgrade from those predicted by Boussinesq. This reduction is proportional to the ratio of the elastic moduli (Mehta, 1990).

$$\Delta = 1.5pa\ Fw / E2 \qquad \text{Equation 2.24}$$

Where;

Δ = Vertical deflection in inches

P = Intensity of applied loading or the contact pressure

A = Radius of circular area of loading

E1 = Modulus of elasticity of the top layer of the pavement structure

E2 = Modulus of elasticity of the lower layer of the pavement structure

Fw = Displacement factor which depends on the thickness of the top layer and the ratio E1/E2 (Ranges from 0.02 to 1.0 for ratios of E1/E2 between about 10000 and 2 respectively).

In Burmister's theory, the following assumptions are made (Burmister, 1943; 1945a):

i. Each layer is homogenous, isotropic, and linearly elastic with an elastic modulus E and a Poisson's ratio μ;
ii. The surface layer is weightless and infinite in extent in the horizontal direction, but finite in vertical direction. The subgrade is infinite in extent in both horizontal and vertical directions;
iii. The surface layer should be free of shearing stress and normal stress beyond the surface loading. The subgrade should be free of stress and displacement at infinite depth; and
iv. Continuity conditions at layer interfaces are satisfied.

2.7.2 Back-calculation Algorithms for Layer Moduli

Deflection measurements with the Falling Weight Deflectometer (FWD), and in turn modulus calculation through back-calculation have been routinely employed in evaluating pavement layers, and the underlying subgrade. Most of the back-calculation procedures in use are based on elastic layer theory to calculate Young's Modulus (modulus of elasticity) for each structural layer within the pavement, such that the difference between the measured and predicted basins is minimal. The purpose of back-calculation is primarily to find out the in-situ elastic moduli (E) of the different pavement layers (Das and Pandey 1998).

Due to the idealizations involved in the back-calculation process of a pavement structure, numerical errors get introduced during iterations, and Therefore, no unique solution may be achieved in the back-calculation process (Ceylan et al. 2005, Chou and Lytton 1991; Hall and Mohseni 1991, Ulliditz and Stubswtad 1985). Nevertheless, various approaches based on closed-form solutions, regression and database search methods, optimization techniques and a combination of these methods have been proposed for back-calculation of layer moduli (Goel and Das 2008; Sharma and Das 2008).

Back-calculation analysis can be classified into several categories, depending on the type of load representation and the type of material characterization. Among all the types of back-calculation methods, the static linear back-calculation is generally preferred in the majority of pavement back-calculation studies because of its simplicity and acceptable error ranges (Setiadji B H, 2009**,** Goktepe et al., 2006).

Fwa (1998), Harichandran et al. (1994) and Goktepe et al. (2006) provided detailed descriptions of the various approaches of the static linear back-calculation currently available for the purpose of back-calculation analysis.

i. One approach makes use of theoretical closed-form solutions to directly compute the elastic modulus of each layer by using layer thickness and deflections from one or more sensors (Li et al., 1996; Fwa et al., 2000).
ii. Another approach of back-calculation involves an iterative process that varies the various pavement layer moduli until a sufficiently close match between the computed and measured deflections is obtained (Hall et al., 1996; Khazanovich et al., 2000; Almedia et al., 1994).
iii. A third approach relies on an appropriate database that pre-calculates solutions based on measured deflections for a large number of pavement sections, and stores them in an organized database. The pavement structure in the database that has its deflection basin that best matches the measured deflection basins is picked as the solution. This approach is often termed as database search algorithm (Lytton, 1989; Uzan, 1994; Tia et al., 1989).
iv. The fourth approach is regression-equation based methods that relate surface deflections to pavement layer moduli using statistical regression techniques (Fwa and Chandrasegaran, 2001; Harichandran et al., 1994).

2.7.3 Back-calculation Software

There are computer programmes available for the analysis of deflection data by back-calculating material response parameters for each layer within the pavement structure from deflection basin measurements. According to the US Department of Transport's Federal

Highway Administration (FHW A-RD-97-076, 1997), these methods and programs can be grouped into four basic categories. These categories are:

- Static (Load Application) - Linear (Material Characterization) Methods;
- Static (Load Application) - Non Linear (Material Characterization) Methods;
- Dynamic (Load Application) - Linear (Material Characterization) Methods;
- Dynamic (Load Application) - Non Linear (Material Characterization) Methods.

Some of the software that has been used to back-calculate layer moduli over the past several years include BISDEF, CHEVDEF, ELMOD, ELSDEF, EVERCALC, ISSEM4, MODCOMP, MODULUS, WESDEF and RoSy Design for Roads. Although many of the software packages have similarities, the results generated from the same set of data by various programs can be different. These differences are a result of the type of iteration scheme used and the modulus calculation routine employed.

Most of these programs are limited by the number of layers and the thickness of those layers within the pavement and are based on linear elastic material assumptions. Consequently, any discontinuity cannot be physically represented by the model. Thus, the calculated layer moduli represent effective or equivalent values that take into account anomalies (such as cracks and voids), thickness variations within each layer, and a combination of layers with similar materials or thin layers with thick layers. Layer thickness is an extremely important feature when back-calculating layer moduli from deflection basin test results. A 10 percent difference in thickness can result in more than a 20 percent change in the calculated modulus. Thus, using accurate layer thicknesses becomes critically important (FHW A-RD-97-076, 1997).

2.7.4 RoSy Software for Road Design

RoSy Design for Roads has been proposed for the back-calculation in this Thesis. The RoSy Design Software is part of the total RoSy Pavement Management System (PMS) for optimum maintenance and control of a specific road network. The system works on the basis of the idea of collecting and storing road data and bridge data in one place and at the same time to organize the data, so that an optimum possibility of data retrieval and import is obtained. The RoSy Design program uses back-calculation method. The calculation of the deflection of a road surface is based on the theory of elasticity and the method of equivalent thickness on the basis of Boussinesq’s equations. The deflection is the sum of the deformation in the layers and the sub-grade. The deformation of one layer is linearly elastic implying that the deformation is directly proportional to the force and the thickness of the layer but inversely proportional to the stiffness of the layer (Grontmij, Carl Bro A/S, 2009).

According to Grontmij, Carl Bro A/S, 2009, the programme back-calculates:

i. E Moduli for road structures consisting of 1-4 layers. In the calculations the deflections measured on the road surface are compared to the corresponding calculated deflections. The sub-grade parameters are first calculated on the basis of the deflection values at the furthest distance from the plate. Then the E Moduli of the pavement layers are calculated by iteration. The iteration is discontinued when a satisfactory conformity between the measured and calculated deflections has been obtained.

ii. The Remaining Life. This is defined as the period from the time of measurement in which the road structure can tolerate the implied traffic load without exceeding the allowable strain in the pavement layers. With the knowledge of the Design Standard Axle Load during the coming year and the traffic growth rate per year, the remaining structural life can be calculated by RoSy.

iii. The Strengthening Layer. This is calculated according to the design period if the remaining life is less than this period.

2.8 Summary of Literature Review

Literature from the Kenya Ministry of Roads and Public Works (MoRPW, 2007) and Kenya Roads Board (KRB, 2012-2014) was reviewed on the Vehicle Axle Loads along Nairobi-Thika Superhighway. It enabled the establishment of the Vehicle Equivalent factors used in the research. The average traffic growth rate of 4.62% was obtained from the average of Kenya GDP growth rates from the year 2003 and 2012 (KNBS, 2012). It was used to obtain cumulative equivalent standard axles (CESA), projected for 5years, 10years, 15years and 20years respectively.

Literature on the pavement design and the as-built records on Nairobi Thika superhighway were reviewed. The Resilient Modulus of Subgrade Reaction (M_R) of the pavement alignment and the as-built pavement materials properties in terms of their Equivalent Pavement Modulus (E_P) were also obtained from the design records (MoRPW, 2007). A comparison between the as-built layer properties and the back-calculated values would indicate the performance of the flexible pavement. Therefore, the thesis provided the baseline performance evaluation of the pavement upon which future research would be based.

3 METHODOLOGY AND DATA COLLECTION

Lanes were appropriately denoted along the Study Area. Each lane was analyzed independently. Homogeneous Sections were identified for each lane. Intervention measures were recommended per homogeneous section per lane.

3.1 Lane denotations

The lanes have been denoted as follows

- Lane 1 – Outer Lane,
- Lane 2 – Middle Lane
- Lane 3 – Inner Lane, next to the median.
- Lane 4 – Inner Lane, after Lane 3 where there are 4 lanes.

Field investigations were conducted to collect relevant data. These investigations focused on the items given in the following sections;

3.2 Data collection and reliability

Traffic survey and analysis was conducted by the author between 07 May 2013 and 13 May 2013 based on the methodology provided in Overseas Road Note 40 and the Average Daily Traffic (ADT) computed. Road condition survey and deflection survey conducted by the Kenya Ministry of Roads (Materials Brach) in the year 2012, which was one year after opening the road to traffic. The equipment used by the Ministry of Roads was the Falling Weight Deflectometer (FWD) for deflection survey and the Bump Integrator for road condition survey. Each of the equipment was properly calibrated to give accurate and reliable data. The timing of the surveys also ensured that the data represented baseline conditions since it was only a year after opening the road to traffic.

Although the Axial Load Surveys were not done, a review of the reports of Axial Loads monitoring by the Kenya Roads Board (KRB) and Kenya National Highways Authority (KeNHA) in section 2.3.1 revealed that overloading practices still existed the Kenyan roads. Therefore, the use of the design Vehicle Equivalent Factors (which were computed from the actual axial load surveys done in the year 2007) was considered reasonable because the design of the flexible pavement was done when overloading practices were very rampant on the Kenyan roads.

3.3 Roughness Measurements

Roughness measurements along the road were taken using a bump integrator. The measurements were reported in terms of International Roughness Index (IRI). A summary of the International Roughness Index for each lane was presented in section 4.1.

3.4 Traffic survey

Traffic survey was conducted at the Nairobi – Thika Superhighway at the Eastern Bypass from Tuesday 7 May 2013 to Monday 13 May 2013. For the seven days, manual classified 12- hour counts were taken on five days, and 24 hour counts taken for two days. From these counts, 12-hour to 24-hour conversion factors were generated as shown in Tables 3.1 and 3.2.

Table 3.1: 12hr/24hr traffic conversion factors - Kenyatta University to Thika

	TUE - 6am - 6pm	TUE - 6pm - 6am	TUE 24 HR	**TUE 24/12Hr Ratio**	SAT 6am-6pm	SAT 6pm-am	SAT-24HR	**SAT-24/12Hr Ratio**
Bus	492	164	656	**1.333**	345	227	572	**1.658**
Light Trucks	587	80	667	**1.136**	473	204	677	**1.432**
Medium Trucks	635	447	1,082	**1.704**	591	774	1,365	**2.310**
Heavy Trucks	773	213	985	**1.275**	701	365	1,066	**1.521**
Articulated Trucks	354	204	558	**1.576**	350	366	716	**2.047**

Table 3.2: 12hr/24hr traffic conversion factors - Thika Town to Kenyatta University

	TUE - 6am - 6pm	TUE - 6pm - 6am	TUE 24HR	**24Hr/12Hr Ratio**	SAT 6am-6pm	SAT 6pm-am	SAT-24HR	**SAT-24Hr/12Hr Ratio**
Bus	453	176	629	**1.389**	446	125	571	**1.279**
Light Trucks	442	97	539	**1.220**	597	129	726	**1.216**
Medium Trucks	662	446	1,108	**1.674**	602	402	1,004	**1.668**
Heavy Trucks	675	203	878	**1.301**	604	183	787	**1.303**
Articulated Trucks	297	161	458	**1.542**	267	139	406	**1.521**

The Average Daily Traffic (ADT) was computed as shown in Table 3.3 and Table 3.4. Traffic counts were based on the methodology provided in Overseas Road Note 40 (TRL, 2004).

Table 3.3: Average Daily Traffic (ADT) - Kenyatta University to Thika

	TUE 24HR	WED 24HR	THU-24HR	FRI-24HR	SAT-24HR	SUN 24HR	MON-24HR	**TOTALS**	**ADT**
Bus	656	494	442	464	572	490	440	**3,558**	**508**
Light Trucks	667	651	677	700	677	184	655	**4,210**	**601**
Medium Trucks	1,082	1,061	1,105	1,128	1,365	395	1,062	**10,761**	**1,028**
Heavy Trucks	985	965	1,004	1,033	1,066	298	970	**8,983**	**903**
Articulated Trucks	558	556	577	560	716	256	543	**3,765**	**538**
Totals	3,947	3,727	3,804	3,884	4,395	1,623	3,670	**31,277**	**3,579**

Table 3.4: Average Daily Traffic (ADT) - Thika Town to Kenyatta University

	TUE 24HR	WED 24HR	THU-24HR	FRI-24HR	SAT-24HR	SUN 24HR	MON-24HR	**TOTALS**	**ADT**
Bus	629	610	569	626	571	530	573	**4,108**	**587**
Light Trucks	539	510	546	573	726	130	551	**3,574**	**511**
Medium Trucks	1,108	1,047	1,135	1,180	1,004	308	1,108	**11,033**	**984**
Heavy Trucks	878	830	889	933	787	210	897	**8,067**	**775**
Articulated Trucks	458	444	414	456	406	376	416	**2,971**	**424**
Totals	3,611	3,442	3,553	3,767	3,493	1,554	3,544	**29,753**	**3,281**

3.5 Calculating the Cumulative Equivalent Standard Axles

A summary of the Vehicle Equivalent Factors adopted in this research are indicated in section 2.3.1. The Actual (Computed) V_{EF} for Nairobi - Thika Superhighway in Table 2.4 were adopted in this research.

Equation 2.6 was used to compute the Equivalent Standard Axle Loads. The directional distribution factor was taken as 1.0 since each carriageway was analyzed independently. A lane distribution factor of 0.6 was multiplied by the Design Equivalent Standard Axles for the three lanes as recommended by AASTHO, 1993.

A reliability factor $\boldsymbol{F_R}$ **of 95%** was selected and is multiplied by the design period traffic prediction ($w_{8.16}$) to produce design applications ($W_{8.16}$) for the design Equation 2.5. The overall standard deviation ($\boldsymbol{S_0}$) adopted for this project is **0.45** for flexible pavements and a **Z value of 1.645** obtained from the Standard Normal Tables (AASTHO, 1993).

Equation 2.7 was used to compute the serviceability. AASTHO (1993) suggested Terminal Serviceability index $\boldsymbol{P_0}$ of 2.5 or higher and Initial Serviceability $\boldsymbol{P_t}$ of 4.2 for the design of major highways.

Therefore, $\Delta \boldsymbol{PSI}$ = ($\boldsymbol{P_0} - \boldsymbol{P_t}$) = **(4.2 - 2.5) = 1.7**

3.6 Deflection Measurements

The deflection measurements were carried out on 25 August 2012 for the study section. The Falling Weight Deflection (FWD) measurements involved testing by simulation of traffic axle loading by an impact and impulse load. Measurements were taken at intervals of approximately 100 m on the three lanes at an offset of about 0.7 m from the edge of the carriageway, along the outer wheel paths. At each drop point, readings were taken for the nine consecutive geophone points of 0, 21, 30, 60, 90, 120, 150, 180, and 210 cm. All the data were computerized to enable simplified analysis. The design software that was employed for evaluation is RoSy Design for Roads.

The deflection measurements were taken at average drop time of 912 micro seconds and at an average air temperature of 28^{0}C and surface temperature of 22^{0}C. The average applied pressure during testing was 700KPa and normalized to 707KPa for ease of analysis of the observed deflections.

Homogeneous sections were identified for each lane. To identify the homogeneous sections, a graph of the cumulative sum of the central deflections (CUSUM) was plotted against the distances along the entire test section. Subsequently, the deflection bowls of the average deflections for each homogeneous section was plotted.

3.7 Structural Evaluation of the Pavement by the RoSy Design Software

RoSy Design software was used in the structural evaluation of the pavement. The back-calculated values of the Residual Modulus of Subgrade Reaction (M_R) were compared with the design values shown in Tables 2.14 and 2.15. Similarly, the back-calculated Residual Moduli of pavement layers were compared with the design values in Table 2.16.

4 RESULTS, ANALYSIS AND DISCUSSIONS

4.1 Roughness

The international roughness index (IRI) measurements were established for the section under consideration and the averages calculated for each lane in both directions. Figures 4.1, 4.2 and 4.3 are the IRI graphs of lanes compared with the corresponding lanes in the opposite direction. The average IRI for individual lanes in both directions is summarized in Table 4.1.

Table 4.1: Mean Roughness Ratings for Nairobi –Thika Superhighway.

IRI Summaries					
Kenyatta University to Thika			Thika to Kenyatta University		
Lane 3 (Inner Lane)	Lane 2 (Middle Lane)	Lane 1 (Outer Lane)	Lane 3 (Inner Lane)	Lane 2 (Middle Lane)	Lane 1 (Outer Lane)
2.3	**2.4**	**2.4**	**2.4**	**2.5**	**2.3**

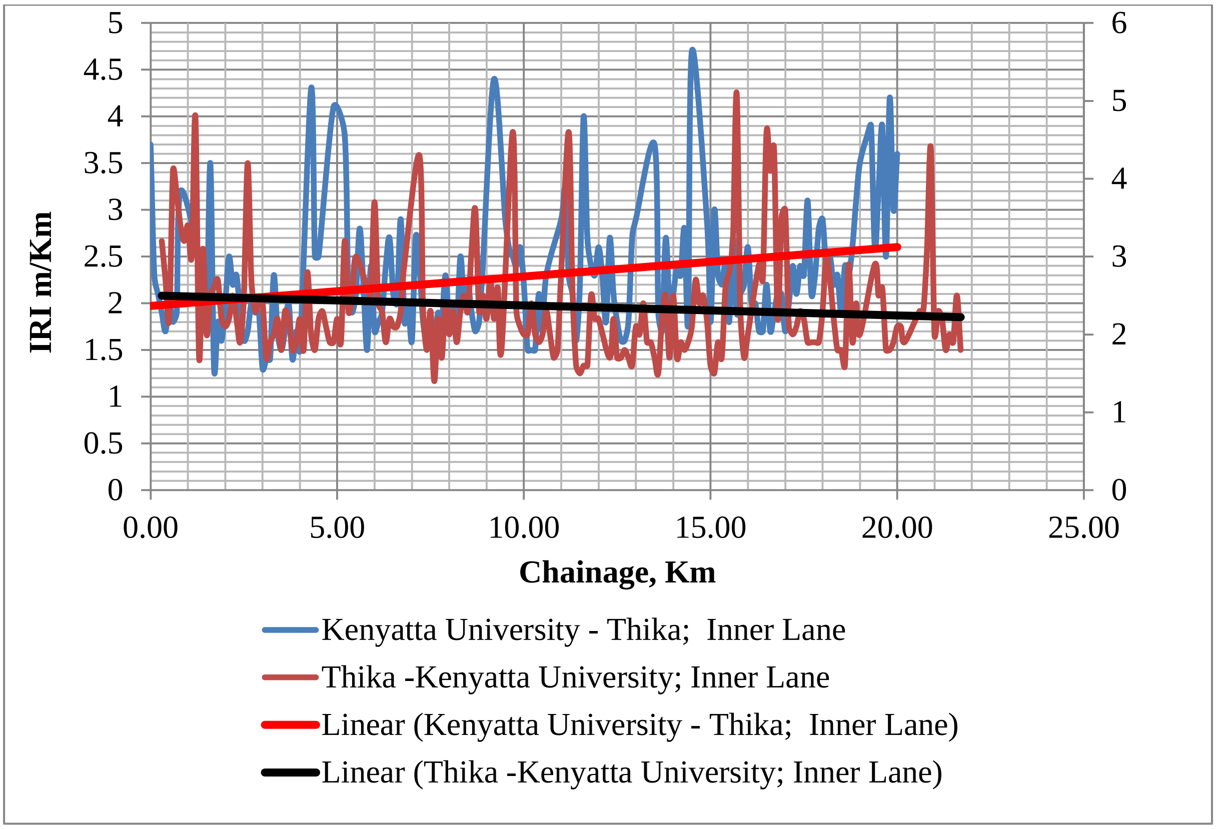


Figure 4.1: Roughness Curves of Lane 1 in both directions

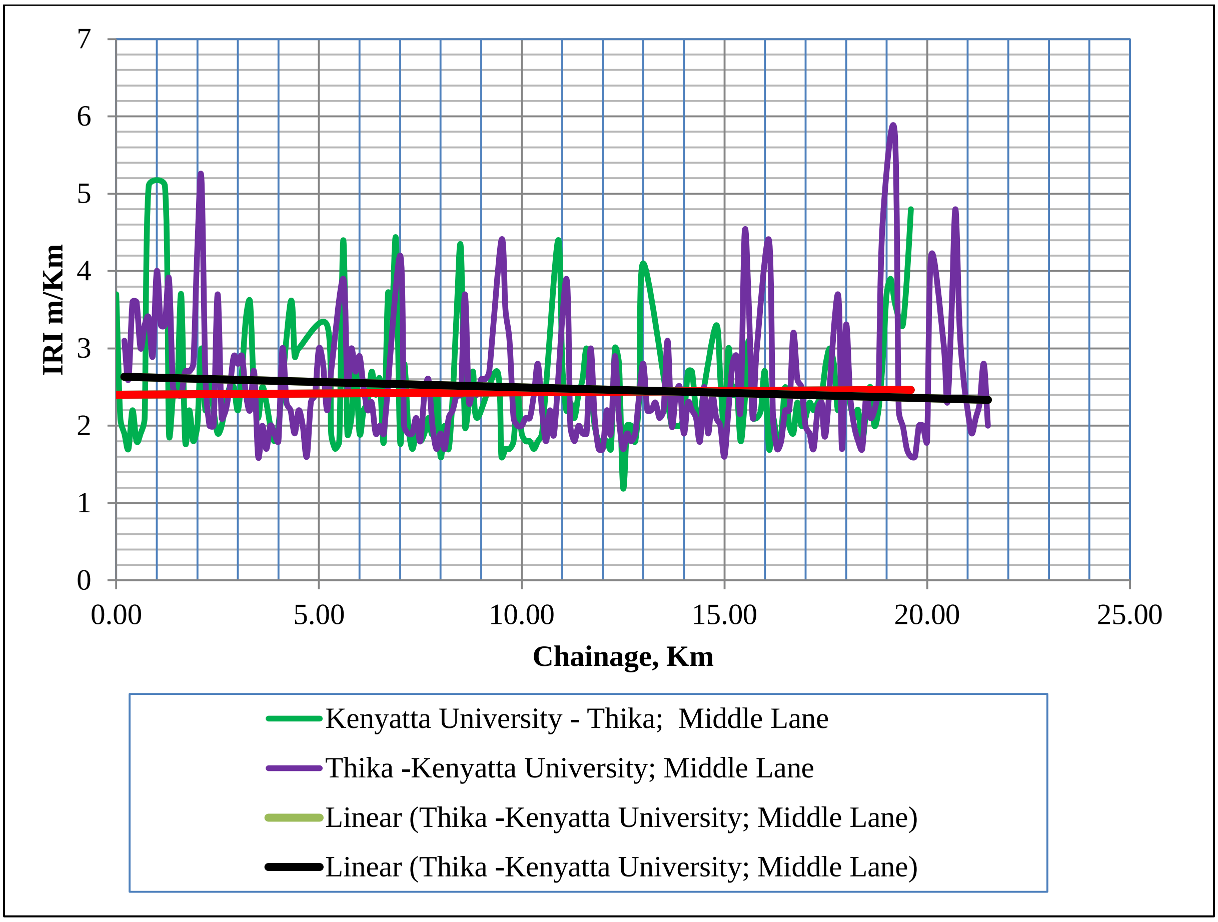


Figure 4.2: Roughness Curves of Lane 2 in both directions

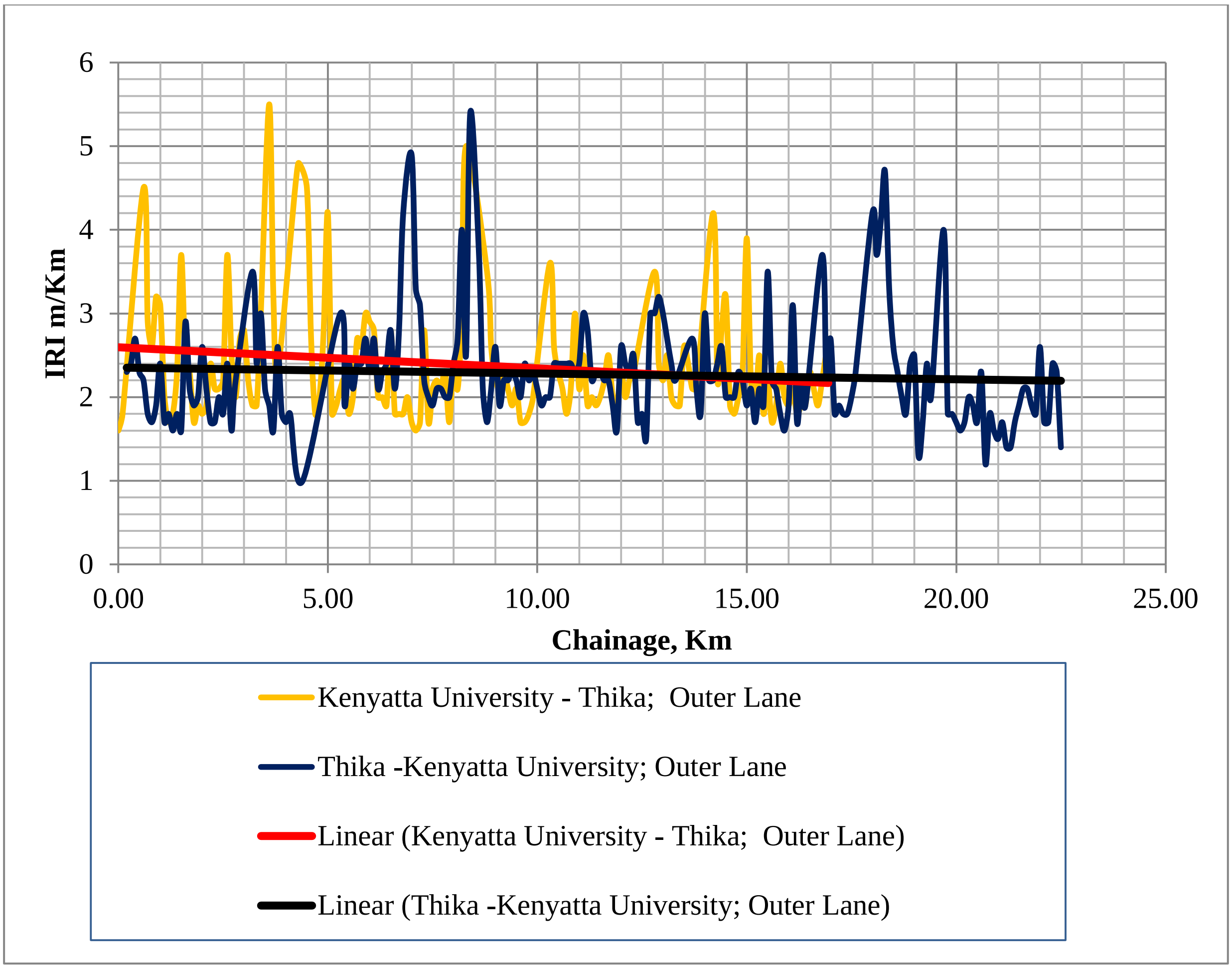


Figure 4.3: Roughness Curves of Lane 3 in both directions

4.2 Discussion of the Roughness measurements

The average IRI for all the lanes in all directions are within IRI values 1.5 to 3.5 as shown in figure 2.2 in section 2. These are the recommended values for optimum performance of new pavements. Therefore, the performance of the Nairobi to Thika Superhighway between Kenyatta University and Thika Town was found to be corresponding to that of a new pavement based on the Roughness measurements.

4.3 Cumulative Equivalent Standard Axles

The Design Equivalent Standard Axles from Kenyatta University to Thika Town was found to be 20,671 for all the lanes. Therefore, 60% of the DESA was 12,402 and was distributed for each of the three lanes and subsequently used in the analysis software RoSy Design for Roads. Table 4.2 is a summary of the Cumulative Equivalent Standard Axles from Kenyatta University to Thika Town.

Table 4.2: Cumulative Equivalent Standard Axles - Kenyatta University to Thika Town

Type of Vehicle	ADT	VDF	DESA	5 year	10 year	15 year	20 year
Bus	**508**	2.12	**1,078**	2,155,954	4,855,545	8,235,854	12,468,528
Light Trucks	**601**	0.04	**24**	48,130	108,396	183,858	278,349
Medium Trucks	**1,028**	4.27	**4,390**	8,783,683	19,782,224	33,554,112	50,798,664
Heavy Trucks	**903**	7.78	**7,024**	14,053,662	31,651,038	53,685,697	81,276,527
Articulated Trucks	**538**	15.16	**8,155**	16,316,365	36,747,000	62,329,340	94,362,419
Total	**3,579**		**20,671**	**41,357,794**	**93,144,202**	**157,988,861**	**239,184,487**
			F_R	**95.0%**	**95.0%**	**95.0%**	**95.0%**
			$\hat{w}_{8.2}$	**39,289,904**	**88,486,992**	**150,089,418**	**227,225,263**
			D_D	**1.00**	**1.00**	**1.00**	**1.00**
			D_L	**0.60**	**0.60**	**0.60**	**0.60**
			$W_{8.2}$	**23,573,943**	**53,092,195**	**90,053,651**	**136,335,158**

Similarly, the Design Equivalent Standard Axle was found to be 17,927 for all the lanes from Thika Town to Kenyatta University. Therefore, 60% of the DESA was 10,756 and was distributed for each of the three lanes and subsequently used in the analysis software RoSy

Design for Roads. Table 4.3 is a summary of the Cumulative Equivalent Standard Axles from Thika Town to Kenyatta University.

Table 4.3: Cumulative Equivalent Standard Axles - Thika Town to Kenyatta University

Vehicle Type	ADT	VDF	DESA	5 year	10 year	15 year	20 year
Bus	**587**	2.12	**1,244**	2,489,095	5,605,831	9,508,470	14,395,182
Light Trucks	**511**	0.04	**20**	40,862	92,027	156,094	236,315
Medium Trucks	**984**	4.27	**4,202**	8,407,939	18,935,989	32,118,750	48,625,622
Heavy Trucks	**775**	7.78	**6,027**	12,059,328	27,159,488	46,067,244	69,742,702
Articulated Trucks	**424**	15.16	**6,433**	12,871,978	28,989,702	49,171,606	74,442,497
Total	**3,281**		**17,927**	**35,869,202**	**80,783,037**	**137,022,162**	**207,442,318**
			F_R	**95.0%**	**95.0%**	**95.0%**	**95.0%**
			$\widehat{w}_{8.2}$	**34,075,742**	**76,743,885**	**130,171,054**	**197,070,202**
			D_D	**1.00**	**1.00**	**1.00**	**1.00**
			D_L	**0.60**	**0.60**	**0.60**	**0.60**
			$W_{8.2}$	**20,445,445**	**46,046,331**	**78,102,633**	**118,242,121**

4.4 Discussion of Pavement Deflections

4.4.1 Maximum central deflections – Kenyatta University to Thika

Graphical representation of the Maximum Central deflections were plotted as shown in figure 4.4, figure 4.5 and figure 4.6. It was observed that on Lanes 1 and 2, the values of the Maximum deflections are randomly spread along the entire stretch. However, towards the end of both Lane1 and Lane 3, higher values (388µm and 405µm) are recorded, indicating a weakness in the pavement layers to be further investigated. On Lane 2, the deflections on the entire stretch indicate there are a number of homogeneous sections along the road section.

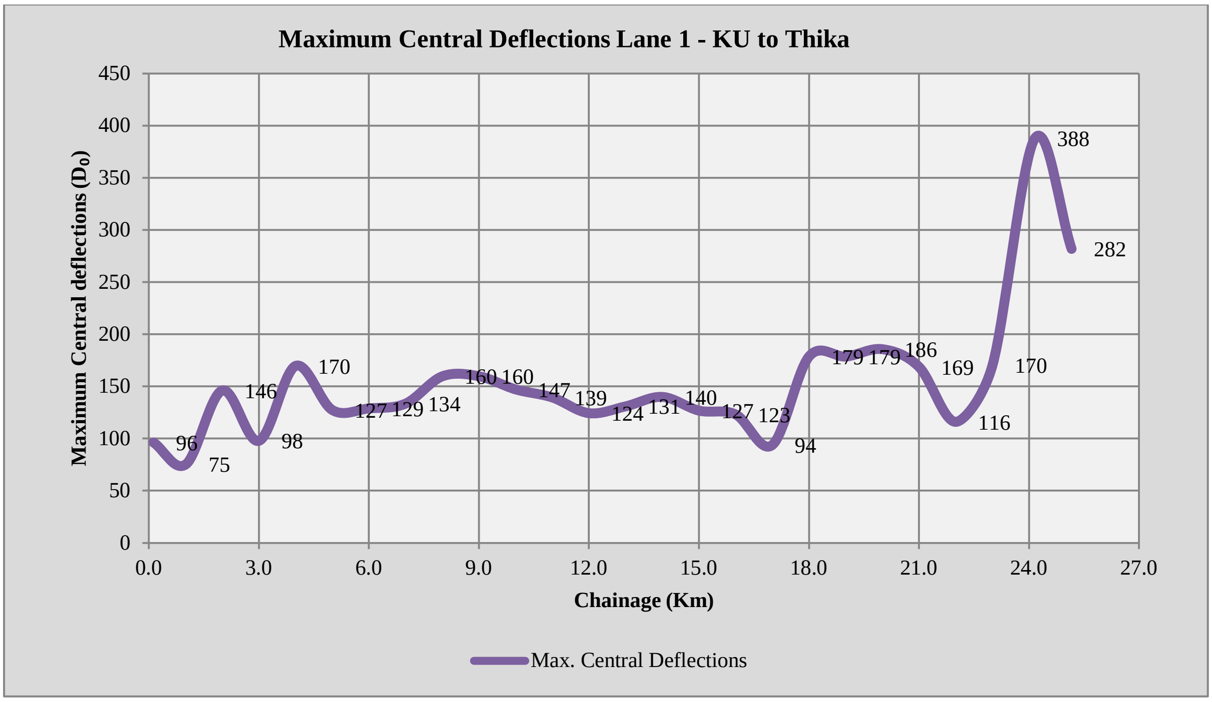


Figure 4.4: Maximum Central Deflections-Lane 1from KU to Thika

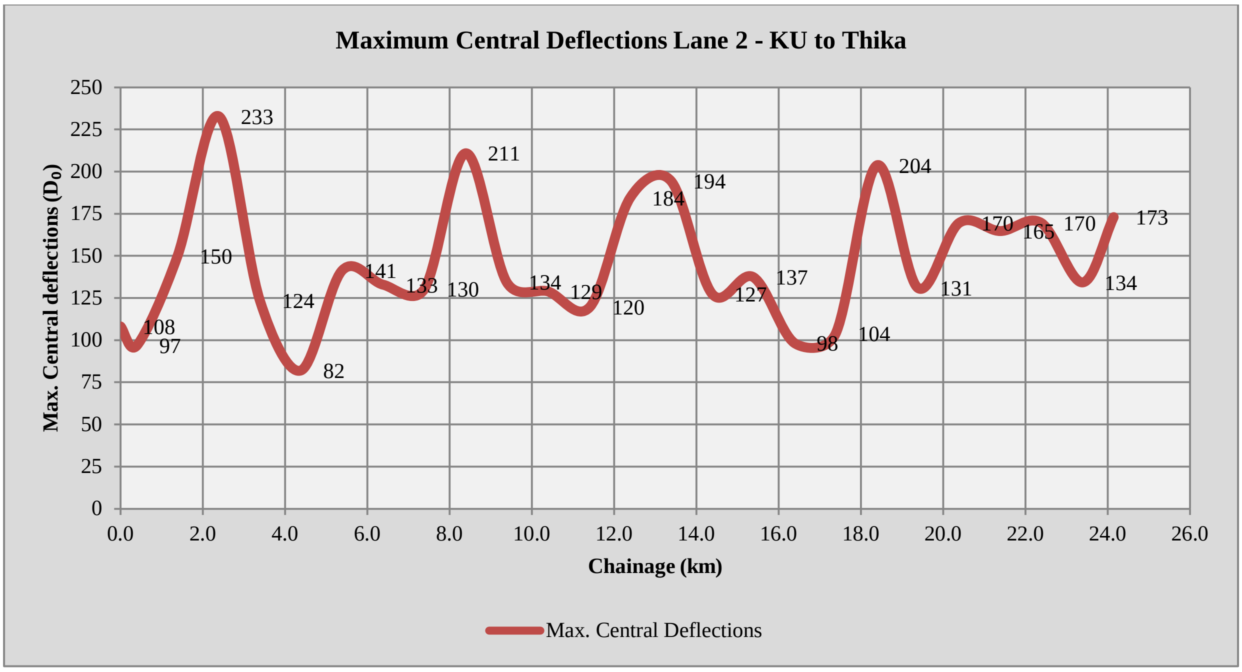


Figure 4.5: Maximum Central Deflections-Lane 2 from KU to Thika

Figure 4.6: Maximum Central Deflections-Lane 3 from KU to Thika

4.4.2 Maximum central deflections – Thika to Kenyatta University

On the Thika to Kenyatta University direction similar graphical representation of the Maximum Central deflections were plotted as shown in figure 4.7, figure 4.8 and figure 4.9. On lane 1, high deflections are spotted throughout the stretch of road under evaluation. However, on Lanes 2 and 3, the deflections are highest at the beginning of the road, also indicating weaknesses in the pavement layers.

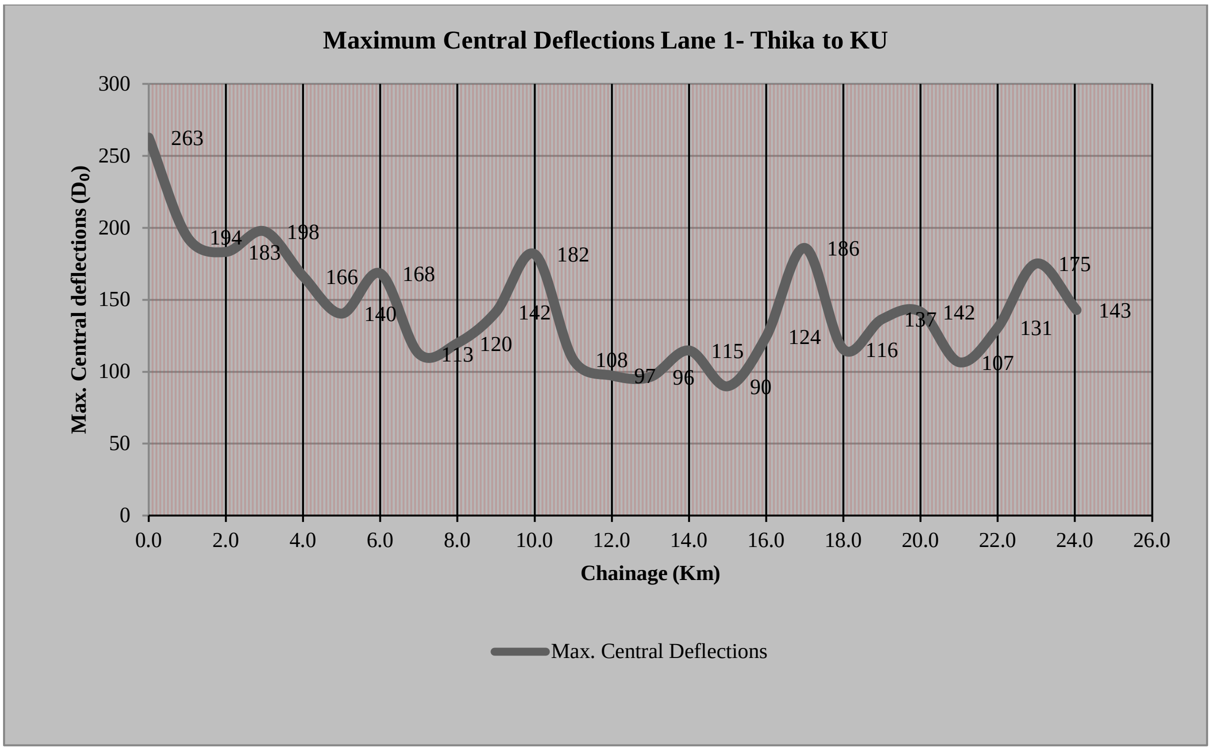


Figure 4.7: Maximum Central Deflections-Lane 1 from Thika to KU

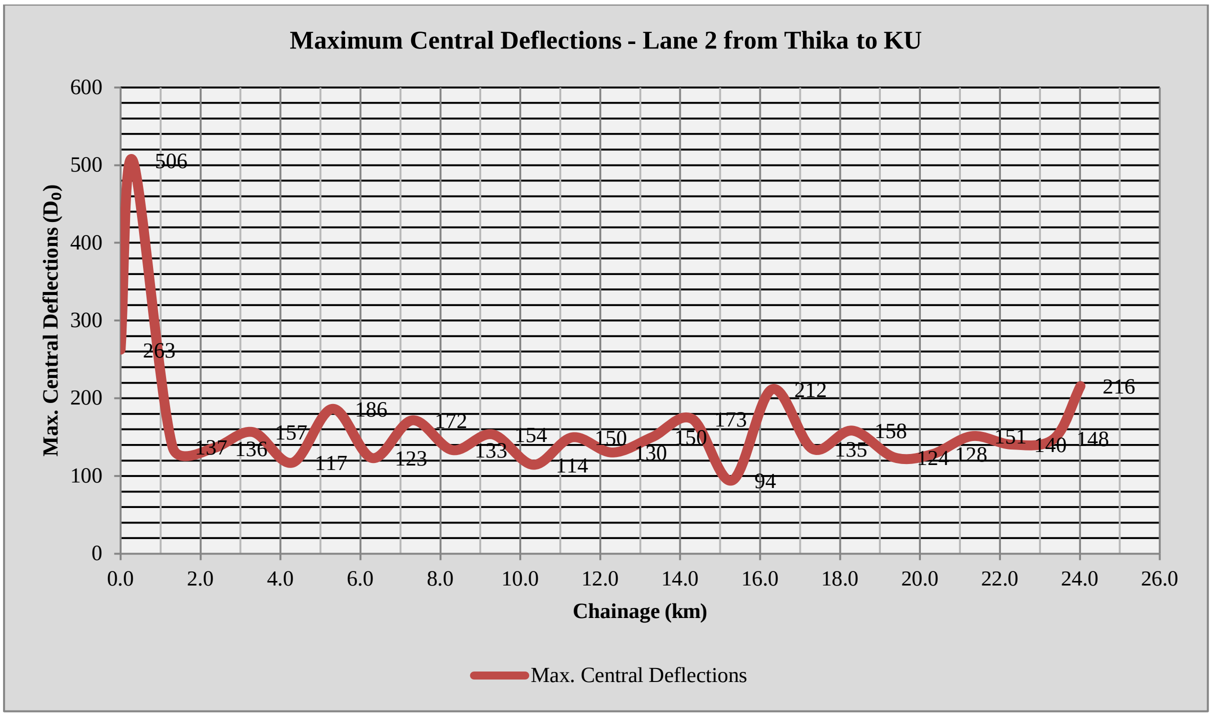


Figure 4.8: Maximum Central Deflections-Lane 2 from Thika to KU

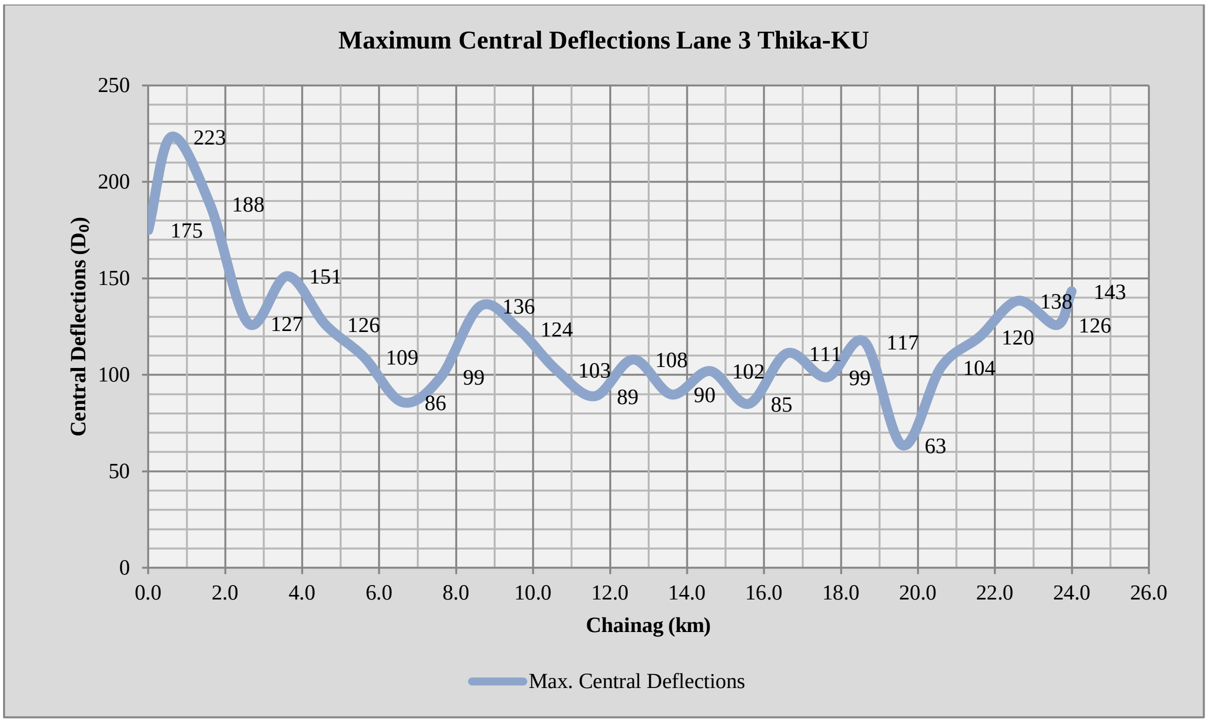


Figure 4.9: Maximum Central Deflections-Lane 3 from Thika to KU

4.5 Homogenous Sections and Deflection Bowls

4.5.1 Homogenous Sections and Deflection Bowls on Lane 1 from KU to Thika

From Figure 4.10, two homogeneous sections have been identified. One homogeneous section is from km 0+000 at Kenyatta University to km 16+000, and the other one is from km 16+000 to km 24+160. The deflection bowls corresponding to these homogeneous sections was plotted in Figure 4.11. It was observed from Figure 4.11 that the average deflections from km 0+000 to km 16+000 are lower than the average deflections between km16+000 to km 24+160, indicating that the pavement layers are weaker in the second scenario.

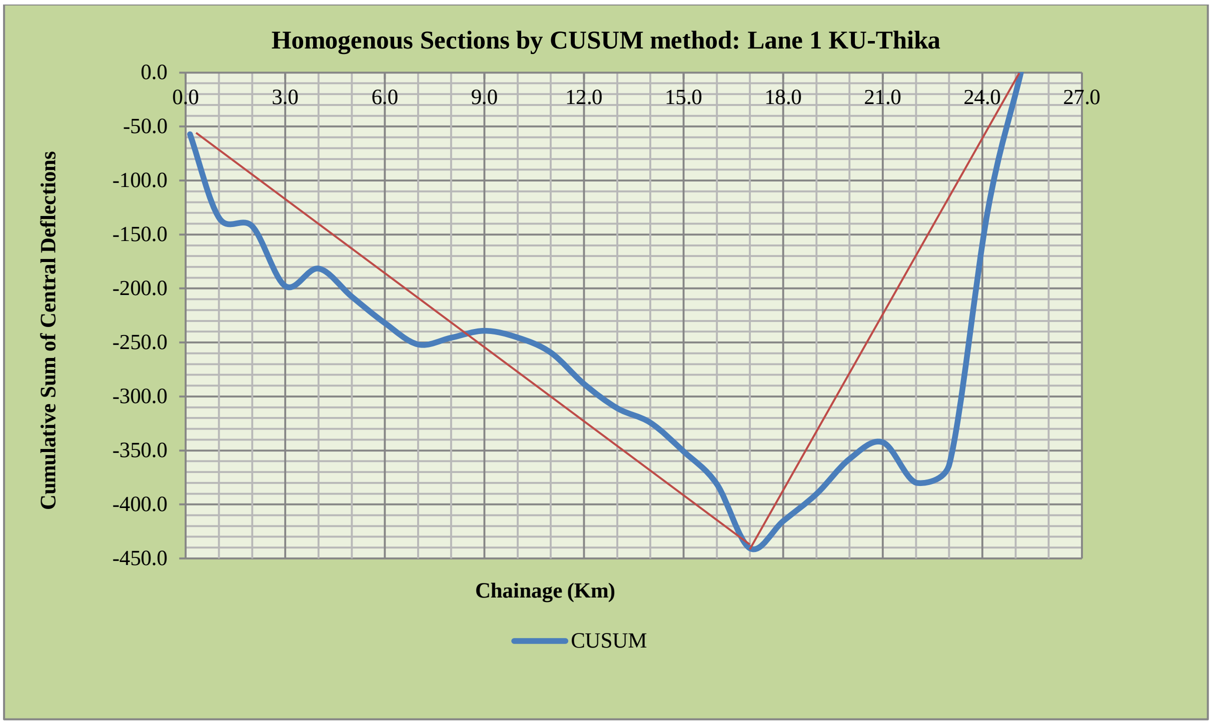


Figure 4.10: Homogeneous Sections on Lane 1 from KU to Thika Town

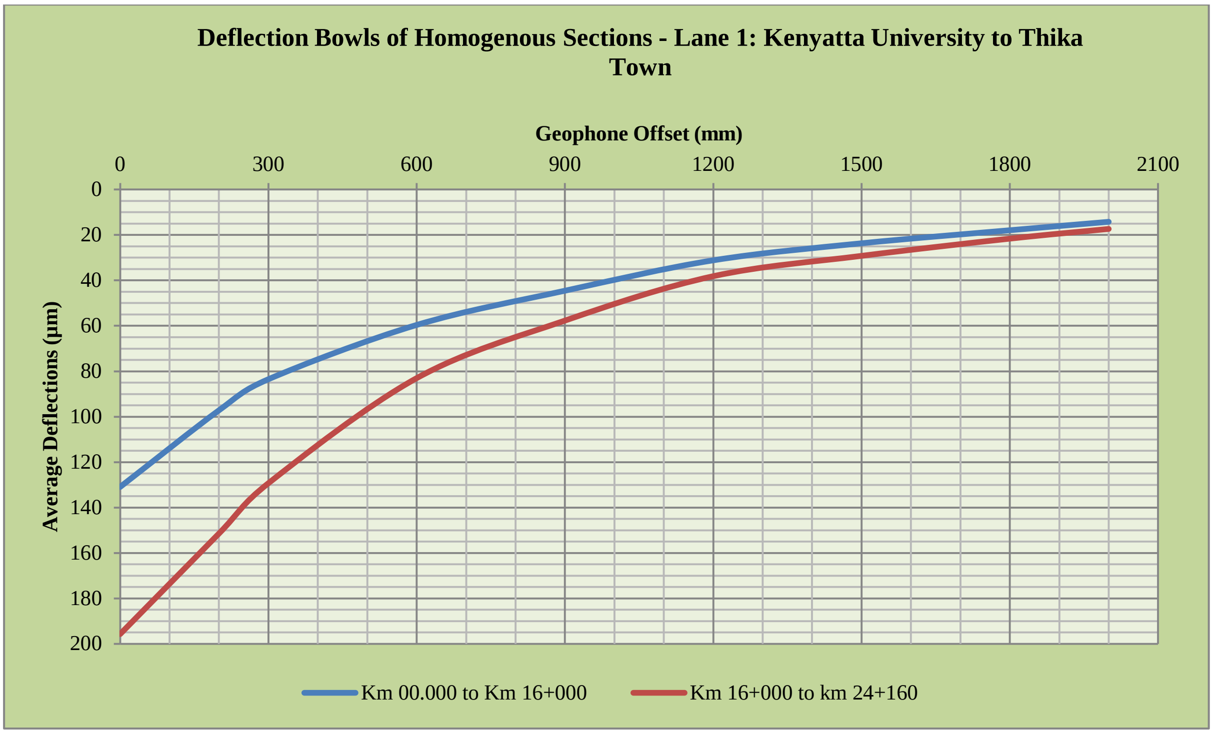


Figure 4.11 Deflection Bowls on Lane 1 from KU to Thika Town

4.5.2 Homogenous Sections and Deflection Bowls of Lane 2 from KU to Thika

Eight homogeneous sections have been identified as indicated on Figure 4.12. The corresponding deflection bowls are presented in Figure 4.13. The section between km 0+000 to 0+400 appeared stronger, going by the deflection bowls; similarly the section between 0+400 and 1+400 is the weakest along this lane going by the same criteria.

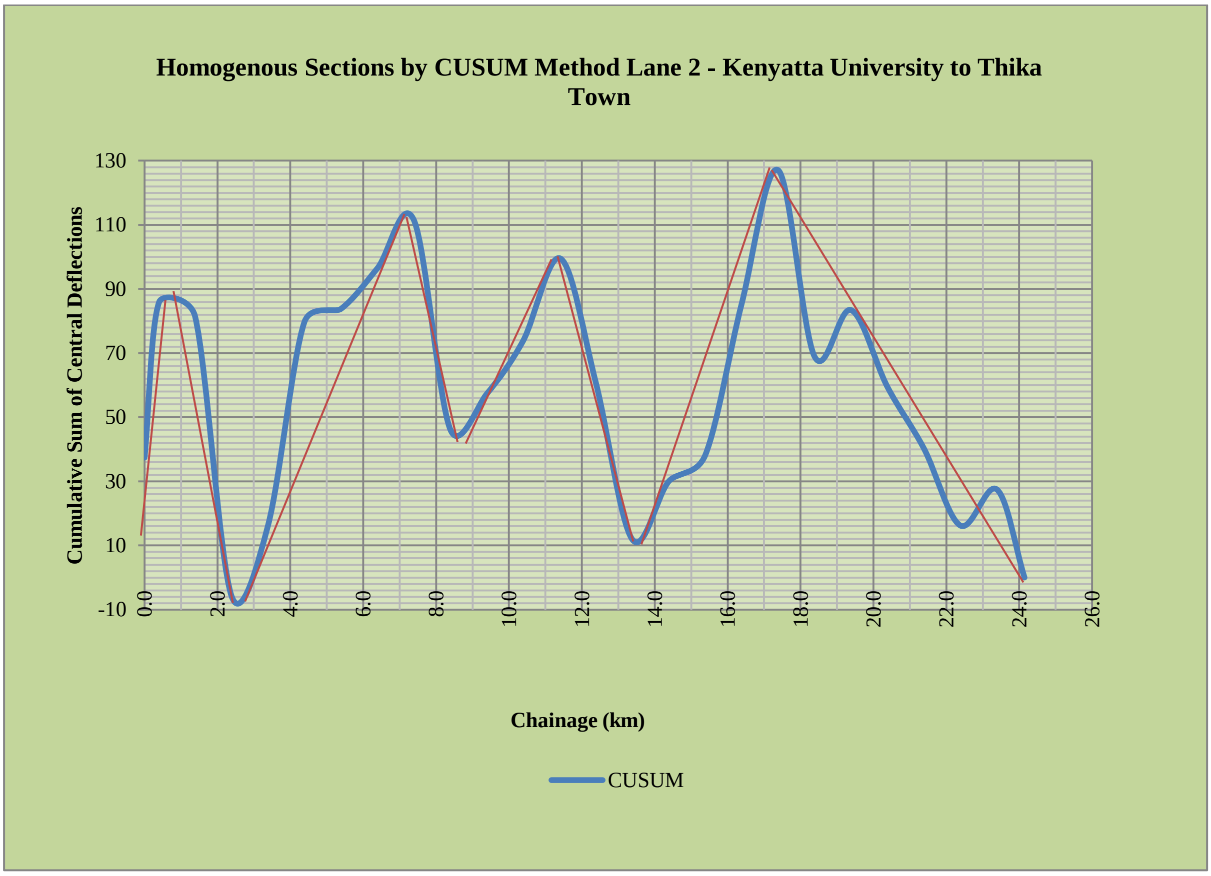


Figure 4.12 Homogeneous Sections on Lane 2 from KU to Thika Town

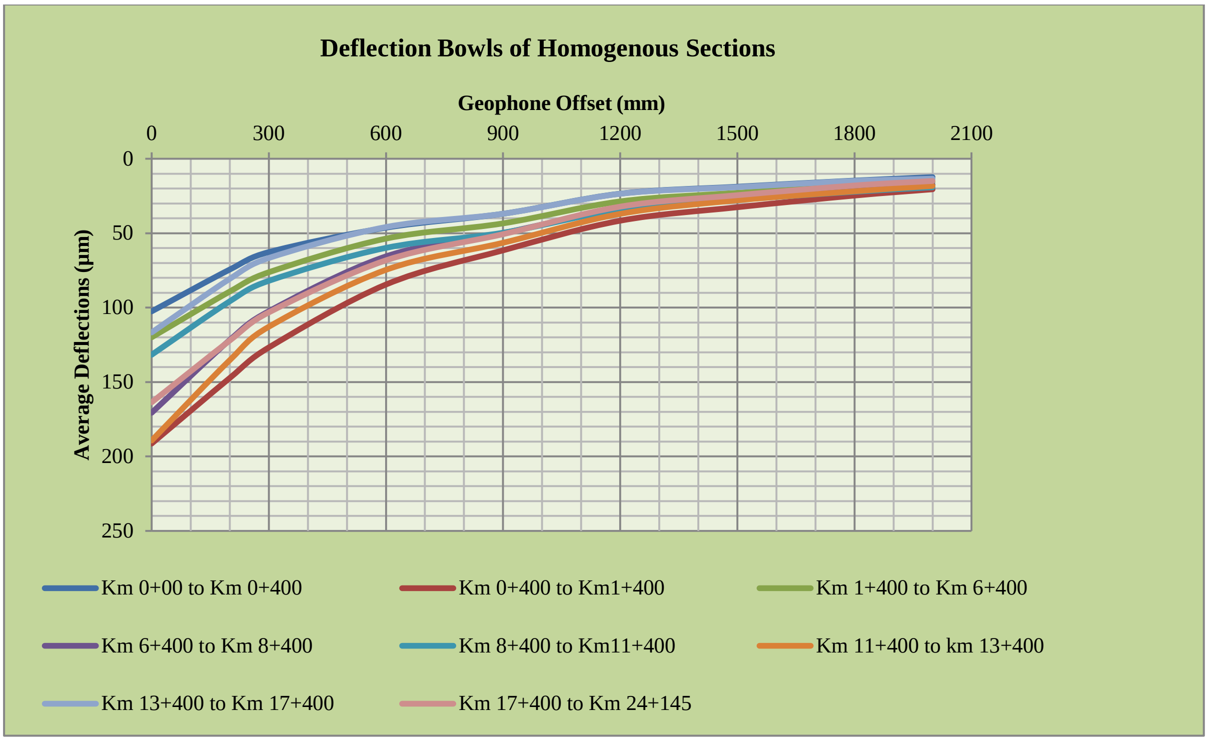


Figure 4.13: Deflection Bowls on Lane 2 from KU to Thika Town

4.5.3 Homogenous Sections and Deflection Bowls of Lane 3 from KU to Thika

Two homogeneous sections have been identified as indicated on Figure 4.14. These are sections between km 0+000 and km 21+500, and from km 21+500 to km 24+500 as shown on the deflection bowls in Figure 4.15.

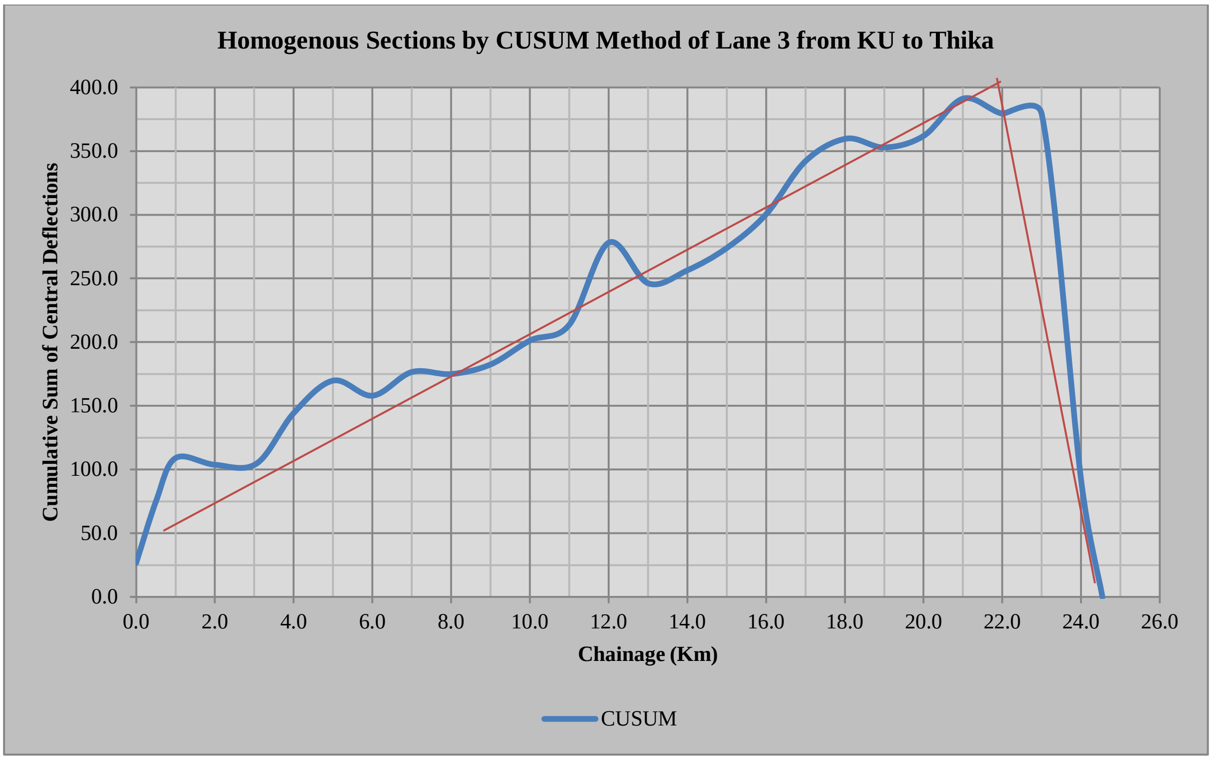


Figure 4.14: Homogenous Sections on Lane 3 from KU to Thika

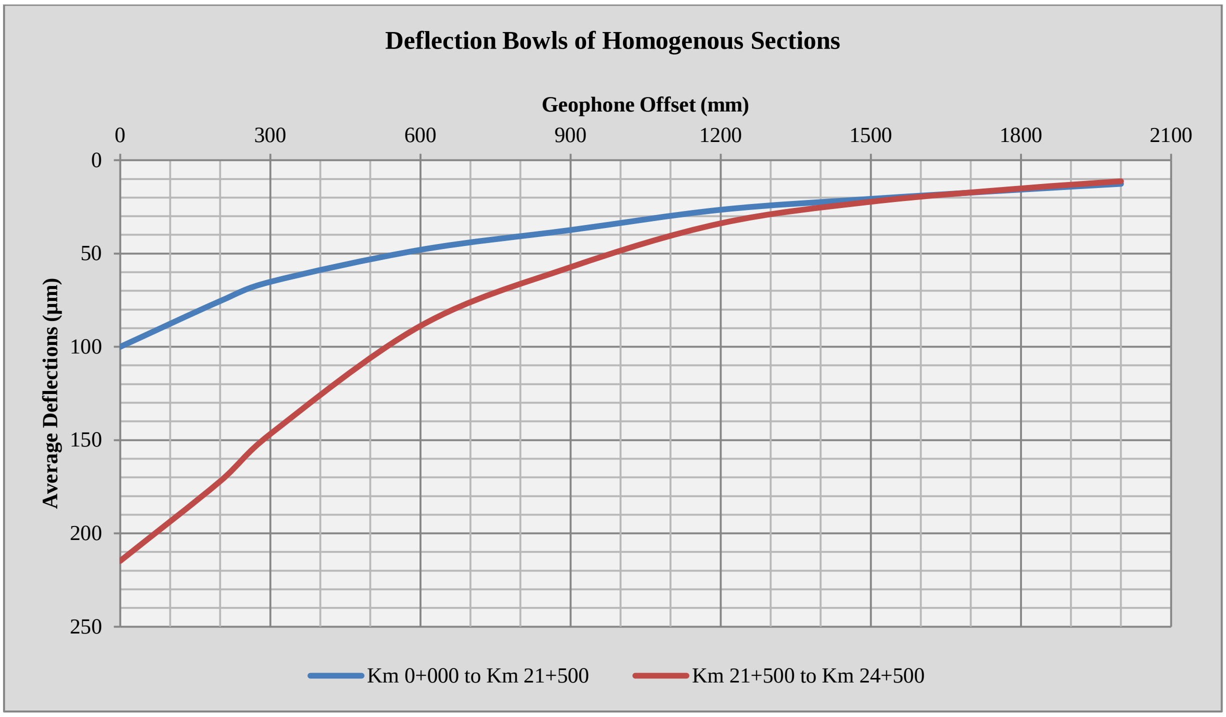


Figure 4.15: **Deflection Bowls on Lane 3 from KU to Thika Town**

4.5.4 Homogenous Sections and Deflection Bowls of Lane 1 from Thika to KU

From Figure 4.16, seven homogeneous sections have been identified. The deflection bowls corresponding to these homogeneous sections was plotted in Figure 4.17. From Figure 4.17, km 0+000 to 0+300 was found to be the weaker section, considering the highest deflections recorded compared with the rest of the road section, whereas the section from km 10+000 to 16+000 was found to be the strongest lane compared with the other sections. For the individual layer performance, refer to section 4.5.

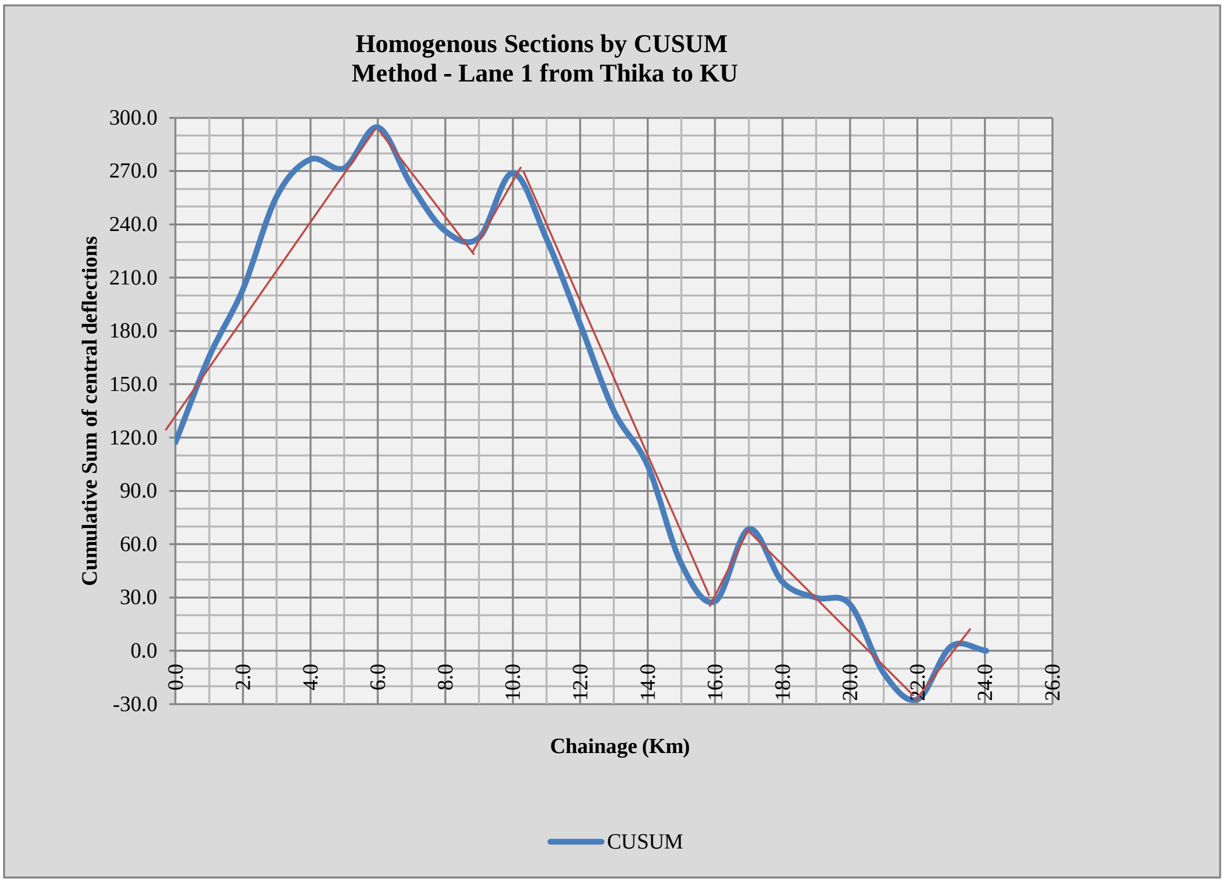


Figure 4.16 Homogeneous Sections on Lane 1 from Thika Town to KU

Figure 4.17 Deflection Bowls on Lane 1 Thika Town to KU

4.5.5 Homogenous Sections and Deflection Bowls of Lane 2 from Thika to KU

Two homogeneous sections have been identified as indicated on Figure 4.18. The corresponding deflection bowls were presented in Figure 4.19. The section between km 0+000 to km 23+000 appeared stronger, going by the deflection bowls; similarly the section between 23+000 and 24+000 was observed to be weaker going by the same criteria. The varying pavement layers strength properties are discussed in more detail in section 4.5.

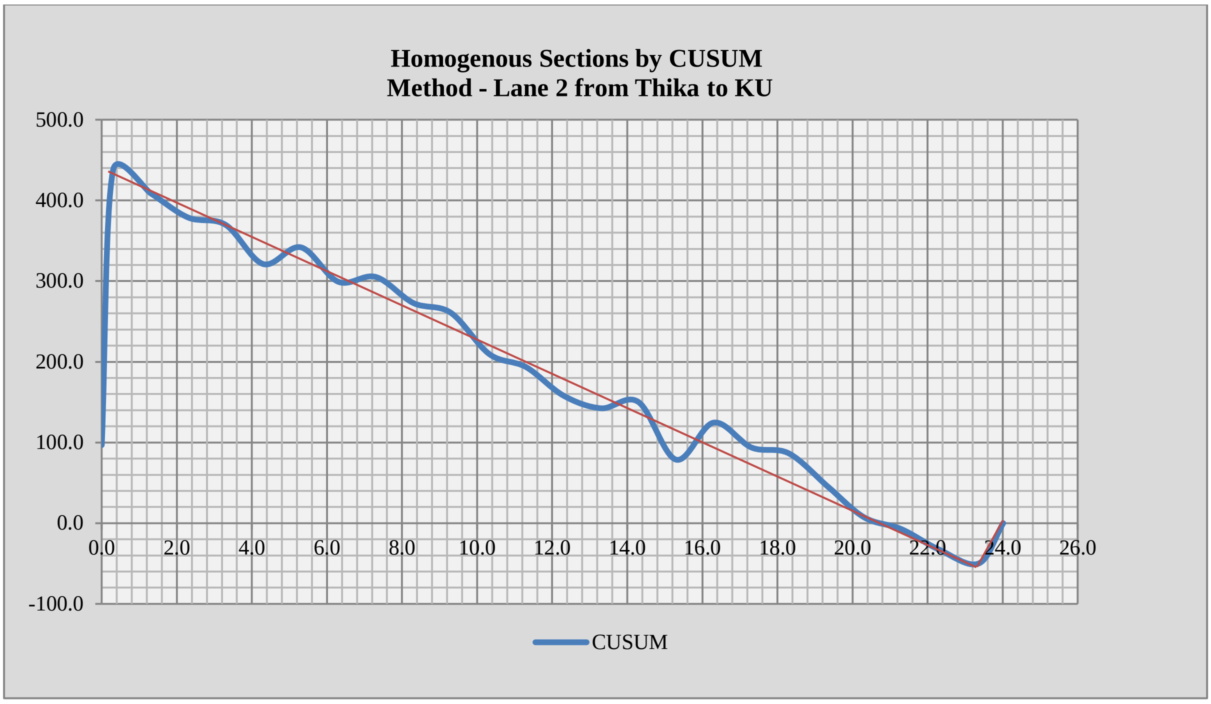


Figure 4.18: Homogeneous Sections on Lane 2 from Thika Town to KU

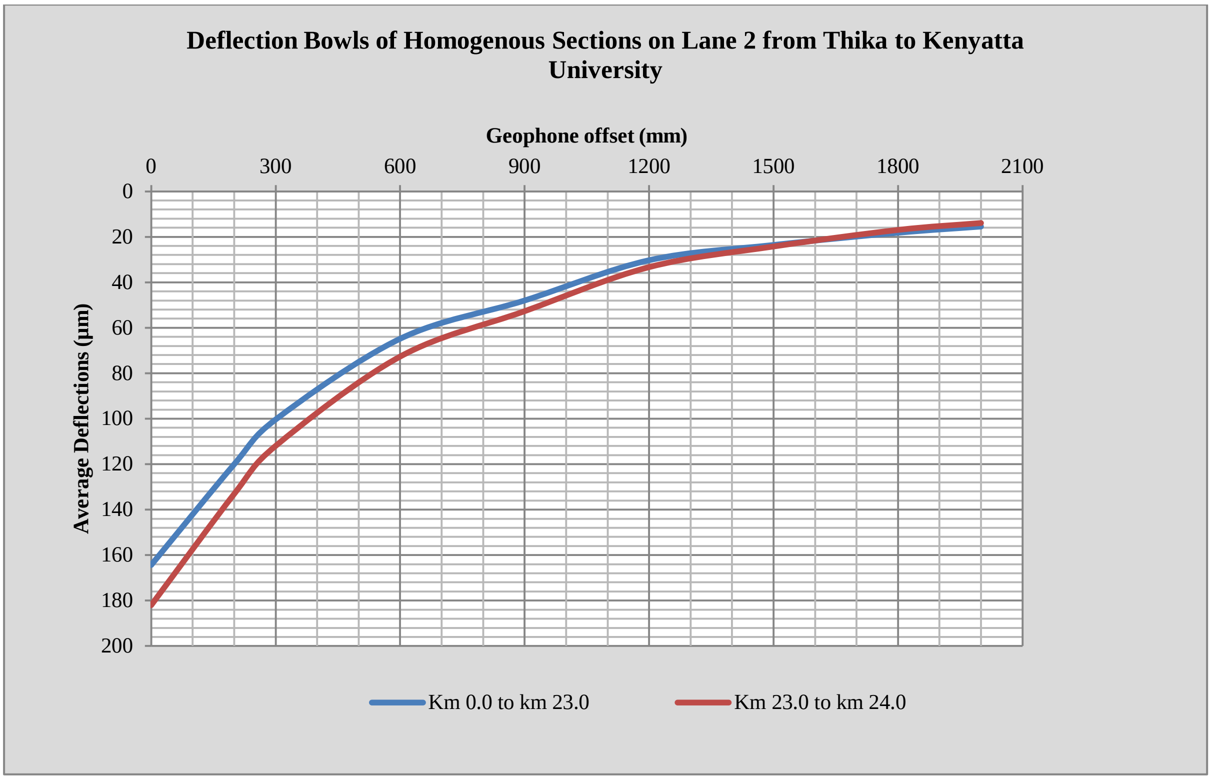


Figure 4.19 Deflection Bowls of Lane 2 from Thika Town to KU

4.5.6 Homogenous Sections and Deflection Bowls of Lane 3 from Thika to KU

Three homogeneous sections have been identified as indicated on Figure 4.20. These are sections between km 0+000 and km 3+600, km 3+600 to 20+600 and from km 20+600 as shown on the deflection bowls in Figure 4.21. The pavement layers strength properties discussed in more detail in section 4.5.

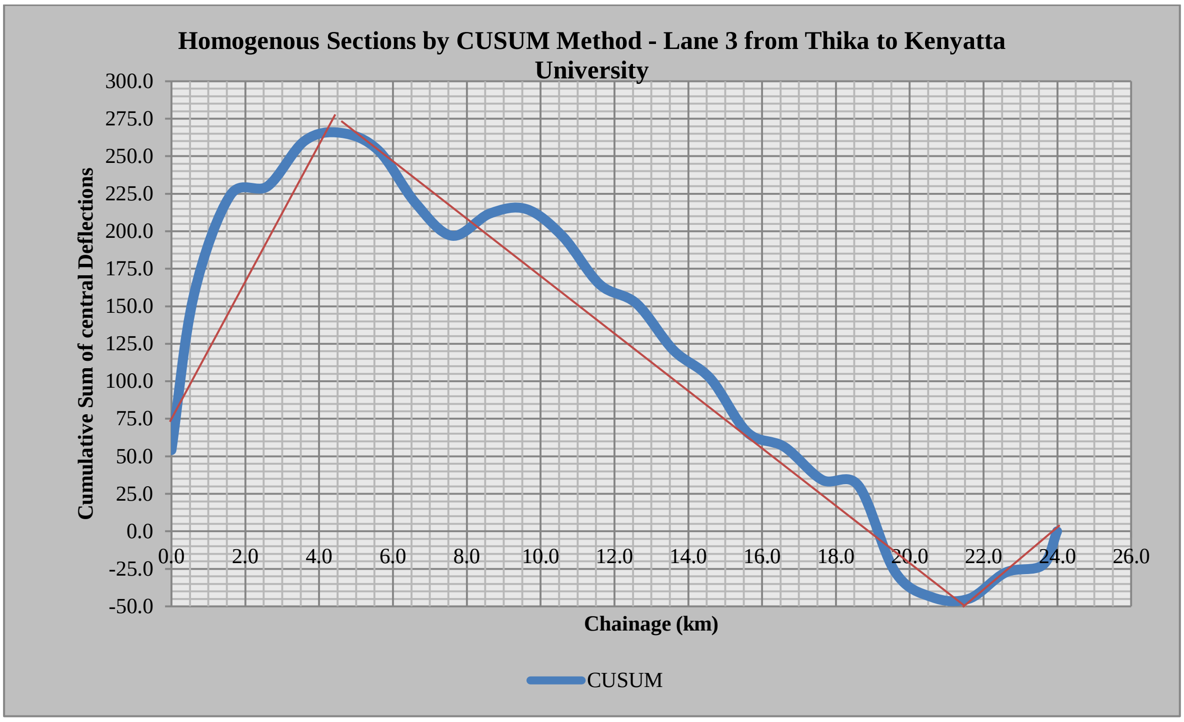


Figure 4.20: Homogenous Sections of Lane 3 from Thika to KU

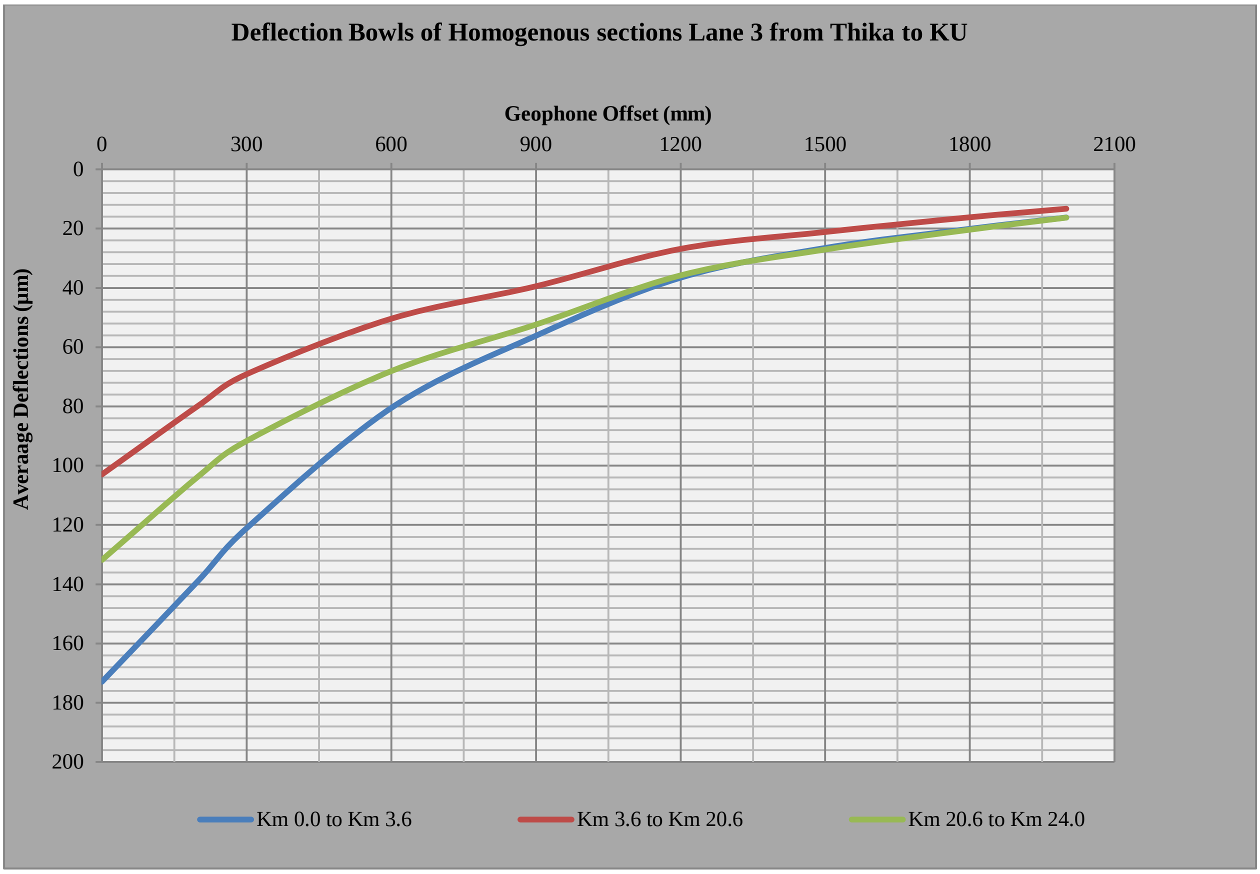


Figure 4.21 Deflection of Lane 3 from Thika Town to KU

4.6 Structural Evaluation of the performance of the Pavement Layers

4.6.1 Structural Evaluation of Lane 1 – from Kenyatta University to Thika Town

The results in Table 4.4 indicate the back-calculated pavement moduli of each layer for lane 1 form Kenyatta University to Thika. The evaluation of the performance of each layer was done by comparing the back-calculated E-modulus of each layer against the design E-modulus values in Table 2.4. The homogeneous sections in Figure 4.10 and Figure 4.11 were slightly adjusted in Table 4.4 to correspond to the actual pavement performance.

- **<u>Asphalt Concrete Surfacing (AC) and Dense Bitumen Macadam (DBM)</u>**

Except for two points (at km 0+000 and at km 4+000) along Lane 1 from Kenyatta University to Thika Town, the performance of the AC and DBM was below average, with values of Modulus of Elasticities falling below the expected value of 5,000MPa. It was observed that some very low values were obtained in homogeneous section H2. For example, 817MPa was obtained at km 23+000. The overall performance of the AC and DBM layers were found to be poor, especially in homogeneous section H2.

Table 4.4: Structural Evaluation of Lane 1 from Kenyatta University to Thika Town

Chainage (km)	Back-calculated E-Modulus			Back-calculated Residual Modulus of Sub-grade Reaction, (M_R)	Design Residual Modulus of Subgrade Reaction (M_R)	5 Years projection		10 Years projection		15 Years projection		20 Years projection		Homogeneous Section
	AC & DBM, *E1* = 5,000 MPa	GCS, *E2* = 4,000 MPa	Cement Improved Subbase, *E3* = 3,000 MPa			Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	
00+000	*6161*	*6272*	*5538*	*316*	171	20	0	20	0	20	0	20	0	H1
00+130	4036	*8074*	*6353*	*586*	171	20	0	20	0	20	0	20	0	
01+000	3479	3355	2510	*248*	171	20	0	20	0	20	0	20	0	
02+000	3546	*5452*	*4115*	*464*	171	20	0	20	0	20	0	20	0	
03+000	2674	2266	*3221*	210	276	14	0	13	0	11	35	10	60	
04+000	*6362*	*5000*	*3000*	237	276	20	0	20	0	20	0	20	0	
05+000	3235	2626	2184	*374*	292	20	0	20	0	18	0	16	35	
06+000	3596	3073	2660	*293*	292	20	0	20	0	20	0	20	0	
07+000	2501	1637	1717	*336*	292	5	0	5	55	4	85	4	100	
08+000	2965	2416	1915	265	292	18	0	16	0	14	15	12	55	
09+000	2679	2810	2610	287	292	20	0	20	0	20	0	19	10	
10+000	3983	3081	2264	262	292	20	0	20	0	20	0	20	0	
11+000	3772	*4227*	2836	*295*	292	20	0	20	0	20	0	20	0	
12+000	3040	3593	*3369*	304	478	20	0	20	0	20	0	20	0	
13+000	2789	2737	2628	317	478	20	0	20	0	20	0	18	20	
14+000	3761	*4105*	2888	290	408	20	0	20	0	20	0	20	0	
15+000	3784	*5000*	*3000*	294	408	20	0	20	0	20	0	20	0	
16+000	3784	*5000*	*3000*	*420*	198	20	0	20	0	20	0	20	0	
17+000	3784	*5000*	*3000*	152	198	20	0	20	0	20	0	20	0	
18+000	2475	2478	2446	*210*	185	19	0	17	0	15	5	13	50	H2
19+000	1784	1232	2077	*298*	270	2	55	2	90	2	110	1	125	
20+000	2515	2125	2039	262	270	12	0	10	0	9	50	8	75	
21+000	4579	*4536*	*3379*	288	322	20	0	20	0	20	0	20	0	
22+000	3224	2359	2855	199	322	17	0	15	0	13	25	11	55	
23+000	817	863	*4708*	123	322	1	95	1	120	1	140	0	155	
24+161	1653	724	1296	164	322	0	100	0	125	0	145	0	160	

Legend: *Red Text in italics – Back-calculated E_P and M_R were equal or exceeded the design values. The layer performed better than the expected design modulus.*

- **Graded Crushed Stone (GCS) Base**

It was also observed that the performance of the GCS base layer was also below average in most sections of the road, with most values of Modulus of Elasticities obtained falling below the expected value of 4,000MPa. It was similarly observed that in homogeneous section H2, some very low values like 863MPa was obtained at km 23+000 and 724MPa obtained at km24+161. Therefore, the performance of the GCS on that lane was also found to be below average.

- **Cement Improved Gravel (CIG) Subbase**

The back-calculated CIG Subbase E-modulus values were observed to be generally below the design value of 3000MPa. However, the value of back-calculated E-Modulus at km 21+161 in homogeneous section H2 (1296MPa) was found to be considerably lower than 3000MPa. The performance of the Cement Improved Gravel subbase was also found to be generally below average.

- **Subgrade**

The back-calculated values of the Residual Modulus of Subgrade Reaction (M_R) were found to be above average for the initial 2kilometres, and then from km 3+000 to 6+000. Generally, the back-calculated M_R values were found to be above or near the design values in both homogeneous sections H1 and H2.

- **Residual Life and Projected Overlay requirements**

The back-calculated Residual Life of the pavement for each section and the corresponding overlay requirements after 5 years, 10 years, 15 years and 20 years of projected traffic were shown in Table 4.4. From the table, it was observed that after 5 years of projected traffic, there was an overlay requirement in homogeneous section H2 between of 55mm and 100mm with a residual life of 1year and 0years at km 23+000 and km24+161 respectively. This meant that homogeneous section H2 required reconstruction after only 5 years of projected traffic.

However, at homogeneous section H1, the residual life at km 3+000 and km 7+000 had reduced to 14years and 5years respectively after 5years of projected traffic. However, the first intervention was required in the 10^{th} year of opening of the road with an overlay requirement of 55mm. Without that intervention, the overlay requirement would increase to 85mm and 100mm in the 15^{th} and 20^{th} years of projected traffic.

4.6.2 Structural Evaluation of Lane 2 – From Kenyatta University to Thika Town

The results in Table 4.5 also indicate the back-calculated pavement moduli of each layer for lane 2 from Kenyatta University to Thika. The evaluation of the performance of each layer was also done by comparing the back-calculated E-modulus of each layer against the design E-modulus values in Table 2.4. The homogeneous sections in Figure 4.12 and Figure 4.13 were also slightly adjusted in Table 4.5 to correspond to the actual pavement performance.

- **Asphalt Concrete Surfacing (AC) and Dense Bitumen Macadam (DBM)**

From Table 4.5, it was observed that the performance of the AC and DBM was generally below average. Most of the back-calculated values of Modulus of Elasticities were found to be below the expected value of 5000MPa except for km 0+400 in homogeneous section H1, the back-calculated E-Modulus was 5710MPa.

- **Graded Crushed Stone (GCS) Base**

It was also observed from Table 4.5, that the back-calculated E-Moduli values for the Graded Crushed Stone were higher than the expected value of 4000MPa in Homogeneous sections H1 and only a few sections in H3, H4, H5, H7 and H8. Lower values of the back-calculated E-Modulus were predominantly obtained in H2, H6 and H8. Therefore, the overall performance of the GCS base on this lane was below average in most sections.

- **Cement Improved Gravel (CIG) Subbase**

The back-calculated CIG Subbase E-modulus values were observed to be above the expected average value of 3000MPa in Homogeneous sections H1 and only a few sections in H3, H4, H5, H7 and H8. Lower values of the back-calculated E-Modulus were predominantly obtained in H2, H6 and H8. The performance of the CIG Subbase was found to be generally below average along Lane 2 from Kenyatta University to Thika Town.

- **Subgrade**

From Table 4.5, it was observed that the back-calculated values of the Residual Modulus of Subgrade Reaction (M_R) were above or near the design average values in homogeneous sections H1, H2, H3, H5 and H7. Even where lower values were obtained, especially in homogeneous sections H6 and H8, they were also close to the design values. Therefore, the subgrade strength for the entire lane 2 was found to be generally consistent with the design considerations.

Table 4.5: Structural Evaluation of Lane 2 from Kenyatta University to Thika

Chainage (km)	Back-calculated E-Modulus			Back-calculated Residual Modulus of Sub-grade Reaction, (M_R)	Design Residual Modulus of Subgrade Reaction (M_R)	5 Years projection		10 Years projection		15 Years projection		20 Years projection		Homogeneous section
	AC & DBM, *E1* = 5,000 MPa	GCS, *E2* = 4,000 MPa	Cement Improved Subbase , *E3* = 3,000 MPa			Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	
00+000	4261	*4875*	*3857*	*351*	171	20	0	20	0	20	0	20	0	H1
00+400	*5710*	*4680*	*3529*	*391*	171	20	0	20	0	20	0	20	0	
01+400	3409	2479	2095	*284*	171	19	0	17	0	15	0	13	45	H2
02+400	2134	1243	1544	*184*	171	2	55	2	90	2	110	2	125	
03+400	4046	*5000*	*3000*	*275*	276	20	0	20	0	20	0	20	0	H3
04+400	4575	*5479*	*5738*	*528*	276	20	0	20	0	20	0	20	0	
05+400	3043	2775	2759	*303*	292	20	0	20	0	20	0	19	15	
06+400	4276	3343	2397	282	292	20	0	20	0	20	0	20	0	
07+400	4046	*5000*	*3000*	258	292	20	0	20	0	20	0	20	0	H4
08+400	1676	1326	1277	*298*	292	3	50	2	85	2	105	2	120	
09+400	3532	3564	2623	*299*	292	20	0	20	0	20	0	20	0	H5
10+400	4046	*5000*	*3000*	288	292	20	0	20	0	20	0	20	0	
11+400	3945	3282	2276	*378*	292	20	0	20	0	20	0	20	0	
12+400	1760	1774	2209	267	478	7	0	6	55	5	80	5	95	H6
13+400	2320	1600	2892	197	478	5	10	4	60	4	85	3	105	
14+400	3301	3364	*3000*	336	408	20	0	20	0	20	0	20	0	H7
15+400	3017	1880	1944	412	408	8	0	7	35	6	65	6	85	
16+400	4735	*5052*	*4199*	*390*	198	20	0	20	0	20	0	20	0	
17+400	4071	*5298*	*4243*	*383*	198	20	0	20	0	20	0	20	0	
18+400	1543	1319	1918	*263*	185	3	55	2	85	2	105	2	125	H8
19+400	3914	*4300*	*3000*	267	270	20	0	20	0	20	0	20	0	
20+400	2950	2135	2242	236	270	12	0	11	0	9	50	8	70	
21+400	2786	2348	2891	230	322	16	0	14	0	13	25	11	55	
22+400	2950	2135	2242	236	322	12	0	11	0	9	50	8	70	
23+400	3709	3276	2984	282	322	20	0	20	0	20	0	20	0	
24+145	1981	1138	2114	221	322	2	65	1	95	1	115	1	130	

Legend: *Red Text in italics – Back-calculated E_P and M_R were equal or exceeded the design values. The layer performed better than the expected design modulus.*

- **<u>Residual Life and Projected Overlay requirements</u>**

The back-calculated Residual Life of the pavement for each section and the corresponding projected overlay requirements projected to 5 years, 10 years, 15 years and 20 years of projected traffic were indicated in Table 4.5.

For the first 5 years after opening the road, homogeneous sections H1, H3, H5, H7 and parts of H8 did not require overlay and the residual life was still 20 years. However, in H2, H4, H6 and parts of H8 overlay was required, the maximum being 65mm in H8. The residual life in H2 was 2years, H4 was 3years and H6 was 5 years. In homogeneous section H8, the residual life reduced from 20 years to 3 years at km 18+400 and to 2 years at km 24+145 within the first five years of projected traffic.

After 10-years of projected traffic, the projected performance of H1, H3, and H5 were still good where the Residual Life of the pavement was still 20 years and there was no need of overlay. But H2 required overlay of 90mm with residual life of 2years, H4 required 85mm overlay with residual life of 2years, H6 required 60mm overlay with 4years residual life, H7 required 35mm overlay and had 7years residual life and H8 required 95mm overlay and had only 1year residual life.

A similar pattern of deterioration was observed after 15 years of projected road use. The performance of H1, H3, and H5 were still good since the Residual Life of the pavement was still 20 years and there was no need of overlay. But H2 required overlay of 110mm with residual life of 2years, H4 required 105mm overlay with residual life of 2years, H6 required 85mm overlay with 4years residual life. H7 required 65mm overlay and had 6years residual life and H8 required 115mm overlay and had only 1year residual life.

After 20years of projected traffic, only homogeneous sections H1 and H5 did not entirely require overlay. The locations that required overlay increased substantially and the thickness of overlay required also became bigger. The road was however considered to have served its full life and was due for reconstruction.

4.6.3 Structural Evaluation of Lane 3 – From Kenyatta University to Thika Town

The results in Table 4.6 also indicate the back-calculated pavement moduli of each layer for lane 3 form Kenyatta University to Thika. The evaluation of the performance of each layer was also done by comparing the back-calculated E-modulus of each layer against the design E-modulus values in Table 2.4. The homogeneous sections in Figure 4.14 and Figure 4.15 were also slightly adjusted in Table 4.6 to correspond to the actual pavement performance.

- **<u>Asphalt Concrete Surfacing (AC) and Dense Bitumen Macadam (DBM)</u>**

It was observed from Table 4.6 that; except for a few locations along lane 3 from Kenyatta University to Thika Town, the performance of the AC and DBM average with values of Modulus of Elasticities falling above or near the expected value of 5,000MPa in homogeneous section H1. However, at homogeneous section H2, the back-calculated values were found to be lower than 5,000MPa, and a very low value of 839MPa was obtained at km 24+000. Therefore, the performance of the AC and DBM layers were found to be average.

- **<u>Graded Crushed Stone (GCS) Base</u>**

From Table 4.6, it was also observed that the performance of the GCS was above average, with most values of Modulus of Elasticities falling above or close to the expected value of 4,000MPa. However, in homogeneous section H2, none of the back-calculated E-modulus was above the expected value of 4,000MPa. A much lower value was obtained at km 24+000 (843MPa). Therefore, the performance of the GCS layer was found to be above average in H1 (from km 0+000 to km 23+000). In H2 (from km 24+000 to 24+542) the performance was substantially below average.

- **<u>Cement Improved Gravel Subbase</u>**

The back-calculated CIG Subbase E-modulus values were observed to be generally above the design value of 3000MPa except at km 6+000, km 13+000 and km 22+000, in homogeneous section H1; and km 24+542 in homogeneous section H2. On this lane the performance of the Cement Improved Gravel Subbase was observed to be very good compared with the expected value.

Table 4.6: Structural Evaluation of Lane 3 from Kenyatta University to Thika

Chainage (km)	Back-calculated E-Modulus: AC & DBM, $E1$ = 5,000 MPa	GCS, $E2$ = 4,000 MPa	Subbase , $E3$ = 3,000 MPa	Back-calculated Modulus of Sub-grade Reaction,	Design Residual Modulus of Subgrade Reaction (M_R)	5 Years projection: Residual Life (Years)	Overlay (mm)	10 Years projection: Residual Life (Years)	Overlay (mm)	15 Years projection: Residual Life (Years)	Overlay (mm)	20 Years projection: Residual Life (Years)	Overlay (mm)	Homogeneous Section
00+000	*8969*	*9897*	*11933*	*349*	171	20	0	20	0	20	0	20	0	H1
00+500	*8846*	*8177*	*6360*	*502*	171	20	0	20	0	20	0	20	0	
01+000	*5267*	*5289*	*4401*	*490*	171	20	0	20	0	20	0	20	0	
02+000	*5232*	*6740*	*5226*	*211*	171	20	0	20	0	20	0	20	0	
03+000	3625	*5000*	*3000*	*316*	276	20	0	20	0	20	0	20	0	
04+000	*7622*	*5527*	*5410*	*466*	276	20	0	20	0	20	0	20	0	
05+000	4827	*4986*	*4729*	*412*	292	20	0	20	0	20	0	20	0	
06+000	3467	2987	2480	*330*	292	20	0	20	0	20	0	20	0	
07+000	*5597*	*6177*	*14379*	*363*	292	20	0	20	0	20	0	20	0	
08+000	3835	*4792*	*3000*	*293*	292	20	0	20	0	20	0	20	0	
09+000	*5044*	*5726*	*4699*	282	292	20	0	20	0	20	0	20	0	
10+000	3753	*5000*	*3000*	*402*	292	20	0	20	0	20	0	20	0	
11+000	*6108*	*6987*	*6614*	252	292	20	0	20	0	20	0	20	0	
12+000	*9630*	*5375*	*5024*	*1050*	478	20	0	20	0	20	0	20	0	
13+000	2601	2393	2587	302	478	17	0	15	0	13	20	12	55	
14+000	*5541*	*5905*	*17296*	304	408	20	0	20	0	20	0	20	0	
15+000	*5151*	*4787*	*4005*	366	408	20	0	20	0	20	0	20	0	
16+000	4898	*5220*	*19479*	*424*	198	20	0	20	0	20	0	20	0	
17+000	*7497*	*7041*	*5334*	*481*	198	20	0	20	0	20	0	20	0	
18+000	3753	*5000*	*3000*	*459*	185	20	0	20	0	20	0	20	0	
19+000	3753	*5000*	*3000*	*279*	270	20	0	20	0	20	0	20	0	
20+000	3965	*4805*	*3134*	*346*	270	20	0	20	0	20	0	20	0	
21+000	*5596*	*5281*	*4702*	*429*	322	20	0	20	0	20	0	20	0	
22+000	4048	2311	1813	*356*	322	16	0	14	0	12	25	11	55	
23+000	3747	3860	*3000*	*346*	322	20	0	20	0	20	0	20	0	
24+000	839	843	*6932*	89	322	1	95	1	120	0	140	0	155	H2
24+542	2005	1033	1942	229	322	1	75	1	105	1	120	1	140	

Legend: *Red Text in italics– Back-calculated E_P and M_R were equal or exceeded the design values. The layer performed better than the expected design modulus.*

- **Subgrade**

The back-calculated values of the Residual Modulus of Subgrade Reaction M_R were found to be generally higher than or near to the expected design values in homogeneous section H1 (from km 0+000 to km 23+000). However, in homogeneous section H2, this was not the case, and a very low value of 89MPa was obtained at km 24+000. Therefore, the subgrade strength for lane 3 was found to be generally consistent with the design considerations except at homogeneous section H2 (from km 24+000 to km 24+542).

- **Residual Life and Projected Overlay requirements**

The back-calculated Residual Life of the pavement for each section and the corresponding projected overlay requirements projected to 5 years, 10 years, 15 years and 20 years of projected traffic were also indicated in Table 4.6. From the table, it was observed that the residual life for homogeneous section H1 (0+000 to 23+000) was still 20years even after 20years of projected traffic implying that there was no need of overlay even after the full life of the road except at km 13+000 and at km 22+000. At km 13+000, the residual life reduced progressively from 17years, 15years, 13years and 12years after 5years, 10years, 15years and 20years respectively of projected traffic. A similar trend was observed at km 22+000 too and an overlay of 55mm was required for homogeneous section H1 (0+000 to 23+000) after 20years of projected traffic..

However, from km 24+000 to 24+542, the road required an overlay of 95mm after 5years of opening to traffic with a residual life of 1year. The same section required an overlay of 120mm after 10years of projected traffic. The section was considered to be due for reconstruction.

4.6.4 Structural Evaluation of Lane 1 – From Thika Town to Kenyatta University

The results in Table 4.7 indicated the back-calculated pavement moduli of each layer for lane 1 form Thika to Kenyatta University. The evaluation of the performance of each layer was also done by comparing the back-calculated E-modulus of each layer against the design E-modulus values in Table 2.4. The homogeneous sections in Figure 4.16 and Figure 4.17 were also slightly adjusted in Table 4.7 to correspond to the actual pavement performance.

- **<u>Asphalt Concrete Surfacing (AC) and Dense Bitumen Macadam (DBM)</u>**

From Table 4.7, the performance of the AC and DBM along the entire lane 1 from Thika Town to Kenyatta University was generally poor with all except one of the back-calculated E-modulus values at km 15+000 in homogeneous section H4 falling below the expected value of 5,000MPa. The lowest value was obtained at km 00+000 (1451MPa) in homogeneous section H1. Therefore, the performance of the AC and DBM layers were found to be below average.

- **<u>Graded Crushed Stone (GCS) Base</u>**

It was observed that the back-calculated E-modulus values of GCS were below the expected with values of Modulus of Elasticities of 4,000MPa obtained in homogeneous sections H1, H3 and H5. Some very low values were obtained at the beginning of homogeneous section H1. Therefore, the performance of the GCS on that lane was also found to be generally below the expected design average.

- **<u>Cement Improved Gravel Subbase</u>**

The back-calculated CIG Subbase E-modulus values were observed to be below the design value of 3000MPa in the homogeneous section H1, H3 and H5. In homogeneous sections H2, H4 and H6, the back-calculated E-Modulus values were above or near the design average. The performance of the CIG Subbase was generally found to be above or near the expected average from homogeneous section H2 onwards (km 6+000 to km 24+042), except at km 17+000 in H7 (1725MPa) where a much lower value was obtained.

- **<u>Subgrade</u>**

The back-calculated values of the Residual Modulus of Subgrade Reaction (M_R) were found to be higher than or close to the expected design values. Therefore, the subgrade strength for the entire lane 1 was found to be generally consistent with the design considerations.

Table 4.7 Structural evaluation of Lane 1 from Thika to Kenyatta University

Chainage (km)	Back-calculated E-Modulus				Design Residual Modulus of Subgrade Reaction (M_R)	5 Years projection		10 Years projection		15 Years projection		20 Years projection		Homogeneous Section
	AC & DBM, $E1$ = 5, 000 MPa	GCS, $E2$ = 4, 000 MPa	Subbase , $E3$ = 3, 000 MPa	Back-calculated Modulus of Sub-grade Reaction (M_R)		Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	
00+000	1451	649	1788	*190*	185	0	105	0	130	0	145	0	160	H1
01+000	2218	1656	1866	*228*	185	6	0	5	55	5	80	4	95	
02+000	2575	1749	1976	*230*	203	7	0	7	40	6	70	5	90	
03+000	2274	1639	2290	200	203	6	0	5	55	5	80	4	95	
04+000	2526	2236	2124	*263*	203	16	0	14	0	12	30	11	55	
05+000	3457	2491	2103	*309*	190	20	0	20	0	18	0	16	35	
06+000	3784	*5000*	*3000*	167	190	20	0	20	0	20	0	20	0	H2
07+000	3190	3471	*3577*	*426*	142	20	0	20	0	20	0	20	0	
08+000	3887	*4828*	*3156*	*295*	142	20	0	20	0	20	0	20	0	
09+000	3756	3459	2571	*251*	142	20	0	20	0	20	0	20	0	
10+000	2246	1980	2562	*222*	115	11	0	10	10	8	55	7	75	H3
11+000	3784	*5000*	*3000*	*359*	126	20	0	20	0	20	0	20	0	H4
12+000	3920	*4690*	2902	*447*	126	20	0	20	0	20	0	20	0	
13+000	3753	*5000*	*3000*	*426*	116	20	0	20	0	20	0	20	0	
14+000	3891	3475	2463	*378*	116	20	0	20	0	20	0	20	0	
15+000	*6112*	*5035*	*4514*	*406*	147	20	0	20	0	20	0	20	0	
16+000	3345	2839	2229	*390*	201	20	0	20	0	20	0	20	0	
17+000	2550	1521	2308	*218*	201	5	10	4	60	4	85	3	100	H5
18+000	4287	*4937*	*3644*	*289*	201	20	0	20	0	20	0	20	0	H6
19+000	3110	2535	2460	*332*	201	20	0	20	0	19	0	16	30	
20+000	3817	3387	2190	*261*	201	20	0	20	0	20	0	20	0	
21+000	3629	*4299*	*3000*	*435*	201	20	0	20	0	20	0	20	0	
22+000	3858	*4884*	*3568*	246	279	20	0	20	0	20	0	20	0	
23+000	3110	2289	1725	223	279	17	0	15	0	14	15	12	55	H7
24+042	3753	*4863*	*3000*	215	279	20	0	20	0	20	0	20	0	

Legend: *Red Text in italics– Back-calculated E_P and M_R were equal or exceeded the design values. The layer performed better than the expected design modulus.*

- **<u>Residual Life and Projected Overlay requirements</u>**

The back-calculated Residual Life of the pavement for each section and the corresponding projected overlay requirements projected to 5 years, 10 years, 15 years and 20 years of projected traffic were also indicated in Table 4.7. From the table, the residual life for homogeneous section H1 was 0years after only 5 years of projected traffic at km 00+000 with an overlay requirement of 105mm. The overlay requirement increased to 130mm (after 10 years), 145mm (after 15years) and 160mm (after 20years). It meant the road required reconstruction after 5years of projected traffic in homogeneous section H1 (from km 0+000 to 5+000). In homogeneous section H2 from km 06+000 to 9+000, and H4 (form km 11+000 to 16+000), the residual life remained 20 years for the 20 years of projected traffic and there was no need for overlay. For homogeneous section H3, H5 and H7, the trend of deterioration was progressive but the worst case was H5. At H5, the residual life reduced to 5years after 5years of projected traffic and progressed to 3years after 20years of projected service. An overlay of 60mm applied from km 1+000 to km 24+042 after 10years of projected traffic would bring the road back to serviceable level. However, the lane required reconstruction from km 0+000 to km1+000 after 5 years of projected traffic.

4.6.5 Structural Evaluation of Lane 2 – From Thika Town to Kenyatta University

The results in Table 4.8 also indicated the back-calculated pavement moduli of each layer for lane 2 form Thika town to Kenyatta University. The evaluation of the performance of each layer was also done by comparing the back-calculated E-modulus of each layer against the design E-modulus values in Table 2.4. The homogeneous sections in Figure 4.18 and Figure 4.19 were also slightly adjusted in Table 4.8 to correspond to the actual pavement performance.

- **<u>Asphalt Concrete Surfacing (AC) and Dense Bitumen Macadam (DBM)</u>**

From Table 4.8, it was observed that all the values of the back-calculated E-modulus obtained were below the expected 5000MPa for AC and DBM along the entire lane. Therefore, the performance of the AC and DBM was found to be below average for this lane.

Table 4.8 Structural Evaluation of Lane 2 from Thika Town to Kenyatta University

Chainage (km)	Back-calculated E-Modulus				Design Residual Modulus of Subgrade Reaction	5 Years projection		10 Years projection		15 Years projection		20 Years projection		Homogeneous Section
	AC & DBM, *E1* = 5, 000 MPa	**GCS, *E2* = 4, 000 MPa**	**Subbase , *E3* = 3, 000 MPa**	**Back-calculated Modulus of Sub-grade Reaction, (M_R)**		**Residual Life (Years)**	**Overlay (mm)**	**Residual Life (Years)**	**Overlay (mm)**	**Residual Life (Years)**	**Overlay (mm)**	**Residual Life (Years)**	**Overlay (mm)**	
00+000	1232	673	1753	*215*	185	0	100	0	130	0	145	0	160	H1
00+300	3914	*5000*	*3000*	83	185	20	0	20	0	20	0	20	0	
01+300	3598	3313	2802	*278*	203	20	0	20	0	20	0	20	0	
02+300	2769	1803	2177	*426*	203	8	0	7	35	6	65	6	85	
03+300	3173	2139	1922	*282*	203	14	0	12	0	11	35	10	60	
04+300	4012	*5000*	*3487*	*308*	190	20	0	20	0	20	0	20	0	
05+300	1935	1350	2091	*282*	190	3	40	3	80	3	100	2	115	
06+300	3383	3597	*3388*	*341*	142	20	0	20	0	20	0	20	0	H2
07+300	3066	3015	2693	*197*	142	20	0	20	0	20	0	20	0	
08+300	3467	3828	*3000*	*291*	142	20	0	20	0	20	0	20	0	
09+300	2663	2200	2405	*299*	115	15	0	13	0	12	30	10	60	
10+300	2680	3303	*4576*	*407*	126	20	0	20	0	20	0	20	0	
11+300	2435	2247	2729	*309*	126	16	0	14	0	13	25	11	55	
12+300	2799	2526	2743	*384*	116	20	0	20	0	18	0	16	30	
13+300	3393	3003	2629	*250*	116	20	0	20	0	20	0	20	0	
14+300	3312	2220	1822	*224*	147	16	0	14	0	12	25	11	55	
15+300	4583	*4927*	*3980*	*433*	201	20	0	20	0	20	0	20	0	
16+300	2137	1399	1829	*205*	201	4	30	3	75	3	95	3	110	H3
17+300	2941	2973	2979	*320*	201	20	0	20	0	20	0	20	0	
18+300	2345	2453	2833	*265*	201	20	0	19	0	17	0	15	40	
19+300	2937	3094	*3125*	*394*	201	20	0	20	0	20	0	20	0	
20+300	3289	3334	2729	*341*	201	20	0	20	0	20	0	20	0	
21+300	2821	2541	2444	*288*	279	20	0	20	0	19	0	16	30	
22+300	3535	2814	2415	*293*	279	20	0	20	0	20	0	20	0	
23+300	2510	2000	2531	*344*	279	11	0	10	5	9	55	8	75	
24+015	1944	1568	2721	172	279	5	0	5	60	4	85	4	100	

Legend: *Red Text in italics– Back-calculated E_P and M_R were equal or exceeded the design values. The layer performed better than the expected design modulus.*

- **Graded Crushed Stone (GCS) Base**

It was also observed that, save for a few locations along the entire lane 2 from Thika Town to Kenyatta University, the performance of the GCS was poor since the most of the values of Modulus of Elasticities obtained through back-calculation fell below the expected average value of 4,000MPa. Although some higher values were obtained in homogeneous sections H1 and H2, some very low values were also obtained especially in H1 at km 0+000 (673MPa), at km 2+300 (1803MPa), at km 5+300 (1350MPa). Similarly, some very low values were back-calculated at Homogeneous section H3 at km 16+300 (1399MPa) and at km 24+015 (1944MPa). Therefore, the GCS on lane 2 from Thika Town to Kenyatta University was found to perform below average.

- **Cement Improved Gravel Subbase**

The back-calculated CIG Subbase E-modulus values were observed to be generally below the design value of 3000MPa. Some values of back-calculated E-Modulus in homogeneous section H1 (1753MPa) and (1922MPa), H2 (1822MPa) and H3 (1829MPa) were found to be substantially lower than the expected 3000MPa. Therefore, the overall performance of the CIG Subbase was generally found to be below average with only about 30% of the back-calculated E-Modulus values above the expected average.

- **Subgrade**

The back-calculated values of the Residual Modulus of Subgrade Reaction (M_R) were found to be higher than the expected values except at km 00+300 in homogeneous section H1 and at km 24+015 in homogeneous section H3. Therefore, the subgrade strength for the entire lane 2 was found to be generally consistent with the design considerations.

- **Residual Life and Projected Overlay requirements**

The back-calculated Residual Life of the pavement for each section and the corresponding projected overlay requirements projected to 5 years, 10 years, 15 years and 20 years of projected traffic were also indicated in Table 4.8. From the table, the residual life of km 0+000 in homogeneous section H1 was 0years after 5years of projected traffic. The overlay requirement was 100mm after 5years, but increased to 150mm after 10 years, 145mm after 15years and 160mm after 20years. Considering the performance of km 5+300 in the same homogeneous section, a reconstruction was required for this section after 5years of projected service. The performance of homogeneous section H2 (from km6+300 to km 15+300) was above average and the maintenance intervention required was an overlay of 30mm after

15years of projected traffic and 60mm after 20years of projected service. However, homogeneous section H3 (from km 16+300 to 24+015) required an overlay of 30mm, 75mm, 95mm and 110mm after 5years, 10years, 15years and 20years of projected traffic respectively.

4.6.6 Structural Evaluation of Lane 3 – From Thika Town to Kenyatta University

The results in Table 4.9 also indicate the back-calculated pavement moduli of each layer for lane 3 from Thika Town to Kenyatta University. The evaluation of the performance of each layer was also done by comparing the back-calculated E-modulus of each layer against the design E-modulus values in Table 2.4.

- **Asphalt Concrete Surfacing (AC) and Dense Bitumen Macadam (DBM)**

It was observed from Table 4.9 that along lane 3 from Thika Town to Kenyatta University, the performance of the AC and DBM below average, with most values of Modulus of Elasticities falling below the expected value of 5,000MPa. It was observed from km 00+000 to km 03+600 in homogeneous section H1 that none of the values back-calculated was or above the expected 5,000MPa. But for homogeneous section H2, the AC and DBM layers were found to be generally performing above or near the expected average. The performance of the AC and DBM layers on Lane 3 from Thika Town to Kenyatta University was found to be average.

- **Graded Crushed Stone (GCS) Base**

From Table 4.9, it was observed that at km 00+000 and at 01+600 in homogeneous section H1, significantly lower values of back-calculated E-modulus were obtained (1372MPa and 314MPa respectively) compared with the expected 4000MPa. The rest of the back-calculated values were found to be above or near the expected average of 4000MPa. The performance of the GCS base in homogeneous section H2 (04+600 to 24+000) in this lane was above average.

- **Cement Improved Gravel Subbase**

The back-calculated CIG Subbase E-modulus values were observed to be generally above the design value of 3000MPa. But it was only from km 00+000 to 00+600 in homogeneous section H1 where slightly lower values were obtained. The entire CIG Subbase for lane 3 from Thika Town to Kenyatta University performed above average with very high values obtained in homogeneous section H2. The highest value was 11460MPa at km 15+600.

Table 4.9 Structural Evaluation of Lane 3 from Thika Town to Kenyatta University

Chainage (km)	Back-calculated E-Modulus				Design Residual Modulus of Subgrade Reaction (M_R)	5 Years projection		10 Years projection		15 Years projection		20 Years projection		Homogeneous Section
	AC & DBM, $E1$ = 5,000 MPa	GCS, $E2$ = 4,000 MPa	Subbase , $E3$ = 3,000 MPa	Back-calculated Modulus of Sub-grade Reaction, (M_R)		Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	Residual Life (Years)	Overlay (mm)	
00+000	2463	1372	1797	*290*	185	3	35	3	75	3	95	2	110	H1
00+600	2813	1314	1758	161	185	3	40	3	75	2	95	2	110	
01+600	2976	2183	2112	187	203	15	0	13	0	12	30	10	55	
02+600	3753	*5000*	*3000*	*256*	203	20	0	20	0	20	0	20	0	
03+600	3146	2524	2428	*267*	203	20	0	20	0	18	0	16	30	
04+600	3970	3737	2503	*299*	190	20	0	20	0	20	0	20	0	H2
05+600	*11521*	*4793*	*3220*	*276*	190	20	0	20	0	20	0	20	0	
06+600	*5568*	*4779*	*4851*	*450*	142	20	0	20	0	20	0	20	0	
07+600	3979	4113	2779	*486*	142	20	0	20	0	20	0	20	0	
08+600	3893	3445	2496	*276*	142	20	0	20	0	20	0	20	0	
09+600	4146	3363	2550	*312*	115	20	0	20	0	20	0	20	0	
10+600	*8215*	*4786*	*4268*	*292*	126	20	0	20	0	20	0	20	0	
11+600	*7064*	*4822*	*4797*	*382*	126	20	0	20	0	20	0	20	0	
12+600	4470	2867	2146	*432*	116	20	0	20	0	20	0	20	0	
13+600	4288	*5833*	*5076*	*449*	116	20	0	20	0	20	0	20	0	
14+600	4668	*5021*	*4041*	*360*	147	20	0	20	0	20	0	20	0	
15+600	*9048*	*9386*	*11460*	*405*	201	20	0	20	0	20	0	20	0	
16+600	*7250*	*4175*	*3000*	*298*	201	20	0	20	0	20	0	20	0	
17+600	*8903*	*4915*	*4034*	*313*	201	20	0	20	0	20	0	20	0	
18+600	3856	*5000*	*3000*	*294*	201	20	0	20	0	20	0	20	0	
19+602	*10592*	*5879*	*5937*	*618*	201	20	0	20	0	20	0	20	0	
20+600	3856	*5000*	*3000*	*384*	201	20	0	20	0	20	0	20	0	
21+600	*5742*	*4583*	*3363*	248	279	20	0	20	0	20	0	20	0	
22+600	3607	3023	2335	*291*	279	20	0	20	0	20	0	20	0	
23+600	3856	*5000*	*3000*	*295*	279	20	0	20	0	20	0	20	0	
24+000	4437	*4756*	*3269*	197	279	20	0	20	0	20	0	20	0	

Legend: *Red Text in italics– Back-calculated E_P and M_R were equal or exceeded the design values. The layer performed better than the expected design modulus.*

- **<u>Subgrade</u>**

The back-calculated values of the Residual Modulus of Subgrade Reaction (M_R) were found to be higher than or near to the expected values for the entire lane. Therefore, the subgrade strength for the entire lane 3 was found to be generally consistent with the design considerations.

- **<u>Residual Life and Projected Overlay requirements</u>**

The back-calculated Residual Life of the pavement for each section and the corresponding projected overlay requirements projected to 5 years, 10 years, 15 years and 20 years of projected traffic were also indicated in Table 4.9.

Considering homogeneous section H1, between km 00+000 to km 03+600, the residual life decreased to 3years after 5years of projected traffic to 2years after 20years of projected service. Similarly, the overlay requirements varied from 40mm (5years), 75mm (10years), 95mm (15years) and 110mm (20years). The section was due for reconstruction after 20years of projected service. Homogeneous section H2 (04+600 to 24+000) displayed very good performance considering that there was no reduction in back-calculated residual life and there were no overlay requirements for the entire 20 years of projected service.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Lanes from Kenyatta University to Thika Town

Conclusion 1

i. On Lane 1 from Kenyatta University to Thika Town, the section from km 00+000 to km 17+000 required an overlay of 55mm after 10 years of projected traffic. But the section from 18+000 to km 24+161 required reconstruction after only 5 years of opening to traffic, and the residual life had reduced to 0years at km 24+161.

ii. Lane 2 from Kenyatta University to Thika Town required an overlay of 55mm from km 0+000 to km 23+400 after 5years of projected traffic. That was because the homogeneous sections across that section were found to be so randomly spaced that spot maintenance would be difficult to execute. At km 24+145, the overlay requirement after 5years of projected traffic was 65mm.

iii. Lane 3 from Kenyatta University to Thika Town from km 00+000 to 23+000 required no improvement after 10 years of projected traffic. For that section, an overlay of 55mm was required after 20years of projected traffic. However, from km 24+000 to km 24+542, the road required reconstruction after 5 years of projected traffic.

Recommendation 1

Based on the performance of Lane 2, an overlay of 55mm was recommended for all the lanes starting at km 0+000 to km 18+000 after 5years of projected traffic, and reconstruction of all the lanes from km 18+000 to km 24+542 after 5years of opening the road to traffic based on the performance of Lane 1 and Lane 3.

5.2 Lanes from Thika Town to Kenyatta University

Conclusion 2

i. Lane 1 from Thika Town to Kenyatta University required reconstruction from km 00+000 to 01+000 after 5 years of projected traffic. However, the section from km 01+000 to 05+000 and from km 17+000 to 18+000 required an overlay of 60mm after 10years of projected traffic.

ii. Lane 2 from Thika Town to Kenyatta University from km 00+000 to km 00+300 required reconstruction after 5years of projected traffic. However, the rest of the sections required an overlay of 80mm after 10years of projected traffic.

iii. Lane 3 from Thika Town to Kenyatta University required 40mm overlay from km 00+000 to 04+600 after 5years of projected traffic. From km 04+600 to km 24+000 there was no need for intervention even after 20years of projected traffic.

Recommendation 2

Based on the performance of Lane 1 and Lane 2, reconstruction was recommended for all the lanes starting at km 0+000 to km 1+000 after 5years of projected traffic, and an overlay of 80mm on all the lanes from km 1+000 to km 24+000 after 10years of opening the road to traffic (also based on the performance of Lane 1 and Lane 2).

Conclusion 3

Since deflection survey was done one year after opening the road to traffic, it was unlikely that the failure of the road was entirely due to overloading practices. Furthermore, the pavement design based on AASTHO method was considered adequate as it captured the actual Design Equivalent Standard Axles which the Kenyan Road Design Manual (III) did not capture (the RDM III considered a maximum of 60million ESA only). The Vehicle Equivalent Factors (V_{EF}) used in design were also the actual factors established by way of axle load survey when overloading practices were rampant. The other likely cause of the pavement failure was due to inadequate pavement layers structure at the identified homogeneous sections probably due to challenges in quality control during construction.

Recommendation 3

More stringent construction quality controls should be implemented to ensure that construction of road pavements are implemented as designed.

Conclusion 4

The use of the speed bumps along the road section reduced speeds of the heavy commercial vehicles thus increasing the transient loads. The severity of the ruts just before the speed bumps was evidence of that fact.

Recommendation 4

It was recommended that the speed bumps erected along the road be removed. Pedestrians and Non-Motorised Traffic (NMT) users should be restricted to their designated facilities and channeled to the safe crossing points at the footbridges. The roads authorities should consider investing in more NMT and pedestrian facilities along the road in order to address the inadequacies that made it necessary to install speed bumps. Further study was recommended to establish the full impact of the reduced transient speeds on the damaging effects of the axle loads effects at the speed bumps.

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