STUDIES OF FLEXIBLE ROAD AND AIRPORT PAVEMENTS IN KENYA

By

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Declaration

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

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

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Abstract

This thesis presents results of a long-term study initiated in the late 1960s on flexible pavements in Kenya. The study was conducted through field and laboratory studies. It was prompted by the need for continuing research and development of rational design, construction and maintenance of a large network of paved roads in Kenya and increasing runway and taxiway pavements for rising air traffic.

The geotechnical properties of some subgrade soils in Kenya have been presented. In addition a method for determining the optimum moisture content and maximum dry density from bulk density and percentage of water added to specimens during experimentation has been developed. The use of locally available construction materials has been found to be acceptable with good compaction and selection of suitable materials. Timely maintenance of flexible pavements and in particular protecting the pavement structures from ingress of water has been found to enhance their supporting capacity.

The properties of the entire pavement structure have been shown to change with age and environment. The bituminous surfacing hardens with age while the base and subbase materials strength characteristics change upon loading. The net effect of all this is a continuously changing structural capacity of flexible pavements. The study of bituminous materials has shown that the voids content of the bituminous structure is a critical consideration. A minimum voids content is required to give the mixture the necessary elasticity while at the same time the voids contents should be minimised for the reduction of hardening of the binder. The hardening of the bituminous materials has been shown to be associated with increasing asphaltene content in the binder. The penetration of bitumen recovered from thin asphalt

surfacings has been found to follow a hyperbolic function with age. For the thick bituminous concrete surfacing, as the air voids content increase the bitumen penetration and ductility reduce with a power relationship.

Evaluation of the pavement structures can be done by quantification of surface condition distress indicators. These include cracking, rutting, roughness and present serviceability index (PSI). A multiple regression equation has been developed showing that PSI is a function of rutting, roughness and cracking. A new maintenance criterion has been developed using a new total rut concept. The current roughness at which maintenance for road pavements was required was found to be high. In effect even when other surface conditions namely cracking, rutting and the present serviceability index indicated the pavement required maintenance, the roughness would pass the road as well performing. A new roughness criterion has therefore been proposed.

The structural capacity can be established using deflection measurements and the dynamic cone penetrometer. Deflection data together with finite element analysis has been shown to be a method of determining pavement deflection and pavement stresses. When finite element is carried out with varying pavement moduli the modulus, which gives the same deflection as the field deflection is regarded as the pavement resilient modulus.

A more relaxed specification for construction of pavements carrying light traffic has been proposed. For the more heavily loaded pavements including runways and taxiways, maintenance is a key element, which seems to have escaped the attention of those responsible for maintaining them. This is because timely maintenance and especially clearing of water under the sealed pavement layers is crucial in maintaining the structural capacity of the entire pavement structure.

Acknowledgements

I would like to acknowledge the guidance and encouragement of my University supervisors. First Professor Francis J. Gichaga who despite his busy office of the vice chancellor found time to discuss the progress of this work from inception to its completion. Secondly, Dr. O. O. Mbeche who made invaluable comments. I would also like to thank Dr. B. N. K. Njoroge the chairman of Department of Civil Engineering for his encouragement at the time of completion of this study. I would further like to extend my thanks to Germany academic exchange service (DAAD) who provided financial support for the research.

These acknowledgements would be incomplete without recognising the many colleagues who have assisted me during the study. There are those who assisted wittingly in the collection of data and test results and those who unwittingly made invaluable contributions in general discussions.

To all of you thank you.

S. K. Mwea
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Dedication

This work is dedicated

To my wife Eleanor

And

To my children Lemmy and Winnie

Their special place in me made it happen.

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Notation Index

The following is the notation of the major symbols used in the thesis

σ' – Effective normal pressure on the failure plane

ϕ' – Effective angle of internal friction

ε - Normal strain

σ - Normal stress

μ - Poisson ratio

γ - Shear strain

τ - Shear stress

AADT – Annual average daily traffic

AC – Asphalt concrete

ACN – Airport classification number

ACV – Aggregate crushing value

Aiv – Air voids content in bituminous mixtures

AIV – Aggregate impact value

ANOVA – Analyses of variance

Asc – Asphaltene content

BC – Bitumen content

BD – Bulk density

C' – effective cohesion

CBR – California bearing ratio

CDM – Compacted density of mix

CESA – Cumulative equivalent standard axles

CH – Clays of high plasticity

CL – Clays of low plasticity

CR – Critical rut

CI – Cracking index

d_0 – Pavement deflection at the centre of the deflection bowl

d_{90} – Characteristic deflection

DCP – Dynamic cone penetrometer

DD – Dry density

DF – Degrees of freedom in analyses of variance

DN – DCP number measured in mm/blow

DSD – Double surface dressing

Duct - Ductility

E – Young's modulus

ES – Experimental site

ESA – Equivalent standard axles

ESWL – Equivalent single wheel load

f - Marshal flow

F – Regression ratio in the statistical F test performed in the analysis of variance

FEM – Finite element method

FI - Flakiness index

Fp – Flash point

GM – Silty GRAVEL

GM-P – Poorly graded silty GRAVEL

GIS – Geographical information systems

GPS – Global positioning system

GC – Clayey GRAVEL

LCN – Load classification number

LAA - Los Angeles Abrasion value

LL – Liquid limit

MST – Marshal stability

MSE – Marshal stiffness

MST– Marshal stability

MDD – Maximum dry density

MBD – Maximum bulk density

ML – Silts of high plasticity

MH- Silts of high plasticity

Mr – Resilient modulus

MS – Ratio of the sum of squares to the degrees of freedom in analyses of variance

OB – Original binder course

OMC – Optimum moisture content

PA – Probability of acceptance

Pen - Penetration

PI – Plasticity index

PM – Plasticity modulus

PSI – Present serviceability index

PSR – Present serviceability rating

r – Coefficient of correlation

r^2 – Coefficient of determination

R – Radius of curvature in the deflection bowl

Ro – Roughness

S – Shear strength

Signf F – Significance level of a regression in analyses of variance

SG – Specific gravity

Sp – Softening point

SPSS – Statistics program for social scientists

SS – Sum of squares in analyses of variance

SSS – Sodium sulphate soundness

Temp - Temperature

TR – Total rut

VTM – Voids in total mix

VFB – Voids filled with bitumen

USC – Unified soil classification system

WC – Wearing course

X – Independent variable in regression analyses

y – Observed value of the dependent variable in regression analyses

Y – Estimated value of y in regression analyses

$\ddot{y}$ - Mean of the observed values of y

Chapter One

Introduction

1.1 General

1.1.1 Road pavements

The history of road development dates back several hundred years. This can be traced in ancient civilisations of India, Egypt, Babylon (Sherrad, 1958), Japan (Road Bureau, Ministry of Construction, 1989). However, road construction became an engineering science based on theory, design parameters and experience in the eighteenth century when French and English engineers started utilising stone bases and gravel wearing courses (Sherrald, 1958). The need for drainage was emphasised at around the same time. The nineteenth century and early twentieth century saw the advent of tar as a surfacing material, initially to reduce dust and then as an integral part of the road pavement required to protect the lower pavement layers against the environment and also to improve the quality of the riding surface.

Towards the end of nineteenth century and the beginning of twentieth century the motor vehicle as a mode of travel became increasingly important (O'flaherty, 1974). The rise in the usage of the motor vehicle called for increased road network. This is true in this country like in many other countries in the world. The rise in the motor vehicle traffic has grown steadily and especially so in the last three decades. In 1992 three hundred thousand vehicles plied the Kenyan roads (Roads Department, 1992). The road network needed to carry these vehicles in 1992 included eight thousand five hundred kilometres of bitumen roads. This network had increased to nine thousand kilometres in 2000 (Morogo 2001). Figure 1.1 shows the annual average daily traffic and its general distribution with the heavily loaded sections carrying over six thousand vehicles per day in 1983.

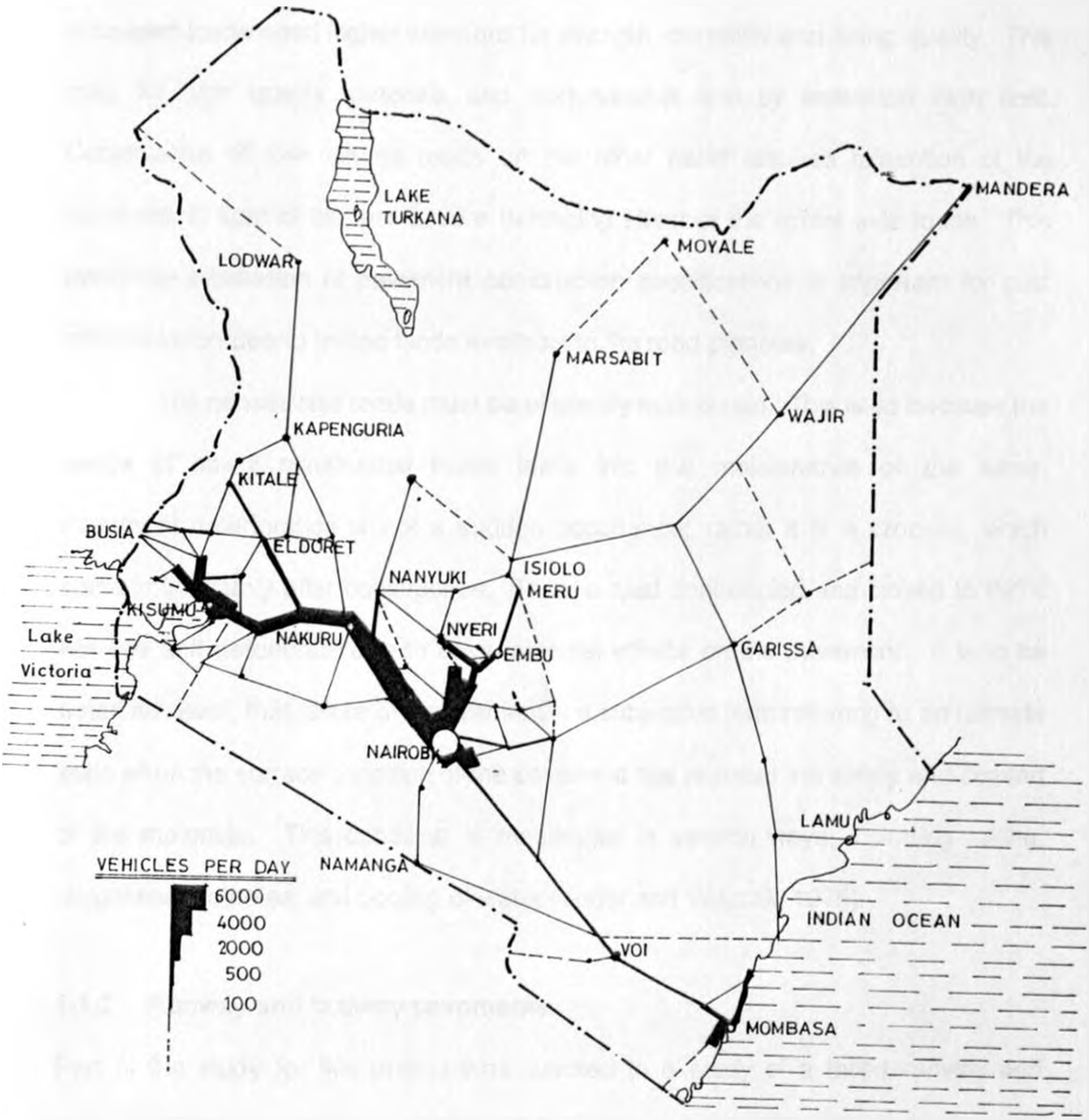


Figure 1.1 Annual average daily traffic in vehicles per day in 1983.

Source: Roads Department (1992)

This large network of flexible road pavements in the transport sector is required for the efficient movement of goods, services and general population. These roads need to be built to the most exacting standards, specifications and rigid quality control during construction. Roads carrying large volumes of traffic with increased loads need higher standard for strength, durability and riding quality. This calls for high quality materials and workmanship and by extension high cost. Construction of low volume roads on the other hand requires relaxation of the standards in light of the less severe damaging effect of the lighter axle loads. This beneficial separation of pavement construction specifications is important for cost rationalisation due to limited funds available to the road planners.

The constructed roads must be efficiently maintained. This is so because the usage of newly constructed roads leads into the maintenance of the same. Pavement deterioration is not a sudden occurrence; rather it is a process, which starts immediately after construction. Thus, a road constructed and closed to traffic will age and deteriorate due to environmental effects on the pavement. It is to be noted however, that failure of a pavement is a subjective term referring to an ultimate state when the surface condition of the pavement has reduced the safety and comfort of the motorists. This condition is manifested in several ways, including rutting, roughness, potholes, and pooling of water (Yoder and Witczak, 1975).

1.1.2 Runway and taxiway pavements.

Part of the study for this project was devoted to a study of a failed runway and taxiway. This enabled the study to cover the entire spectrum of flexible pavements in Kenya starting from low volume flexible road pavements through the heavy-duty flexible road and airport pavements. It is to be noted that the design construction and performance of flexible runway and taxiway pavements follow the same fundamental principles as the road flexible pavements. The main difference in the design is in the loading which is generally higher in the case of runway and taxiway pavements due to the heavier wheel loads of the present day aircraft (Horonjeff and Mckelvey 1983).

For this purpose therefore, a preliminary study was conducted to establish the air traffic growth in Kenya. The preliminary study is summarised below.

Air traffic growth in Kenya

With the invention of the aeroplane in 1903 (Ibid. 1983), air transportation took a key place in the transportation for goods and persons. This can be attributed to the advancement in the aircraft technology, which has resulted in fast and safe air journeys. Additionally the otherwise inaccessible parts of the world are reached by plane.

Data collected from the Kenya Airports Authority over the period 1991 to 1999 is shown on Table 1.1 and Figure 1.2. The data shows that there has been a steady rise in the movements and cargo at the four major Kenyan airports. The passenger traffic has levelled out at about four million passengers from 1994. The general drop in the mail can be attributed to the general increase in the usage of the electronic mail. Overall however, there is increment air traffic over the period.

The increasing usage of the airports leads into the need of pavement evaluation and maintenance. In the recent years the three older international airports namely Jomo Kenyatta International Airport Nairobi, Moi International Airport, Mombasa and Wilson Airport in Nairobi have had to receive major rehabilitation works. The runways and taxiways have necessitated the rehabilitation works after outliving their service lives.

Table 1. 1 Air traffic handled at the main airports in Kenya (1991-1999)

	Det.	1991	1992	1993	1994	1995	1996	1997	1998	1999
JKIA	Mov.	28.5	33.0	39.3	45.1	40.6	41.5	42.1	41.5	45.6
NBI	Pas.	1.86	1.88	1.87	2.80	2.79	2.68	2.55	2.35	2.67
	Car.	67.1	69.1	71.3	87.6	76.7	75.0	75.6	116	126
	Mail	1.74	1.77	1.93	1.82	1.57	1.49	1.54	1.39	1.07
Moi	Mov.	26.4	35.9	30.6	28.6	31.0	24.6	28.9	21.3	19.2
MSA	Pas.	.752	.888	.865	.972	.885	.927	.865	.812	.890
	Car.	1.36	1.25	1.04	1.26	1.57	2.45	3.30	3.14	2.35
	Mail	.076	.072	.058	.084	.078	.112	.064	.034	.026
WLS	Mov.	53.3	62.8	54.4	52.0	50.1	59.8	67.1	104	98.7
NBI	Pas.	.159	.188	.171	.190	.196	.213	.201	.234	.221
	Car.	1.06	2.64	3.33	4.29	5.39	3.59	3.63	4.77	3.07
ELD	Mov.	-	-	-	-	-	-	.163	1.45	1.67
	Pas.	-	-	-	-	-	-	.002	.017	.023
	Car.	-	-	-	-	-	-	.100	6.20	4.21
Sum	Mov.	108	132	124	125	122	126	138	168	165
	Pas.	2.77	2.96	2.91	3.96	3.87	3.82	3.62	3.42	3.80
	Car.	69.5	73.0	75.7	92.2	83.7	81.0	82.7	130	135
	Mail	1.82	1.84	1.98	1.91	1.65	1.60	1.61	1.42	1.09

Legend: JKIA NBI, - Jomo Kenyatta International Airport Nairobi, MSA - Mombasa, WLS - Wilson ELD - Eldoret, Mov. - movementsX10^3, Pas - Passangersx10^6, Car. - Cargox10^6kg, Mailx10^6 kg,

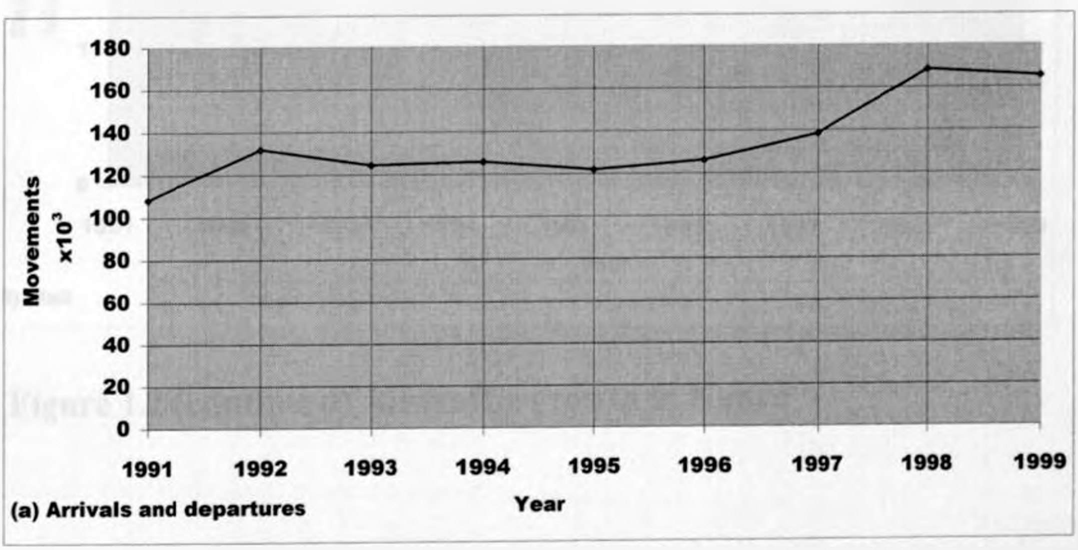


Figure 1.2 Air traffic growth in Kenya

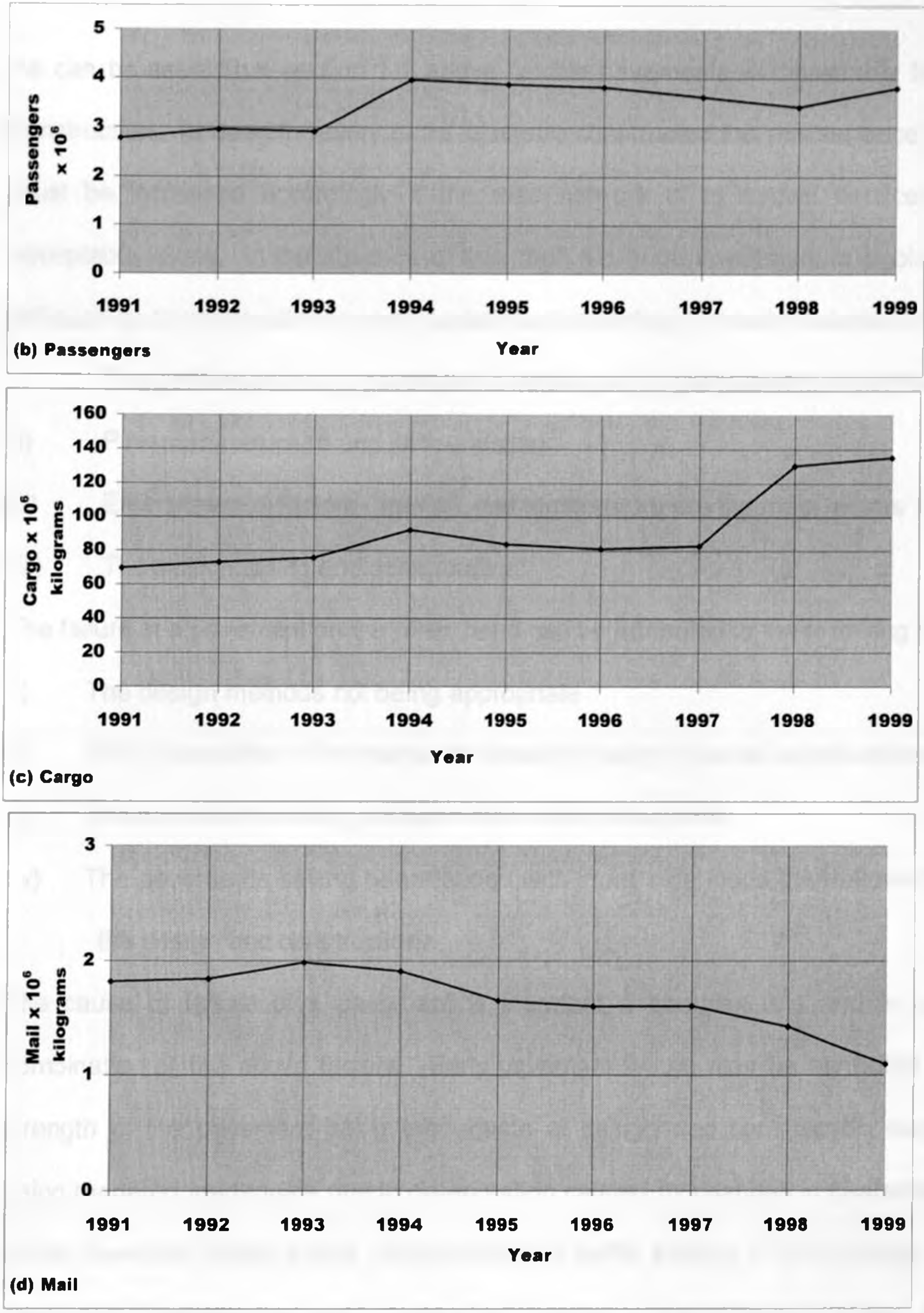


Figure 1.2 (continued) Air traffic growth in Kenya

1.2 Statement of the problem

As can be seen from section 1.1 above flexible pavements will inevitably fail after construction. In deed for every extra kilometre constructed the maintenance budget must be increased accordingly if the road network is to remain serviceable at acceptable levels. In the absence of this, then the huge investment in public funds will soon go to waste with the once paved roads reverting to barely motorable roads.

The performance of a pavement is influenced by the following broad factors.

(i) Pavement structure and its foundation

(ii) Environmental factors (rainfall, and temperature are the main factors)

(iii) The traffic loading and configuration.

The failure of a pavement on the other hand can be attributed to the following factors:

(i) The design methods not being appropriate

(ii) The construction of the roads not following design material specifications.

(iii) The pavements having not been maintained adequately.

(iv) The pavements having been loaded with more axle loads than allowed for in the design and construction.

The cause of failure of a pavement is therefore a complex one and is often a combination of the above factors. Early pavement failure may be attributed to the strength of the pavement being inadequate at design and construction stages or being rendered inadequate due to deterioration caused by changes in characteristics of the pavement layers and/or unexpected high traffic loading. The changes in the pavement characteristics may include age hardening of the bituminous layers, loss in strength of the pavement and subgrade layers due to ingress of water or excessive loading from traffic.

1.3 Specific objectives of the study

The previous sections have shown an existing problem that needs continuing research for more information of the design, construction and maintenance of the country's flexible pavements. It was therefore, pertinent to set out specific objectives to address the various issues raised in the stated problems. Identifying specific objectives, which can be summarised as follows, did this.

- ***Non bituminous pavement materials***

To study the geotechnical properties of non-bituminous materials in order to influence current flexible pavement construction practice.

- ***Bituminous materials***

To study the performance of bituminous materials under the Kenyan environment. This will include age hardening and re-healing behaviour of the bituminous mixtures. This will enhance the understanding of their behaviour under different Kenyan traffic and environmental conditions.

- ***Study of pavement conditions of test roads.***

To establish the behaviour of selected existing pavements in various areas of the country. This will enable early detection of pavement deterioration.

To evaluate long term behaviour of in service pavements studied over several years and the thereafter assess the impact of maintenance operations.

- ***Case study of flexible airport runway and taxiway pavements.***

To evaluate the performance of flexible airport pavements and therefore compare the performance of the entire spectrum of pavements from low volume pavements through high volume road pavements to heavy duty airport pavements

- ***Finite element analysis***

To carry out structural analysis using finite element methods to enable comparison of theoretical and field deflections

1.4 Scope of the study

Considerable research has been carried out in the world and continues to be done in the areas of flexible pavement design, construction, maintenance and performance. The interest in this area of research has been geared towards generating comprehensive data and results, which can influence present design standards and construction practices in an attempt to reduce the incidence of flexible pavement failures.

The different environmental, economic and traffic conditions for each country require specific details not common to all countries to be identified for incorporation in the design and construction specifications. This calls for development and continuous updating of the design and construction practices to meet various specific conditions. Within the same country the diversity of the said factors call for their identification for the various parts of the country. In this light the scope of this study has attempted to cover a representative portion of Kenyan low volume and high volume roads which have been studied together with runway and taxiway pavements. Subgrade materials have been sampled from many parts of the country. The details and location of each of the study areas are shown in the relevant sections of the methodology and data collection in chapters three and four.

A study of the flexible pavement performance requires one to look into all aspects, which lead into pavement deterioration. Such a study should examine the basic components of the pavement and their performance under various traffic and environmental conditions. In this light the pavement structure as shown in Figure 1.3 representing typical road section needs a thorough understanding for design, construction, and maintenance specifications.

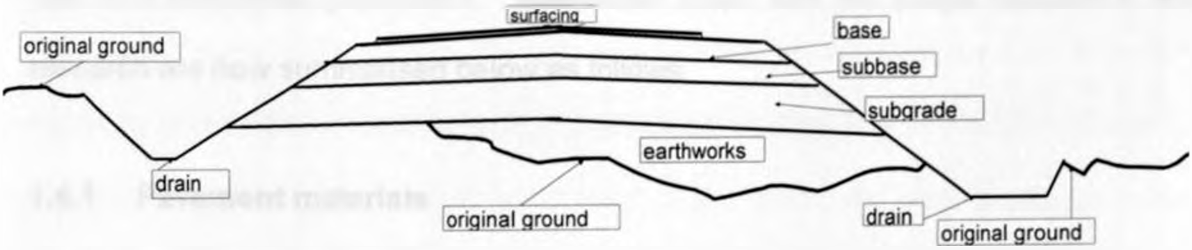


Figure 1.3 Typical pavement structure

The pavement structure is designed such that the stresses are distributed to the lower layers without excessive deformation. The stresses decrease with depth and consequently the stronger materials are placed at the top with the weaker materials being placed deeper down the pavement structures. The materials, which form the pavement structures, were studied to yield salient design strength characteristics.

The Roads Department (1987) design manual adopted a computer program that calculates the following parameters required for the design and specification of the pavement structures: -

(i) Horizontal tensile stresses and strains at the bottom of each layer of the bound material.

(ii) The vertical compressive stress and strain in the surface of each layer

(iii) The deflection at the surface of the pavement.

In order to understand the behaviour of the various materials under the above stresses and strains the various pavement materials used in the country were studied in order to identify their strength characteristics and hence assess the present design standards. The entire performance of the pavements was studied by pavement evaluation techniques while finite element analysis was carried out for comparison of field and theoretical predictions. The broad areas and the scope covered in this research are now summarised below as follows.

1.4.1 Pavement materials

(a) Natural materials

Natural subgrade materials including the widely distributed black cotton soils, clayey loamy soils, red coffee soils and gravel soils were studied.

(b) Lime treated gravels

Natural gravels when sufficiently strong are used as subbase and base materials. However, gravels whose strength requirements in their natural state can not sustain the traffic loads can be improved by addition of lime. Accordingly lime improved materials were tested and their performance has been recorded.

(c) Bitumen

Bitumen is a naturally occurring hydrocarbon found in combination with mineral aggregate and produced as a by-product from distillation of petroleum (O'flaherty, 1974; Roads Department, 1986). In Kenya, bitumen is used in the surface course and in the base construction for the heavily trafficked roads. When used in base course construction it may be used either in the form of dense bitumen macadam or dense bitumen emulsion. It has been observed that due to the age hardening of bitumen the use of grade 40/50 bitumen is not recommended in Kenya. Generally

60/70, 80/100 and 180/200 penetration bitumen are used in proportions ranging from 4.5 to 7.0%. The rest of constituents consist of graded aggregate and suitable filler. The Roads Department (1987) design manual on the other hand describes dense bitumen emulsion as a cheap alternative.

1.4.2 The performance of the pavements

The performance of the pavements was evaluated by the measurement of deflection, cracking and rutting. Data collected in previous studies in study sections was re-evaluated to determine possible relationship of environmental factors and pavement performance. The low cost low volume test sections were studied by visual inspection, measurement of rutting, cracking and recovery of pavement materials for laboratory testing.

1.4.3 The finite element analysis

The finite element analysis has been used to analyse the pavement structures. This enabled theoretical predictions to be compared with the field observations. In order to carry out a realistic finite element analysis the physical characteristics of the road pavement layers were used in the analysis.

1.5 Justification of the study

The study has dwelt on a key area of the transportation sector in the country. The findings promise development of rational use of locally available materials in recognition of the local traffic and environmental conditions. The test results on the locally available materials form a database for use by researchers and designers. The age hardening of bitumen has been researched and recommendations on the suitability of various binders for the Kenyan environment will provide a base for further research besides the results being available for immediate use by the road engineer in practice.

Analysis and interpretation of performance data has been presented. A useful criterion for analysing field data has been proposed. The relationship of pavement condition and present serviceability index has been established and this can be utilised in estimating the maintenance requirements.

Chapter Two

Literature Review

2.1 General

The flexible pavement comprises of various thickness of bituminous bound or cement bound or unbound aggregates resting on cohesionless or cohesive subgrades. All these materials must be characterised under all possible in-service loading and environmental conditions if a rational approach to design, construction and maintenance procedures for flexible pavements is to be developed. There is need to continuously revise design and construction procedures in consideration of fundamental properties on pavement materials and the response to loading of the entire pavement under the local in-service conditions.

It is therefore necessary to continuously carry out laboratory and full-scale pavement tests on pavement materials. In addition over-all performance of the pavement structures requires to be done alongside the above testing. In light of the above the literature review is presented in several parts as follows: -

(i) Environmental conditions in Kenya relevant to pavement performance.

(ii) Common embankment materials.

(iii) Pavement materials.

(iv) Pavement performance

The review of pavement performance is key in that it touches on the maintenance of pavements, which is necessary for virtually all constructed pavements.

(v) Airport pavements

(vi) Pavement management and geographical information systems

(vii) Finite element analysis as a tool is reviewed in light of the recent developments in the use of the computer (Younger 1990).

2.2 Environmental conditions in Kenya

The main principle environmental factors affecting road pavements in Kenya are moisture content and temperature (Hall 1979). The primary effect of temperature and moisture content are with respect into the response of pavement materials to load. For non-bituminous pavement materials the effect of rain is increase in material moisture content with corresponding reduction of the strength of the pavement layers. Temperature variations result in variation of the strength and chemical properties of the bituminous bound materials. It is the main contributor to the ageing of the binder in the bituminous mixtures.

There are marked differences in annual rainfall in Kenya which range from less than 250 mm in northern and eastern areas to over 2000 mm on the high mountain ranges (Survey of Kenya, 1970 and Roads Department 1987). Temperature variations between night and day range from 6°C to around 36°C. With this variation of environmental factors the performance of flexible pavements varies with the location of the roads even when the other conditions of traffic and pavement structure are the same. Figure A2.1 and A2.2 in appendix A show the variation of rainfall and temperature in the country respectively (Roads Department, 1987).

2.3 Common embankment materials

Embankment construction is generally made of locally available materials. This is generally economical and rarely would design engineers need to import embankment materials from long distances. The embankment geometry revolves around the choice of an appropriate slope. Only in very high embankments does the strength and settlement characteristics play a key role in the slope stability computations. In effect therefore, the height of embankments do not affect the choice of materials (Roads Department, 1987). The standard slopes have been determined by experience and monitoring of in-service slopes. The recommended slopes (vertical : horizontal) are as follows: -

(i)	Cohesionless materials	1:3 if $h \leq 1$ meter
		1:2 if $h > 1$ meter
(ii)	Other materials	1:3 if $h \leq 1$ meter
		1:2 if $1 < h < 3$ meters
		1:1.5 if $3 < h < 10$ meters

Where: - h is the height of the embankment.

However, sufficient strength in the embankment material is required to prevent the settlement and deformation of the embankment layers which would result in the loss of support of the pavement structure. The engineering properties, which determine the design of embankment for support of pavements, include grain size distribution, atterberg limits, moisture content, dry density, shear strength and the California Bearing Ratio (CBR). These properties are described in many textbooks (Craig, 1987; Gichaga and Parker, 1988; Smith and Smith, 1998). Their determination is explained in BS 1377 (1990)

In brief however, the particle size distribution of soils and the plasticity as measured by the atterberg limits lead to the classification of the soil. The particle size distribution influences the arrangement of particles in the compaction of soils. A well-graded soil will compact to lesser voids than a poorly graded soil. The effect is that the well-graded soils will have an increased density and improved properties such as bearing capacity on compaction. Additionally the greater the soil density the less its water absorption tendencies and the better improved resistance to settlement. These enhanced strength characteristics of compacted fills are important because of the increasing need of road embankments capable of supporting pavement structures under heavy traffic loads.

The strength of a compacted fill is derived from the effective cohesion (*C'*) and the effective angle of internal friction ($\varnothing'$) and can be expressed in the classical Coulomb equation (Craig, 1987): -

$$S = C' + \sigma' \tan\phi' \qquad [2.1]$$

Where: *S* is the shear strength

σ' is the effective normal pressure on the failure plane.

The biggest problem of the Coulomb equation is that *C'* and ϕ' are dependent on the loading conditions, stress history and testing conditions. In the field this will vary with the drainage which vary with changing environmental conditions. An embankment, which is constructed in materials, which change in strength on saturation, has strength variation over the usage period. This is typical of the fine cohesive materials and loose cohesionless soils.

Shear strength mobilisation on the coulomb equation has been explained for a long time (Rosenquit, 1959). The physical component of the shear strength in soils contributes to the frictional resistance and interlocking between the particles, when large scale interlocking between the particles is required. This movement is normal to the shear plane and is accompanied by volumetric expansion for failure to occur. Small scale interlocking due to particle surface roughness requires small movement for failure to occur. These factors are proportional to the effective normal stress on the failure plane and are of significance primarily between the granular particles. Although described as a physical factor the friction may also be described as a physical chemical factor. Thus when two particles are pressed together the contact is initially between a few grains. These grains deform elastically to develop van der Waal forces. The van de Waal forces arise from electrical forces developed in atoms bonding the particles together (Singh and Prakash, 1985). They hold the particles together and when they are brought closer more forces have to be overcome during shearing. The total contact area developed by the normal forces is directly proportional to the effective stress. Hence tanϕ represents the magnitude of the adhesive bond and the normal stress causing the contact bond. When the effective stress is removed from the material which develops its strength in the manner described above the elastic stresses within the particles will rebound. The particles attain a parking denser than the original configuration but looser than the one developed by the effective stress. This is because some of the deformation will be unrecoverable and can be regarded as plastic. The effect is loss of adhesion and shearing strength.

The other important physical factor contribution to the mobilisation of the shear strength is the interlocking between particles. In dense materials, grains move over adjacent grains during shearing deformation causing dilation. The dilation tends to decrease the pore water pressure and hence increases the effective stresses and consequently increases the frictional resistance.

The behaviour of soils containing clay particles is different from that of granular soils. Their behaviour is related to the arrangement of the clay minerals, the inter-particle forces and the nature of soil water (Lambe, 1958). The clay particles are generally less than two microns thin and flat. The surface area to the mass ratio is high and forces at the particle surfaces influences the behaviour of the atoms in the minerals. The net effect is electrical field at the particle surfaces. These two fundamental properties determine to a great extent the engineering behaviour of a compacted clay.

The above literature survey on embankment soils has dwelt principally on theoretical basis of mobilisation of strength for embankment soils. The bridge between theory and practice is achieved by conducting control tests during embankment construction. The principal tests include classification, tests (grading and atterberg limits) and relative density tests. Conducting the CBR test checks the strength of the compacted earthworks. The CBR test is by far the most widely used parameter of characterising the bearing capacity for earthworks and unbound granular pavement layers. This is principally because the deformation of the specimen as penetration is effected is fundamentally shear deformation. The test therefore provides an indirect measure of shear strength described above.

In order to retain the strength during the usage of pavement structures the Roads Department (1987) has specified construction procedures of embankments, which include: -

(i) The lower embankment layers are filled in one hundred fifty millimetre compacted layers. The compaction is required to be at least ninety five per cent standard BS compaction.

(ii) The top three hundred-millimetre of the embankment forms the subgrade of the pavement. Its strength determines the pavement thickness and quality. It is considered together with traffic loading for pavement design. For this top layer the compaction is raised to one hundred per cent.

(iii) Excluding potentially weak soils which have CBR values less than two or plasticity index exceeding fifty per cent or containing more than five percent by weight of organic matter or material with moisture content greater than one hundred and five per cent of the optimum moisture content.

(iv) Placing the expansive soils in the lower sections of fill where the weight counteracts the expansion of the soil.

The above construction safety measures compare well with those of other road networks as can be seen from the requirements of California State in USA as outlined below (State of California, Department of Transportation, 1992).

(i) Treatment of expansive soils with lime for reduction of expansion tendencies.

(ii) Replacement of weak soils with non-expansive materials.

(iii) Provision of surcharge over the expansive clays.

(iv) Placing the surface courses after the underlying materials have been allowed to expand and stabilise.

(v) Limiting the entry into the pavement layers of water by geo-textile and asphalt membranes

(vi) Relocation of project to areas with favourable conditions.

(vii) Compaction of earthworks to ninety five percent relative compaction

In conclusion the need of good compaction of earthworks and subgrade can not be underestimated. If the subgrade is not well compacted then it is not possible to effectively process and compact the overlying pavement layers. Besides, a poorly compacted subgrade will not offer effective support for the pavement layers. Material for subgrade should be selected from the stronger earthwork materials.

2.4 Pavement materials

2.4.1 Introduction

The pavement foundation material consists of the subgrade soils. These are basically the top layers of the earthworks. When the existing subgrade soils are weak an improved subgrade consisting of selected stronger earthworks are incorporated in the works to form an improved base for the pavement structure. Alternatively the weak subgrade soil may be stabilised. The subbase and the base layers are the main load carrying layers of the pavement structure. These usually consist of granular materials constructed either neat or improved with lime or cement. Where suitable granular materials are absent consideration of using suitably treated cohesive materials is made. In the case of the very heavily loaded pavement structures, crushed stone, lean concrete or bitumen bound materials are used in place of the granular materials.

The literature review with respect to the pavement materials is now presented. The presentation starts with subgrade materials continuing with subbase and base materials and ends with bituminous bound materials.

2.4.2 The subgrade soils

The subgrade soils support the pavement structure. The support strength of a subgrade on which a flexible pavement is built is the main factor controlling its design. This is so because when the subgrade deflects the overlying flexible pavement also deflects and in deed deforms to a shape, which is similar or different depending on the subgrade/pavement interaction. The fundamental design criteria is the depth of pavement thickness required to distribute the applied traffic load to the subgrade. Such a distribution of the applied load should not over-stress the pavement and cause it to deform to a greater extent than it can deform without exceeding its own structural capacity.

In order to enhance the supporting strength of the subgrade more processing than that which would have gone into the earthwork construction is generally required for the subgrade. This would be in the form of higher compaction levels with or without mechanical or chemical stabilisation of the locally selected earthwork soils. The chemical stabilisation is principally in the form of cement or lime stabilisation.

O'Reilly et al (1968) and Gichaga and Mwea (1990) showed that moisture contents at subgrade level are generally at or slightly drier than optimum moisture content. This state is true when the surfacing has not cracked to let in rain and surface runoff water. The net effect of drying out of the subgrade layers is a general increase in the subgrade strength,

O'Reilly (1974) studied the state of compaction methods in road bases and earthworks in East Africa. He found out that the states of compaction achieved by normal compaction methods in Kenya generally complied with the specifications for BS compaction test with the two and half kilogram rammer. In the field the construction is governed by moisture condition of the materials, thickness of the layer under compaction and the compacting effort of the construction and compaction plant. Gichaga (1982) studied common subgrade soils; namely red coffee soils, murram and black cotton soils. He noted that substantial loss in stiffness of the subgrade soils occurred with loading time and varied with the type of the soil. The loss in stiffness effectively reduces the load carrying capacity of the subgrade with cumulative loads.

It is therefore visible that the strength of the subgrade soils can be preserved if the pavement structure above is capable of carrying the traffic loads and thereby reducing its influence on the subgrade. This is generally achieved by an engineering combination of selecting a suitable depth of pavement and the right specification of material quality and strength. The upper layers should also prevent the ingress of the water to the subgrade. In this way the strength of the subgrade improves with the compaction effort of the traffic being beneficial. The assumption here is that the upper layers maintain their integrity.

Pavement design revolves around obtaining a pavement cover over a subgrade soil. The cover is dependent on the subgrade strength as measured by CBR. Subgrade materials in Kenya comprise of natural soils found in different geological state and conditions varying across the breadth and width of the country (Roads Department, 1987). Stiff subgrades are those with CBR values over 30%. These are found in well-drained areas where erosion has given way to rock outcrops, decomposed rock, compact gravels and boulders. Pavements constructed on these subgrades will generally be durable if they are well constructed and maintained.

Commonly available residual soils have weathered from the original rock and have CBR values ranging from 5% through 30%. Roads on these subgrades perform well subject to good-engineered pavement and good drainage, which keeps the runoff away from the pavement layers. When the subgrade strength falls below 5% then a weak foundation of the pavement results. There is then need for large-scale stabilisation and/or replacement for such subgrade soils. Table 2.1 shows the broad classification of subgrade soils in Kenya (Roads Department 1987).

Table 2.1 shows the broad classification of subgrade soils in Kenya

Type of material	CBR After 4 days Soak	CBR At OMC
Black cotton soils	2-5	15-30
Micaceous silts (decomposed rock)	2-5	7-13
Other alluvial silts (decomposed rock)	5-10	10-18
Red friable clays	7-13	15-30
Sandy clays on volcanic	7-18	15-30
Ash and pumice soils	7-18	15-30
Silty loams on gneiss and granite	10-18	15-30
Sandy clays on basement	10-18	15-30
Clayey sands on basement	10-18	15-30
Dense sands	10-18	10-30
Coastal sands	10-18	15-30
Weathered lava	10-30	10-30
Quartizitic gravels	10-30	10-30
Soft weathered tuffs	10-30	10-30
Calcareous gravels	10-30	10-30
Lateritic gravels	10-30	10-30
Coral gravels	10-30	10-30

Source: Roads Department (1987)

Table 2.2 shows the recommended modulus values for the Kenyan subgrade soils. The values shown allow for repeated loads. Table 2.2 can be idealised into Equation 2.2. This power equation enables the determination of the resilient modulus for the various subgrade soils upon determination of the CBR.

$$Mr = 3.74CBR^{1.20} \qquad [2.2]$$

Table 2.2 Resilient modulus values for subgrade soils

Soil Class	CBR (%)	Mr (kN/m^2 x 10^3)
S1	3.5	15
S2	7.5	50
S3	10.0	65
S4	14.0	90
S5	22.5	125
S6	30.0	250

Source: Roads Department (1987)

Equation 2.2 can be compared to Equations 2.3 and 2.4 developed by Henkelom and Foster (1960) and Powell et al (1984) respectively.

$$Mr = 11CBR \qquad [2.3]$$
$$Mr = 17.6CBR^{0.64} \qquad [2.4]$$

The resilient moduli that are suggested for natural subbase and base materials by Roads Department (1987) are $200x10^3$ and $300x10^3$ kN/m^2 respectively. These two values do not fit in the curves provided by the Equations 2.2 through to 2.4 when their specified CBR values of thirty and eighty are considered. The equations can therefore be considered as lower bound relationships for CBR values in the range of two to thirty. Figure 2.1 shows the graphical comparisons of the three equations. It is visible that the modulus calculated by the linear relationship (Equation 2.3) is generally higher than that calculated from the other two relationships for corresponding CBR values.

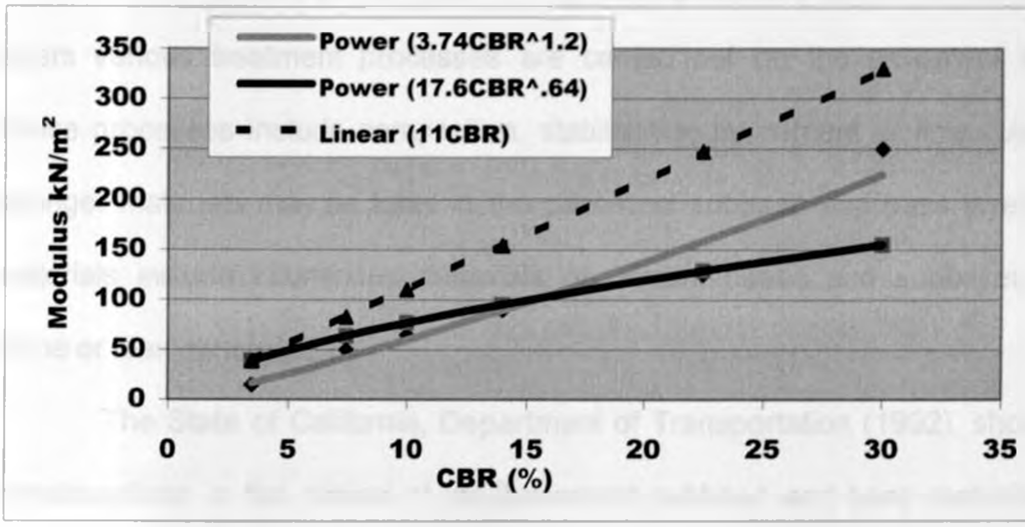


Figure 2.1 Variations of resilient modulus with CBR

2.4.3 Non bituminous subbase and base materials

The main load carrying layers of the pavement structure are the subbase and base layers. The base layers receive the traffic stresses from the wheels and consequently must safely transmit them to the lower layers. The deformation of these layers on being stressed should not lead into excessive deformation of the surfacing which would ultimately reduce the serviceability of the pavement. In addition, these layers protect the subgrade by distributing and reducing the stresses and strain to acceptable levels (Wijk, 1997).

Flexible pavement bases and subbases usually vary in thickness from one hundred millimetres to two hundred twenty five millimetres (Roads Department, 1987). The subbase layers normally consist of untreated materials. These are usually gravels, soft stones, and graded crushed stones. Depending on the traffic loading the base may be designed in untreated materials. Untreated base and subbase materials are widely used in the construction of pavements carrying light traffic.

However, in order to improve the bearing capacity of the various pavement layers various treatment processes are carried out on the pavement structures. These processes include compaction, stabilisation by cement or lime. Additionally stronger materials may be used in the pavement subbase and base layers. These materials include bituminous materials or cement bases and subbases, crushed stone or lean concrete.

The State of California, Department of Transportation (1992), shows similar considerations in the choice of the pavement subbase and base materials. Thus improved bases and subbases are generally used after comparison with the performance of untreated material. Treated materials include cement-stabilised materials, lean concrete and asphalt bound materials. These materials do not significantly differ from the Roads Department (1987) subbase and base materials. The main difference in the entire pavement structure however lies in the inclusion of a treated permeable layer in virtually all the pavement structures in the former case. The material properties, specifications and observed behaviour are now presented.

(a) Subbase and base material properties

The material grading, plasticity, and plastic modulus (the product of plasticity index and the percentage by volume passing 0.425 sieve size) categorise the gravel mechanical properties. Conducting the California bearing ratio (CBR) assesses the strength and bearing capacity. When stabilised with cement the assessment is by the unconfined compression test.

Large deformations of gravel and crushed stone layers can be attributed to insufficient strength as the modulus changes with time and load intensity. This is due to the non-linear elastic behaviour of the materials.

The grading of the aggregates, the crushing ratio, strength tests and their affinity to bitumen binder, assess the properties of aggregates for use in bituminous bound materials. The suitability of the binder is on the other hand assessed by its ability to produce an acceptable riding surface and more importantly to seal the pavement layers from the ingress of water. When used in thickness greater than fifty millimetres these layers contribute to the pavement structural strength. The properties of these bituminous bound materials are comprehensively covered in section 2.4.4 below.

(b) Subbase and base specifications

The CBR design method adopted for design by the Roads Department (1987) ensures that the pavement cover over the subgrade is adequate over the design period. The design ensures minimum structural failure of the pavement, namely cracking, rutting and also ensures that the riding quality and skid resistance do not impair the usage of the pavement. The subbase and the base should take the pavement through the design period. This period is between fifteen and twenty years. The specifications of subbase and base materials are based on the following three major factors (Wijk, 1997).

(i) *Mechanical properties*: The grading and the plasticity are limited to maximum and minimum values. The limits are in most cases dependent on observed behaviour on existing pavement sections.

(ii) *Strength of the materials*: The strength of the materials is considered in order to meet the loading expected on the pavement. The strength of the pavement is usually assessed by the CBR. In the case of graded crushed stones (GCS) the strength is assessed by the aggregate strength tests namely aggregate crushing value (ACV), and Los Angeles abrasion value (LAA). Additionally the GCS is checked for shape and flakiness by checking for the crushing ratio (CR) and flakiness index (FI) respectively.

(iii) *Durability*: - Durability is a key consideration in pavement design. For this purpose density specifications are made to ensure that the pavement layers are compacted to at least one hundred per cent BS relative compaction. The aggregate based layers have also to withstand environmental weathering. For this purpose the sodium sulphate soundness (SSS) test is carried out.

The specific specifications for the various subbase and base materials are now highlighted.

(i) *Natural materials*: - The specifications for subbase and base require that the CBR of the gravels at ninety five percent relative density (AASHTO compaction using the 4.5 kilogram hammer) and four days soak be a minimum of thirty percent for the subbase. The higher strength requirements for the base require the CBR to be raised to eighty percent. Clayey and silty sands meeting the subbase criterion may be used for subbase but not for base even if they satisfied the base criterion.

(ii) *Cement and lime improved materials*: - Cement and lime improved materials, namely, gravels, sands, and clayey sands can be used in base and subbase layers. Their enhanced strength enables them to be used for higher traffic levels than the natural materials. Stabilisation with cement is only justified when the gravel or coarse clayey sands are utilised in the base layers.

(iii) *Graded crushed stone*: - The graded crushed stone (GCS) is one of the most widely used bases in Kenya. The GCS is also utilised in the subbase where no suitable gravel can be found. The main design criterion of the GCS is an appropriate grading, typically 0/40 and 0/60 for the subbase and 0/30 and 0/40 for the base (Figure 2.2). Additionally the strength requirements shown on Table 2.3 should be met. The different classes of stones are selected in

the design process in cognition of the traffic loading. The class A stones are naturally used for the heavier sections.

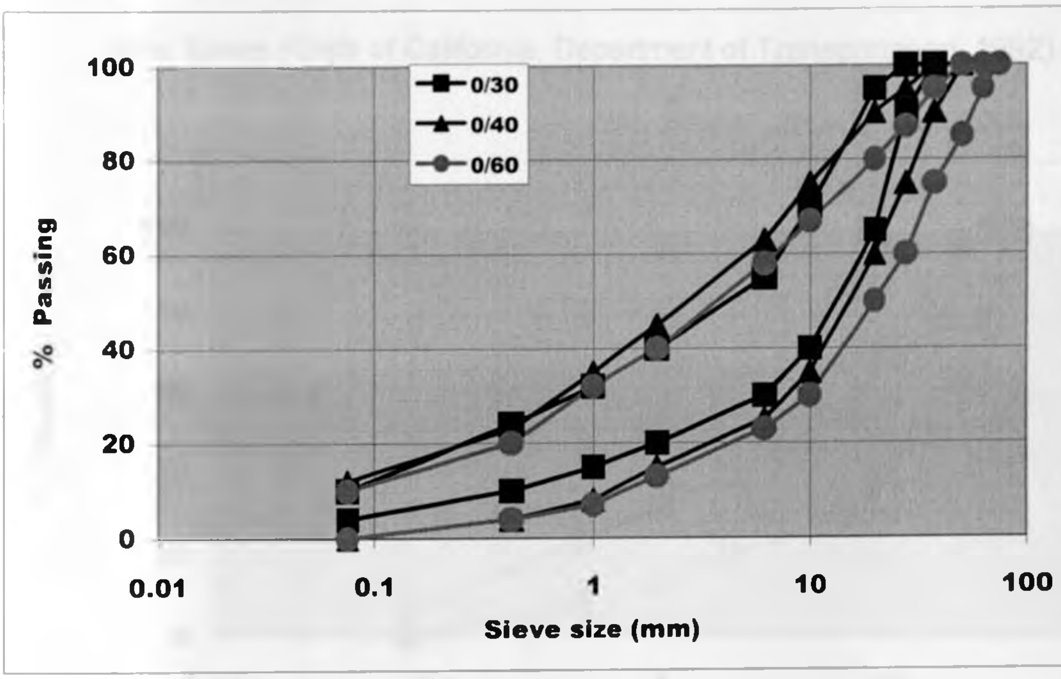


Figure 2.2 Grading envelope for the graded crushed stone courses

Source: roads department (1987)

Table 2.3 Strength requirements for GCS base and subbase layers

	Base			Subbase		
	Stone class			Stone class		
	A	B	C	A	B	C
LAA max	30	40	45	40	45	50
ACV max	25	30	32	30	32	35
SSS max	12	12	12	20	20	20
FI max	25	30	30	35	35	35
CR min	100	80	60	30	30	-

Source: Roads Department (1987)

(iii) *Lean concrete:* - The heavier traffic bases require the use of dense bitumen macadam (discussed in section 2.4.4) and lean concrete. The grading for coarse aggregate used in the lean concrete is shown on Figure 2.3. The strength requirements for the coarse aggregate are shown on Table 2.4. The low cement contents of three to six percent would suggest that the cost of the

lean concrete is significantly less than that of ordinary concrete. The cost benefit analysis of this type of pavement should be worked on a project by project basis since their maintenance is considerably less than that of the other bases (State of California, Department of Transportation, 1992).

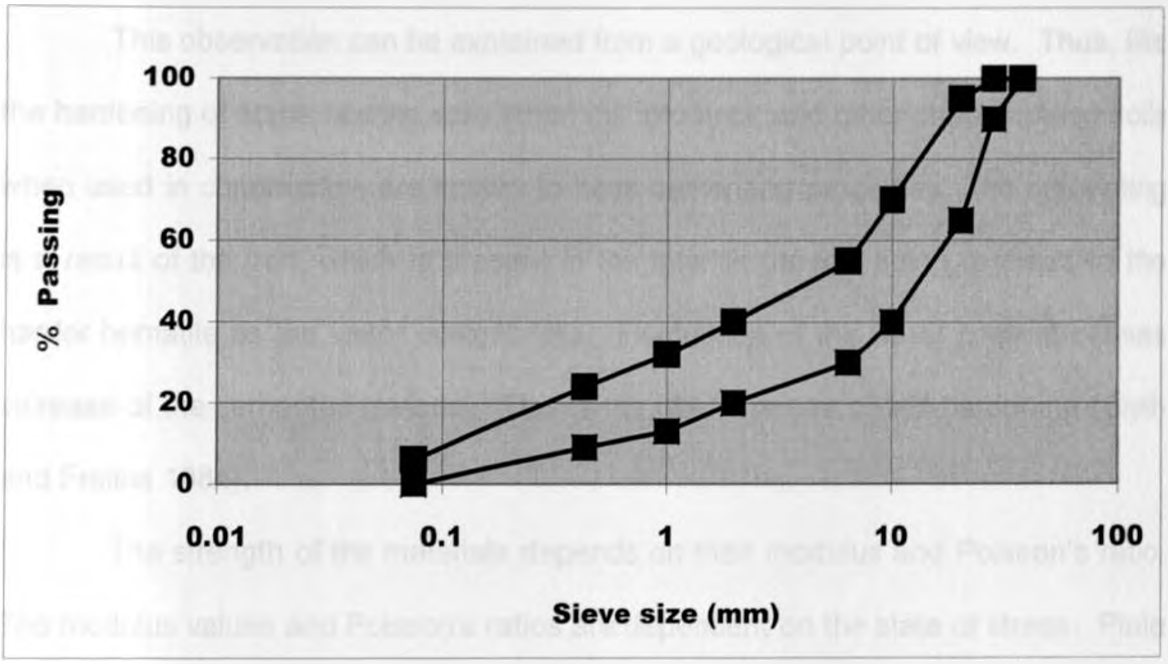


Figure 2.3 Course aggregate grading envelope for lean concrete

Source: Roads Department (1987)

Table 2.4 Strength requirements for the coarse aggregates used in lean concrete

Crushing Ratio (%) min	80
Flakiness Index (%) max	25
Loss Angeles Abrasion Value (%) max	35
Aggregate Crushing Value (%) max	28

Source: Roads Department (1987)

(c) Observed subbase and base behaviour

Satisfactory behaviour for lateritic gravel bases beyond theoretical predictions based on grading, plasticity and deflection have been recorded in Kenya and Malawi (Grace and Toll 1987) in Australia (Cocks and Hamony 1988) and Brazil (Queiroz et al, 1991). While it is difficult to make generalised conclusion for the above reported performance, two definite observations are common. The first is that in the above cases the bases were compacted at OMC and allowed to dry back. The second observation was that there was evidence of self-hardening occurring in the materials. This results in increase of strength for materials, which were otherwise mechanically unstable and too plastic.

This observation can be explained from a geological point of view. Thus, like the hardening of some lateritic soils when cut into brick and other shapes these soils when used in construction are known to have cementing properties. The cementing is a result of the iron, which is present in the lateritic gravels being oxidised to the harder hematite as the water content falls. Fluctuation of the water content causes increase of the cemented material. This is the phenomenon of self-hardening (Blyth and Freitas 1984).

The strength of the materials depends on their modulus and Poisson's ratio. The modulus values and Poisson's ratios are dependent on the state of stress. Plate loading test by Gichaga (1979) on saturated bases confirmed a non-linear relationship of stresses and strains.

To improve the strength of granular soils, cement improvement is widely used for subbase and base materials. The soil cement is however weaker in tension than in compression. Hence, they are prone to cracking under tensile loads as shown by Bofinger (1970). However, Jones and Smith (1980a) in their study of pavement performance in Kenya showed that despite the loss of tensile strength of a stabilised road base routine maintenance ensured satisfactory performance. In some of the test sections studied by Jones and Smith (1980a), in-situ CBR's were in excess of one hundred per cent and little cracking of the surfacing despite very low values of tensile strength of the cement stabilised bases. The significance of maintenance is evidenced from this observation in that the maintained surface does not lose the supporting ability of traffic loads which is attributed to the shearing strength of the material. The shear strength theory discussed for embankment materials (Section 2.2) also applies for the bases in either improved condition or not. In particular the application of CBR for the assessment of strength is also the same as that for subbase and base materials. In deed it is the magnitude of CBR, which increases for the subbase and base layers

The loss in tensile strength would result in the drop of performance when the surfacing has broken up to such an extent that environmental effects of weather affect the stabilised layers. It could also occur in extreme cases of a completely broken up surfacing when the traffic loads are applied directly on the stabilised pavement layers. This is a case where maintenance of the pavement is completely absent and the once paved roads are turned into barely motorable tracks!

Lime improvement is alternatively used on the cohesive soils (Roads Department, 1987). The lime content required to improve the physical properties of soil (reduction of plasticity) varies between three and ten per cent. The strength of lime stabilised soils is measured by the unconfined compression test or the CBR test. Like the cement-improved material discussed above the lime-improved materials increase in both compressive and tensile strength. The compressive strength increase is attributed to the formation of cementitious products and the effect of aggregation, which results in greater interlocking and rougher surfaces (Sastry and Gobal 1990)

(d) Resilient modulus and Poisson's ratio

Determination of resilient modulus and Poisson's ratio of non-bituminous subbase and base materials require accurate simulation of traffic and field conditions. This means simulation of stress and time variations. Cyclic triaxial tests are done to simulate these field conditions. Under the testing, the stresses due to the dead weight of the upper layers are simulated by applying a constant confining stress σ_3. The cyclic vertical stress σ_d consists of a vertical stress σ_s and a cyclic part σ_c to model out repeated axial stress from traffic (Brown and Hyder, 1975). Figure 2.4 shows the stress conditions of the triaxial specimens. This loading can be modelled out in a large triaxial cell, which, can take large specimens usually of the order of 400-mm diameter, by 800-mm length.

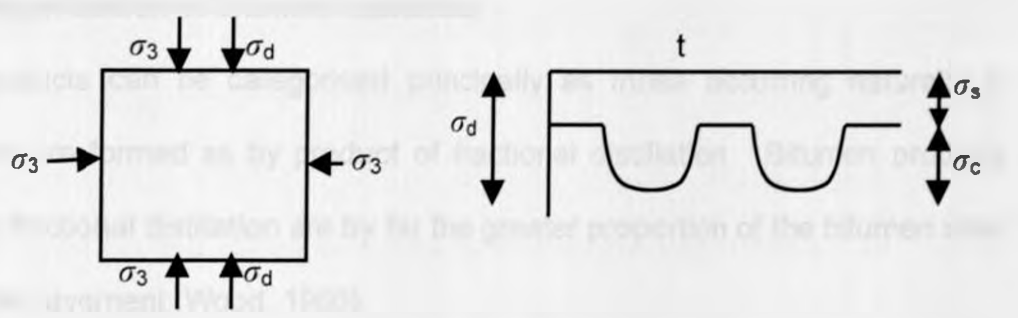


Figure 2.4 the stress condition in a cyclic triaxial specimen

Application of cyclic load to the triaxial specimen requires sophisticated pneumatic or hydraulic loading equipment and electronic devices for monitoring the cyclic specimen deformation (Muraya, 2000). Displacement transducers mounted on the specimen measure the axial and radial deformation. The resilient modulus is then calculated as the quotient of the applied axial deviator stress and the measured strain (Equation 2.5).. The resilient modulus is applied in calculations with a constant value of Poisson's ratio between 0.25 and 0.35 to predict deformations (Opiyo, 1995).

$$Mr = \sigma_d \div \varepsilon \qquad [2.5]$$

Where: ***Mr*** is the resilient modulus

σ_d is the cyclic vertical stress

ε is the averages axial resilient the strain

2.4.4 Bituminous bound materials

The bituminous bound materials literature survey is presented below. The survey attempts to bring to the fore aspects of categorisation distillation, chemical composition and properties of bituminous mixtures. The use of the words asphalt and bitumen or asphaltic and bituminous has been done interchangeably since they refer to the same product.

(a) Categorisation of bitumen materials

Bitumen products can be categorised principally as those occurring naturally or those, which are formed as by product of fractional distillation. Bitumen products obtained by fractional distillation are by far the greater proportion of the bitumen used in the flexible pavement (Wood, 1960).

Natural bitumen is found in many forms and states. Some materials like asphaltine exist almost entirely as hydrocarbon products with little or no mineral matter. Others like the lake and rock asphalt are associated with appreciable quantities of mineral matter (Broome, 1965). Natural bitumen may be found on the surface of the earth, while others have to be mined at depth. Most of the bitumen used in this country comes from the Mombasa oil refinery. However, with the liberalisation of the energy sector the market has had to do with directly imported bitumen.

(b) Fractional distillation of bitumen

Petroleum crude from different sources yield varying percentages of bitumen. The bitumen recovered from various crudes will also vary in chemical composition. The different crudes generally require different refining conditions. The different refining conditions are in most cases dictated by the boiling characteristics of the oils present and those to be removed (Corbett, 1965 and Asphalt Institute, 1970).

The distillation of the petroleum crudes involves vaporising the crude above its boiling point. The vaporised material is subsequently condensed to liquids by cooling back to normal temperatures. The various hydrocarbon materials are separated out at different temperatures depending on the material boiling points. The residue is normally straight run bitumen.

In the event that the crude contains constituents of high boiling temperatures the fractionation is done under reduced pressure and steam injection in the fractionating column. The residue bitumen products are generally of different grades and further processing includes air-blowing, blending, compounding and admixing with other ingredients to make a variety of asphalt products used in paving, roofing, waterproofing, coating and sealing materials for industrial applications.

The air blowing process is a major process in which soft bitumen residues are heated to high temperatures in an oxidation tower where air is forced through the residue. The process results in dehydrogenation and polymerisation of the residue. This process is generally controlled to limit the residue temperatures so that the desired characteristics of the product are achieved. It is normal to blend two or more residues to produce a material of the desired properties (Asphalt Institute, 1970).

(c) Bitumen types used for pavement structures.

The bitumen used for pavement construction can be divided into two fundamental types. Firstly the penetration grade bitumen which is heated to varying temperatures before mixing with hot or warm aggregates (O'Flaherty, 1974 and Roads Department, 1987). Secondly when it is inconvenient or uneconomical to handle hot asphalt it is possible to use liquid binders. These liquid binders are prepared by emulsifying the asphalt cement in an aqueous medium called bitumen emulsion. Selected petroleum solvents are used to cutback solid and semi-solid asphalt materials (Asphalt Institute, 1970).

Cutback bitumen are grouped into three types depending on the volatility of the solvents used to reduce the viscosity of the penetration grades, namely: - rapid curing (RC), medium curing (MC) and slow curing (SC). Each type of cutback bitumen is subdivided into several grades characterised by their viscosity. The viscosity is controlled by the quantity of the cutback solvent applied on the penetration grade bitumen. Alternatively an emulsified bitumen may be prepared by dispersing very small drops of asphalt particles in water in the presence of an emulsifying agent to promote the dispersal and stability of the bitumen water mixture. These types of bitumen are called anionic and cationic emulsions. The emulsion breaks when sprayed or mixed with mineral aggregates in the field construction process. The water is subsequently removed and asphalt remains as a film on the surface of the aggregates.

(d) Specifications for bitumen used for flexible pavements in Kenya

The Roads Department (1986), specifies a wide range of penetration grade bitumen, cutback bitumen and bitumen emulsions. However, the most commonly used road binders in Kenya are 80/100-penetration bitumen, cutback MC 3000, cationic emulsion K1-60 and K1-70 (Roads Department, 1987).

The straight run bitumen are required to comply with the specifications shown on Table 2.3. Further, the rapid curing cutbacks are required to comply with the requirements of AASHTO standard specification M81 (ASTM D 20 28) while the medium curing cutbacks should comply with AASHTO standard specification M82 (ASTM D 20 27). The MC 3000 in the Kenyan case is 80/100-penetration bitumen blended with 15% diesel or 14% kerosene. The cationic emulsions are required to comply with the requirements of BS 434. The K1-60 and K1-70 are prepared by emulsifying 80/100-penetration bitumen with amine based emulsifier mixed with water. The percentage of bitumen in the two emulsions is 60% and 70% respectively. The percentage of the emulsifier is usually 1% of the total mix.

Table 2.3 Specification for straight run bitumen in Kenya

Penetration grade	20/30	40/50	60/70	80/100	180/200
Penetration (0.1mm)	20-30	40-50	60-70	80-100	180-200
Softening point (°C)	59-69	52-60	48-56	45-52	37-43
Fp (°C)	250	250	250	225	200
Minimum ductility (cm)	30	100	100	100	100
Max. loss on heating (%)	0.2	0.2	0.2	0.5	0.5
Min. pen. of residue (%)	80	80	80	80	80
SG at 25 °C	1.02-1.07	1.01-1.06	1.-1.06	1.-1.05	1.-1.05
Minimum solubility (%)	99	99	99	99	99

Legend: Fp - Flash point (Cleveland point), Loss on heating - after 5 hrs at 163°C, - Pen. of residue - % of initial penetration, SG - Specific gravity, Solubility - in carbon tetrachloride.
Source: Roads department (1986)

(e) Chemical composition of bitumen

The physical properties of bitumen are closely related to their chemical composition. Elemental analysis of bitumen from various sources by Peterson (1984) shows that besides the major carbon and hydrogen compounds accounting for over ninety per cent, minor components of principally nitrogen, sulphur, oxygen, vanadium and nickel exist in bitumen. It is to be noted that the minor components make disproportionate large contribution to the difference in physical properties among bitumen from different sources. This is primarily so due to the atoms of the minor components imparting functionality and polarity to asphalt molecules.

For engineering purposes it has been found satisfactory to study bitumen composition by separation (Rathor and Ramaswany, 1987). Earlier on Peterson (1984) had shown that the separation techniques led to homogenous fractions. Selective solvents, chromatography and chemical precipitation affect fractionating. Use of this method has led to the isolation of the asphaltene fraction as the component of bitumen, which is primarily responsible for asphalt viscosity and colloidal behaviour. The asphaltenes are kept dispersed by the peptising ability of the nitrogen bases. The peptised asphaltenes are in turn dispersed by the resinous acidaffin fractions and gelled by the paraffins.

The fluidity of the asphalt is imparted by saturate and naphthalene aromatics and asphaltene fractions. The polar aromatic fraction imparts ductility to the asphaltenes while the saturates and naphthalene aromatics in combination with the asphaltenes produce flow properties in the asphalt.

The variation of chemical composition depends on the crude source and the manufacturing process. This can be seen in Table 2.5, which presents chemical composition of bitumen from Venezuela, USA, Mexico and the Middle East. Table 2.6 presents chemical composition of bitumen produced by different levels of distillation.

Table 2.5 Effect of crude source on chemical composition

Source	Pen.	Sp °F	Sat. %	Na %	Pa %	Asp. %
Venezuela	90	114	14.0	34.5	36.3	14.1
USA	92	114	10.5	38.5	33.4	16.8
Mexico	88	116	8.5	29.6	42.6	18.3
Mid East	85	116	8.0	38.5	37.0	15.5

Legend: Pen - Penetration at 77°F, Sp - Softening point, Sat – Saturates, Na - Naphthalene aromatics, Pa - Polar aromatics, Asp – Asphaltenes.

Source: Corbett (1969)

Table 2.6 Effect of Vacuum reduction on chemical composition

Level of Distillation	Pen.	Sp °F	Sat. %	Na %	Pa %	Asp. %
Flux	300+	83	16.7	41.4	29.8	11.5
Binder	89	114	13.5	34.7	37.4	14.6
Pitch	5	184	3.7	27.2	45.8	23.0

Legend: Pen - Penetration at 77°F, Sp - softening point, Sal – Saturates, Na - Naphthalene aromatics, Pa - Polar aromatics, Asp – Asphaltenes.

Source: Corbett (1969)

The flow properties of bitumen have been recognised for a long time. Traxler (1961) divided bitumen on the basis of their flow properties. He described the bitumen as sole type where the system contains sufficient resins or protective colloids to fully peptise the heavy asphaltenes. He referred to them as gel type when their system lacked enough stabilising agent to disperse the asphaltenes. Well-dispersed asphalt is susceptible to temperature change, ductile and exhibits low rate of age hardening on the other hand an asphalt in which the asphaltenes are not well dispersed possesses low susceptibility to temperature change low ductility and rapid age hardening. Bitumen possessing intermediate properties is described as sole–gel bitumen

(f) Properties of bituminous mixtures

Bituminous mixtures are composed of mineral aggregates, binder and air voids. The aggregates are generally divided into three groups based on their size, namely coarse, fine and filler. These mixtures are widely used in the surfacing and bases of paved roads. The performance of these materials is critical in that they affect both the structural strength of the pavement structure and rideability of the road. The behaviour of the mixture is however complicated by the change of the physical and chemical state of the bitumen with environmental change.

The strength of the bituminous mixtures decreases during hot periods. This is a direct result of the decreased viscosity of the bitumen binder. The decreased strength causes a loaded bitumen binder to undergo varying distortion. Increased loads are transferred to the lower pavement and subgrade layers. During the cold periods the materials tend to increase in strength due to the increased viscosity. The bituminous materials consequently tend to carry larger proportion of load. However, the materials in this state are brittle and can fracture and disintegrate under traffic loading. This behaviour of bituminous materials is a manifestation of an elastic and visco-elastic material. Pell and Brown (1972) and Gichaga (1979) evaluated various methods of analysis and concluded that classical theory of elasticity provides a practical approach in the study of flexible pavement behaviour.

In either elastic or visco elastic state the exhaustion of the pavement load bearing capacity is the result of their magnitude, repeatability, resilient strength, environmental and climatic factors. The climatic factors in the Kenyan case are principally rainfall and temperature.

Motumah (1973) found that the pavement temperatures vary between zero and fifty degrees centigrade in Kenya. From the foregoing discussion of changes in properties of bituminous mixtures with temperature, it is visible that the temperature effect on the pavement performance is large and contributes significantly to pavement deterioration. An understanding of the effects of temperature on bituminous mixtures is therefore required to enable rational design and construction of the bitumen bound surfacing and bases.

The stiffness of bitumen and aggregate mixtures is of paramount importance in determining how well a pavement performs and is a fundamental and essential property for the analysis of pavement response traffic loading (Sousa et al 1991). The fundamental properties measuring stiffness namely Young's modulus and Poisson's ratio need accurate and simple methods of determination. Whereas the behaviour of bitumen will vary under different conditions of stressing from viscous to elastic, the aggregate in the mix behaves elastically under all conditions. The grading and interlock make the behaviour of the combined bitumen and aggregate complex.

The strength of the pavement materials at static or near static loading depend on the grading of the pavement particles, inter-particle friction and binder cohesion. The grading influences flexibility, workability and durability of bituminous mixtures. Their mechanical strength must satisfy performance requirements of pavement structure, surface texture, and particle shape resistance to polishing and skidding. Indeed with any hot-mix asphalt pavement the applied loads are ultimately carried by the aggregates. Consequently in addition to meeting minimum aggregate quality requirements the proper aggregate grading must be used if the hot-mix asphalt pavement is to perform satisfactorily in service (Lynn et al, 1999).

The deterioration of the bituminous bound pavement materials is linked to the stresses and strains. Excessive stresses result in micro cracking of the materials (Ullidtz and Stubstand, 1990). These micro cracks reduce the supporting area of the pavement and subsequently increase the stresses of the remaining supporting pavement materials. This causes further increase in micro-cracks. The micro-cracks eventually increase to form macro-cracks, which are manifested in ruts and/or roughness on the pavement surface

The stiffness modulus of a bituminous mix is dependent on the stiffness of the bitumen binder and the volume ratios of the different mix components comprising of aggregates, binder, and air (Harvey and Monsmith, 1993). The stiffness of asphalt is thus sensitive to binder type, air void content, and mixing viscosity. It can therefore be concluded that in the design of an asphalt mix, the stiffness of the mix should be considered. In addition, as the bituminous mixtures are subjected to load repetition from traffic they must be resilient and possess good fatigue response to load.

The diametral test which involves loading a cylindrical specimen by a strip loading has been used for the determination of the stress state, the modulus and Poisson's ratio of bituminous mixtures (Adedimila, 1980; Tangella et al 1990 and Sousa et al, 1991). The test is also known as indirect test. Adedimila (1980) derived Equations 2.5 through 2.7 for the 102-mm diameter specimens (typical of the Marshal specimens).

$E_x = LDH\,(2647+9807\mu)/\,t$ **[2.5]**

$E_y = LDV\,(35183\text{-}615\mu)/\,t$ **[2.6]**

$St = 60.35Pf/\,t$ **[2.7]**

Where: E_x is the tensile modulus (kN/m^2)

E_y is the compressive modulus (kN/m^2)

St is the tensile strength (kN/m^2)

Pf is the load at failure (kg)

t is the thickness of the specimen in mm

LDV is the average slope of the load in kg against horizontal deformation

LDH is the average slope of the load in kg against vertical deformation

μ is the Poisson's ratio

Adedimila (1980) also proposed Equations 2.8 and 2.9 for the determination of the compressive modulus and tensile strength from the Marshal stiffness for the wearing course. The Marshal stiffness is the ratio of Marshal stability to flow in routine Marshal testing.

$E_y = 7751 + 350MSE$ **[2.8]**

$St = 97 + 1.13MSE$ **[2.9]**

Where: ***MSE*** is the Marshal stiffness in kg/mm

Tangella et al (1990) considered the test simple to conduct and can be considered to be an effective way of characterising bituminous materials in terms of their fundamental properties. He enumerated the following as the advantages of the test.

(i) The test is simple

(ii) Design of mixtures and pavements for fatigue is possible.

(iii) The equipment is applicable for other tests for example resilient modulus and tensile strength.

(iv) Failure is initiated in a region of relatively uniform tensile stress.

(v) A biaxial state of stress exists. It approximately represents the field stress conditions under traffic.

(vi) The test can be also performed on field core samples

The diametral testing equipment was not available for this project. To complete the analysis of Marshal test data obtained during the data collection (Chapter Three) it was necessary to estimate the modulus and the tensile strength of the test specimens. For this purpose the relationships presented in Equations 2.8 and 2.9 were utilised.

(g) Resilient modulus and Poisson's ratio for bituminous materials

The determination of resilient modulus for the bituminous mixtures is done by use of repeated loading on diametrical specimens (ASTM 1987, Sousa et al 1991 and Aire and Fyve 1994). The repeated loading models out traffic loading. Equations 2.10 through 2.12 are used for the estimation of the resilient modulus, tensile strength and Poisson's ratio for repeated loading on the diametrical specimens.

$$Mr = P(\mu + 0.27)/Ht \qquad [2.10]$$

$$St = 2P/\pi Dt \qquad [2.11]$$

$$\mu = 3.59H/V - 0.27 \qquad [2.12]$$

Where: ***Mr*** is the resilient modulus in kN/m^2

P is the Repeated load in kN

H is the Total horizontal deflection of the specimen in mm

D is the diameter of the sample on millimetre

t is the specimen thickness in mm

μ is the Poisson's ratio

V is the vertical deflection in mm

2.4.5 Effects of changes in chemical composition of bitumen

The previous sections of literature survey have dwelt on the physical and strength properties of the flexible pavement structure. This section deals with the chemical change of bitumen and its effect on the performance of the pavement structure.

The difference in chemical composition of various types of bitumen due to their processing and relationships have been recognised. Durability is determined by the physical properties of the bitumen which in turn are determined directly by the chemical composition. An understanding of changes in chemical composition on ageing is thus fundamental to an understanding if bitumen durability.

An ideally durable bitumen is one which possesses suitable chemical and physical properties to produce a pavement with acceptable performance and resistant to changes during environmental ageing in the field service (Bezabeh, 1992). However, bitumen composition changes with time due to oxidation, polymerisation and volatilisation when weathering factors act on it. As a result it hardens and stiffens with time. Wright (1965) cited the following chemical changes during both accelerated and natural weathering:

(i) The oil component produced asphaltenes, water-soluble products and volatile products.

(ii) The asphaltene content increased due to oxidation of the oils rather than polymerisation of the oils.

(iii) The bitumen lost weight due to the loss of water-soluble products, volatile products and ethanol soluble materials.

These chemical changes are closely associated with the hardening of bitumen binders. Gotoloski et al (1971), found that changes in the percentage of asphaltene fraction correlate with changes in penetration and absolute viscosity. Thus, bitumen hardened with time based on physical conditions and asphaltene content. They pointed out that the air void volume in a bituminous mixture is a major factor influencing pavement durability.

Vallerga and Halstead (1971) showed that the increase in asphaltene and decrease on acidifies were generally related to the air voids content of the mixture. They quantified the effect of air voids by showing that in pavements of below two per cent air voids field ageing during eleven to thirteen years of service was negligible. Above the two per cent hardening increased with higher air voids. It is to be noted that not only are higher air voids responsible for excessive hardening of the bitumen, they also make stripping defined as weakening and loss of adhesive bond more severe. This is a result of water finding its way in the air voids (Maupin, 1999).

Lee (1973) used the change in asphaltene content as an important parameter in his study of asphalt durability. He observed that: -

(i) Ageing is accompanied by increase in asphaltene content.

(ii) The change in asphaltene content is a hyperbolic function of time.

(iii) The rates of asphaltene formation are different for various types of asphalt.

He also showed differences resulting from voids content during pavements by establishing time-equivalency correlation curves for different voids levels. The relative positions indicated that as percentage of voids decreases, longer laboratory ageing time is required to reach equivalent field service hardening.

Bitumen binders in service for eighteen years were sampled and analysed by Corbett and Merz (1975). The analysis of chemical changes indicated that the liquid component (naphthene aromatics) decrease and the solid components (polar aromatics and asphaltenes) increase due to ageing in a Michigan test road. The percentage increase of asphaltene content was consistent in the top three layers and the next six millimetres layer. However the increase of asphaltene in the top layer was more than double the increase of asphaltene in the next layer. There was little distinction among the binder sources in ageing resistance in the bottom layer. It was further noted that the two binders which had the highest asphaltene contents before and after ageing showed greater change in physical properties and more wear and weathering. Green et al (1976) found the contribution of asphaltene and second acidaffin fractions to be the most important in asphalt performance. Since asphaltenes are the solid component, an increase in asphaltenes results in harder asphalt.

Rathor and Ramaswamy (1987) studied the chemical changes upon ageing of bitumen binder under Singapore conditions. They found out that asphaltene ratio increases with time, while the aromatics ratio decreases correspondingly. The drop in penetration was correlated with the increase in the formation of asphaltene compounds as the bitumen ages. The aromatics and polars are components of bitumen, which provide workability as they enable the asphaltenes and saturates to mix. The loss in ductility is thus caused by decrease in both aromatics and polars.

2.4.6 Changes in physical properties of bitumen on ageing

Cracking, disintegration and abrasion manifest the deterioration of flexible pavements early in their service life. This is a direct result of loss in penetration, ductility with increase in softening point and viscosity. It has been found that when the penetration of bitumen reaches values of twenty to thirty, the pavement will show indications of cracking and will have approached the end of its useful life (Bright and Reynolds, 1962). The decrease in penetration, which is a measure of hardness starts in the hot mixing operation of the asphalt concrete. Hatherty and Leaver (1967) suggested that the mixing time should be the minimum required to give uniform coating of the aggregate particles.

Kenis (1962) studied experimental pavements under Delaware (USA) conditions. In the experiment two asphalt of 60/70-penetration grade from Venezuela and Middle East were tested. Samples were taken during hot mixing, after one and two years of service. The physical characteristics are presented in Figure 2.5. The penetration of bitumen decreased by over 40% immediately after construction principally due to oxidation and volatilisation. In the forthcoming period the penetration continued to decrease while the softening point increased.

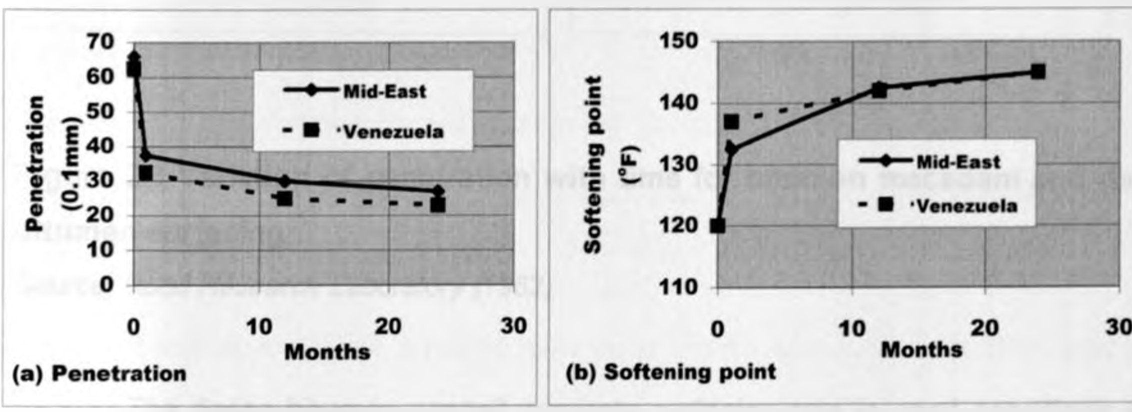


Figure 2.5 Asphalt characteristics versus time of ageing
Source: Kenis (1962)

The Road Research Laboratory (1962) reported similar tests. The penetration values plotted versus ageing time are shown in Figure 2.6. The figure shows considerable reduction in penetration for the bitumen macadam, which was basically, penetration asphalt allowing air and water to penetrate. The bitumen under test included refinery bitumen and Trinidad Lake asphalt. In subsequent monitoring the carpets made with Venezuelan, Mexican and Californian bitumen were all in fairly good conditions after eighteen years of service. The Trinidad Lake asphalt showed some disintegration signs after ten years and subsequently failed after fourteen years.

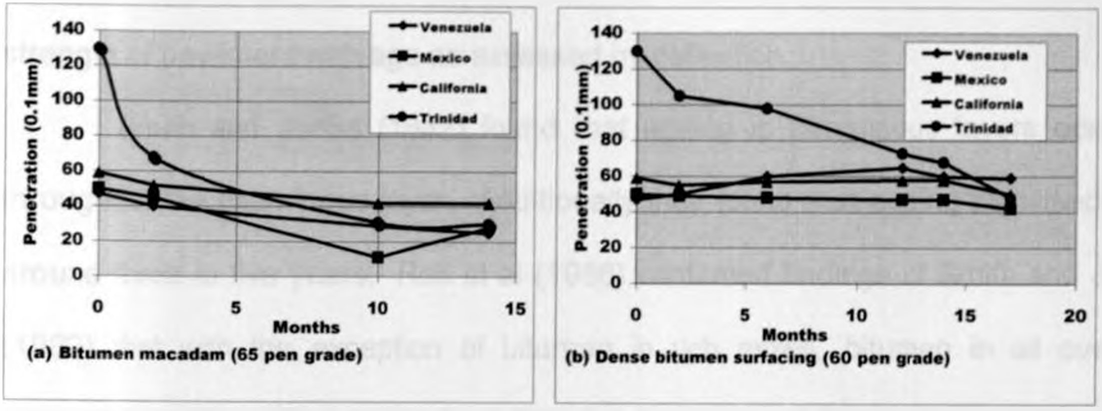


Figure 2.6 Variation of penetration with time for bitumen macadam and dense bitumen surfacing

Source: Road Research Laboratory (1962)

The dense bitumen asphalt concrete surfacing was in good conditions after seventeen years. The penetration values had remained almost unchanged over the years of service. The good performance for these asphalt can be attributed to the impermeability of the dense bitumen mixture. The heavy traffic improved the performance by further compaction while in service. Subsequently the agents of weathering, air and water did not have an effect on the surfacing.

2.4.7 Ageing of bitumen under Kenyan environments

Hall (1979) found out that 80/100-penetration grade bitumen, which is commonly used in Kenya, undergoes substantial hardening during the first eighteen months under traffic. Thereafter, there is a period of more hardening at a reduced rate until stable conditions are reached at around three years after laying.

Ageing of bitumen under Kenyan conditions was studied by Jones and Smith (1980b). They showed that ageing of bitumen influenced the amount of plastic deformation within the bituminous bound bases and surfacing. The deformations were found to be high for new surfacing but low for aged ones. This investigation reconfirmed findings by Gichaga (1979) that suggested that there is increase in strength of pavement with age as assessed by deflection.

Smith and Jones (1982) found that ageing in bituminous layers occurred throughout the bituminous layer. Additionally they found that ageing stabilised after around three to five years. Rolt et al (1986) confirmed findings of Smith and Jones (1982) that with the exception of bitumen in rich mixes, bitumen in all overlays hardened rapidly. They pointed out that the predominant form of deterioration, and cracking was correlated to the ageing of bitumen. Ageing was found to occur throughout the depth of the overlays, but was extremely severe in the top few millimetres.

Smith et al (1990) reported extensive investigations on laboratory and field performance of three grades of bitumen, namely 80/100, 60/70, 40/50 penetration bitumen. Hardening was measured by viscosity test on recovered bitumen samples. Exposure time was found to be the dominant factor. It was found out that the mix properties had little effect on ageing of bitumen. Sealing and surface dressing of asphalt concrete surfacing was found to be a very effective way of reducing bitumen hardening. Thus within two years service the sealed road sections were in good conditions. While many of the unsealed sections built in asphalt concrete surfacing developed cracks at the surface of the pavement structure. Increasing bitumen content to reduce the rate of hardening resulted in extensive deformation, poor surface texture and yet little reduction in the rate of hardening.

Experience with bituminous surfaced roads in Kenya indicates a deterioration of bitumen surfacing as manifested mainly by early cracking at their service life (Mwea and Bezabeh, 1994). The typical failure pattern is that environmentally induced hardening of binder in surfacing results in brittle skin, which is prone to cracking under stress of traffic and temperature. Water finds its way through the cracks and weakens the lower pavement layers.

2.5 Pavement performance

2.5.1 General.

The previous sections of literature review have dealt with the materials that are used in the construction of pavements. Once a pavement has been constructed then it has to withstand traffic loading under environmental conditions specific to the road section. This section will review how the performance of pavement is evaluated during its service life.

Presently, the designer is required to estimate the traffic loading in terms of cumulative standard axles (8200kg). The design of the pavement then requires that the road alignment soils are evaluated to give their CBR values. Subsequently the subgrade classification is done. Based on the available materials, which meet design specifications, a pavement structure is selected from a catalogue of fourteen structures (Roads Department 1987). The resulting pavement thickness prevents excessive deformations (Roads Departments 1989). The performance of a pavement is evaluated on the basis of its ability to carry over a period of time the designed traffic volume. Poor performance is often characterised by the formation of distress parameters on the pavement. The common distress parameters include potholes, raveling, cracks, ruts, corrugations and distortions. However, pavements in seldom fail suddenly (Visser et al, 1989). They may continue to carry traffic for a long time after they have become severely damaged and attract attention only once normal maintenance is no longer effective.

Different categories of pavement perform differently under their service loads. The ability of the subgrade soils to carry their loads is the single most crucial factor in determining the life of the selected pavements. In addition, Maree and Visser (1994) revealed that the performance of low volume roads depend to a large extent on the quality and strength of pavement materials.

The objective of pavement evaluation is therefore to carry out the following tasks as suggested by Visser et al (1989):

(i) Identify the pavement characteristics by determining functional class, length and width, surfacing, traffic and climate.

(ii) Make a functional assessment of the pavement by determining, riding quality, skid resistance and surface drainage.

(iii) Assess the ability of the pavement structure to resist traffic and environmental forces. This is done in consideration of:-

- Defects permitting the ingress of water.
- Defects indicating inadequate structural strength.
- Defects indicating loss of surfacing integrity, such as stone loss and surfacing patches.

Pavement evaluation may follow non-destructive or destructive methods. In the non-destructive methods, the integrity of the pavement is not affected while during the destructive procedures, pavement layers are dug up in order to obtain access to various layers. After exposing the various layers in-situ test may be carried out or alternatively samples are recovered for laboratory testing or both.

2.5.2 Pavement characteristics

The functional class identifies the class of road within a road classification system. In Kenya, the road classification system includes;-

(i) Class A – International trunk roads

(ii) Class B – National trunk roads

(iii) Class C – Primary roads

(iv) Class D – Secondary roads

(v) Class E – Minor roads

In addition to the above-classified network, there are several non-classified roads comprising roads to government institutions, rural access roads, tea, sugar and other minor rural roads.

The pavement characteristics include the length, width and the type of surfacing. The length and width measurements are intended to quantify the scope of works. The surfacing type is particularly valuable in that, different types of surface, call for different maintenance needs. The different types of surfacing commonly used in Kenya include asphalt concrete, single and double surface dressing (Roads Department 1987).

In addition to the above characteristics traffic and climate assessment help to complete the pavement characteristics in an evaluation exercise. The traffic estimation is utilised in the assessment of the traffic already carried by the pavement. The remaining capacity at the time of evaluation can then be estimated. The climate assessment helps in the environmental evaluation, which allows complete understanding of the pavement condition.

2.5.3 Functional assessment

(a) Present service rating

The riding quality and the condition of the surface of the pavement is a key factor in pavement evaluation. The functional assessment is evaluated by assigning a Present Service Rating (PSR) of a section of the road together with conditional evaluation quantified by measurements of physical distress features. The PSR is attributed to a single rater.

The riding quality influences the user opinion, which ultimately affects the quality of the ride. The user opinions are generally correlated with measurements of pavement roughness, patching, rutting and cracking.

(b) Present serviceability index

The quantitative measure of opinions was established by a concept developed during the AASHO road test (Carey and Irick, 1960). The concept revolves around the determination of present serviceability index (PSI). PSI is simply the mean PSR of the raters for a test section. The PSI, which quantifies the subjective user opinions, can then be correlated with the objective measurements of pavement roughness, patching, rutting and cracking.

The general consideration in the earlier design of pavements gave little consideration to the level of service (Janoff, 1982). The practice in Kenya has not been taking rideability into account. However, the Roads Department (1988a) introduced the PSI concept in the evaluation and design of rehabilitation and improvement works for existing pavements.

The objective of present serviceability index is to devise a system where the end users contribute to the improvement of the pavement surface. This is made possible because serviceability is a psychological quantity and/or experience which is quantifiable (Janoff, 1982). The main reason for conducting PSI exercise is that, it improves pavement evaluation and subsequent design of strengthening and improvements of existing pavements. It is assumed that:

(i) The opinion of the users influences the prioritisation of the sections to be maintained and maintenance programs.

(ii) A relationship between objective pavement measurements and subjective ratings by the users can be developed.

(iii) Measurements at various times during the life of pavement can be made for historical performance.

(iv) Parameters for defining pavement condition can be developed.

The concept removes bias as a result of individuality when evaluating pavement conditions. The usual practice is to use Adjective Comparison Scale (ACS) to obtain the psychological data from the raters. The ACS rating employs an adjective comparison from 0-5 with word cues very poor, poor, fair, good and very good respectively. Each analysis yields a single mean rating value for each rated section (Roads Department, 1988a). The mean PSR for the section is the PSI for that section. A Present Serviceability Index (PSI) of 2.5 for trunk roads and 2.0 for secondary roads is generally acceptable.

Statistical analysis of a rated section is required to relate the subjective data to the objective data, which consists of measurements of roughness, rutting, patching and cracking. The objective data being easier and cheaper to obtain is then used to predict PSI for the entire scheme.

During the AASHO road test by the National Research Council (1962) the need for a device to measure the wheel path profiles was found necessary. The AASHO road test profilometer developed at the time provides a continuous analogue of the pavement with respect to horizontal plane and indicates the pavement slope. For each test section the mean squared deviations of the measured slope was taken as the slope variance. The variation of the slope causes the differences in comfort and hence affects the serviceability index. The original serviceability equation (Equation 2.13) for flexible pavements developed on the AASHO road test is given by Yoder and Witczak (1975).

$$PSI = 5.03 - 1.91 lg\,(1+SV) - 0.10\,\sqrt{(C+P)} - 1.38\,RD^2 \qquad [2.13]$$

Where: ***SV*** is the slope variance

C is linear feet of major cracking per 1000 square feet

P is the bituminous patching in ft^2 per 100 square feet

RD is the rut depth in feet (both wheel trucks measured with a four-foot straight edge.

The AASHO test, equation has been modified to suit different countries. In Zimbabwe, Grant (1982) found out that cracking and patching do no affect the ability of a road to carry traffic adequately, unless the pavement surface is cracked or patched areas have become uneven enough to make it unacceptable. The value of RD^2 was judged small and subsequently PSI was by far affected by the slope variance. Equation 2.14 was suggested as being more applicable.

$$PSI = 4.95 - 2.01 \log (1+SV) \qquad [2.14]$$

The PSI minimises personal bias as compared to personal knowledge and judgement (Nakamura, 1963; Yoder and Witzack,1975). However there is still considerable variation among raters due to difficulties in harmonising the opinions and hence the ratings of the different panel members. This is principally because different engineers and users have divergent opinions to the classification, state of stress, comfort and convenience of a pavement (Wakyendo,1991).

In order to carry out the PSI exercise a panel of at least five persons is selected (Yoder and Witczak, 1975). The Canadian Good Roads Association (1963) used only highway engineers in the evaluation panel. However, Nakamura (1963) found out that the amount of knowledge and experience in engineering was of no significance in rating. Yoder and Witczak (1975) however, indicated that highway engineers were more critical than typical road users.

2.5.4 Objective pavement distress parameters

Pavement roughness, rutting and cracking contribute to the PSI values. These objective distress parameters are directly related to the maintenance activities of flexible pavements. Further, the method of maintenance directly influences these factors. These objective distress parameters are now discussed.

(a) Roughness

Roughness is quantified as the sum of deviation from the horizontal profile in mm/km (Hodges et al 1975, Gichaga and Parker 1988). Hodges et al (1975) found a roughness of 3750 mm/km to represent a critical value for pavements with stabilised road bases and bituminous seals. Roads Department (1988a) proposes that the critical roughness depend on traffic levels, operating costs and the cost of overlaying the road. It proposes a minor overlay and resurfacing at a roughness of 2800 and 3400 respectively for trunk roads.

Surface dressing generally increases the roughness of a flexible pavement, while slurry seals and overlays will reduce it. These findings are contained in Roads Department (1984). It was reported that timely resealing with either surface dressing or slurry seal limited patching and maintained an acceptable level of roughness under high traffic loading.

The results of a long term study by Smith et al (1980) on Nairobi-Mombasa road indicated that the most important factor affecting roughness was the durability of surface dressings. Equally important was their ability to prevent the ingress of water, which leads into the separation of the surface dressing and localised degradation of the top of the base.

(b) Rutting

Rutting are permanent depressions, which develop on the wheel tracks. Ruts develop in a pavement as a result of progressive accumulation of plastic strains in each layer of pavement. The strain is occasioned by each passage of axle load. The cause of rutting is a combination of densification, consolidation and shear deformation (Barksdale, 1977). In addition, permanent deformation of the subgrade causes permanent deformation of the surface.

The subgrade strain criterion is important for pavement design. There is a finite number of standard wheel loads which when passed results in subgrade permanent deformation. The Roads Department(1987) uses this criteria in the determination of the pavement cover in the design of flexible pavements. If the compressive strain is excessive, permanent deformation of subgrade will subsequently cause rutting at the surface.

The bituminous materials have been shown to influence the amount of rutting. Thus, Gichaga (1979) found out that increasing the thickness of surfacing leads to higher compressive stresses in surfacing and the base. This ultimately manifests as ruts. The bituminous mixture stiffness was shown to influence the depth of rutting with the harder and stiffer binders presenting less depth of ruts. In addition, asphalt pavements constructed in Kenya have been found to be prone to plastic deformation and consequent formation of ruts, where the air voids were too low to allow secondary compaction under traffic (Wambura et al, 1999).

Environmental effects on pavement surfacing also affect rutting. While the cited references on rutting place emphasis on accumulation of plastic strains in the pavement layers and subgrade, hot climates have different scenario. In the hot Saudi Arabian environment rutting has been associated with the top 100mm of asphalt bound layers (Noureldin et al,1994). In line with the other references cited above rutting in Saudi Arabia has been found to be severe on the slow truck lanes. It has been found that increasing the air voids content to 5% reduces the incidence of rutting.

(c) Cracking

Cracks are vertical cleavages in a pavement layer. Cracks may be longitudinal, transverse or interconnected to form blocks. The Roads Department (1988a) explains that cracking may be caused by excessive vertical loading, fatigue and ageing, thermal and/or moisture change in the subgrade or shrinkage. More often than not therefore cracking is a result of complex combination of the above effects.

Traffic loading repeatedly flexes the pavement layers causing fatigue and eventually cracking.(Barksdale 1977). Such cracking can be significantly reduced by increasing the elastic modulus of surfacing, base and subbase (Gichaga,1979). The environmentally induced cracks however have been reported to be extremely significant in the Kenyan environment (Smith et al 1990). This type of cracking which results from loaded and aged hardened bitumen does not correlate with strength or traffic loading.

Cracking itself does not significantly affect the serviceability rating of a pavement. If a cracked pavement remained intact without deformation then the rating would basically remain unchanged. However it is the eventual effect of ingress of water and loading which results in accelerated failure of a cracked zone, which eventually reduces the serviceability rating (Uzan et al 1972).

Cracking and rutting are related. This is so because as rutting increases then cracking is also manifested. Murunga (1983) found that rutting and cracking were positively correlated. On a rutted surface, cracking becomes apparent at a rut depth of about 10mm. The extent of rutting and cracking are used as criteria for maintenance and rehabilitation of roads by the Roads Department (1988a).

2.5.5 Deflection

(a) *Introduction*

As traffic traverses the flexible pavement the axle loads induce a downward deflection of the pavement service. The pavement however, bounces back as soon as the load passes a section. This downward deflection is measured by tracing the profile of the surface behind loaded wheels of a vehicle moving at creep speed. The magnitude of this deflection has been related to the strength of the pavement and has therefore been used for the assessment of pavement structural condition.

(b) *Geometry of the deflected bowl*

The deflection as the axle loads pass is almost elastic for the medium and lightly loaded vehicles. In the case of the heavily loaded vehicles a small amount of deflection does not bounce back and permanent deformation results. It is the accumulation of these deformations, which ends up as ruts and cracks.

Several parameters have been picked from the deflected bowl in the analysis of deflection test results. The parameters used include maximum deflection (d_o) and radius of curvature (R). Analysis of the deflected profile shows Equation 2.15 as the relation of deflection and radius of curvature (Roads Department, 1988a and Wakyendo, 1991). It is seen that after the measurement of d_x and corresponding values of x, the value of the radius of curvature may be obtained.

$$d_x = (2Rd_o^2) / (2Rd_o+x) \qquad [2.15]$$

Where: d_x is the deflection at a distance x from the loaded point

d_o is the maximum deflection, usually at the loaded point

R is the radius of curvature

(c) Pavement structural assessment

The Roads Department (1988a) has developed the concept of equivalent modulus (Eq) from Burmisters two-layer analysis. The expression for defining equivalent modulus is given in Equation 2.16. The various variables in Equation 2.16 are determined by Equation 2.17 through 2.23

$$Eq = 10^{\alpha-1}\ E_1 (R/ d_o)^{\alpha/2} \qquad [2.16]$$

Where: Eq is the equivalent modulus of existing pavement kg/cm^2

E_1 is the elastic modulus of the upper layer kg/ cm^2

$$\alpha = 1/(1+\log (E_1/1018)) \qquad [2.17]$$

$$E_2^{B} = (R/0.056)^* (d_o /58000)^{A} \qquad [2.18]$$

$$A = (1-X)/(1-Y) \qquad [2.19]$$

$$X = 0.86 \log h - 0.474 \qquad [2.20]$$

$$Y = 0.493 \log h - 0.71 \qquad [2.21]$$

$$B = 1-A \qquad [2.22]$$

$$(E_1/E_2)^{(x-y)} = Rd_0/3248 \qquad [2.23]$$

Where: E_2 is the elastic modulus of lower infinite layer (kg/ cm^2)

h is the thickness of top layer (i.e. existing pavement in mm)

From the above it is observed that when the value of the thickness of the pavement structure (h) and the maximum deflection d_o are known then Eq may be determined. A value of 1500 kg/cm^2 is interpreted to mean failure and the need for an overlay.

The Canadian Good Roads Association (1963) related failures for maintenance purposes on the basis of rebound deflection. Information from the deflection bowl showed that the extent of weakened sections can be quickly identified. The pavement distress was found to be greater near the outer wheel where the confining effect of the shoulders was either absent or reduced.

Nicholas (1963) found that pavements exhibiting deflections of the order of half a millimetre to one millimetre were either new or had been rehabilitated with asphalt bound bases or soil cement bases. Pavements with low deflection values were judged to have no appreciable maintenance problems.

The data obtained from deflection measurements can be used for quality control during construction. Henderson et al (1994) conducted deflection tests on subbase and base layers during construction of a highly trafficked road in South Africa. In order to carry out deflection on the granular subbase a temporary seal was applied. The deflections achieved on the subbase were 0.26 and 0.43mm. The radius of curvature varied from 109 meters to 139 meters. The low deflections and high radii of curvature indicated that the subbase layers had sufficient strength to support the base. When deflection at the top of the asphalt surfacing was carried out the maximum deflection was 0.2 mm.

The elastic deflection has also been related to subbase strength. On the Thika-Nairobi road, the sections, which had poor drainage were, investigated (Roads Department 1989). Surface deflections on these sections were found to be high. These sections were also found to have extensively failed.

Atibu (1986) conducted deflection and condition survey on the Uplands-Limuru road. He established a fair correlation of rebound deflection with cumulative standard axles. The measured deflection was reduced by about 35% after overlaying.

Long term measurements by Gichaga (1979) and Murunga (1983) established that deflections increased with age and pavement air temperatures. Additionally the deflections for the slow lanes were always large than those for the fast lanes. The lateral deflection profiles were severer than the longitudinal profiles.

(d) Determination resilient modulus from deflection bowl measurements

The modulus of an in-service bituminous layer generally decreases over the design life as the service deflection increases (McMullen al 1990). Majidzadeh (1982) used the deflection of the pavement structure for estimation of resilient modulus. Under the study the measured deflection parameters were used to calculate moduli of the pavement layers for the evaluation of the pavement structural condition.

Murunga et al (1985), recognised that the modulus of a given pavement material would change with repeated application of wheel road and such change would be a function of the fatigue properties of pavement materials. In their study Finite Element methods (Ibi et al 1985) gave various deflections for differing assumed modulus. The resilient modulus for the pavement was obtained as the modulus corresponding to the benkleman beam field deflection.

2.5.6 Dynamic cone penetrometer measurements

The dynamic cone penetrometer (DCP) is a simple equipment, which has been used to obtain the in-situ strength of a pavement structure (Kleyn and Savage 1982). The instrument weighs approximately twelve kilograms consisting of sixteen-millimetre diameter rod with 60^0 cone at one end, which is driven using an eight-kilogram in-built hammer falling 575mm (Figure 2.7) .

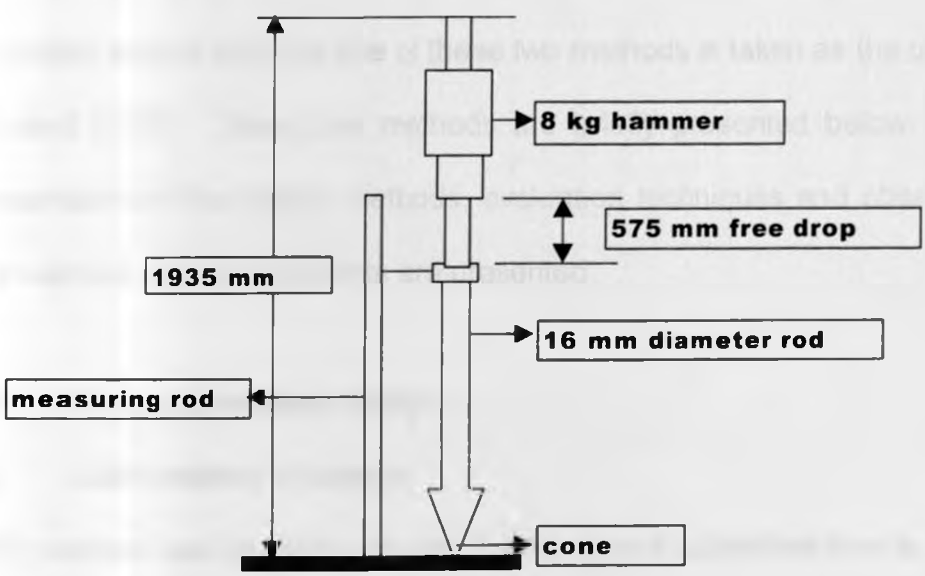


Figure 2.7 The portable dynamic cone penetrometer (DCP)
Source: Kleyn and Savage (1982)

The penetration rate of the DCP is usually measured in mm/blow. This penetration rate is referred to as the DCP number (DN). DN gives an indication of the in-situ shear strength of the penetrated strata and has been highly correlated to CBR (Kleyn 1975). Equation 2.24 developed by the Transport and Road Research Laboratory (1990) is used for estimation of CBR from penetration data.

$$Log_{10}(CBR) = 2.632 - 1.28 \log_{10}(DN) \qquad [2.24]$$

Maree and Visser (1994) linked DCP measurements to the pavement condition. In their study of road conditions for light pavements in South Africa the mean DN-value of the poor sections varied from 5.0 to 8.4 mm/blow. The DN-value reduced to 4.3 for the good sections.

2.6 Airport pavements

There are several methods of design of airport pavements. The most widely used methods are the CBR method and the Federal Aviation Administration (FAA) method. These methods are also used in Kenya. In some countries like Italy the average thickness arrived at by the use of these two methods is taken as the design thickness (Moraldi 1970). These two methods are briefly presented below. Following the presentation of the design methods, evaluation techniques and observed behaviour of in-service airport pavements are presented.

2.6.1 Airport pavement design

(a) CBR method of design

CBR method was originally adopted by the airport authorities due to its simplicity in procedures for testing the subgrade and pavement components, a satisfactory record and adoption to airport problems in a reasonable time (Horonjeff and Mckelvey, 1983). Like in the CBR design for road pavements the subgrade CBR after a four-day soak is adopted for design since at that time the subgrade is at its weakest condition.

In the airport pavement design it is the equivalent single wheel load (ESWL) which yields the same maximum deflection as a multiple wheel load that is used. The contact area of the ESWL is equal to the contact area of one of the wheels in the multiple wheel assembly. The thickness of the flexible pavement required for varying subgrade CBR and different aircraft type is shown on Equation 2.25

$$t = \alpha i \sqrt{(ESWL/8.1CBR - A/\Pi)} \qquad [2.25]$$

Where: t is the design thickness in inches

A is the tire contact area in square inches

αi is the load repetition factor which is dependent on the number of wheels in the main landing gear used to compute ESWL (Figure 2.8)

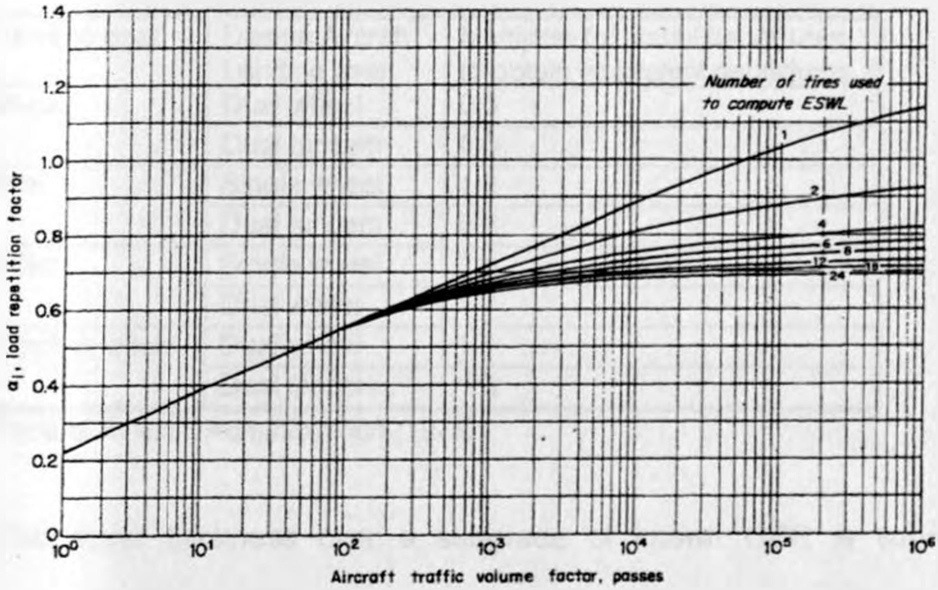


Figure 2.8 Composite plot of load repetition factors versus passes

Source: Horonjeff and Mckelvey (1983)

(b) FAA method of design

This method was developed by the Federal Aviation Administration (1975). The method requires that a forecast of the annual departures by each aircraft be made. The maximum takeoff weight of each aircraft for use in the design is taken as ninety five percent of its weight. This is assigned to the main landing gear. The equivalent annual departures of the design aircraft are determined by summing the equivalent departures of each aircraft by use of Equation 2.26 developed by Federal Aviation Administration (1975).

$$Log\ R_1 = Log\ R_2\ (W_2/W_1)^{0.5} \qquad [2.26]$$

Where: R_1 is the equivalent annual departures by design aircraft

R_2 is the annual number of departures by an aircraft in terms of the design aircraft landing gear configuration (Table 2.6)

W_1 is the wheel load of the design aircraft

W_2 is the wheel load of the aircraft being converted

Table 2.6 Factors for converting annual departures by aircraft to equivalent annual departures by design aircraft

Aircraft landing gear	Design Aircraft Landing gear	Multiplier for actual departures to obtain equivalent departures
Single wheel	Dual wheel	0.8
	Dual tandem	0.5
Dual wheel	Single wheel	1.3
	Dual tandem	0.6
Dual tandem	Single wheel	2.0
	Dual wheel	1.7
Double tandem wheel	Dual wheel	1.7
	Dual tandem	1.0

Source: Federal Aviation Administration (1975)

The cover thickness over a subgrade of known CBR is subsequently obtained from design charts for various wheel gear configurations. The chart for the single wheel gear configuration is shown on Figure 2.9

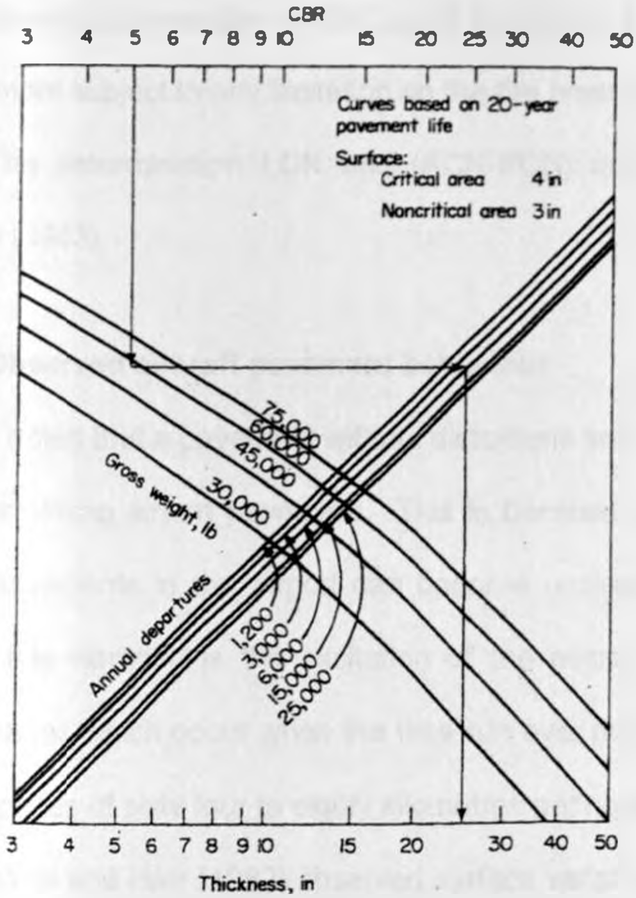


Figure 2.9 Flexible pavement design chart for critical areas for single wheel gear
Source: Horonjeff and Mckelvey (1983)

2.6.2 Airport pavement evaluation

The Air Ministry of the United Kingdom formulated a load classification number (LCN) Horonjeff and Mckelvey (1983). In this evaluation technique the LCN of a pavement depends on the design ESWL and the contact area. It is a measure of the pavement supporting capacity. On the other hand the LCN of an aircraft is based on the gear geometry, tire pressure and composition and thickness of the pavement. Thus if the LCN of an airfield pavement is larger than the LCN of the aircraft the aircraft can safely use the pavement.

The International Civil Aviation Organisation (1983) on the other hand devised the aircraft-pavement classification number system (ACN-PCN). The system reports a pavement classification number (PCN) which indicates an aircraft with aircraft classification number (ACN) equal to all less than the PCN can operate on the pavement subject to any limitation on the tire pressure.

The determination LCN and (ACN-PCN) are contained in Horonjeff and Mckelvey (1983).

2.6.3 Observed aircraft pavement behaviour

It is to be noted that a pavement without distortions and uneven surface is needed for a well performing airport pavement. This is because vibrations experienced during aircraft movements in the airport can become unpleasant. The main contributory factor in this vibration is the excitation of the aircraft body by rapidly changing impulse forces which occur when the tires run over poor uneven pavements even at modest speeds of sixty four to eighty kilometres per hour (Bonney 1970).

White and Harr (1982) observed surface variations with response to transient aircraft loading in USA. Due to the wide pavements at the test sites, aircraft would wander on pavement surface. Instrumental gauges placed in the pavement structure showed up-lift and downward deflection at different parts. In effect a wave was being transmitted on the pavement surface. The surface layers would therefore approach unsupported state as the aircraft passes. This therefore reduces the confinement of the underlying layers and lowers the pavement performance. In the same study theoretical estimation of deflection was done by arbitrary change of elastic moduli and Poisson's ratio to arrive at field deflections

Pinard et al (1990) showed that the methodology used in the evaluation of road pavements was equally applicable in the airport pavements. The methodology adopted consisted of visual conditional survey, non-destructive testing, deflection testing, DCP measurement and selective test pitting of a runway in Botswana. Under the testing program, sections exhibiting similar visual conditions were demarcated and subjected to analysis. The mean DN-value for sections analysed to have less than two years remaining life were found to be 4.1 mm/blow. The DN-value reduced to 3.2 mm/blow for sections with over ten years remaining life.

For the particular airport it was found that the base had degraded principally due to the environmental conditions in Botswana, which led to destruction of lime and cementing compounds in the lime, improved layers. The untreated subgrade and subbase layers were found to have been performing well. In addition the FAA method gave smaller thicknesses of pavement compared to the values based on calculated modulus and remaining pavement life. The outcome of this investigation is that pavement design models must match the local environment, material and pavement structure conditions appropriate to the different areas.

Bush and Thompson (1990) reported that an airfield pavement, which is not structurally adequate, exhibited distresses such as rutting and cracking. These two features indicate that the pavement would be in need of strengthening. Localised distresses would generally require patching while distresses covering large sections of the pavement require that the entire pavement be strengthened. Under the testing program eleven pavement test sections were trafficked to failure using a model aircraft load. Tests carried out before, during and after the testing showed that base course failures occurred as a result of loss of strength. This was attributed to increased moisture content, decrease in density and/or loss of confining stress. It was observed that water infiltrated the pavement through the cracked pavement layers,

In general, distress was characterised by cracking of the pavement followed by rapid increase in rut depth. It was found out that double surface dressing treatment resulted in shallow rutting under the wheel paths. This was a manifestation of deformation of the subgrade layers. When asphalt concrete was used rutting was found to be wider under the wheel paths indicating failure attributable to the base layers

2.7 Pavement management systems and geographical information systems

2.7.1 Pavement management systems

(a) Introduction

This section reviews the application of pavement management systems (PMS) in the management of pavement design and maintenance. This is done in light of the crucial role maintenance takes in the performance of flexible pavements. The World Bank (1992) noted that nearly half of Africa's paved roads were in fair to poor condition due to poor policy on maintenance. This scenario was due to the fact that maintenance operations principally depend on institutional capacities and mobilisation of resources rather than technical matters. The benefit of a properly working pavement management system was demonstrated by Heggie (1992) when he noted that on a road carrying 3000 vehicles per day a reduction of $ 2400 in annualised maintenance expenditure increased the annualised road costs by nearly $ 7000. Experience with implementing pavements management systems in South Africa showed remarkable improvement in road networks where PMS was implemented (Visser et al 1989).

In essence the PMS is an essential planning and design tool. The design of a maintenance system requires detailed information about the state of pavements. In addition PMS operate at a number of different levels. The level of service is dependent on the class of road in the network and the desired level of service. Accordingly the assessment needed is only appropriate to the detail of information corresponding to each level of service required.

The aim of a pavement management system can be summarised as a system that allows maintenance authorities to: -

i) Estimate appropriate level of funding required for maintenance.

ii) Realise the benefits accruing from the application a PMS model.

iii) Realise the consequences of restricting the required funding.

iv) Decide which pavements should have the highest priority for maintenance under limited funds.

v) Decide the best maintenance for each situation.

(b) Road network evaluation

Function class, length width type of surfacing, traffic and climate (Linda et al, 1982), uniquely define each road in the network. These factors enable the costs of the pavement at the start of implementation of the PMS. For existing pavements, estimated construction costs are used, while for new works, the actual construction costs are applied.

Once the initial costs are known, it is then necessary to forecast the costs associated with maintenance of the pavements and road user costs. For each simulated year, the deterioration of the pavement is estimated from relationships contained in the PMS model being used. The road deterioration built into the PMS predicts the level of deterioration at increasing axle loading. The World Bank (1985) HDM III predicts cracking, rut depth and roughness from consideration of loading and age. The cost attributed to vehicle users is attributable to the level of vehicle maintenance and is predicted by relationships of roughness against fuels and maintenance parts. The application of the HDM III deterioration costs is contained in Gichaga and Parker (1988). In making comparative cost estimates and economic evaluation of different policy options the model is provided with detailed specifications of various sets of alternatives along with the unit costs.

In a case study on Yala-Busia road in Kenya after building up the construction cost for the new road, the road was demarcated to nearly homogenous 12 sections. The maintenance costs were based on patching at a cracking index of $5m/m^2$; clearing drainage ditches and surface dressing every 5 years. Annual maintenance operations consisted of surface and base patching besides shoulder and drainage maintenance. The speed, fuel consumption, and operating costs were evaluated on the basis of section gradient, predicted roughness, and road curvature and vehicle specifications. The overall result for this maintenance scheme was discounted costs of construction cost, maintenance and vehicle operation cost (Gichaga and Parker 1988). Different maintenance strategies were tested using different standards by repeated application of the model to compare the costs to the road users to facilitate the most acceptable combination of maintenance and vehicle operation costs. The outcome of the PMS was that the vehicle operating costs dominate the total cost even at low to moderate traffic volumes. It is imperative that maintenance schemes should aim at improving the pavement condition to mitigate vehicle-operating costs

(c) Pavement monitoring

PMS applications require periodic pavement monitoring during the pavement usage. In the monitoring, a structural and functional assessment is made. In the structural defect assessment the degree and extent of surfacing defects including cracks, pumping, deformation, patching and overall failures such as potholes is made. The functional assessment is given in terms of riding quality, skid resistance drainage and shoulder or kerb conditions. Figure 2.10 shows a typical assessment form (Visser et al 1989). The effectiveness of application of the PMS is therefore made during the pavement usage and the necessary adjustments in the PMS Model are therefore done during the monitoring.

Pavement Assessment											
TOWN:						DATE					
SUBURB						CODE					
STREET						CLIMATE					
SECTION:						ROUTE					
LENGTH/WIDTH:/_____m.						TRAFFIC	VL	L	M	H	VH
Structural Assessment											
SURFACING TEXTURE		COARSE			MEDIUM			SMOOTH			
TYPE		DEGREE					EXTENT				
		SMALL		SEVERE			RARE				GREAT
SURFACING DEFECTS	0	1	2	3	4	5	1	2	3	4	5
SURFACING FAILURES											
SURFACING PATCHES											
SURFACING CRACKS											
DRY/BRITTLE											
AGGREGATE LOSS											
FLUSHING/LOSS											
STRUCTURE		DEGREE					EXTENT				
		SMALL		SEVERE			RARE			GREAT	
CRACKS/GENERAL	0	1	2	3	4	5	1	2	3	4	5
STABILISATION											
SUBSIDENCE											
LONGITUDINAL											
CROCODILE											
PUMPING	0	1	2	3	4	5	1	2	3	4	5
DEFORMATION	0	1	2	3	4	5	1	2	3	4	5
RUTTING											
WAVING/SETTLEMENT											
PATCHING	0	1	2	3	4	5	1	2	3	4	5
FAILURES/POTHOLES	0	1	2	3	4	5	1	2	3	4	5
Functional Assessment											
RIDING QUALITY		V. Good		Good		Fair		Poor		V. Poor	
SKID RESISTANCE		V. Good		Good		Fair		Poor		V. Poor	
DRAINAGE (SURFACE)		Satisfactory			Inconsistent				Poor		
KERBING/SHOULDER		Yes			Partially				No		

Figure 2.10 Pavement assessment rating form
Source Visser et al (1989)

2.7.2 Geographical information systems for pavement maintenance

Geographical information systems (GIS) enable the input, management, analyses and output of geo-referenced data and information (Mulaku 1998). For road pavement maintenance, the road sections under the system are co-ordinated by the Global positioning system (GPS) or other surveying techniques. Under the system the uniform sections are summarised into one co-ordinated geo-referenced point. The co-ordinates of the various points are then linked to a database about the surface condition. The database is descriptive about the surface condition. The detail of database may include photographic description of the sections.

The combination of the GPS and database becomes a digital road pavement inventory, which can then be used to make decisions on maintenance strategies, planning and maintenance schedules. Inventories taken at different times are overlaid to create a pavement condition monitoring system.

2.8 Finite element method of evaluating pavements

Finite element method (FEM) when utilised for pavements uses basic elastic theory to determine stresses and strains. Younger (1990) showed that normal pavement base layers loaded conditions approximated the Bousinesq distribution of stress and strain. FEM for pavement structure assumes that a continuum can be represented by a series of elements whose behaviour can be specified by finite number of elements (Atibu 1986). The structure is idealised into a finite number of elements where the nodes are fixed at appropriate positions. The nodal force vectors are computed by lumping the total load on the nodes. The solution is achieved by consideration of stiffness of the elements and the assembled matrix for the pavement structure.

The usefulness of FEM in analysing pavement structures, has been recognised for sometime now. Murunga et al (1985) found the results from FEM gave deflections, which compared favourably with those obtained from field measurements using benkleman beam.

Gichaga (1979), Murunga et al (1985) and Atibu (1986) have studied the pavement performance of Nairobi-Thika road using theoretical analysis by FEM. It has been observed that

(i) Crushed stone base gave higher deflections than stabilised murram base.

(ii) The contribution of surface thickness below fifty millimetres to the structural capacity was insignificant.

(iii) For base thickness below one hundred millimetres stresses and strains in the surfacing were relatively insensitive.

(iv) The maximum surface deflection was relatively insensitive to variations in subbase thickness for thickness below one hundred millimetres.

The theoretical estimation of deflection by FEM requires moduli of pavement layers and their Poisson's ratio. The change of moduli and Poisson's ratio with environmental factors complicate their accurate estimation. Different estimations of these fundamental characteristics of the pavement inevitably leads to different conclusions (Abe,1982). However, Wakyendo (1991) explained that change in Poisson's ratio have little effect on strain and stress in the pavement and subgrade.

Opiyo (1995) showed that the stiffness of the layers had considerable effect on the structural performance of the various layers. A reduction in the stiffness of the base layer results in more load being carried by the asphalt layers. The effect is more severe in the thin asphalt surfacing layers. If on the other hand a pavement has high stiffness, then the risk of shear failure in the subgrade layers is reduced but the surfacing layer will fail in fatigue.

In summary therefore, the pavement materials must be accurately characterised to model out the pavement. The loading of the pavement must additionally be accurately estimated in order to use realistic load in the FEM.

2.9 Literature review conclusion.

2.9.1 Overall conclusions

In conclusion therefore the pavement performance for a given traffic loading depends on the several factors including the environment, the type of subgrade and the pavement structure. The subgrade strength, which is the main criteria in the section of a pavement structure, should be constructed as per design and specifications. In the field, the subgrade being weaker than the subbase and base layers should be protected from deformation by the top layers. The elastic modulus for the subgrade layers can be estimated from the Equation 2.2, when the CBR of the material is known.

The properties of bitumen relevant for the evaluation of pavement structures can be done by isolating the constituents into asphaltenes, aromatics and saturates. As bitumen ages the proportion of asphaltene increases when the aromatics and saturates are oxidised and vaporised. Increased asphaltene content leads to high viscosity and brittleness. This contributes to early pavement failure.

The behaviour of bituminous mixtures in service depends on the environment. Decreased viscosity during the hot periods reduces the strength of the pavement. The fundamental properties of a bitumen mixture namely resilient modulus and Poisson's ratio change rapidly with changing temperature. The strength of the pavement is therefore changing all the time. Pavement design models therefore, must match the local environmental conditions and materials.

The Marshal stiffness, which can be obtained from the Marshal testing of bitumen mixture samples can be used for estimation of modulus of elasticity and tensile strength of a bituminous mixture from Equations (2.8 and 2.9 respectively).

The variation of strength of a bituminous mixture depends with the source of the binder. This means that with the liberalised Kenyan economy, bitumen coming into the country requires to be routinely tested in order to ascertain its characteristics for use under the local environment.

The functional assessment PSI is correlated with pavement distress parameters. Which means that by conducting a pavement conditional survey the PSI is indirectly being assessed. By use of established correlation a level of service can be reached for an acceptable surface. Rutting and cracking therefore can be used for maintenance and rehabilitation of bitumen bound flexible pavements.

The geometry of the deflected bowl in the benkleman beam test can be used to estimate the equivalent modulus of the pavement. When the equivalent modulus has reduced below a critical value 1500 kg/cm^2, then the pavement would usually require rehabilitation. The dynamic cone penetrometer was found to be a cheap and easy to use equipment for determination of in-situ strength of the pavement structure.

In the case of airport pavements the design revolves around the ESWL for the design aircraft in the case of the CBR method or the annual departures of the design aircraft in the case of the FAA method. The thickness of the pavement layers in the two cases depends on the CBR of the subgrade and that of the pavement materials.

With the aid of computers the FEM is used for estimation of stress and strain in the pavement structures. The main handicap is the estimation of elastic modulus and Poisson's ratio applicable to the various layers at the time of testing. This is in recognition of the variation of strength properties with age and loading for the different environmental conditions encountered in the country.

2.9.2 Areas that this research thesis aims to address

After an evaluation of the research objectives and the available literature in flexible pavement design, construction and maintenance, gaps in knowledge with particular reference to the flexible pavement performance under the Kenyan environment were found. The specific gaps addressed in this research include: -

i) Determination of physical, strength and compaction properties for the Kenyan subgrade soils in their neat and improved conditions.

ii) The strength behaviour of bituminous materials with age and environment.

iii) The present serviceability index (PSI) relationship with pavement condition.

iv) The use of field deflection and FEM in computation of stress and strain.

Chapter Three

Methodology and Data Collection - I

Flexible Road Pavements

3.1 Introduction

3.1.1 General

This chapter deals with the method used for data collection with respect to flexible road pavements. The data was necessary for the eventual analysis to establish the properties of pavement materials and pavement performance under different Kenyan environments. Chapter four deals with data collection with respect to the case airport flexible pavements. Relevant literature review was presented in chapter two. The present chapter therefore, outlines the methodology adopted to achieve the objectives. The data collected during the various field surveys and laboratory testing is presented. Chapter five then presents detailed analysis and discussions that can be drawn from the test results of both chapters three and four.

The material in this chapter is presented in the following order.

(i) Laboratory testing of the pavement materials

(ii) Effect of environmental and traffic factors on pavement materials

(iii) Non destructive pavement evaluation of high and low volume roads.

Though presented in this chapter the data collected under laboratory testing is also relevant to the flexible airport flexible pavements. It is acknowledged that field tests are often expensive and difficult to mount. On the other hand even laboratory tests which are well controlled can miss vital field performance modelling. This is due to boundary conditions of the test specimens being at variance with the field conditions. Testing of laboratory and field samples were carried out with a view of establishing performance characteristics of the studied flexible pavements.

3.1.2 Analyses of data

Because of the diversity of data contained in this chapter and chapter four the developed relationships of the various parameters have been tested to check the degree of correlation (r) by calculation of the coefficient of determination (r^2). The relationships have additionally been checked by analysis of variance (ANOVA) to check the reliability of the developed equations. For this purpose after evaluating the capabilities of the present day statistics packages the data was analysed by the statistics program for social scientists (SPSS). The package has been used to perform the F test on all the regression equations developed.

Under the package, for every observed value of the dependent variable (y), there is a corresponding estimated value (Y) and a mean for all the observed dependent values ($\bar{y}$) (Figure 3.1). In the regression analyses the variation of the dependent variable is explained by the sum of squares (SS). The part attributable to the regression line is $\Sigma(y-Y)^2$, while the part explained by the residuals is $\Sigma(Y-\bar{y})^2$. MS is the sum of squares divided by the degrees of freedom (DF). The ratio F performed in the F test is the MS attributable to the regression divided by MS attributable to the residuals.

Under the test, when the F ratio returned by the analyses is large, the relationship is significant. Alternatively the signif F shows the significance level of the regression. Low values of signif F show regressions, which are highly significant.

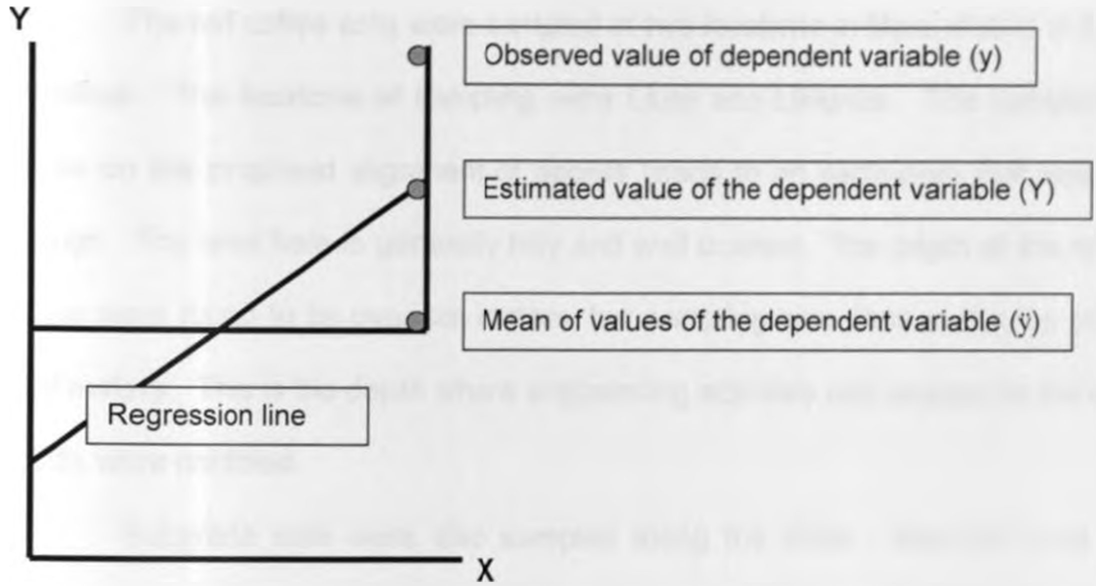


Figure 3.1 Values of the dependent variable in regression analyses

3.2 Laboratory testing of pavement materials

Pavement building materials were collected from various parts of the country (Figure A3.1 in appendix A) and tested to give a database for the engineering properties and performance of the materials. The data collected was in all cases associated with road projects, which were at the design stage of implementation.

Laboratory testing was also carried out on bituminous mixtures representing bituminous bound materials. An attempt was also made to simulate field and environmental conditions in the laboratory.

3.2.1 Sampling and laboratory tests on subgrade soils

(a) Sampling

Black cotton soils were sampled from Bunyala division of Western province. In this area the roads being designed were to serve a large irrigation project. The area sampled is on the River Nzoia flood plains. The soils here are predominately black cotton soils. To fully identify and characterise the soils for the roads in the irrigated areas a sampling grid of one kilometre was adopted.

The red coffee soils were sampled at two locations in Meru district in Eastern Province. The locations of sampling were Liutu and Likiundu. The samples were taken on the proposed alignment of access roads to an earth-dam that was under design. The area here is generally hilly and well drained. The depth of the red soils cover were found to be over ten meters, but sampling was done in the top one and half meters. This is the depth where engineering activities with respect to the access roads were confined.

Subgrade soils were also sampled along the Wote - Makindu road. This earth/gravel road passes through a semi-arid part of Makueni district in Eastern Province. The sixty five-kilometre road passes through predominantly sandy loamy silt clays. Sampling was carried out at one-kilometre intervals to depth of one and half meter depth. The road was being designed with an aim of improving it to a bitumen-surfaced road.

Lateritic gravels were also obtained from Western province divisions of Mumias and Amagoro. These were to be used in the construction of the irrigation scheme roads. Several quarries along the Wote-Makindu road were also investigated. The quarries were to be the source of base and subbase materials for the improved road.

(b) Laboratory testing

Samples from the field were subjected to identification, compaction and strength tests. The summary of the tests carried out is presented on Tables A3.1 through A3.5 in appendix A. Lateritic gravels are usually used in the subbase and the base layers of the pavement structures. For these soils therefore further testing was required to assess their suitability in the upper pavement layers. The lateritic gravel samples obtained from Wote-Makindu road were therefore tested after lime improvement in addition to the above tests. The test results with respect to lime improvement are presented on Table A3.6 also in appendix A.

3.2.2 Bituminous materials

(a) Introduction

The main objective of testing the bituminous materials was to study the index properties of bituminous mixtures and compare them to the fundamental physical and chemical properties of bitumen and bituminous mixtures. The index properties included penetration and softening point of bitumen binder. The Marshal testing of bituminous mixtures was also done. The physical properties are those which are intended to give the engineering characteristics and performance of bituminous mixtures. In this study the physical properties determined included the Young's modulus and the tensile strength. The chemical properties studied for bituminous materials were those related to separation of bituminous binder into its various components. An assessment of the changing chemical composition with age was also carried out.

The testing program was varied to simulate varied conditions of traffic environment and different types of materials. An attempt was also made of correlating the index properties with the physical properties. This was in recognition of the fact that pavement design basically utilises index properties (Sargious, 1975).

The material types were varied in the experimentation. The variables included aggregate types and grading, bitumen type and content, and temperature. Age hardening was evaluated by studying the pavement materials recovered from past surfacing and asphalt concrete of pavements in service. The selected sites were part of long-term study sections constructed for the purpose of studying the environmental and traffic effects on flexible pavements by the Ministry of Public

Works (Roads Department 1988b). In addition high volume sections were included purposely for this study. Section 3.3 below deals with this topic.

(b) Laboratory tests on aggregates for bituminous mixtures.

Aggregates form a major proportion of the bituminous mixtures. The effect of aggregate on the bituminous mixture is related to the type of aggregates and the gradation used. A testing program of the aggregates used in the laboratory study was therefore done in order to characterise them.

In order to impact, good performance characteristics in the bituminous mixture the Road Departments (1987) has set strength limits of aggregates for use in bituminous mixtures. It was therefore necessary to ensure that the aggregates used in the laboratory study complied with this standard. Further, tests were carried out in aggregates recovered from in-service pavements.

In order for aggregates to perform properly, the qualities, which they require to possess, include high crushing resistance, high impact resistance and high resistance to abrasion. These properties enable the bituminous mixtures to resist static and impact loads in addition to traction forces of the tyres, which induce abrasive forces on the aggregates. Additionally, the aggregates should possess low water absorption tendencies. The shape of the aggregates should further ensure that high stability of the resultant bituminous mixture is achieved. For the purpose of mix design, individual aggregates falling into the nominal sizes of 20, 13, 6 and 3mm were obtained in sufficient quantities for testing of bituminous mixtures. The grading of the four types of aggregates is shown on Figure 3.2

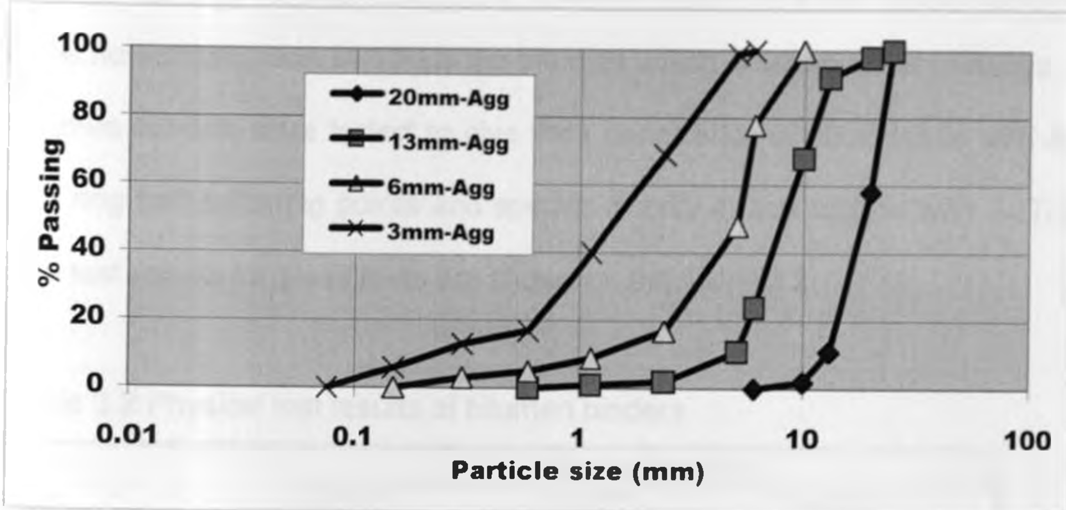


Figure 3.2 Nominal aggregate grading

The aggregates were subjected to physical and strength tests. This was done to ensure that they complied with the Roads Department (1987) requirements. Table 3.1 shows the physical and strength test results of the aggregates and the acceptable Roads Department (1984) ranges. It can be seen that the aggregates satisfied the acceptable criteria for dense bitumen macadam.

Table 3.1 Test results with respect to aggregates and acceptable criteria for dense bitumen macadam

Test	Results	Range *
Value (ACV)	18	10-35
Aggregate impact value (AIV)	16	10-35
Loss Angeles abrasion value (LAA)	19	Max 25
Water absorption test	2	1-4
Elongation index	25	Max 25
Flakiness index	11	Max 20
Angularity No.	10	Max 10

** Source: Roads Department, (1987)*

(c) Laboratory tests on binders used in the bituminous mixtures

Penetration grade bitumen 60/70, 80/100 and 180/200 were used for the bitumen mixture test. The variation of the grade of bitumen was deliberately done to cover the performance of the hard grade 60/70 bitumen to the softer 180/200 bitumen. It is to be noted that grade 80/100 is the bitumen which is widely used in Kenya. Sample bitumen binders were tested to give their penetration in accordance with ASTM D5 and ring ball softening points and specific gravity in accordance with ASTM D2398. The test results for these tests are shown on the Table 3.2.

Table 3.2 Physical test results of bitumen binders

Test	Grade of bitumen		
	60/70	80/100	180/200
Penetration (.1mm)	65	90	190
Softening point (°C)	52	49	40
Specific gravity	1.06	1.05	1.05

(d) Bituminous mix design

In order to produce results which could be compared to the expected performance of in-service bituminous mixtures, the preparation and testing of the specimens attempted to duplicate field procedures for production of bituminous mixtures in the country. Four blended aggregates were chosen from the Nairobi City Council (1969) standard specifications. A comparison of the gradations and that of the Roads Department (1987) is shown on Table 3.3 and Figure 3.3. It can be seen that the Roads Department combined 0/30 and 0/40 envelops tends to have finer gravel fraction and coarser sand fraction. The Roads Department combined envelope is narrower than the envelope created by the four gradations in the gravel range. In the sand range the created envelope is narrower.

Each of the four gradations was mixed and compacted with the three grades of bitumen binder: -namely 60/70, 80/100 and 180/200 penetration grades in turn. The bitumen content in the samples was varied from 3% to 8%. The specimens were cured in air for seven days before Marshal testing on the eighth day. Prior to the testing the weight and dimensions of the samples were taken. The collected data was then used for calculation of compacted density of mix (CDM), air voids in the compacted total mixture (VTM) and voids in the filled with bitumen (VFB). The test results are presented on Table A 3.7a in appendix A. Table A3.7b also in appendix A shows the Marshal test results of the samples.

Table 3.3 Gradations of Blended Aggregates

Sieve size	Blended aggregate type (% passing)				Roads Department (1987 envelope)	
(mm)	A	B	C	D	0/30	0/40
40	100	100	100	100	100	95-100
25	100	100	100	100	90-100	70-94
20	86	84	100	88	71-95	
13	81	66	94	56	58-82	56-76
10	67	61	83	48		
6	46	54	62	46	44-60	44-60
4	39	50	49	46		
2.4	27	36	31	31	26-40	25-40
1.18	17	23	21	20	20-33	20-33
0.60	9	12	13	12		
0.30	5	11	10	10	7-21	7-21
0.15	2	6	4	6		4-15
0.075	1	1	2	2	2-8	2-8
Filler	3	4	4	6		

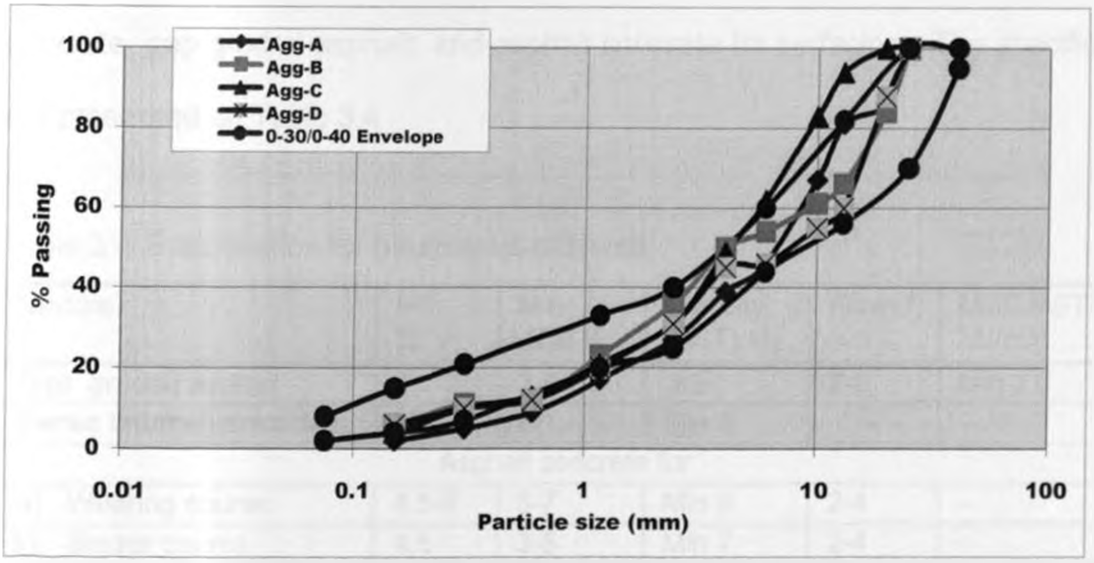


Figure 3.3 Nominal aggregate grading

The optimum bitumen content is a compromise of a maximum compacted density, which ensured an impervious dense mixture, a maximum stability, which is a measure of strength, maximum flow and voids content, which measures an allowable deformation. Further the voids content is significant in the performance of bituminous mixtures as it measures the void space in the mixture. It is the void space which forms the conduit if air and water which are the key parameters in the age hardening of bituminous mixtures under the Kenyan environment (Mwea and Bezabeh, 1994). In addition a minimum percentage of air voids is required to give the bituminous mixture capacity to elastically deform without rutting (Noureldin et al, 1994).

In order to arrive at an appropriate bitumen content, graphs showing the variation of CDM, VTM and VFB with bitumen were developed and are presented on Figure A3.2 in appendix A. The corresponding graphs of stability, flow and Marshal stiffness are presented on Figure A3.3. These test results were then used for the mix design as explained below.

The Roads Department (1987) has different specifications for the various bituminous mixtures when used as dense bitumen macadam, for base, asphalt concrete, gap graded asphalt, and asphalt concrete for surfacing. The specifications are presented on Table 3.4

Table 3.4 Specification for bituminous mixtures

Mixture	BC %	Max. VTM %	Stability (MST) kN	Flow (f) mm	MSE MST/f kN/mm
Gap graded asphalt		3-9	3-9	2-9	Min 2.0
Dense bitumen macadam	Min-4	7	Min 5	- -	- -
Asphalt concrete for					
a) Wearing course	4.5-8	3-7	Min 9	2-4	-
b) Binder course	4.5	3-5	Min 7	2-4	-

Legend BC – bitumen content, VTM – voids in total mix, MSE – Marshal stiffness

Source: Roads Department (1987)

. The specifications additionally require that the 60/70-grade bitumen is used in areas where the road temperatures exceed 40^{0}C and the 180/200-grade bitumen is laid in attitudes above 2500 meters. It is to be noted that the gap graded asphalt is essentially for low traffic and the current specification is unsure of its suitability in heavily trafficked roads.

It was therefore necessary to develop specific targets from Table 3.4. For this purpose the optimum bitumen content was found by obtaining the arithmetic mean of the bitumen content corresponding to maximum stability, maximum compacted density, five percent voids in total mix (VTM) and seventy-five percent voids filled with Bitumen (VFB). The optimum content was checked for compliance with a range of flow of 2-5mm.

Based on this criteria which also complied with criteria proposed by Asphalt Institute (1970) Table 3.5 showing the optimum bitumen content and various mix design and strength variables was developed for further testing.

Table 3.5 Mix Design Results

	Aggregate grading/Penetration grade of bitumen											
	A			B			C			D		
	180/ 200	80/ 100	60/ 70	180/ 200	80/ 100	60/ 70	180/ 200	80/ 100	60/ 70	180/ 200	80/ 100	60/ 70
BC	6.6	6.4	6.2	5.7	5.6	6.6	7.0	6.8	6.6	5.2	5.0	4.8
CDM	2.23	2.22	2.23	2.24	2.24	2.19	2.18	2.17	2.19	2.28	2.28	2.3
VTM	4.15	5.5	4.8	5.2	5.2	6.2	6.3	6.8	6.2	4.8	4.5	4.6
VFB	77	71	72	70	70	68	70	67	68	70	70	70
MST	7.23	7.11	6.80	8.43	9.60	6.67	5.07	5.51	.67	7.74	10.2	8.6
Flow	4.06	4.17	4.06	4.06	4.06	4.06	4.06	4.8	4.06	4.06	4.06	4.1
MSE	1.78	1.71	1.67	2.08	2.37	1.64	.28	1.15	1.64	1.91	2.52	2.1
Filler	0.5	0.5	0.5	1.1	0.6	0.6	0.6	0.6	0.6	1.0	1.20	1.2

Legend: - BC – bitumen content (%), CDM - compacted density of mix (kg/m^3), VTM – void in total mix (%), VFB – voids filled with bitumen (%), MST – Marshal stability (kN), Flow – (mm), MSE Marshal stiffness kN/mm, filler- (%)

The Marshal specimens tested for mix design were prepared and immersed in hot water bath maintained at 60^0 C for 30-40 minutes. The Marshal-breaking head used in breaking of the samples was simultaneously immersed in the bath. The specimens were subsequently loaded with the breaking head advancing at the rate of 50mm per minute. As is normal in the Marshal testing, the Marshal stability was recorded as the maximum load at which the specimen failed while the flow was recorded as the vertical deformation of the failed sample (Asphalt Institute 1970).

Gichaga (1987) showed that bituminous mixtures undergo considerable healing after failure. Thus when bituminous mixtures with 80/100 and 60/70 penetration bitumen were re-tested after being kept in oven for sixteen hours at varying temperatures, the maximum recovery was found to occur around 40^0 C (Figure 3.4).

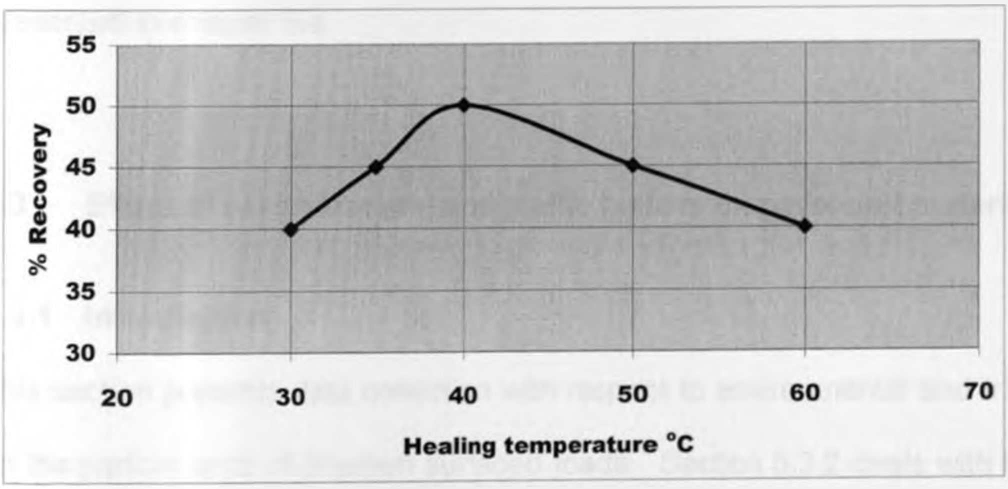


Figure 3.4 Healing of bituminous samples

Source Gichaga (1987)

For this project re-testing the failed samples after placing them in an oven maintained at 40°C for sixteen hours carried out re-healing tests. Table A3.8 in appendix A shows the Marshal test results after healing. The corresponding results before healing are presented on Table A3.7 (b) in appendix A.

(e) Variation of stability, stiffness and flow with temperature

Using the optimum bitumen content shown on the Table 3.5, three samples were prepared at each binder content. By varying the three samples through the four aggregate gradations, one hundred and eighty four samples were prepared for testing at 20^0, 30^0, 40^0, 50^0 and 60^0C. The variation of temperature during the testing models out the effect of varying pavement temperatures as is common with in-service pavement structures. In addition to the usual measurement of Marshal stability and flow the time it took the Marshal to fail from the commencement of loading to failure of the samples was recorded as the loading time. Table A3.9 in appendix A shows the test results. The variation of Marshal stability, flow and stiffness and loading time is tabulated against varying temperature and gradation. The analysis of this data is presented in chapter five.

3.3 Effect of environmental and traffic factors on pavement materials

3.3.1 Introduction

This section presents data collection with respect to environmental and traffic effects on the performance of bitumen surfaced roads. Section 3.3.2 deals with low volume test roads, which were spread around the country. Under this section data on ageing of bituminous surfacing has been presented. In addition the performance of the section constructed in the same bases and subbases but varying surfacing bitumen and chippings is presented. Section 3.3.3 presents the high volume test sections. For these sections the physical and chemical behaviour on thick asphalt concrete surfacing was evaluated. The laboratory work associated with the samples recovered from the field is presented in section 3.3.4.

3.3.2 Low volume bitumen roads

Flexible pavement design involves the selection of materials for construction of a road that will carry traffic safely, conveniently and economically during its design life. It has however been observed that roads fail before they obtain their desired design life even after well-organised construction and maintenance. Further, when the same engineering standards are maintained over a length of road, parts of the road will fail and leave other sections in a serviceable condition. It would therefore appear that environmental factors have an effect on the performance of road pavements.

Construction of lightly trafficked roads using normal design standards leads to undue emphasis of high standard materials. This leads to high unnecessary construction costs (Maree and Visser 1994). Consequently data on construction and performance of low volume pavements was collected for analysis with an aim of investigating the possibility of using low quality materials on lightly loaded pavements.

Several test sections were identified on existing roads and others were constructed for research purpose. Figure A3.4 in appendix A shows the location of these test sections. The test sections represented different formations, different air temperatures, subgrade materials and strength. The traffic volumes in the test roads (Roads Department 1988b) fell in the class T5 in accordance with the Roads Department (1987). Class T5 traffic attracts 0.25-1.0 million cumulative standard axles over the design period.

Table 3.6 shows the environmental and traffic characteristics of the test sections. The test sections were studied over periods ranging from six to ten years. The data collected has included rational analysis and determination of age hardening characteristics of the investigated bitumen binders.

Table 3.6 Environmental and traffic characteristics of test sections

Section/Road classification	Province	Alt.(m)	AAR	Temperatures		AADT
				Max ^{0}C	Min^{0}C	
Marich Pass/A1	Rift Valley	1100	1000	34	>22	*
Narok/C12	Rift Valley	1900	730	27	8	113
Kalokol/D348	Rift	460	170	34	>22	*
Majengo/D291	Western	1620	1500	30	16	26
Oyugis/D220	Nyanza	1400	1075	30	16	53
Kisii/C21	Nyanza	1530	2183	26	15	741
Kwale/C106	Coast	90	850	34	>22	182
Ndumberi/D409	Central	1540	1016	28	12	678

Legend: -, AAR - Average annual rainfall, Alt – Altitude, AADT- Annual average daily traffic, * New sections

The test sections were divided into subsections. On the same test section, the base and subbase had the same specifications. Variations were however made in the surfacing types and binders. Plate A3.1 through A3.3 in appendix A shows three such test sections. The difference in surfacing aggregates can be seen in Plate A3.1 for the Kwale test site where the two surfacing aggregates have different colours. Table 3.7 shows the construction details. The surfacing aggregates included hard strong stones (phonolite, basalt and sandstone), soft stones (quartz, trachite, weathered rock and coral), and laterite modules. Medium curing cutback (MC3000) was dominantly used as binder. In a few cases cationic emulsions (K1-60 and K1-70), short residues 200/300, 200/500 penetration bitumen grades and soft bitumen MB 5000 were used. The MB 5000 was imported from Norway for the research purpose while the other bitumen binders are generally used in Kenya for surface dressing. The MB 5000 has a penetration range of 400 to 500.

The medium curing cutback MC3000 used in Kisii, Oyugis and Majengo trial sections was prepared by blending 80/100 penetration bitumen with heavy diesel fuel, while in the other section it was blended with kerosene. To improve the adhesion property of the bitumen 1-% ant-stripping amine was added to the MC 3000 at the same sections. The surfacing layers were constructed in double surface dressing except for the Narok site, which was single surface dressing.

Table 3.7 Construction details for test sections for the low volume sealed sections

Section /Road	Year of Co.	Surfacing		Base			Subgrade/ subbase. PI/ CBR
		Binder Type	Agg. LAA/ ACV (%/%)	Type	LAA/ ACV	PI/CBR (%/%)	
Marich Pass – (A1)	1978 1984 1984	MC 3000 MC 3000 MB 5000	- - -	Quartz	- - -	-/46	10 /11
Narok (C12)	1985	80 /100 MC 3000 KI 60 KI 70	Trach .(33 /22 Quartz 39 /26	Quartz	39 / 24	16 /15	Calcarious Loam. 19 / 12
Kalokol (D348)	1985	200/500	Quartz 45/26	Quartz	45 / 30	14/27	Quartz 15 / 24
Majengo (D291)	1984	MC 3000*	Quartz 56/33 Phon. 17 / 15 Lat. 74/57	Quartz	56 / 33	17 /35	(Subbase) Laterite 21 / 40
Oyugis (D220)	1984	MC 3000*	Lat. 65 / 55 W/Rock 60/ 36 Basalt 12 /9	Laterite W/rock	/54 /54	17 / 61 15 / 30	Sandy Clay 23/13
Kisii (C21)	1984	MC 3000*	Lat. 65 / 55 W/Rock 60 36 Basalt 12 /9	W/rock Laterite W/rock	- / 12 60/36 60 /36	21/22 13/33 21 /75	(Subbase) W/rock 23 / 38
Kisian – Bondo (C27)	1981	80 /100 KI/60 200/300	-	-	-	-	-
Kwale (C106)	1985	MC 3000*	Lat. 54 / 42 Coral 38 /32 San. 36 /22	Laterite	-	15 / 28	Sand /Silty / gravel 19 / 15
Ndum-beri (D409)	1984	MC 3000 80/100	Phono. 20 /20	Laterite	- / 45	21/29	(Subbase) Laterite 27 /16

Legend: Co. – Construction -* 1% ant-stripping amine added, - W / Rock - Weathered Rock,- MB - Soft Bitumen, Trach – Trachite, Phon. – Phonolite, Lat – Laterite, Agg. - Aggregate

The investigation therefore, focused on both high quality and low quality aggregate. Additionally a wide range of bitumen binders was investigated. The entire construction of the test section was done under close engineering supervision to ensure design specifications were achieved. The relative densities in the base and subbase layers were around 100% of the standard compaction immediately after construction.

During the subsequent study after construction, visual inspection, supplemented by field and laboratory tests were carried out for ten years. After three years of service all the potholes that had developed in Ndumberi, Kisii, Oyugis, Majengo and Kwale test sections were patched. The Narok test section was sealed. The study in the fourth year included counting potholes, which had developed in the intervening period. In effect the performance of the test sections was evaluated. Table 3.8 shows the performance indicators (potholes and penetration of bitumen). Additionally it shows the aggregate strength characteristics and the traffic that had been carried at the time of recording the performance. The sections then remained without maintenance up-to the final year of study.

Table 3.8 Pavement performance of the low volume test sections

Road Section	Surfacing Aggregate Type/LAA/ACV		Traffic CESA x 10^6	Pen. of rcvd binder	Surfacing thickness (mm)	Number of Potholes	General surface condition
Kwale	Laterite	54/42	0.067	18	12	0	Good
	Coral	38/32		21	12	0	Good
	Sandstone	36/22		20	12	0	Good
Narok	Trachyte	33/22	0.049	23	15	0	Good
	Quartz	39/26		20	15	0	Good
Ndumberi	Phonolite	20/20	1.625	11	15	43	Fair
Kisii	Laterite	65/55		21	8	265	Failed
	W/rock	60/36	2.111	40	14	20	Fair
	Basalt	12/9		32	14	15	Fair
Oyugis	Laterite	65/55	0.159	23	12	31	Critical
	W/rock	60/36		31	17	194	Failed
	Basalt	12/9		45	15	0	Good
Majengo	Laterite	74/57	0.564	12	5	45	Critical
	Quartz	56/33		35	16	0	Good
	Phonolite	17/15		57	16	0	Good

Legend: CESA Cumulative equivalent standard axles, Pen of rcvd - Penetration of recovered binder.

During the final year of study monitoring of the test sections included collection of bitumen surfacing samples. The samples were recovered by peeling off the bound surfacing layer. The exposed base and subbase were then tested with the dynamic cone penetrometre to obtain the DCP strength. The CBR was then calculated by use of a correlation developed by the Transport and Road Research Laboratory (1990). Field densities for the pavement layers were also taken. The test result for the bituminous materials are presented in section 3.3.4(i) below while those for the base and subbase are presented in section 3.3.4(ii)

3.3.3 High volume asphalt concrete pavements

The test sections for the high volume asphalt sections were not constructed for investigation purposes. They were located on existing high volume sections of Mombasa-Nairobi-Nakuru road (Figure A3.4 in appendix A). Their details were retrieved from construction records and are shown on Table 3.9

Table 3.9.High volume asphalt concrete test sections

Road Section	Year of Construction	Surfacing
Ullu –Sultan Hamund	1977	100AC
Nyayo Stadium – Airport Road	1985	40AC
Uplands –Longonot	1984	50 AC
Nakuru Highway	1985	50AC

Legend : AC - Asphalt concrete

For these high volume roads core samples were taken at the edge and along the wheel path on each traffic direction and at the centreline of every road section. The test results for these sections are consolidated with those of the low volume roads and presented in the section 3.3.4(i) below.

3.3.4 Laboratory work

(a) Bituminous materials

Bitumen samples were recovered from the collected surfacing and asphalt samples. The recovery of the bitumen was conducted in the line with AASHTO T 170 (ASTM D1856). Plate A3.4 shows the extraction apparatus. The extracted samples were then subjected to ductility at 25^0c (AASHTO T51, ASTM D113) (Plate A3.5), penetration tests at 25^0C (AASHTO T49, ASTM D5), and ring and ball softening point (AASHTO T53,ASTM D2398) (Plate A3.6). The kinematic viscosity of the recovered bitumen was determined to AASHTO T 201 (ASTM D2170) with the canon type apparatus. Table A3.10 in appendix A shows the chemical and physical properties of the bituminous samples before chemical separation.

Further chemical analysis on the bitumen binder conducted in accordance with ASTM D4124-84 standard procedure. The bitumen samples were first separated into n-heptane insoluble asphaltenes and n-heptane soluble petrolenes following digestion of the asphalt in n-heptane. Petrolene fractions were then absorbed on alumina and fractionated into the saturate naphene, aromatic and polar aromatic fractions by downward solvent elution in a glass chromatographic column. The fractions were then obtained by removing the element solvents following standard laboratory distillation procedure. The net weight of each fraction was taken and the percentage of the fractions was calculated as the weight percent of the original sample (Tables 3.10 and 3.11).

Table 3.10 Chemical fractions of the recovered bitumen samples for surface dressed low volume roads

	Ndumberi	Kwale	Narok	Majenjo	Kisii	Oyugis
Age at testing (Years)	7	6	6	7	7	7
Penetration (0.1mm)	12	16	23	35	32	45
Saturates %	23.4	20.6	22.9	24.0	24.2	24 8
Naphathalene Aromatics %	20.9	25.1	35.5	23.3	24.2	21 3
Polar Aromatics %	23.9	24.2	16.4	23.3	21.1	23.0
Asphaltenes %	31.8	30.1	25.2	29.4	30.5	30.9

Table 3.11 Chemical fraction of the recovered bitumen samples for high volume asphalt surfacing

	Ulu-Sultan Hamud Rd	Stadium – Airport Rd	Uplands – Longonot Rd	Nakuru Highway
Testing Age	15	7	8	7
Penetration (0.1mm)	89	15	23	40
Saturates (%)	37	28	26	29
Naphathalene Aromatics (%)	40	24	20	28
Polar Aromatics (%)	5	16	25	16
Asphaltenes (%)	19	32	29	28

(b) Non bituminous subbase and materials

During the fieldwork on the environmental and traffic survey on the low volume roads, the base and subbase pavement materials were subjected to classification and strength tests. Conducting CBR tests on samples remoulded at field moisture content and field dry densities assessed the strength of the base and subbase. The field CBR was assessed by the dynamic cone penetrometre (DCP). This procedure modelled out the field strength conditions at the time of sampling. In recognition of the variation of the CBR with the changing moisture content, further CBR tests were carried out at optimum moisture content (OMC), ½ OMC and OMC with four days soak. This variation of testing condition modelled out the strength of the pavement from construction to the drying out periods and then to flooding. The case of flooding is modelled out by the testing of soaked samples. The flooding of a cracked pavement courses a rise in moisture content of the base and subbase materials. The field and laboratory test results are presented on the Table 3.12. The table shows that at the time of testing, relative densities were generally high except for Majengo whose relative compaction was 84 and 92 percent for base and subbase respectively..

Table 3.12 Field and laboratory test results for the base and subbase materials for the low volume roads

Test Section	Age (yrs)	Layer	Class	PI %	FMC %	FDD kg/m3	OMC %	MDD kg/m3	Field CBR by DCP	CBR at				Relative Density
										FMC (%)	½ OMC (%)	OMC (%)	4 day soak	
Oyugis	7	Base	GW	13	15	1670	17	1770	49	38	52	38	23	94
		Subbase	SC	18	19	1550	21	1620	22	19	20	17	9	96
Majengo	7	Base	GW	14	8	1630	12	1930	43	40	51	40	18	84
		Subbase	SC	20	17	1550	18	1690	24	19	22	19	12	92
Kwale	6	Base	SM	9	7	1980	10	1990	32	50	42	21	11	100
		Subbase	SM	12	8	1820	12	1860	25	43	32	12	7	98
Kisii	7	Base	GC	18	7	2010	13	1870	50	45	45	18	11	107
		Subbase	GC	20	17	1510	25	1530	53	61	25	10	3	99
Narok	6	Base	GC	18	3	2030	10	2020	126	120	46	33	20	100
		Subbase	SC	19	12	1540	21	1600	89	94	26	12	9	104
Ndum-beri	7	Base	GC	19	16	1800	17	1830	87	100	32	18	10	102

Legend: PI – Plasticity index, FMC – Field moisture content, FDD – Field dry density, OMC – Optimum moisture content, MDD – Maximum dry density, CBR – California bearing ratio, yrs - years

3.4 Non destructive pavement evaluation of high and low volume roads

3.4.1 General

Non destructive evaluation consisted of present serviceability rating, rutting, cracking, roughness and deflection measurements. Present serviceability, rutting and cracking, known as present surface condition was carried out on three heavily trafficked roads at two-kilometre intervals. The test roads were Nairobi-Thika (forty-two kilometres), Nyayo Stadium - Jomo Kenyatta Airport (twelve kilometres), Jomo Kenyatta Airport – Athi River (sixteen kilometres) and Jomo Kenyatta Airport – Nyayo stadium (twelve kilometres). The total length of the road investigated was therefore eight two kilometres. Figure A3.5 in appendix A shows the location of test roads and sites for the non-destructive tests.

Along the test roads, five test sites established by Gichaga (1979) and Atibu (1986) were subjected to deflection testing. Four other testing sites were also added to the deflection-testing program. Of the four test sites two tests were on the heavily trafficked roads (Langata road, and Limuru - Uplands road), while the other two were on low volume roads along Gatura – Mataara road. The construction details for these test sites are shown on the Table 3.13. The table shows these historical sites, namely ES1 through ES10 with an exception of ES8, which was not visited during this research. Plates A3.7 through A3.10 show some of these historical sites. Table 3.14 shows the traffic and maintenance history for the high volume test sections.

In addition to the deflection, rutting and cracking measurements, the low volume test sites on Gatura-Mataara road (ES9 and ES10) were visited in April 2000. Their condition was assessed by visual assessment and DCP soundings. This established the condition of the surfacing, the base and the underlying layers.

Table 3.13 Construction details for the deflection test section

Test site	Date of constr.	Location	Surfacing	Base	Subbase	Subgrade
High volume deflection test sections						
ES1	1961	Fox cinema Nairobi-Thika (A2)	DSD	300mm Cement stab gravel	450mm Natural gravel	Natural gravel
ES2	1965	Langata road	DSD	225mm Hand Packed stone	225mm Natural gravel	Rock
High volume test sites continued						
ES3	1977	Belle-Vue Cinema Airport-Lusaka (A107)	100mm AC	130mm GCS	200mm crushed stone	300 Improved subgrade
ES4	1977	General Motors Lusaka Airport	100mm AC	165mm GCS	165mm GCS	300mm Improved subgrade
ES5	1977	Kenchic Airport Athi river	100mm AC	200mm Hand packed stone	200mm soft stone	300mm Improved subgrade
ES6	1977	KMC Airport Athi River	135mm AC	200mm Hand packed stone	200mm soft stone	300mm Improved subgrade
ES7	1981	Limuru Uplands(km27)	50mm AC	150mm DBM	250mm Natural gravel	Red coffee soil
Low volume test sections						
ES9	1974	Gatura- Mataara (km 75)	DSD	130mm Crushed stone	100mm gravel	Murram
ES10	1974	Gatura- Mataara (km 80)	DSD	150mm Lateritic Gravel	-	Murram

Legend Stab – Stabilised, DSD – Double surface dressing, AC – Asphalt concrete

Table 3.14 Traffic and maintenance details for high volume test sites

	Traffic		Maintenance	
Test Site	Years of service	CESAx10^6	Years of service	Type of maintenance
ES1	29	16.0	25	135mm AC overlay
ES2	24	4.5	22	Slurry Seal
ES3	12	12	8	35mm AC overlay
ES4	12	7.32	8	35mm AC overlay
ES5	12	28.57	8	35mm AC overlay
ES6	12	28.57	-	-
ES7	8	8.52	-	-

Legend: AC – Asphalt concrete, CESA – Cumulative equivalent standard axles

3.4.2 Surface condition survey

(a) Present serviceability rating

Roads Department (1988a) spells out the rules for carrying out the present serviceability rating. People who understand the purpose of pavement rating and the rating method after explanation should carry out the rating. Accordingly the team of eight civil engineers and one typical road user carried out the PSR. The rating rules were spelt out to the raters. These rules are reproduced here as follows:-

i) The rater should consider only the present condition of the service.

ii) The rating should be based on the fact that the pavement carries mixed traffic under all types of weather conditions.

iii) The geometric characteristics of the road should be ignored.

iv) The skid resistance of the road should be ignored.

v) The rater should be concerned primarily with longitudinal and transverse distortion of the surface, potholes, cracking and patching.

vi) The rater should not give consideration to isolated conditions such as bumps, due to settlement at culvert and bridges, rough railway crossings etc.

vii) The rater should keep in mind the class of road on which the section is located when answering the question" Is the pavement of acceptable quality?"

viii) The rater should not refer to previously rated sections, or discuss the ratings or seek advice from anyone on the condition or the design of the section being rated.

The standard forms are shown on Figure 3.5. The ratings from 0 to 5 are assigned to each two-kilometre section. The value of 0 is assigned to very poor sections while 5 is assigned to the very good sections.

Present serviceability form

Road

Section   Date

Rating Scale		Is the pavement of acceptable quality?	
5		On the trunk system	Yes
....very good			No
4			Undecided
.........good		On the secondary	Yes
3			No
.........fair			Undecided
2			
..........poor			
1			
......very poor			
0			

Factors affecting your rating	None	Minor	Major
Longitudinal distortion			
Transverse distortion			
Cracking			
Others			
Remarks			

Figure 3.5 Present serviceability form.

Source: Roads Department (1988a)

(b) Pavement rutting measurement

Rutting was measured during the surface condition and deflection survey. The pavement deformation on the pavement surface was measured using a stiff 1.8-meter straight edge.

During the condition survey the rut depths were measured from the end of the straight edge at nine points viz.: - 0, 300, 600, 750, 900, 1050, 1200, 1500 and 1800-mm points. The rut straight edge was placed at the wheel path during the survey on the test roads. However during the deflection survey (section3.4.3) the straight edge was placed directly on the line of the rear wheels of the defection measuring truck. The rut measurements were carried out ahead of the deflection measurement to ensure that latter did not affect the rut depths.

In practice the rut depth is recorded as the deepest rut along the 1.8-metre edge. For this project the deepest rut together with the sum of the rut depths at the nine points were recorded. This summation is referred to as rut sum. The total rutting measured as rut summations across the section of four wheel paths were recorded as the total rut. Figure 3.6 shows the placement of the rut straight edge on the wheel paths for a two-lane road.

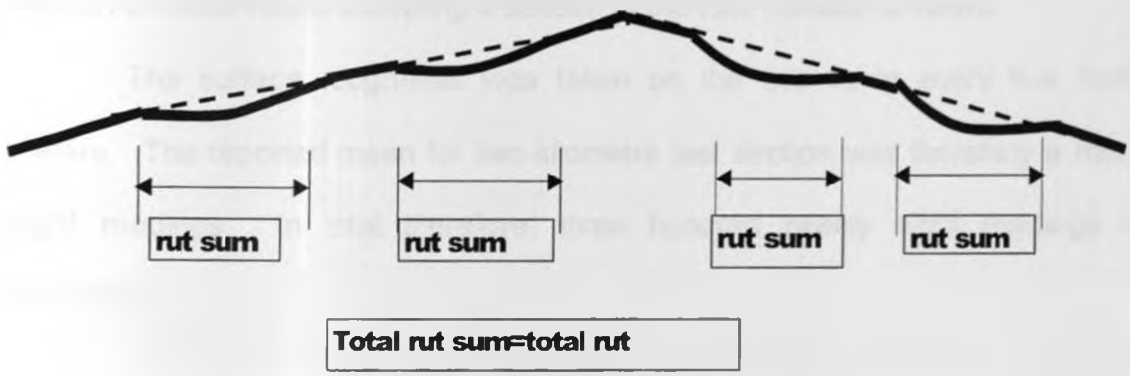


Figure 3.6 Placement of straight edge on wheel paths for rut measurement

(c) Pavement cracking

Cracking was measured during both the surface condition and deflection surveys using a 1mx1m square frame. The total length of cracks enclosed by the frame when placed at the test section was recorded as the cracking index in mm/m^2

(d) Surface roughness measurements

The towed bump integrator used by the roads department was used in the measurement of roughness. The downward movement of the wheel axle relative to the chassis is integrated to give the roughness of a five hundred-meter long wheel track. The test was conducted at a standard speed of thirty-two kilometres per hour (Roads Department, 1984).

(e) Surface condition evaluation results

A summary of the evaluation results for the present serviceability rating, rutting, cracking surface roughness and probability of acceptance is shown in appendix A, Table A 3.11. The mean of the present serviceability rating (PSR) for the five raters is the present serviceability index (PSI). The reported rut includes the rut on the two wheel paths and the total rutting. The surface cracking is the mean of cracking for the two wheel paths in a two-kilometre section. The probability of acceptance is the fraction of those raters accepting a section to the total number of raters.

The surface roughness was taken on the two lanes every five hundred meters. The reported mean for two-kilometre test section was therefore a mean of eight readings. In total therefore, three hundred twenty eight readings were assessed.

3.4.3 Deflection measurements

The equipment for measuring deflection was the benkleman beam developed in the USA and used in the AASHO road test between 1958 and 1960. In the Roads Department (1988a) specifications, which was also used in this study, a two axle loaded truck was used for deflection measurements. The rear axle load was adjusted to 6350 kilograms.

The points at which the deflection measurements were required were marked, the truck was then positioned parallel to the verge and with its wheels pointing straight ahead and its rear wheels placed directly on top of the pre-marked position (CSI). With the truck on this position two, the benkleman beams were inserted between the dual wheels. Figure 3.7 and Plate A3.11 in appendix A shows the wheel and beam arrangements. The beam measures 3.66 meter and is pivoted at 2.44 meters from the end next to the test truck.

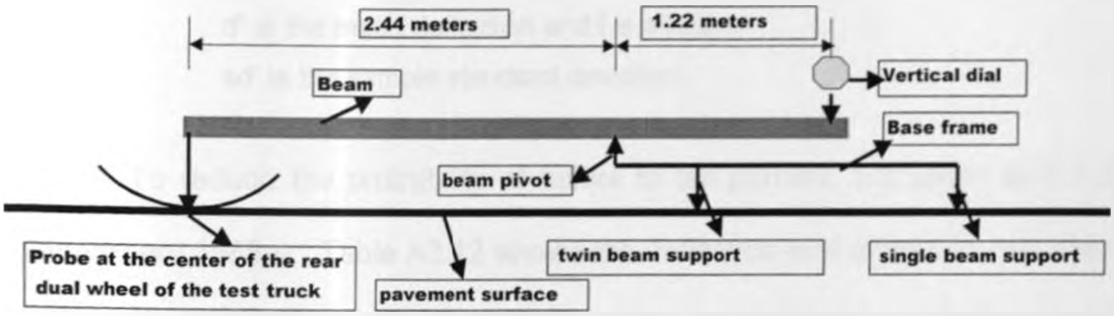


Figure 3.7 Benkleman beam arrangement

The truck was subsequently driven slowly and halted at 10 centimetre intervals to 50 centimetres and then to 1500 centimetres. Dial readings were taken at each stop. This enabled the deflection of the surface to be measured as the beam rotates about the pivot. The 1500 centimetre point was regarded as the second cross section (CS2). The whole procedure was repeated again, up to CS5, for one lane and then repeated for the second lane.

For every test site twenty readings were done. The performance of a pavement section is associated with the weaker areas of section. These are the sections, which are unable to carry the axle loads and subsequently deform. The deflection (d) measured by the benkleman beam arrangement is associated with the structural performance of the pavement. If the average value of d were taken, then evaluation of the pavement using this average value would result in using a deflection whose value is exceeded in half of tested sections. This would result in failure of the pavement structure. To counter this a characteristic deflection D_{90} is obtained from Equation 3.1

$$D_{90} = d' + fsd' \qquad [3.1]$$

where: D_{90} is the characteristic deflection

d' is the mean deflection and f is a factor,

sd' is the sample standard deviation

To reduce the probability of failure to ten percent, f is taken as 1.3 (Roads Department 1988a). Table A3.12 shows the deflection and offsets for test sites. The deflections and offsets are in relation to the deflected bowl created by the dual wheels of the test truck as shown on Figure 3.8 which shows a typical deflected bowl.

In line with Hall (1979) no measurements were taken when the temperatures soared above 45^{0}C. At those high temperatures the delay effects associated with the visco-elastic nature of asphalt are very significant.

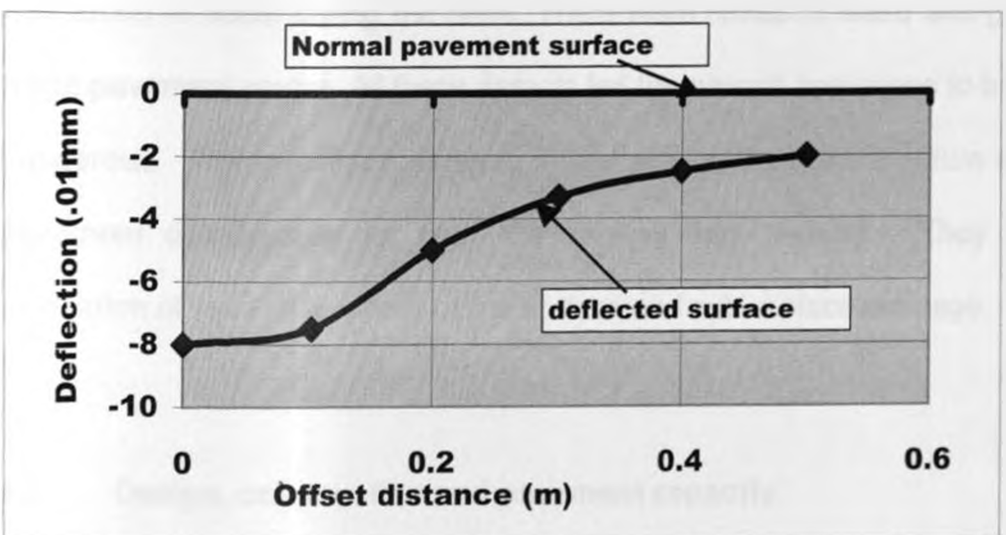


Figure 3.8 Deflected bowl

Chapter Four

Methodology and Data Collection – II

Case Study: Airport Flexible Pavements

4.1 Introduction

This chapter deals with investigation carried out on runaway and taxiway flexible pavements. The case study was conducted on an airport located approximately at the equator and 37^0 east. The airport pavements were found to have deteriorated due to cracking, crazing, distortion and disintegration. Due to the distortions pooling was found to occur during the rains. There were cases of weed and grass growing along pavement cracks. All these defects led the aircraft operations to be difficult and dangerous. The investigations were aimed at establishing the nature and extent of pavement deterioration for both the runway and taxiway. They included an evaluation of the effectiveness of the surface and sub-surface drainage.

4.2 Design, construction and pavement capacity

4.2.1 Design and construction

The runway and taxiway were designed in 1971 for a length of 2800m.The construction started in 1972 and was completed in 1974. A design to extend the runway to 4000m was done in 1976 and construction took place between 1977 and 1978.

During the initial construction fill material was got from one end of the runway. Classification, compaction and strength tests showed that the materials were light brown silty clay. The tests showed that these materials were slightly expansive during CBR tests on soaked specimens prepared at 100% BS compaction. The materials however were found suitable for use as fill. Gravel selected from quarries along a nearby river was used as improved subgrade material. Rock for use as crushed rock in bases and asphalt surfacing was got from a private quarry. Tests carried out on this rock gave the following results:

ACV: 16-17%,	LAA: 19%,	SSS	3%
FI: 26-35%,	Bitumen affinity:		good

The pavements were constructed on a minimum fill thickness of 500mm. From as built drawings the pavement structure was as shown on Figure 4.1. The design was carried out in accordance with the International Civil Aviation Organisation (ICAO), (1965) aerodromes manual part 2. The drainage during this first design consisted of base drains connected to 150 take off pipes below the pavement structure for the runway.

The runway was extended two years after opening. The embankments were formed from the light brown silty clay material used in the first construction phase. The materials were found to have a plasticity index between 20 and 54%. The liquid limit was ranging from 43 to 118%. The material had an average CBR of 8%. Observation of the two-year-old section was judged satisfactory. However due to the high liquid limit and plasticity index the material was compacted wet of optimum moisture content. This reduced the likelihood of swelling and reduction in bearing capacity upon the material receiving water from surface runoff through pavement cracks.

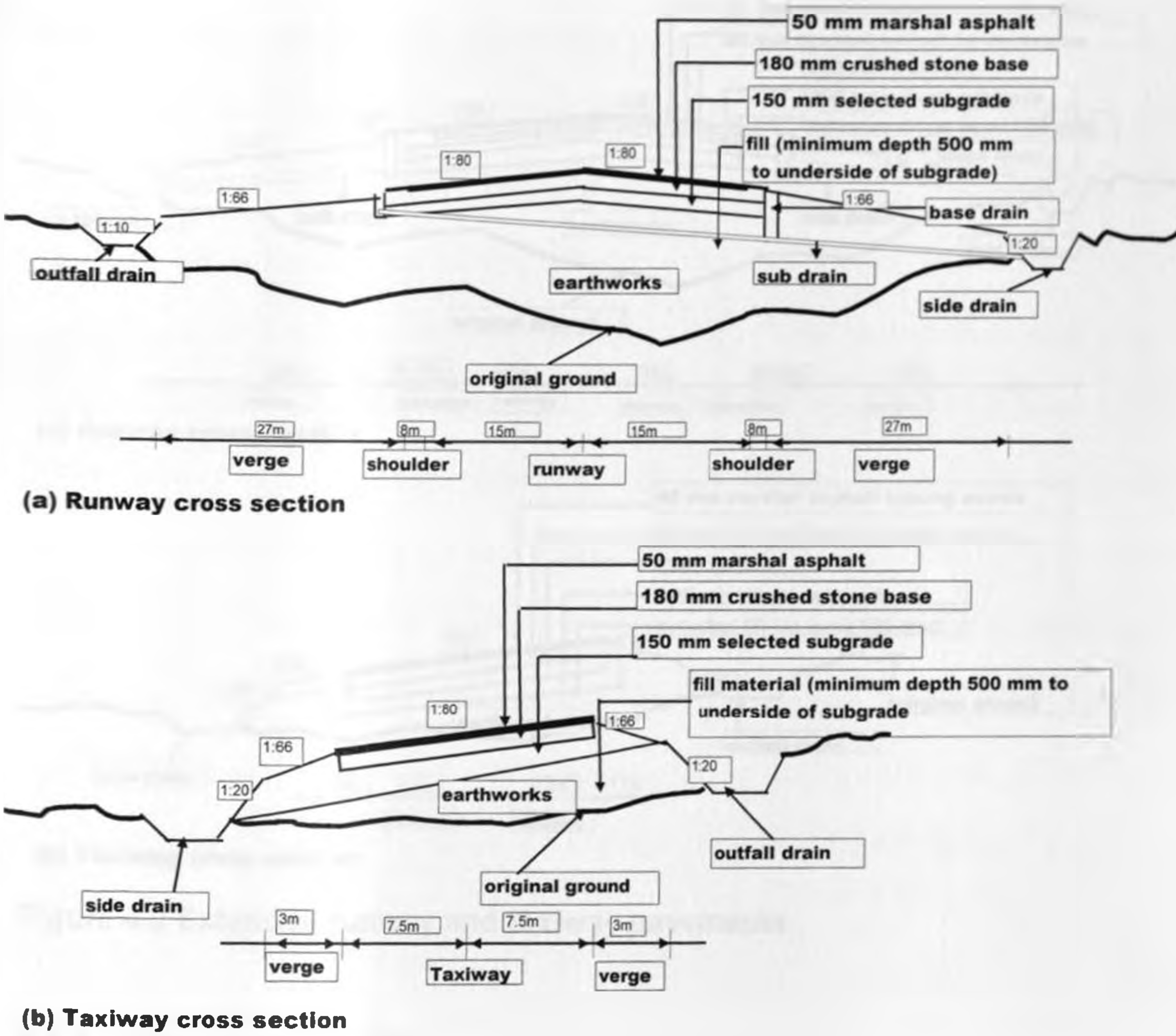
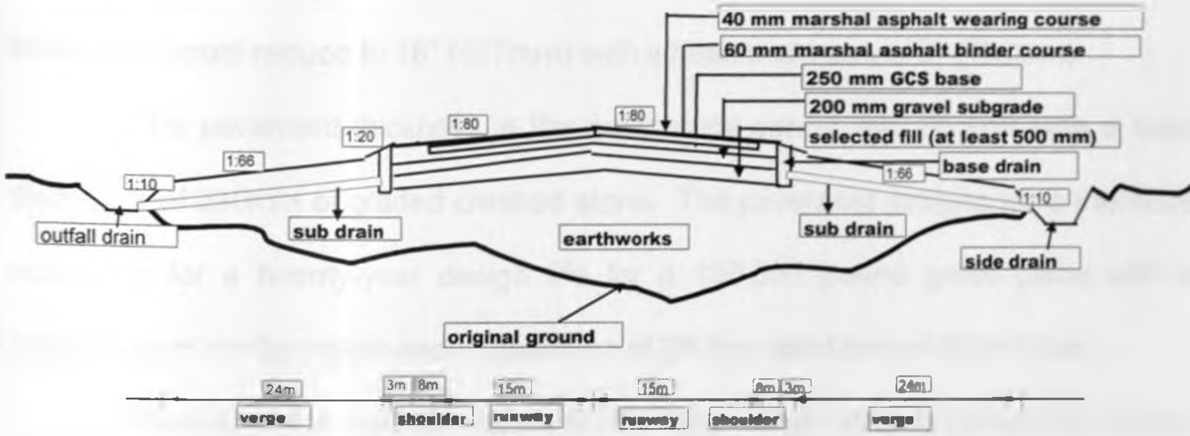


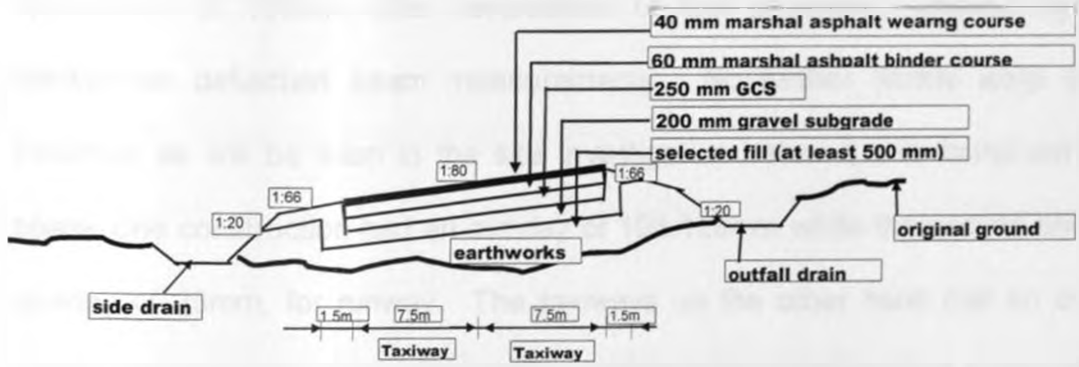
Figure 4.1 Flexible pavement cross sections after 1974 construction

A three meter verge to the pavement shoulders and a perforated drain connection through the verges to the out-fall was expected to shed off the water from the pavement surface. Sub-surface drains below the runway were not introduced in the new section. This reduced the possibility of water leakage from the sub-surface drains through the pipe perforations. The consultant recommended that the installed pipes in the phase one construction (km 0 to 2+800) be removed and the pavement repaired appropriately.

The pavement structure during this second phase is shown on Figure 4.2



(a) Runway cross section



(b) Taxiway cross section

Figure 4.2 Extended runway and taxiway pavements

4.2.2 Pavement capacity

Basic load and aircraft wheel gear configuration used in the design of the airfield pavements was not available. However by use of FAA method of design it was possible to do a back analysis and evaluate the structural capacity of the pavements. This was possible by use of the measured subgrade CBR of eight. Thus if it is assumed that the pavements were designed for a dual wheel gear, and if the gross aircraft mass of 50,000lb is considered as the design aircraft, the total pavement thickness required would have been 15 inches (380mm) with a base thickness of 6" (150mm). This would have twenty five thousand annual departures. If the gross aircraft of 100,000lb for dual wheel gear configuration is considered the total pavement would have been 24" (610mm) with a base thickness of 8" (200mm). For the same gross weight of 100,000 pound and a tandem gear configuration the total thickness would reduce to 18" (457mm) with a base thickness of 6" (150mm).

The pavement thickness in the case study airport was 550mm with a base thickness of 250mm of graded crushed stone. The pavement designs were therefore adequate for a twenty-year design life for a 100,000 pound gross plane with a tandem gear configuration and a maximum of 25 thousand annual departures.

Perusal of the materials report for the latter construction proposed an overlay application of 100mm after revaluation of the pavement strength by use of benkleman deflection beam measurements. No further details were available. However as will be seen in the site investigation below it was found out that the phase one construction had an overlay of 100-125mm while the second one had an overlay of 25mm, for runway. The taxiways on the other hand had an overlay of 175mm for the phase one construction and 50mm for the phase two construction.

4.3 Visual inspection

4.3.1 Cracking and surface disintegration

Surface cracking was the most significant form of distress for the pavements. Many types of cracks were evident. The most widespread was longitudinal cracking. In one case a 50mm wide cracking extending 150mm into the surfacing was noticed.

Many of the longitudinal cracks appeared on the construction joints, which seemed to have been improperly done during construction. In other cases the longitudinal cracks formed due to differential vertical movements between pavement slabs. Painted areas marking the runway and taxiway pavement line edges showed severe longitudinal cracking. This cracking was associated with differential expansion between the bituminous material and the paint. It was also observed that some cracking is associated with the landing lights.

The pavement surface also showed coarse and fine crazing, isolated stripping and surface disintegration. The bituminous wearing coarse appeared to be open textured. It was clear some of the aggregates used to produce the mix had weathered and appeared slightly brown and decomposed.

Plate A4.1 in appendix A shows typical cracking noted during the visual inspection.

4.3.2 Pavement distortion and ponding

Cases of depression on the runway and taxiway were noted. These depressions resulted in ponding of rainwater. In some cases the depressions were accompanied by cracking. Plate A4.2 in appendix A shows such typical ponds. A surface heave on the pavement surface was observed at chainage 3700m.

4.3.3 Drainage system

The drainage system for the pavements consisted of surface and sub-surface systems (Figures 4.1 and 4.2). Large culverts carrying storm-water from the base with an out-fall discharging into a side drain located between the runway and taxiway were the main exit drains.

From chainage 0 to 2800 meters the surface run-off collected on the base drain which, was in form of a subsoil drain. The subsoil drain was then provided with manholes located at low points (Plate A4.3 in appendix A). The extended runway from chainage 2800 meters to 4000 meters had the drain done in pre-cast concrete drains. The pre-cast drains emptied into manholes (Plate A4.4 in appendix A). The surface drainage runs parallel to the taxiway and the runway. The area was found to be heavily overgrown with weed. It was not receiving regular maintenance.

Take-off pipes were provided to collect water from the base drains running parallel to the runway and the taxiway. They were intended to drain into the side drains. After opening collection manholes along the pavement edge, it was found out that some drains were not working. The drains were not regularly cleaned. Some of the manhole covers could not be opened suggesting that they had not been opened for a long time.

Overall it appeared that the surface water was finding its ways into the pavement structure through the many cracks. In a number of cases weed and grass were seen growing along such cracks (Plate A4.5 in appendix A). The weed appeared to have resulted from presence of mud slurry along the cracks.

4.3.4 General condition of the pavement

It was evident that the pavement structure and the drains were not subjected to effective and regular maintenance. There were cases of slurry seal, which was meant to make up the depressed areas, having stripped. Basic weed cutting, weed control and crack sealing were absent or poorly done.

Despite the many distress features and poor maintenance bordering on neglect, the runway and taxiway pavements appeared to be in reasonable geometry. The main distress features required to be identified for immediate rehabilitation. Surface and sub-surface drainage required improvement and opening up to enable any water trapped within the pavement structures to be drained away.

4.4 Preliminary investigations

4.4.1 Introduction

The general users of the runways and taxiways are pilots. For these investigations, interviews with the pilots who use the facility were conducted. This was followed by car running tests at 100km/hr to model out planes at high speed. The preliminary investigations were concluded by a ground survey to establish the cross section survey details at 20-m intervals. In areas identified by visual inspection, pilots and car running tests as severely distorted the survey intervals was reduced to five meters. The results of these preliminary investigation can be summarised as follows:

4.4.2 Runway

(a) Cross section 175-575 meters

The pavement in this section was found to have been severely distorted. There were many depressed areas, which pooled after rains. Longitudinally there were several distortions, which gave the feeling of humps when the car was at high speed. This section was generally very busy for both take-off and landing. The pilots identified the section as dangerous.

(b) Cross section 1380-1480 meters

The pavement around this section was found to suffer from a longitudinal distortion around the construction joints. It was found out that the distortion could be removed by scarifying the surface and reconstruction of the section

(c) Cross-section 3170-3270 meters

As in the previous section the pavement here suffered longitudinal distortion in the form of a hump.

(d) Cross section 3520-3820 meters

The section was found to be severely distorted with many areas attracting pools of water. Again the longitudinal distortions gave the feeling of humps when driving at a high speed. This section was described by the pilots as dangerous.

4.4.3 Taxiway

Cross-sections 1250-1600,1925-2150 and 2350-2550 meters

These sections of the taxiway showed severe distortion, indicating the pavement was severely distressed. The pavement was found to have widespread cracking.

4.4.4 Overall assessment

Safe for the above seriously distorted sections geometric analysis revealed that the geometric design of the pavements were generally within the design proposals presented in section 4.1

4.5 Field and laboratory investigations

4.5.1 Introduction

The field investigations were aimed at establishing the pavement design and construction sufficiency. In addition the maintenance condition of the pavement structure, paved shoulders, verges and the drainage system was to be determined. The data obtained together with subsequent laboratory testing enabled a rational analysis and recommendation of remedial measures. The experience gained here can be drawn for similar cases.

4.5.2 Field sampling and testing for non-bituminous materials

Sampling was done in the field for the following elements.

i) Materials below the bituminous pavement layer

ii) Base drains, verges and side drains

iii) Pitting for the water table

(a) Materials below the bituminous pavement layer

Materials below the bituminous layer were sampled by core drilling. The location of the boreholes is shown on Figure 4.3. During the core drilling exercise the standard penetration test (SPT) was carried out to determine the bearing capacity in terms of SPT count (N). The SPT specimens were taken to the laboratory for moisture content determination.

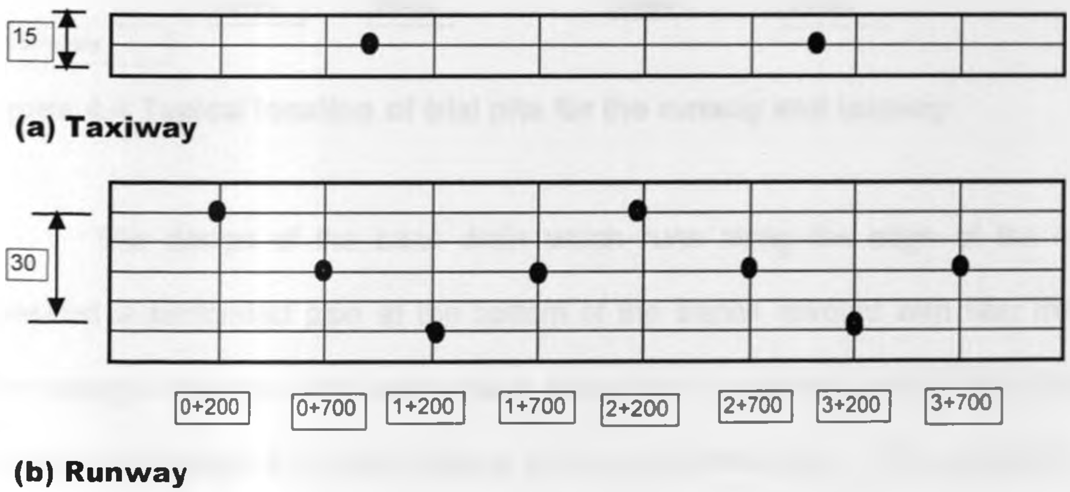


Figure 4.3 Location of core drilling points for the taxiway and runway

(b) Base drains, verges and side drains

Sampling for the base drains, verges and side drains were carried out by trial pitting. The cross sectional location of the trial pits is shown on Figure 4.4. During the sampling, in-situ densities were determined. The materials recovered were visually described to provide information on their thickness, texture, and colour etc. Samples for laboratory test were taken for the determination of the field moisture contents, particle size distribution and permeability on samples remoulded at field density, for the cohesionless samples. Undisturbed tube samples for the cohesive materials were taken for laboratory testing. The significant observations during the sampling and laboratory testing are discussed in chapter five.

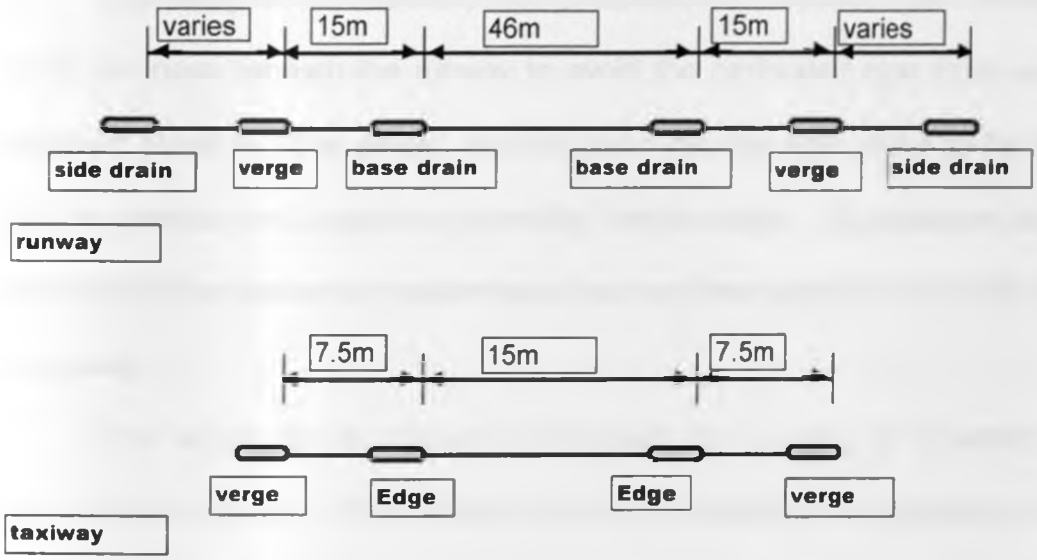


Figure 4.4 Typical location of trial pits for the runway and taxiway

The design of the base drain which runs along the edge of the runway specified a perforated pipe at the bottom of the trench covered with filter material. The design assumed that sub-surface water from pavement would drain into the base drain through the filter material to the perforated pipe. The perforated pipe would then drain into manholes, which were located along the edges of the runway from where normal drainpipes would direct the water to the side drains (Figure 4.2)

The base drains were excavated at intervals along the runway to establish their properties. In general the base drain had a top layer of about 100-mm made up of black clay soil. The filter media was mainly river aggregate with pebbles. There were however sections of the pipe which were mixed up with clayey silt and sandy silt. The location of the perforated pipe was in some places found misplaced horizontally and sometimes broken. In a number of cases it was found out that water had accumulated at the bottom of the base drain away from the perforated pipe. It appeared that the base drain was not effectively taking the water from the pavement structure.

The design in the second phase of construction required that the base drains should not pass beneath the runway to avoid the perforated pipe from wetting the pavement structure. The design also required that the filter material for the base drain be granular and capable of providing free drainage. Observations at the field indicated that the above key requirements had not been satisfied in the first 2000m of the runway.

The verges for the runway and taxiway were made up of sandy silt with traces of clay materials. The results for field and laboratory investigations are shown on Table 4.1

Table 4.1 Properties of verge materials

			Runway			**Taxiway**	
X-Section (m)	LL (%)	PI (%)	Permeability (cm/sec)	Field		Field	
				MC (%)	CBR (%)	MC (%)	CBR (%)
1000L	52	16	$2x10^{-6}$	16	38	22	33
R	39	7	$1x10^{-6}$	11	16	14	28
R	44	13	-	19	16	-	-
1500 L	49	18	$2x10^{-5}$	15	33	31	17
R	48	16	$2x10^{-5}$	18	22	18	35
2000 L	45	26	$1x10^{-5}$	18	32	15	50
R	46	11	$2x10^{-5}$	18	30	15	47
3000 L	43	15		15	48	28	80
R	42	16		14	43	31	60
3500 L	62	11		34	14	23	30
R	97	48			16	18	26

Legend L – Left verge, R – Right verge

The results show that the verge materials were silts of low plasticity except for the verge materials at 3500m, which were clays of high plasticity. The CBR as estimated by DCP soundings (TRRL, 1990) showed strong verge materials with CBR values ranging between fourteen and sixteen. The strength can be attributed to the generally low field moisture content, which was found to lie below the plastic limit for materials. The verge materials had low permeability values in the range $1x10^{-6}$ cm/sec to $2x10^{-5}$ cm/sec.

(c) Pitting for water table

It was found necessary to confirm whether the high moisture content encountered below the pavement was from rain precipitation through cracks or was from underground. For this purpose bore-holes to establish the presence of the water table were dug on the runway verge at cross section 200m and 3200m. The borehole at 200m was dug using a hand auger to a depth of 3 metres. The hole was then left overnight capped at the top for collection of water at the bottom. In case of cross section 3200 the hole was dug by picks due to hard strata which could not be penetrated by the hand auger. Again the hole was left overnight for water to collect at the bottom

In both cases no water had collected at the bottom and it was therefore concluded that the water table was deeper than 3m below the ground surface. This reconfirmed findings reported in the site investigation for design of the pavement when no water table was encountered up-to a depth of 3.6 metres.

4.5.3 Field sampling and testing of bituminous materials

(a) Introduction

Pavement core drilling was carried out to obtain information on the pavement structure. This procedure enabled samples of the pavement structure to be collected from the field intact (Plate A4.6 in appendix A). These samples were then transferred to the laboratory for various tests. The samples were separated into materials from the wearing courses, binder courses and the original layer.

The tests carried out on the samples included determination of the density, bitumen content, voids in total mix and grading of the aggregate fraction in the bituminous mixtures. The recovered bitumen was also subjected to determination of penetration and softening points. Figure 4.5 shows the mean grading of the aggregate portion of the bituminous layers. The material grading fell in the 0/40

Roads Department (1987) envelope. Table 4.2a shows the relationships of bitumen content (BC) with voids in total mix and bulk density while Table 4.2b shows the variation of penetration with voids in total mix and softening points for the samples.

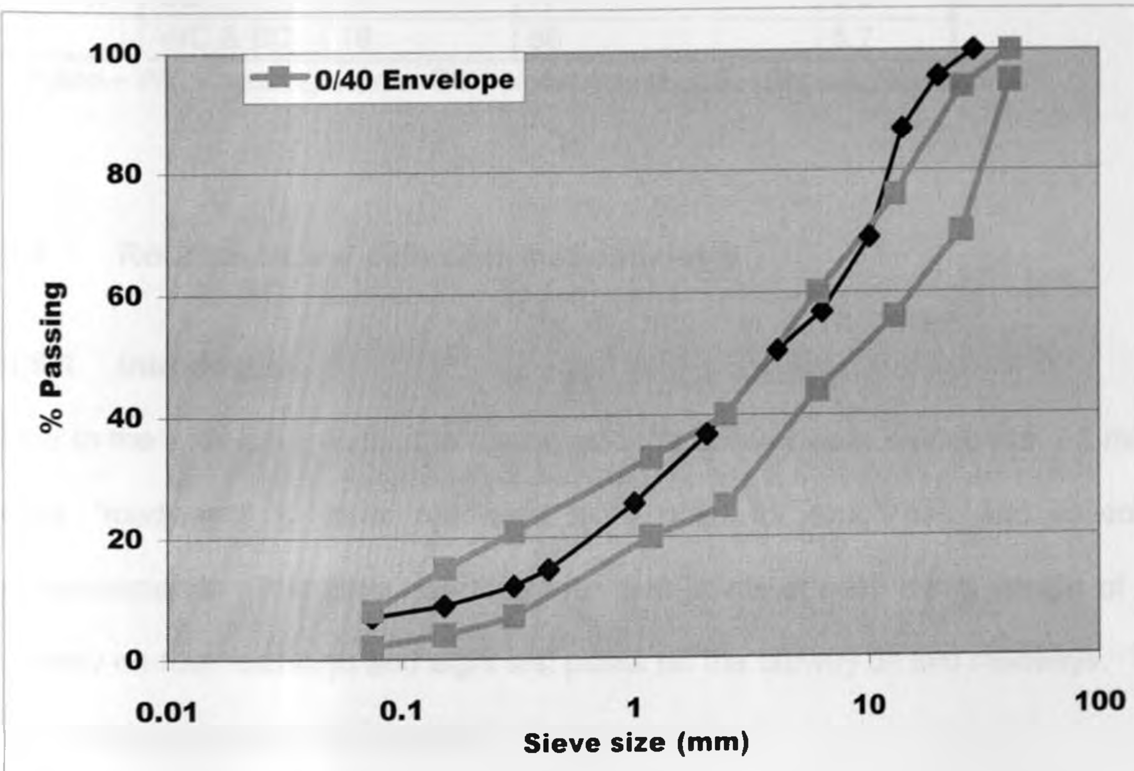


Figure 4.5 Mean grading of the aggregate fraction of the bituminous mixtures

Table 4.2 Test results for bituminous pavement materials

a) Bitumen content, voids in total mix and density

B/ No.	Cross Section	Wearing			Binder Course			Original before overlay		
		BC %	VTM %	BD gm/cc	BC %	VTM %	BD gm/cc	BC %	VTM %	BD gm/cc
i) Runway										
1	200	6	5.7	2.19	5.5	6.5				
2	700	6	1.7	2.29	5.5	6.5	2.21	5.9	6.1	2.19
3	1200	6	4.6	2.22	5.5	7.2	2.19	5.9		2.16
4	1700	6	11.6	2.06	5.5	8.5	2.16	5.9		2.27
5	2200	6	4.3	2.22	5.5	6.2	2.22	5.0	9.4	2.22
6	2700	6	2.5	2.27	5.5	7.2	2.19	5.0	10.2	2.09
7	3200	6	6.80	2.17	5.5	7.2	2.15	5.0		2.23
8	3700	6	7.30	2.16	5.5	9.2	2.25	5.0		2.17
ii) Taxiway										
	1100	6	3.7	2.24	5.5	6.5	2.21	5.9	6.9	2.17

Legend: - BC – binder content, VTM – Voids in total mix, BD – Bulk density

b) Penetration, softening point and mean voids in total mix

B/Nos	Layer	Penetration	Softening Point (°C)	VTM %
2,3,4	WC	19	69.6	6.0
2,3,4	BC	17	70.1	7.4
2,3,4	OB	16	70.1	6.1
5,6,7,8	BC	11	78.0	7.5
5,6,7,8	OB	11	77	9.8
1	WC & BC	19	68	5.7

Legend – WC – wearing course, BC – binder course, OB – Original base-course

4.6 Roughness and deflection measurements

4.6.1 Introduction

Due to the wide pavements, the runway and the taxiway were divided into 7.5 meter wide "roadways". These roadways were used for roughness and deflection measurements. This gave rise to sixteen test points at each cross section of the runway on four roadways and eight test points on the taxiway on two roadways. The test points are shown on Figure 4.6.

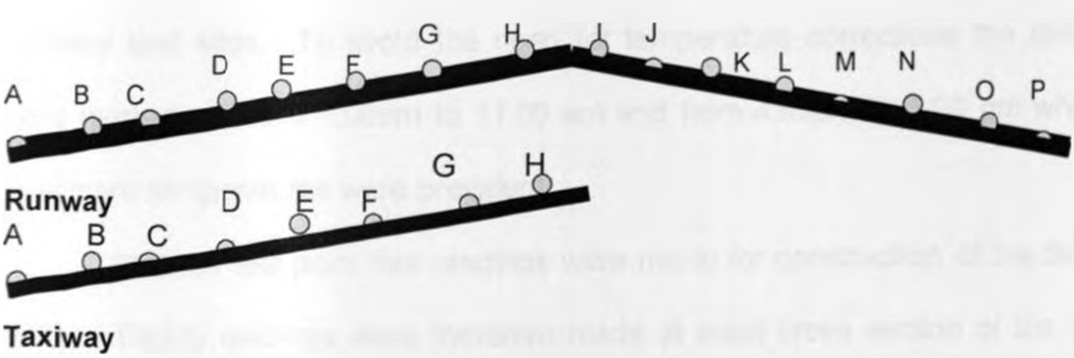


Figure 4.6 Test points for roughness and deflection tests

4.6.2 Roughness

The bump integrator equipment used in the highway test sites was used for roughness assessment in the airport flexible pavements. The results of roughness measurements (Ro) are shown on the histogram and cumulative frequency of roughness on Figure 4.7.

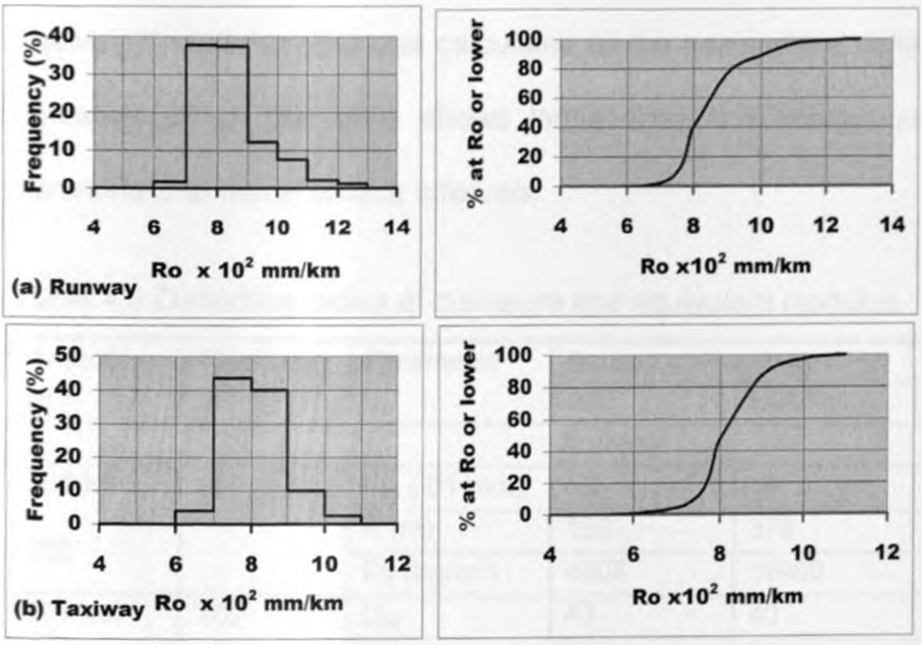


Figure 4.7 Roughness scatter for runway and taxiway

4.6.3 Deflection measurements

Pavement deflection was measured using the same equipment as that for the highway test sites. To avoid the need for temperature corrections the deflections were carried out from 7.00am to 11.00 am and from 4.00pm to 6.00 pm when cool pavement temperatures were prevalent.

At each test point five readings were made for construction of the deflection profile. Eighty readings were therefore made at each cross section of the runway. The taxiway had forty readings on each cross section readings. In both runway and taxiway test cross sections were spaced at 100m intervals over the four kilometre lengths. In total the runway had 37 test sections with 2960 readings while the taxiway had 38 test sections with 1520 readings. With this high volume of data, statistical analysis was justified to arrive at pavement deflection characteristics.

Table 4.3 shows processed test results for four roadways of the runway and two roadways of the taxiway. As for the highway test results the characteristics deflection (D_{90}) is defined as the mean deflection plus 1.3 standard deviation. The equivalent modulus (Eq) was calculated on the basis of the deflections and radius of curvature (R). The table shows longitudinal and lateral variation of the three parameters at half kilometre intervals.

Table 4.3 Deflection radius of curvature and equivalent modulus

X-Section (m)	Thickness (mm)	Parameter	Section of Roadway			
			A-D	E-H	1-L	M-P
			Runway			
200 to 600	150	D_{90} (.01 mm)	53	44	47	58
		R (m)	159	379	253	213
		Eq (kg/m^2)	4008	18460	8738	7436
700 to 1100	175	D_{90}	43	40	35	46
		R	288	290	56	244
		Eq	8361	8190	1646	6559
1200 to 1600	175	D_{90}	45	33	42	45
		R	302	438	262	305
		Eq	9356	14985	7066	9504
1700 to 2100	150	D_{90}	36	27	34	33
		R	311	490	330	341
		Eq	10594	20815	11614	12068
2200 to 2600	150	D_{90}	42	36	41	41
		R	202	250	221	206
		Eq	5352	7039	6206	5481
2700 to 3100	150	D_{90}	58	49	55	56
		R	167	252	274	183
		Eq	5968	12346	17037	7034
3200 to 3600	125	D_{90}	58	49	55	56
		R	167	252	274	183
		Eq	5965	12346	17037	7034
3700 to 3800	125	D_{90}	57	25	20	53
		R	62	560	818	239
		Eq	1077	35567	64794	12027

X–Section (M)	Thickness (mm)	Parameter	Section of Roadway	
			A-D	E-H
Taxiway				
325 to 800	225	D_{90}	68	56
		R	76	150
		Eq	1205	2698
825 to 1300	225	D_{90}	49	38
		R	193	416
		Eq	3697	10249
1325 to 1800	225	D_{90}	39	42
		R	319	174
		Eq	7042	3151
1825 to 2300	225	D_{90}	35	76
		R	343	487
		Eq	7569	16649
2325 to 2800	225	D_{90}	33	34
		R	196	332
		Eq	3563	7177
2825 to 3300	225	D_{90}	55	48
		R	114	144
		Eq	2326	3178
3325 to 3800	150	D_{90}	55	47
		R	68	109
		Eq	1179	2058
3825 to 4000	150	D_{90}	43	32
		R	172	359
		Eq	4102	12767

Chapter Five

Analysis and Discussion of Test Results

5.1 Introduction

5.1.1 General

The methodology and data collection with respect to flexible road pavements and flexible airport pavements was presented in chapter three and chapter four respectively. This chapter presents the analysis and discussion of the test results of both the road and airport pavements in the following order:

- Subgrade, and non bituminous subbase and base materials
- Bituminous mixtures
- Effect of environmental factors on flexible pavements
- Pavement condition surveys
- Performance and remedial measures of test roads
- Deflection data analysis
- Case airport flexible pavement investigations
- Finite element analysis
- Conclusions of test results and analysis

5.1.2 Analyses of data

Statistical quality control and in-situ measurements have pointed to large variability, which exists in properties of pavement materials after construction. Even if perfectly homogenous and uniform materials were tested, some variability would occur (Swami et al 1991). The variability in engineering properties of pavement material depends upon many factors, such as traffic loading forecast variations, natural moisture content, methods of sampling and testing, characteristics of the material, geographical and environmental factors. Data in this research has been collected over a long period and across the breadth and width of Kenya. In regression analyses by SPSS, linear and non-linear relationships are tested. The relationships returning high coefficient of determination and low signf F are chosen and are reported in Appendix B. The main inference of the test results is however made in the sections of the analyses below. Good correlations have been accepted when high r and r^2 are accompanied by signf F less than 0.05.

5.2 Subgrade and non-bituminous subbase and base materials

5.2.1 Geotechnical properties of Kenyan subgrade materials

(a) Introduction

A large volume of geotechnical data was processed to yield the pavement characteristics. In light of this large volume of data, a by-product of the main research was conceived. This was to determine the geotechnical properties of the analysed soils. The properties established include classification and compaction characteristics. In addition a quick method of determination of optimum moisture content and maximum dry density for earthworks has been proposed.

Samples for testing of subgrade materials were collected from the field, below the existing pavement structures and under different environmental conditions. Test results for these samples exhibited more variations than would have been expected if the materials were collected from areas with the same environmental conditions. However the test results have returned good correlations for the developed relationships in line with correlations reported by Swami et al (1991) .

(b) ***Classification of fine-grained soils (black cotton soils and red coffee soils).***

The results of the plasticity test are plotted on the plasticity chart for the unified classification system (Smith and Smith, 1998) on Figure 5.1a and 5.1b.

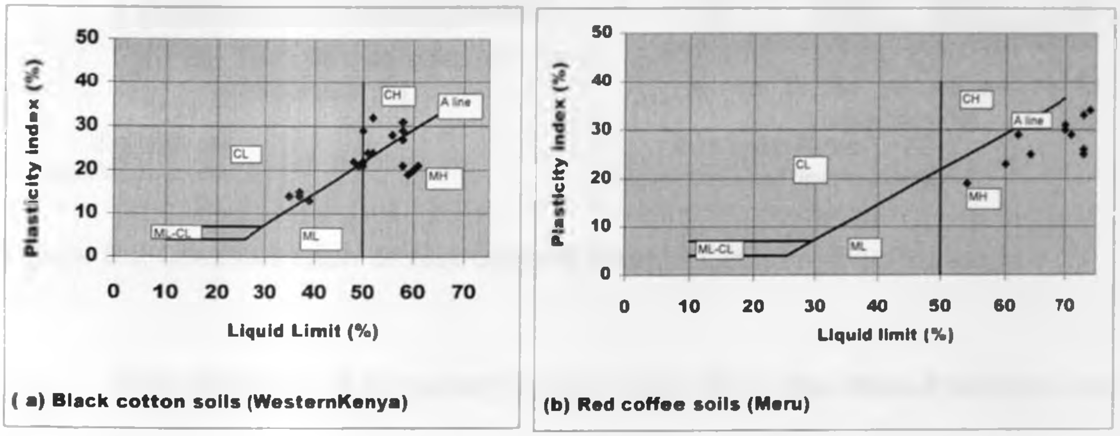


Figure 5.1 Classification of the fine grained soils

For the black cotton soils from Western Kenya it was found that the soils band around the A line. The majority of the samples were found to lie above the 50% LL. These soils are therefore, clays or silts of high compressibility (CH and MH) Equation 5.1 shows the correlation of PI and LL for the investigated soils. The red coffee soils from Meru district were found to plot below the A line also above the 50% LL. These soils are therefore silts of high compressibility. Equation 5.2 again shows the correlation of PI and LL for the red coffee soils.

$$PI = -0.5 + 0.45LL \quad (r = 0.64) \qquad [5.1]$$

$$PI = -15.62 + 0.64LL \quad (r = 0.84) \qquad [5.2]$$

Comparison with Nairobi red coffee soils which were investigated by Gichaga et al (1987) and Kipwen red coffee soils (Mwea and Ngware 1996) in Uashin Gichu district of the Rift valley shows that these soils are similar as can be seen on Figures 5.2a and 5.2b.

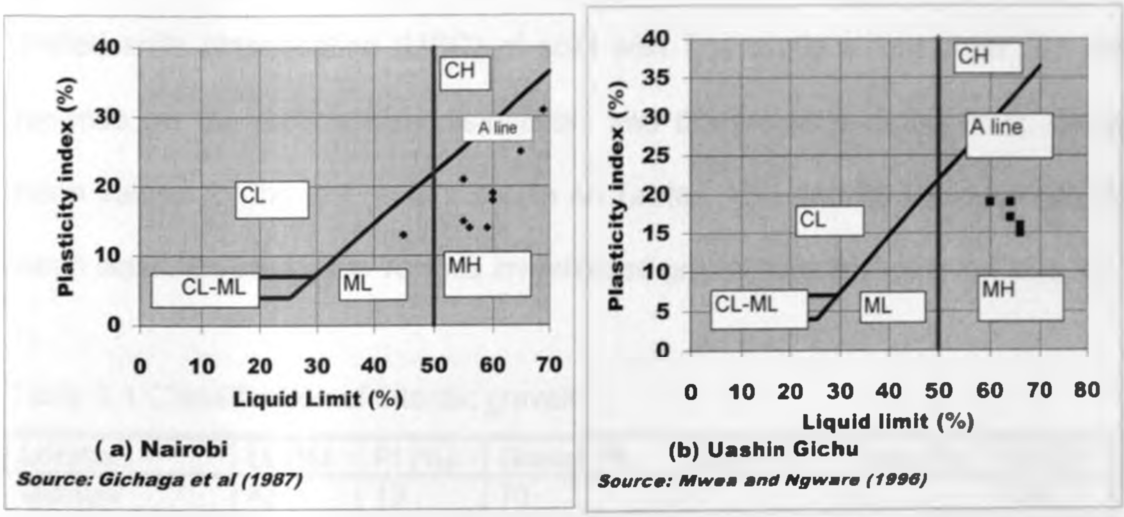


Figure 5.2 Classification of Nairobi and Uashin Gichu red coffee soils

(c) Classification of the sandy loamy soils along the Wote-Makindu road

The variation of the geotechnical properties of the subgrade soils along the sixty five-kilometre road corridor was found to be large (Table A3.3 in appendix A). The fine portions of soils were found to be of low plasticity with an average PI of 14%. Overall, the sand proportion was found to be high (58%) with high proportion of fines plotting above the A line and below the 50% liquid limit mark (Figure 5.3). The soils can therefore be classified as clayey SANDS (SC).

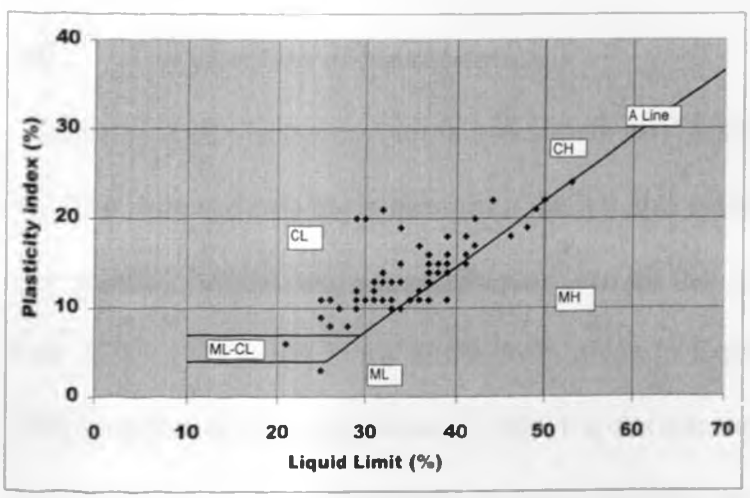


Figure 5.3 Classification of sandy loamy soils on the Wote – Makindu road

(d) Classification of the lateritic gravel soils

Unified soils classification (USC) of soils with fine portions less than fifty percent depends on the particle size distribution, and the plasticity of the fines. Using the mean values for the test results shown on Tables A3.4 and A3.5 in appendix A, the mean classification for the various investigated gravel soils is shown in Table 5.1

Table 5.1 Classification of lateritic gravels.

Location	LL (%)	PI (%)	Gravel (%)	Sand (%)	Fines (%)	USC
Mumias	42	13	70	17	13	GM
Amagoro	35	11	73	16	11	GM-P
Wote-Makindu	38	15	50	32	18	GC

Legend: – LL – Liquid limit, PI – Plasticity index

Gravel from Western Province (Mumias and Amagoro) was found to have lower percentage of fines than the gravel from quarries located along Wote-Makindu road in Eastern Province. The Mumias gravel can be classified as silty GRAVEL (GM), while that from Amagoro had less than 12% fines but more than 5%. It is therefore described as a poorly graded silty GRAVEL (GM-P). The Wote-Makindu quarries had a higher percentage of sand and fines. These gravels are classified as silty GRAVEL (GM).

(e) Compaction characteristics

The results of compaction tests are shown on Tables A3.1 through A3.5 in appendix A. The optimum moisture content (OMC) is the water content at which the maximum dry density (MDD) has been achieved. In all the tested soils the variation of OMC with MDD was found to be of the form given in Equation 5.3. Statistical analysis of OMC versus MDD was done. Table 5.2 shows the statistical constants related to Equation 5.3. The results for the general equation is shown graphically on Figure 5.4

OMC = A – MDD/B ***[5.3]***

Table 5.2 Statistical constants for MDD Vs OMC (Equation 5.3)

Soil Type	A	B	r
Black cotton Soils	71	32	-.85
Red coffee soils	84	27	-.92
Lateritic gravel soils	66	38	-.89
Clayey sands	54	47	-.92
General equation	66	37	-.93

Legend – A & B –constants, r – coefficient of correlation

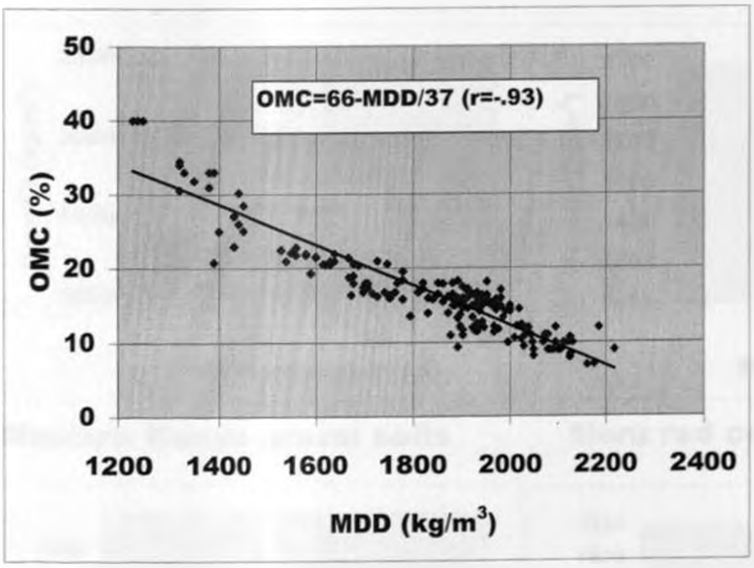


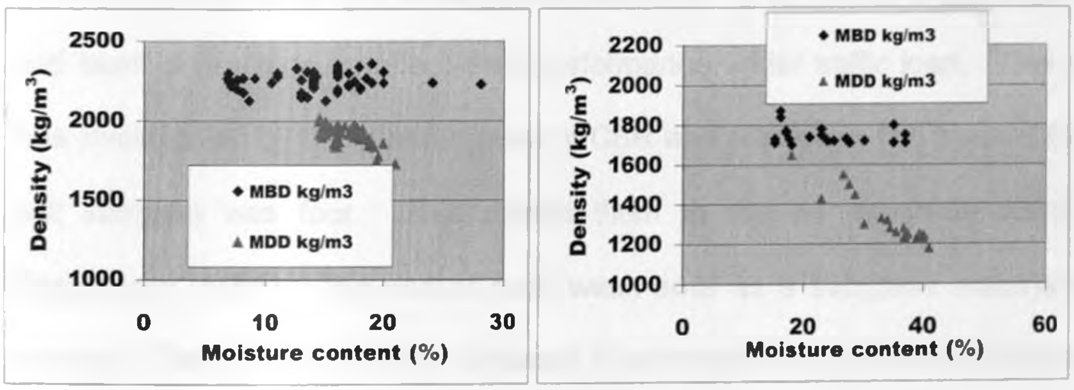
Figure 5.4 OMC vs MDD for Kenyan subgrade soils

(f) Quick determination of optimum moisture content for earthworks

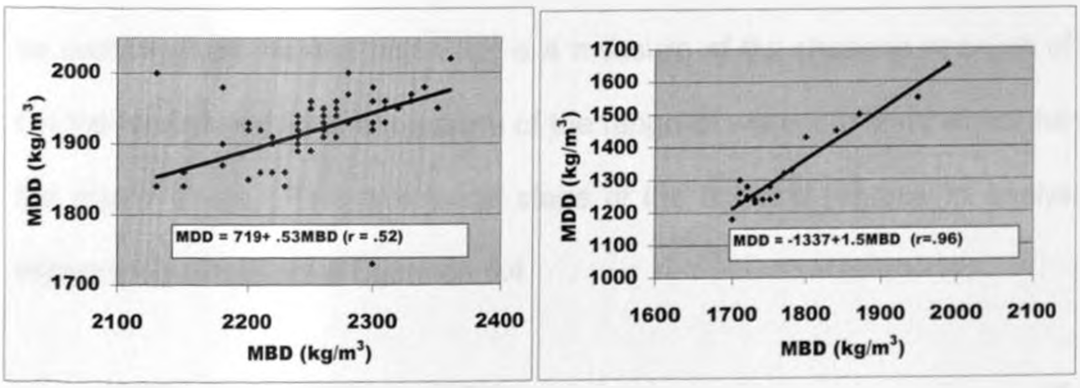
BS 1377(1990) spells out that in the determination of moisture content, oven dried samples should be weighed every four hours. When the drop of the weight of the container and sample is less than 0.1% the value of moisture content is recorded.

The above procedure is not commonly used in practice. Instead, the samples are left overnight in ovens up-to sixteen hours. The result is that there are delays in processing results for the purpose of approval or rejection of compacted earthworks. There is also attendant high consumption of energy required to keep the ovens at 105^0C through the drying period, even when the four-hour interval is observed.

Raw compaction data was processed to give the percentage of added water to the bulk weight during dry density determinations. This percentage is referred below as quick moisture content (QMC). The bulk density (BD) plotted versus the QMC gives a quick plot, which has been lifted above the normal dry density (DD) versus moisture content (MC) plot (Figure 5.5).



Western Kenya gravel soils **Meru red coffee soils**



(a) Western Kenya gravel soils **(b) Meru red coffee soils**

Figure 5.5 Density – Moisture content relationships

The aim of the analysis was to formulate a relationship of obtaining MDD and OMC from the quick plot which, is done minutes after completing the proctor test. A plot of QMC versus BD on all the test results showed a maximum BD referred as

MBD at a QMC referred to as OQMC. Figure 5.5 shows the graphical presentation of MDD versus MBD and inset equations which, can be used for determination of MDD for lateritic gravels and red coffee soils. Equation 5.3 with the relevant statistical coefficients from Table 5.2 would be used for the determination of OMC.

5.2.2 Properties of the fine-grained soils with respect to pavement performance

(a) Black cotton soils

The black cotton soils tend to be problematic soils. Their low strength, high plasticity and swell characteristics affect their performance under traffic load. Their strength was investigated by the measurement of CBR and plasticity. The mean CBR for the test samples was four. This placed them in the S1 subgrade class (Roads Department, 1987). The use of such weak soils as a subgrade material is to be avoided. They should be either replaced or an improved subgrade is used as cover.

The CBR was found to be linearly related with plasticity index (PI). This can be explained by the fact that CBR is a measure of the shearing strength of the soil. On the other hand PI is a measure of the range of water contents when the soil is in the plastic stage. This is a weak stage of the soil and regression analysis of the experimental data gave Equation 5.4

$$CBR = 11 - 0.33PI \; (r = 0.82) \qquad [5.4]$$

(b) Red coffee soils

The investigated red coffee soils had their CBR values in the range of 9-49 (Table A3.2 in appendix A). This places them in the subgrade regions of S3-S5 (Roads Department, 1987). These fine-grained soils are suitable pavement foundation. The pavement design over these soils revolves around the selection of a pavement structure depending on the traffic loading and available pavement materials.

Unlike the black cotton soils these soils were found to have the CBR not well correlated to their PI. The poor correlation is due to their coarser particles and the gradation becoming increasingly important in the strength characteristics. Indeed for the lateritic gravels it was found out that strength was now inversely related to the percentage of fines as discussed in the next section.

5.2.3 Properties of lateritic gravels with respect to base and subbase construction.

Detailed results for the gravel tests are presented on Tables A3.4 and A3 5 for the Western and Wote-Makindu roads gravel quarries respectively. The results show considerable variation in plasticity and strength characteristics. The strength of the soils was found to be inversely proportional to the plasticity modulus (PM). The plasticity modulus is defined as a product of PI and % passing 0.425mm sieve size. The strength was also found to be inversely proportional to the percentage of fines. Figure 5.6 shows the trend of CBR with PM and percentage of fines.

Table 5.3 shows the Roads Department (1987) requirements for gravel when used as base or subbase material. From Figure 5.6 and Tables A3.4 and A3.5 in appendix A, it can be seen that a large percentage of gravel in the area of investigation does not meet the plasticity and strength characteristics. Indeed for the twenty quarries investigated along the Wote-Makindu road none met the design requirements for the base while only 60 % were suitable for the subbase.

Table 5.3 Gravel characteristics required for subbase and base

Layer	Minimum CBR (%)	Maximum PI (%)	Maximum PM (%)
Base	80	15	250
Subbase	30	15	250

Legend: PI – Plasticity index, PM – Plasticity modulus

Source: Roads Department, 1987

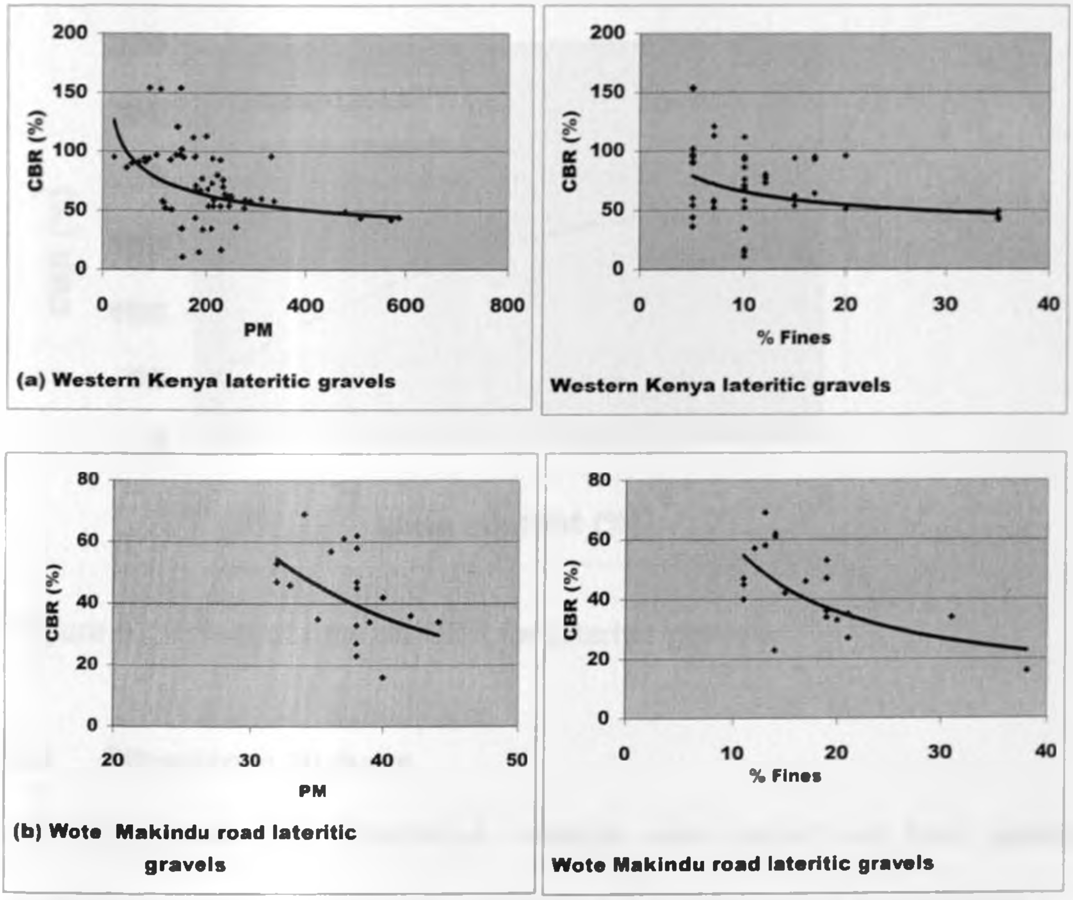


Figure 5.6 Variation of CBR with plasticity modulus and % fines

Lime improvement was performed on the samples from Wote-Makindu road. The test results are presented on Table A3.6 in appendix A. In all the cases the effect of addition of lime was to reduce the plasticity and plasticity modulus. This was accompanied by an increase in the strength of the gravel. All the samples had their PI and PM values falling considerably. For the four and six percentage contents, the samples were non-plastic and hence PM reduced to zero.

The reduction in plasticity enhanced the strength but with diminishing effect as the percentage of the lime increases. This can be seen in Figure 5.7 that was developed from Table A3.6 test results. The trend in the figure is power relationship with a coefficient of determination (r^2) of 0.68 and a very significant relationship between CBR and lime content as shown by ANOVA signf F of .000

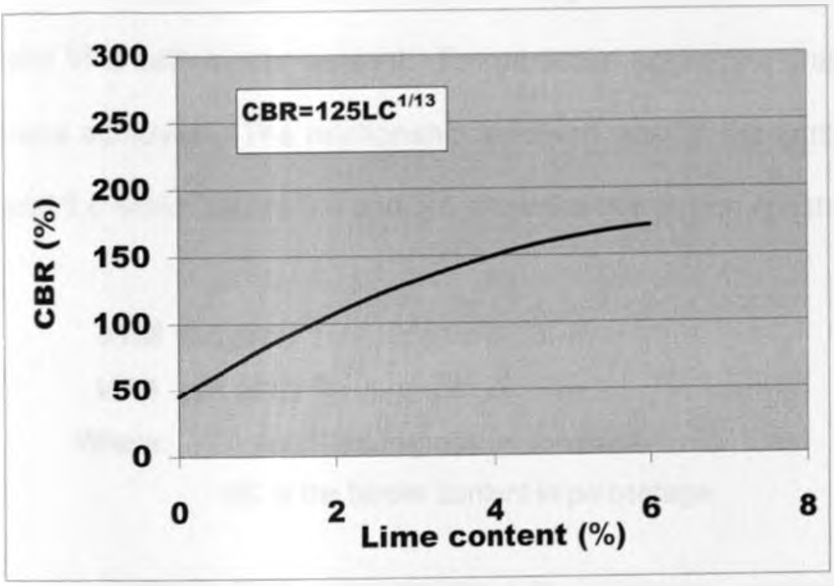


Figure 5.7 Effect of lime on CBR for lateritic gravels

5.3 Bituminous Mixtures

Laboratory testing of bituminous materials was carried out from well-controlled samples and has returned high correlations of the developed relationships.

5.3.1 Effect of binder content

The physical properties of the Marshal test specimens as measured by the compacted density of mix (CDM), voids in total mix (VTM) and voids filled with bitumen (VFB) were found to depend on the binder content as can be seen in Figure A3.2 in appendix A.

The CDM was found to have a peak value. Thus at low binder contents, the CDM was low. Increasing the binder content was found to increase the CDM up to maximum value. Further increase of the binder content resulted in a decrease in the CDM. This behaviour is similar to the water content/dry density relationship for soils where an optimum water content is required to give a maximum dry density (Smith and Smith 1998)

Power regression analysis was performed to establish relationships of VTM and VFB with binder content. For particular aggregate gradations good correlations were achieved. The relationship achieved was of the form shown in Equations 5.5 and 5.6 while Tables 5.4 and 5.5 show the regression constants.

$$VTM = A\,(BC)^{B} \tag{5.5}$$

$$VFB = A\,(BC)^{B} \tag{5.6}$$

Where: A and B are regression constants

BC is the binder content in percentage.

Figures 5.8a and 5.8b show the graphical representation of the developed relationships. It can be seen from the figures that no appreciable difference in VTM and VFB was recorded for the three grades of bitumen

Table 5.4 Power regression factors VTM versus binder content (BC)

Aggregate grading	Bitumen grade /Regression factors								
	180/200			80/100			60/70		
	A	B	r	A	B	r	A	B	r
Gradation A	56.36	-1.36	-.98	22.92	-.793	-.95	35.59	-1.08	-.99
Gradation B	95.62	-1.70	-.91	14.72	-.554	-.72	14.72	-.554	-.72
Gradation C	32.22	-.848	-.98	18.82	-.529	-.98	18.82	-.529	-.98
Gradation D	15.77	-.653	-.81	9.35	-.405	-.77	27.15	-1.04	-.94
All	36.74	-1.09	-.80	15.74	-.570	-.58	36.87	-1.18	-.72

Table 5.5 Power regression factors VFB versus binder content (BC)

Aggregate grading	Bitumen grade/ Regression factor								
	180/200			80/100			60/70		
	A	B	r	A	B	r	A	B	r
Gradation A	18.08	.754	.98	24.53	.571	1.00	15.47	.831	.97
Gradation B	17.01	.795	.97	27.41	.514	.93	29.06	.572	.96
Gradation C	12.98	.874	.97	21.63	.589	.99	17.26	.723	.99
Gradation D	25.79	.565	.93	36.08	.394	.94	19.26	.742	.94
All	17.08	.774	.91	27.22	.517	.87	21.42	.671	.87

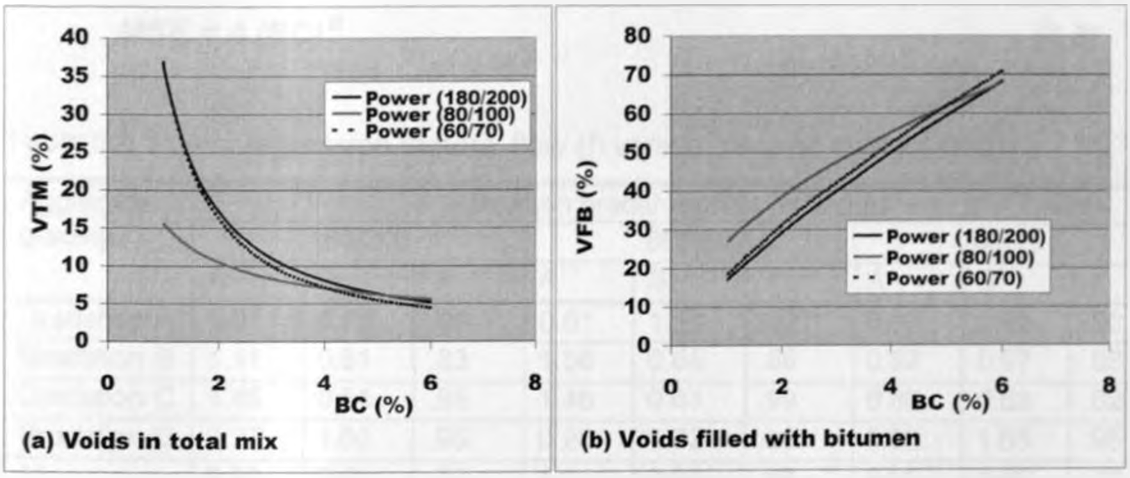


Figure 5.8 Voids in total mix and voids filled with bitumen trends with bitumen content

Further analysis of the Marshal specimen test results is shown on Figure A3.3 in appendix A. The figure shows the trend of Marshal stability (MST), flow (f) and the Marshal stiffness (MSE) with binder content (BC). It can be seen that, the binder content had a significant effect on the strength and deformation of the samples. The strength was assessed by measurements of Marshal stability while the deformation was assessed by the Marshal flow (f). The combination of strength and deformation was assessed by the Marshal stiffness.

For the three grades of bitumen, the Marshal stability was found to fall with binder content in the tested range of 4% to 8%. The softer 180/200-grade bitumen was however found to have a peak stability on the lower binder content ranges. The flow values showed consistent increase with increasing binder content. Power regression analysis was performed on flow and Marshal stiffness against binder content and good correlation was obtained. The relationships obtained were of the forms shown on Equations 5.7 and 5.8. Tables 5.6 and 5.7 shows the regression constants A and B while Figures 5.9a and 5.9b shows the graphical representation of the developed relationships.

$$f \quad = A\,(BC)^{B} \qquad [5.7]$$

$$MSE = A\,(BC)^{B} \qquad [5.8]$$

Table 5.6 Power regression factors, flow (f) versus bitumen content (BC)

Aggregate grading	Bitumen grade/ regression factors								
	180/200			80/100			60/70		
	A	B	r	A	B	r	A	B	r
Gradation A	1.01	0.75	.99	0.61	1.11	.92	0.90	0.92	.92
Gradation B	1.11	0.81	.83	1.56	0.65	.86	0.92	0.97	.88
Gradation C	1.45	0.54	.85	1.46	0.61	.99	0.80	0.89	.82
Gradation D	0.37	1.60	.95	0.28	1.72	.98	0.32	1.65	.95
All	0.90	0.91	.73	1.11	0.68	.83	0.95	0.90	.78

Table 5.7 Power regression factors Marshal stiffness versus bitumen content (BC)

Aggregate grading	Bitumen grade/ regression factors								
	180/200			80/100			60/70		
	A	B	r	A	B	r	A	B	r
Gradation A	7.03	-.792	-.80	28.53	-1.62	-.87	15.28	-1.34	-.98
Gradation B	33.46	-1.74	-.90	107.8	-2.38	-.94	191.4	-2.70	-.94
Gradation C	5.50	-0.78	-.74	38.01	-1.86	-.97	22.56	-1.45	-.87
Gradation D	185.9	-0.30	-.95	344.3	-3.21	-.98	60.96	-2.32	-.95
All	18.93	-1.49	-.73	79.65	-2.26	-.90	33.94	-1.80	-.86

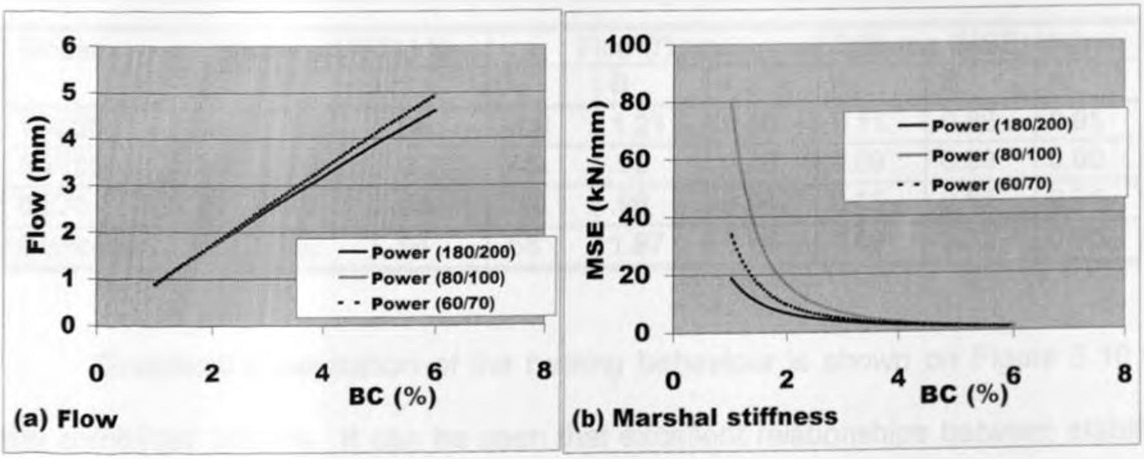


Figure 5.9 Marshal flow and stiffness (MSE) trends with bitumen content

5.3.2 Re-healing behaviour

Test results of failed samples reheated at 40^{0}C showed considerable re-healing. Table A3.7 shows the initial Marshal test results while Table A3.8 shows the results after initial testing and re-healing. Graphical representation of the results is shown in Figure A5.1 in appendix A. An examination of the tables and graphs shows that the healing behaviour spreads across the tested binder content ranges.

Linear regression analysis of the initial test results and the re-healed test results showed good correlation's as can be seen in Table 5.8. The relationships for re-healing as measured by Marshal stability, flow and Marshal stiffness are of the form shown in Equations 5.9 through 5.11.

$$MST2 = A + B*MST1 \quad [5.9]$$

$$f2 = A + B*f1 \quad [5.10]$$

$$MSE2 = A + B*MSE1 \quad [5.11]$$

Where: MST1 & MST2 are the Marshal stability before and after healing respectively
f1 & f2 are the Marshal flow before and after healing respectively
MSE1 & MSE2 are the Marshal stiffness before and after healing respectively
A & B are the linear regression constants

Table 5.8 Linear regression coefficients: Healing behaviour at 40^{0}C

Binder	Stability (MST) kN			Flow (f) mm			Stiffness (MSE) kN/mm		
	A	B	r	A	B	r	A	B	r
180/200	.24	.80	.98	-.08	1.21	0.90	0.11	0.66	0.95
80/100	.24	.80	.92	.45	.99	0.86	0.09	0.83	0.90
60/70	.81	.73	.96	.88	.96	0.94	0.13	0.65	0.94
Combined	.45	.80	.94	.68	1.97	0.84	0.09	0.72	0.90

Graphical presentation of the healing behaviour is shown on Figure 5.10 for the combined binders. It can be seen that excellent relationships between stability, flow and stiffness before and after healing exist. This means that for the pavements with temperatures of around 40^{0}C, as commonly found in Kenya (Motumah 1973) if the loading interval is about sixteen hours a completely deformed mixture will heal but can not regain the original strength.

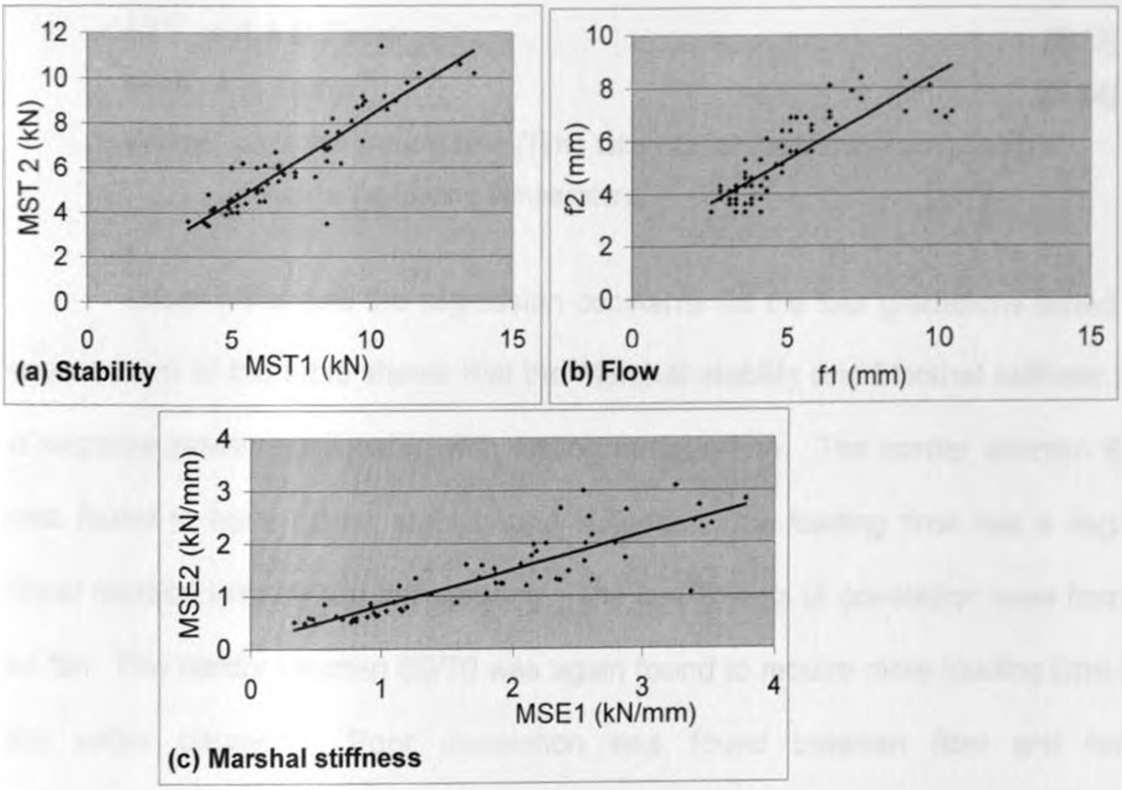


Figure 5.10 Re-healing behaviour of bituminous mixtures at 40°C

5.3.3 Effect of temperature

Temperature has a major effect on the performance of bituminous mixtures. In the hot climates, the effect of temperature has a large proportional effect on the deterioration of the riding surface. This was reported by Nouredin et al (1994) for an investigation done in the hot climatic conditions of Saudi Arabia.

The effect of temperature on Marshal stability, stiffness, flow and loading time is presented in Figure A5.2 in appendix A. From the graphs it is visible that there is a general drop in stability and stiffness with increasing temperature. Similarly as temperature rises the loading time decreases. Further, the structural capacity of the asphalt concrete is reduced and consequently the likelihood of deformation upon loading is increased. This is in line with Ibi (1994). Regression analysis of testing temperature against stability, Marshal stiffness, testing time and flow gave relationships of the form shown in Equations 5.12 through 5.15.

$$MST = A\ Temp^{B} \quad [5.12]$$

$$LT = A + B\ Temp \quad [5.13]$$

$$MSE = A\ Temp^{B} \quad [5.14]$$

Where: LT is the loading time (Time taken to fail the Marshal specimen)
Temp is the testing temperature

Table 5.9 shows the regression constants for the four gradations tested. An examination of the table shows that the Marshal stability and Marshal stiffness have a negative power relationship with testing temperature. The harder bitumen 60/70 was found to have higher stability and stiffness. The loading time has a negative linear relation with testing temperature. The coefficients of correlation were found to be fair. The harder bitumen 60/70 was again found to require more loading time than the softer bitumen. Poor correlation was found between flow and testing temperatures suggesting that flow was independent of the testing temperatures. However, a flow of between 3mm and 5mm was generally achieved in the test results. The overall equations for the commonly used 80/100 binder is summarised in Equations 5.15a-c

$$MST = 506Temp^{-0.97} \quad (r=-0.82) \quad [5.15a]$$

$$LT = 18.1-1.02Temp \quad (r=-0.55) \quad [5.15b]$$

$$MSE = 103Temp^{-0.92} \quad (r=-0.86) \quad [5.15c]$$

Table 5.9 Regression coefficients-Effect of temperature on Marshal test results

(a) Marshal Stability (MST = A*Temp ^{B})

Binder	Aggregate grading/regression coefficients											
	A			B			C			D		
	A	B	r	A	B	r	A	B	r	A	B	r
180/200	6.51	-1.1	-.99	953	-1.2	-.99	1326	-1.31	-1.0	5219	-1.65	-.95
80/100	4.28	-.97	-.96	590	-.99	-.98	358	-.96	-.98	728	-.98	-.99
60/70	4.27	-1.5	-.99	3197	-1.44	-.99	4536	-1.58	-99	2080	-1.21	-.97

(b) Marshal stiffness (MSE = A *Temp ^{B})

Binder	Aggregate grading/regression coefficients											
	A			B			C			D		
	A	B	r	A	B	r	A	B	r	A	B	r
180/200	171	-1.15	.83	126	-.94	.86	247	-1.24	.99	165	-1.05	.94
80/100	122	-1.02	.89	77	-.83	.96	76	-.86	.96	158	-.97	.96
60/70	508	-1.38	.99	3113	-1.78	.99	384	-1.24	.98	4629	1.84	.96

(c) Loading time (LT = A+B* Temp)

Binder	Aggregate grading/regression coefficients											
	A			B			C			D		
	A	B	r	A	B	r	A	B	r	A	B	r
180/200	21.6	-.19	.74	20.0	-.18	.96	16.8	-.11	.96	21.6	-.15	.78
80/100	18.8	-.10	.88	18.4	-.10	.94	14.6	-.10	.76	20.6	-.11	.97
60/70	28.6	-.29	.85	26.2	-.25	.97	22.6	-.23	.93	26.2	-.17	.75

5.3.4 Elastic modulus determination

For the purpose of structural evaluation of pavement structures, an accurate assessment of elastic modulus is required. However the estimation of this key strength parameter remains difficult due to the change of strength of bituminous mixtures with temperature, strength and grading of the aggregate and type of binder. However by using correlations developed by Adedimila (1980) as shown on Equation 2.8 and 2.9, Table A5.1 in appendix A was developed. The Table shows variation of compressive modulus and tensile strength with temperature. The variation of Marshal stiffness with temperature is also shown.

Regression analysis of the strength variables shown on Table A5.1 with temperature is presented on Table 5.10. As can be seen from the correlations and the graphical presentation (Figure 5.11), the elastic modulus and the tensile strength of the bitumen mixtures falls with increasing temperature. The harder bitumen 60/70 is found to have higher modulus, than the softer 80/100 and 80/200. However as the testing temperature rises the modulus for the three grades of the bitumen tends to merge. While the above trends are true, the modulus calculated tend to be lower than that recommended by Roads Department (1987). The reason here is that the Marshal specimens are highly stressed with longer loading time, than would apply to vehicles traversing the pavement surface.

Table 5.10: - Temperature vs. compressive modulus and tensile strength

(a) Compressive modulus: - $E_y = A*Temp^B$

Binder	Aggregate grading/regression coefficients											
	A			B			C			D		
	A	B	r	A	B	r	A	B	r	A	B	r
180/200	4.68	-1.06	-.83	13.3	-1.30	-.98	6.94	-1.15	-.99	5.035	-.989	-.94
80/100	3.68	-.96	-.89	2.44	-.785	-.96	2.34	-.810	-.96	4.930	-.925	-.97
60/70	14.3	-1.3	-1.0	89.1	-1.7	-.99	11.7	-1.17	-.98	138.42	-1.78	-.95

(b) Tensile strength $St = A*Temp^B$

Binder	Aggregate grading/regression coefficients											
	A			B			C			D		
	A	B	r	A	B	r	A	B	r	A	B	r
180/200	11.1	-.91	.83	27.6	-1.09	.98	14.06	-.97	.98	17.91	-.957	.95
80/100	8.52	-.81	.90	6.31	-.682	.97	5.79	-.69	.96	12.22	-.817	.97
60/70	28.5	-1.11	.99	178.	-1.54	.99	27.05	-1.06	.98	30.40	-1.64	.95

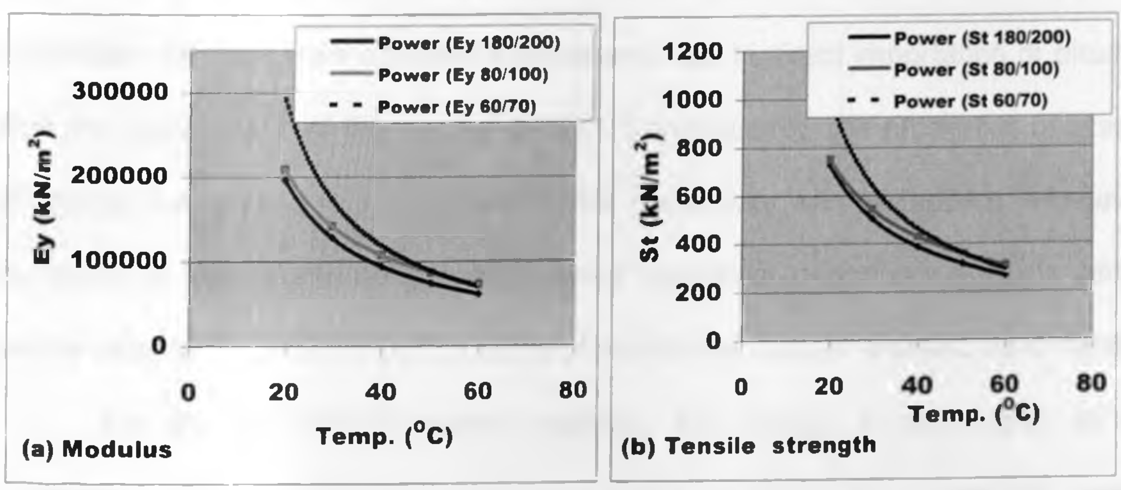


Figure 5.11 Typical modulus and tensile strength Vs temperature relationships

5.4 Effect of environmental factors on flexible pavements

The study of environmental factors established that there was a distinct difference of the two principal types of the surfacing layer. These two types include the bituminous surfacing and the asphalt concrete. The bituminous surfacing consists of a thin layer made up of surface dressing (referred to as single seal or double seal). This type of surfacing is usually used as the surfacing layer on the low volume flexible pavements. The asphalt concrete on the other hand is thick bituminous mixture used as the surfacing layer for the high volume flexible pavements. The analysis of the two types of the surfacing layers is now presented.

The data in this section relates to bituminous data collected from many parts of the country. Different environmental conditions were considered in the analysis of this data. To reduce variability some of the test sections were constructed for the purpose. The test results have returned good correlations for the developed relationships.

5.4.1 Ageing of bituminous surfacing

Table A3.10a in appendix A shows the properties of bitumen used in surface dressed test sections of various ages. The bitumen used in Kenya varies considerably due to importation of crude from different sources and due to direct importation of bitumen after the liberalisation of the energy sector. Consequently, the properties of grades of bitumen being used in the Kenyan flexible roads vary with the source. However, the study on low volume pavements showed consistent performance trends which, can be used to interpret the performance of bituminous surface dressed pavements.

For the low volume sealed sections, the change in penetration of the commonly used MC 3000 bitumen was found to follow the hyperbolic model suggested by Brown in 1957 (Lamina et al, 1962 and Lee, 1973). The hyperbolic model suggested is shown in Equation 5.16

The model is rearranged to form Equation 5.17. This has enabled regression analysis. The predictive models calibrated by the linear regression analysis are summarised in Table 5.11. The coefficient of determination r^2 lying above 0.96

except for the Oyugis test section where it stood at 0.94 shows an excellent model for time variation of penetration of thin bitumen surface dressing. The variation of penetration against service age is shown in Figure A5.3 for the test sections. Figure 5.12 shows the results for all the test sections tested with MC3000.

$$\Delta pen = T/(A+BT) \qquad [5.16]$$

$$T/\Delta pen = A+BT \qquad [5.17]$$

Where: Δpen = change in penetration during a time T (years). A and B are constants.

Table 5.11 Summary of regression constants for Equation 5.14

Test section	A	B	r^2
Majengo	0.0814	0.0054	0.96
Oyugis	0.0771	0.0119	0.94
Kisii	0.1043	0.0067	0.98
Ndumberi	0.0486	0.0094	1.00
Kwale	0.0707	0.0094	1.00
Narok	0.0967	0.0087	0.99
Marich Pass	0.0831	0.0088	0.98
Average	0.018	0.0086	

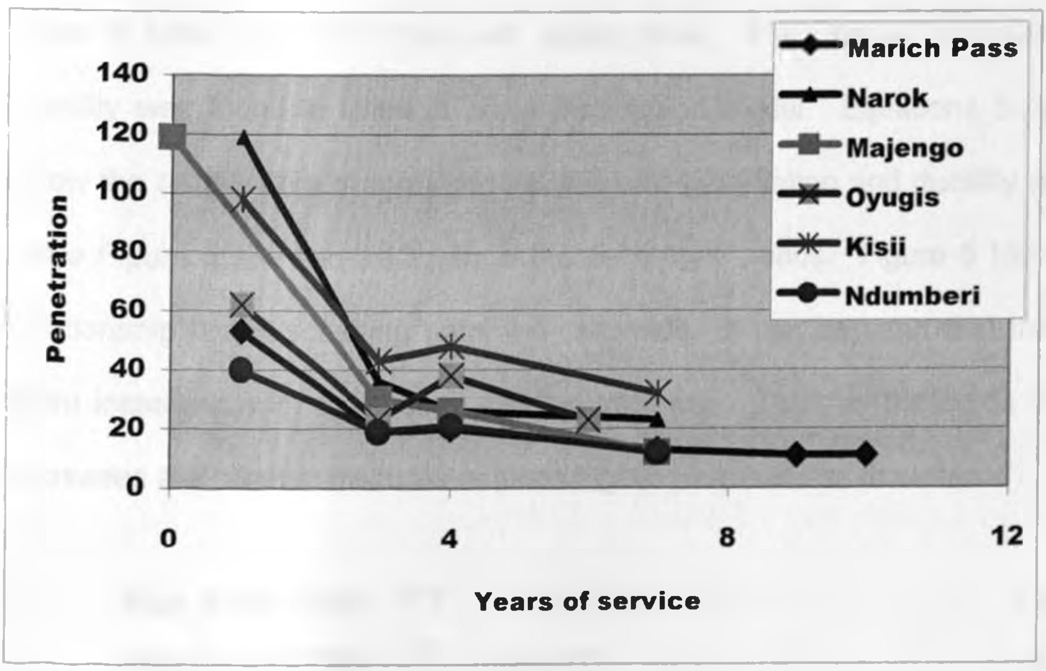


Figure 5.12 Penetration Vs time for MC 3000

The rate of hardening as measured by decrease in penetration is significant in virtually all the test sections for the first two years. Subsequently the change in penetration is either insignificant or zero. What is apparent is that the trends in the hardening for MC3000, K1-60, K1-70 and 80/100-penetration bitumen differ up-to three years. Beyond the three-year service period the hardening trends converged. The trends for the softer bitumen MB5000 and 200/500 short residue showed higher values of penetration after four years of service when compared to the other bitumen which are commonly used in Kenya. Further, the test sections with MC 3000 prepared with heavy diesel hardened at a slower rate than those prepared with kerosene.

5.4.2 Ageing of asphalt concrete pavements

Table A3.10b shows the properties of the recovered bitumen samples. These were recovered from asphalt concrete high volume roads. Trial regression analysis showed that the penetration and ductility of the samples was more related to the voids in total mix (VTM) than with ageing time. The change in penetration and ductility was found to follow a power regression model. Equations 5.18 and 5.19 show the relationship of voids in total mix with penetration and ductility respectively while Figure 5.13a and 5.13b show the developed trends. Figure 5.13c shows the relationship of the softening point with air voids. It can be seen that the softening point increases with increasing voids in total mix. Thus as the voids in total mix increases the bitumen mixtures requires higher temperatures to soften.

$$Pen = 105\,(VTM)^{-0.72} \quad (r=-0.87) \qquad [5.18]$$

$$Duct = 109\,(VTM)^{-1.22} \quad (r=-0.81) \qquad [5.19]$$

Where: Pen is the penetration in 0.1 mm

Duct is the ductility in cm and

VTM is the voids in total mix

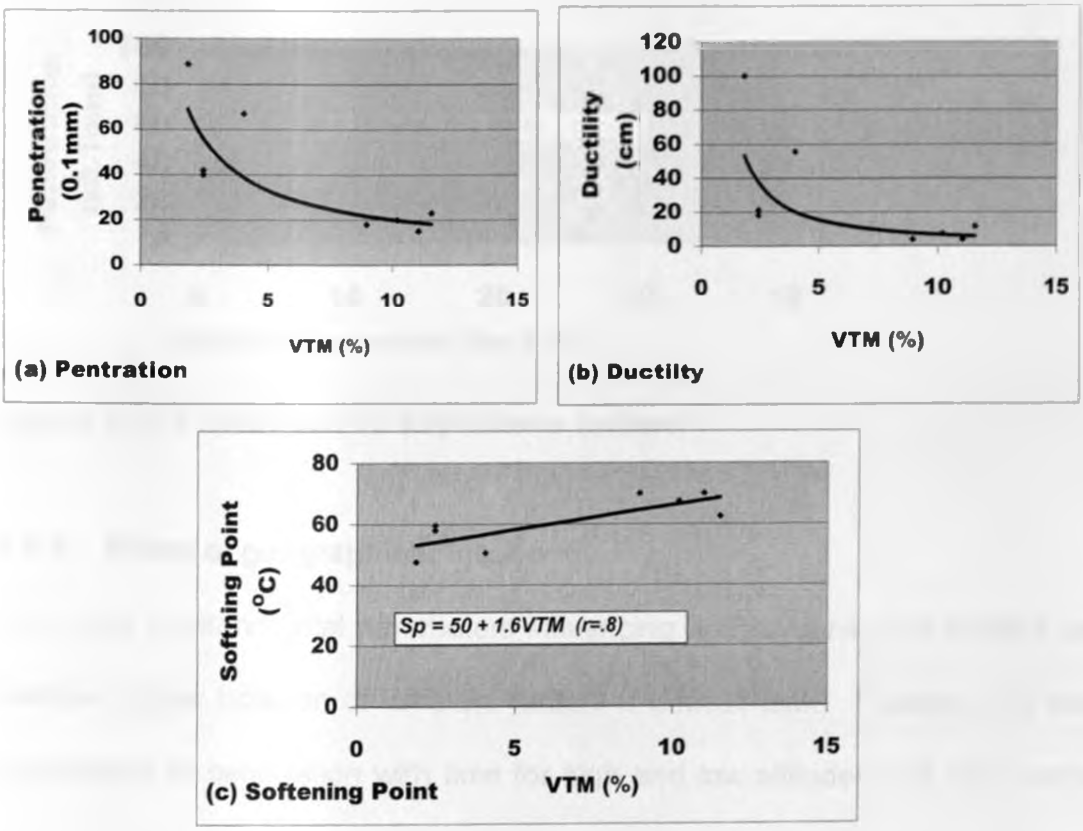


Figure 5.13 Penetration, ductility and softening point Vs voids in total mix

5.4.3 Chemical composition and penetration

Figure 5.12 shows consistent decrease of penetration with age for surface dressings, while Figure 5.13 shows consistent reduction in penetration with increasing voids in total mix for the thick asphalt concrete. The hardening process has been explained in the literature review. In a nutshell it occurs as a result of readjustment of the chemical composition. Thus, increase in asphaltene content and corresponding reduction of the saturates and aromatics are responsible for the decrease in penetration. Results presented in Tables 3.10 and 3.11 are shown on Figure 5.14. It is seen that as the asphaltene content increases, the penetration reduces.

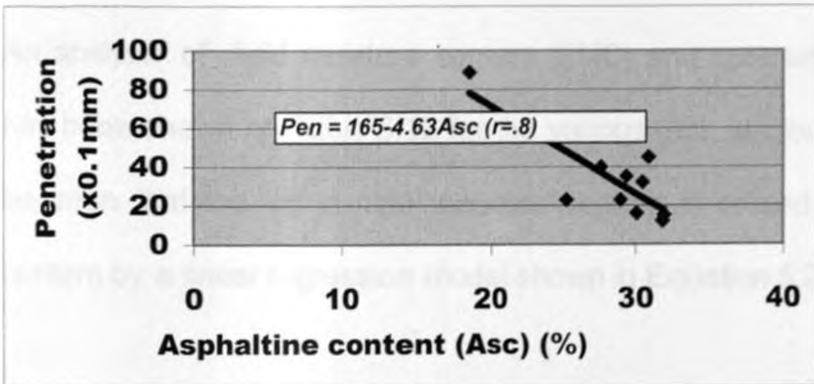


Figure 5.14 Penetration Vs asphaltene content

5.4.4 Effect of geographical location

The many environmental parameters influencing age hardening for surface dressed sections make isolation of different factors a difficult task. Figures 5.15 shows a comparison of penetration with time for high and low altitudes, hot and cold areas, wet and drier sites for the MC3000 binders. The fall in penetration was found to be rapid for test sites in hot and drier areas. The effect of the altitude does not come out distinctly. Perhaps it has been masked by the other environmental factors.

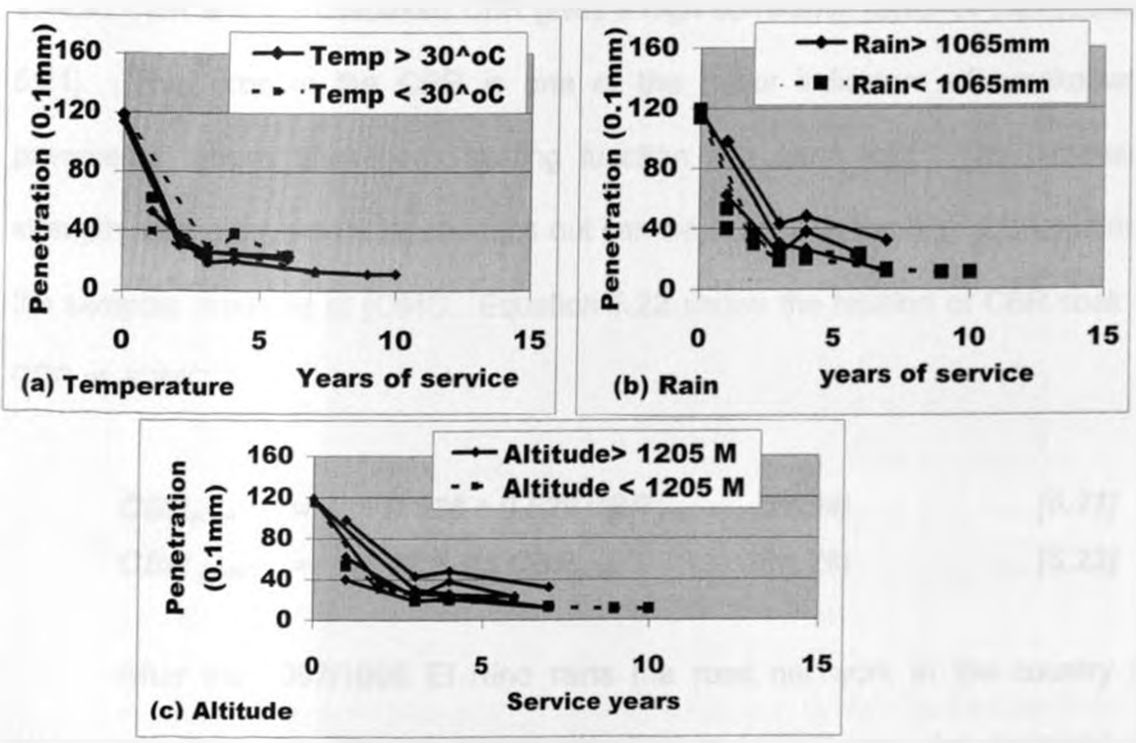


Figure 5.15 Effect of environmental factors on penetration

5.4.5 Behaviour of non-bituminous subbase and base materials

An analysis of field moisture content (FMC) and optimum moisture content (OMC) has been shown on Table 3.12 for the various test sections at different ages. It can be seen that the fall in field moisture content is related to the optimum moisture content by a linear regression model shown in Equation 5.20

$$FMC = -2.86 + 0.91\ OMC \qquad (r=0.85) \qquad [5.20]$$

Thus the Field moisture content had dropped slightly lower than the optimum moisture content during the final year of monitoring. This is in line with findings of Hall (1979) for sealed roads where the surfacing remained intact over the service years. In the event of large storms typical of the Kenyan environment, the above scenario changes rapidly in a failed surfacing layer, which lets in water to the base and subbase layers. In this case the moisture content would rapidly increase with corresponding reduction in strength. As can be seen from Table 3.12 there is a large drop in CBR for the four-day soaked samples. The linear regression model for the soaked CBR and the unsoaked CBR gives a high correlation factor of 0.94 (Equation 5.21). This drop in the CBR is one of the major indicators of breakdown of pavements whose bituminous sealing function has been lost. The increase in strength as the pavement layers dries out can be seen from the high CBR values for the samples moulded at $\frac{1}{2}$OMC. Equation 5.22 shows the relation of CBR soak and CBR at $\frac{1}{2}$OMC

$$CBR_{soak} = 0.628 + 0.529\ CBR_{omc} \qquad (r=.94) \qquad [5.21]$$

$$CBR_{.\frac{1}{2}OMC} = 17 + 1.5\ CBR_{soak} \qquad (r=.76) \qquad [5.22]$$

After the 1997/1998 El Nino rains the road net work in the country was extensively damaged. The low volume sites ES9 and ES10 were also damaged after the deflection surveys. In order to determine the deterioration of unbound pavement bases and the underlying subgrade layers due to the loss of surfacing, DCP soundings were carried out two weeks after the onset of the April 2000 long rains. The soundings were carried out at three points at ES9. At the first DCP site the surfacing was still intact. At the second site a cracking index of 1 m/m2 was recorded while at the third site a pothole had developed (Plate A5.1 in appendix A). At ES10 the surfacing was washed away (Plate A5.2 in appendix A). The soundings were done on the exposed gravel base

The test results for this exercise are presented in Figure 5.16a and 5.16b. It can be seen that the beneficial effect of the surfacing was quickly wiped out in the potholed and bare sections for upper pavement layers. The CBR of the pavement layers dropped by seventy five percent at the pothole due to the pooling of the water. At ES10 where the surfacing was wiped out, the CBR for the pavement layers were higher than that of the potholed section. The CBR for this section was also higher than that of the intact and cracked areas from a depth of 200 millimetres. This scenario can be attributed to the good drainage of the bare section.

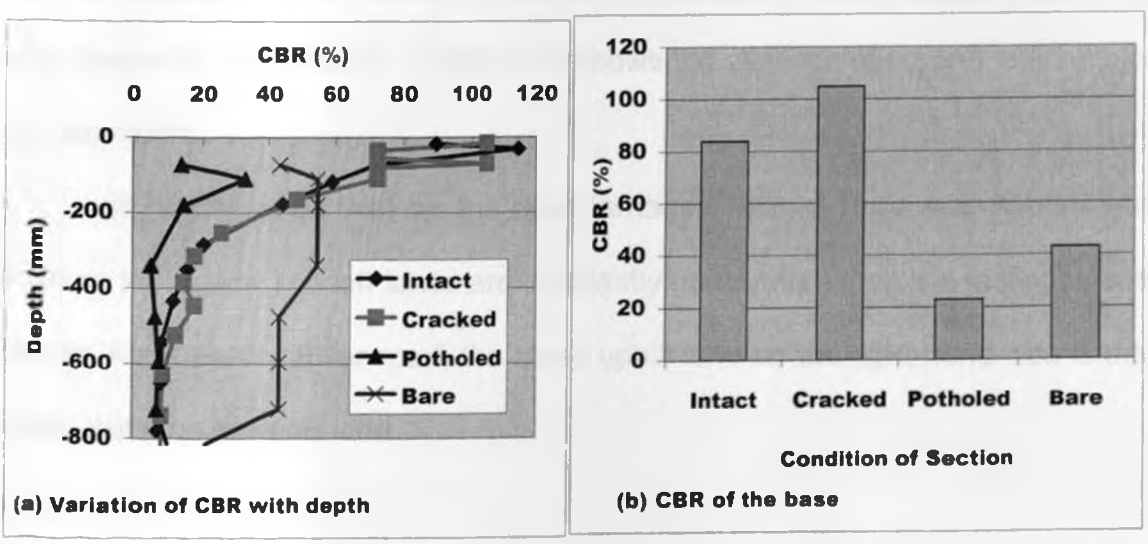


Figure 5.16 Variation of CBR at ES9 and ES10 after damage by El Nino rains

5.5 Pavement condition surveys

The data with respect to pavement condition surveys and present serviceability index (PSI) returned low values of r and r^2. Similarly a large variation of ANOVA signf F values between .000 and .293 was found. The poor correlations reflect the difficulties in relating the various pavement condition parameters, and PSI. This is a result of the large number of factors that affect the pavement condition. These factors include many environmental conditions, type of pavement structure, variability of the pavement structure, variability of traffic load and change of grade. Such low correlations have been reported by Claesson et al (1977). However the large volume of data used in the study makes the use of the developed relationships acceptable.

5.5.1 Rutting

Rutting was measured on the individual lanes and the maximum ruts recorded on each lane for the test roads. The test results are shown on Table A3.11 in appendix A alongside PSI cracking, roughness and probability of acceptance. A new concept of the total rutting is explained in chapter three. The total rut is the sum of ruts on the four-wheel paths at the various points on the standard rutting straight edge and is also shown on Table A3.11. Figure 5.17 shows the variation of rut and total rut along the test roads

It can be seen that on the dual carriages Nairobi-Thika and Airport-Nyayo stadium test roads, the left lanes are constantly more rutted than the fast right lanes. On the Airport –Athi River road, the steep uphill lane on the right hand side is more rutted than the downhill lane.

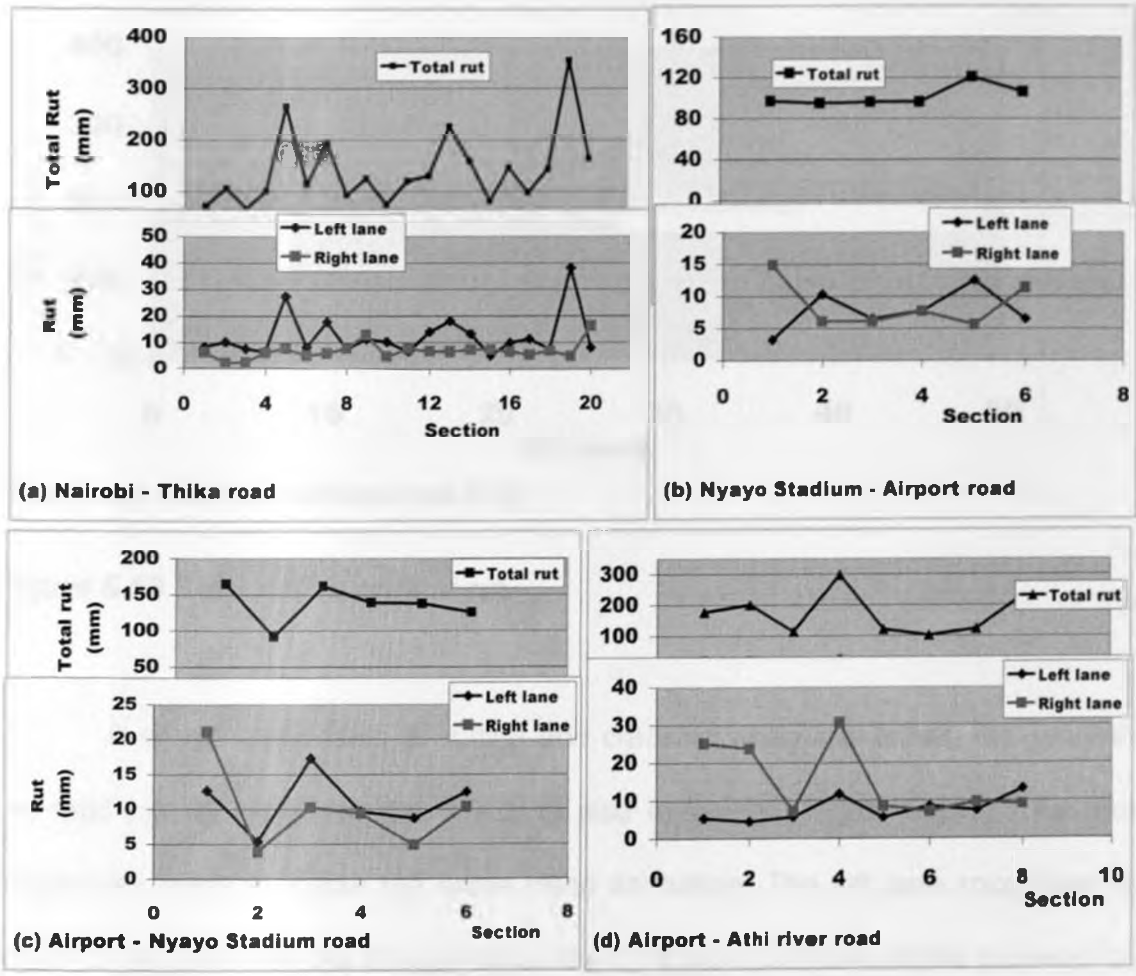


Figure 5.17 Rut variation along the test roads

5.5.2 Relationship of total rutting to cracking and roughness

The three variables, rutting cracking and roughness were found to be related to the total rutting (TR). The critical rut (CR) is defined as the more severe of the rut values for the left lane and right lanes for the dual carriageway. For the two-way test roads CR is the more severe of the rut values for the climbing and downhill lanes. A good relationship of CR and TR (Equation 5.23) shows that as the critical rut increases, then the entire road is heavily rutted. Figure 5.18 shows a graphical relationship of CR and TR for all the test roads.

$$TR = 40.9 + 7.5CR \quad (r=.87) \qquad [5.23]$$

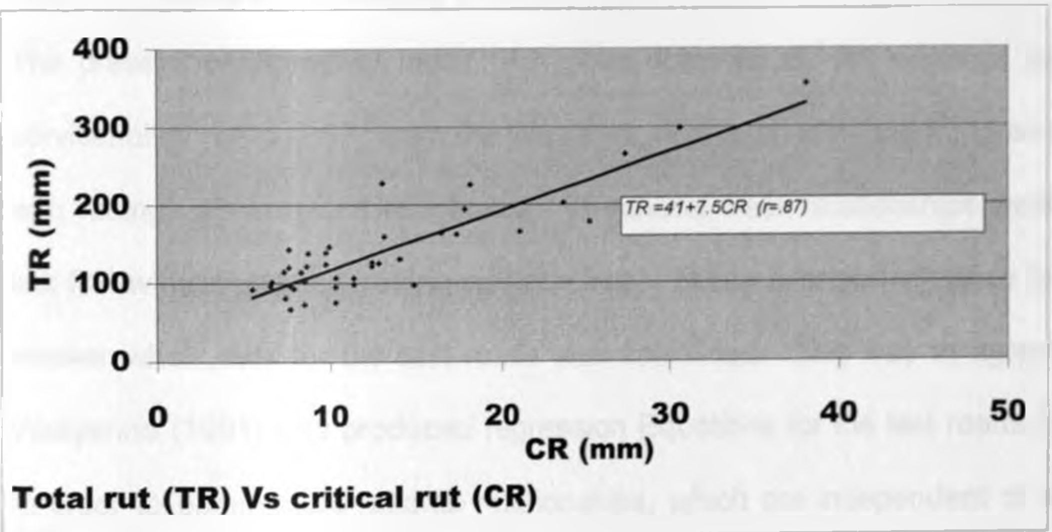


Figure 5.18 Total rut Vs critical rut

A weak relationship of rutting and cracking was established. In general as the total rutting increases, the cracking also increases (Figure 5.19a). Pavement roughness tends to follow the same trend as rutting. The left lane roughness was found to be more than the roughness on the right lane. Similarly on the two-way road, the climbing lanes have more roughness than the downhill lanes. Again, a weak relationship was established between total rut and roughness (Figure 5.19b).

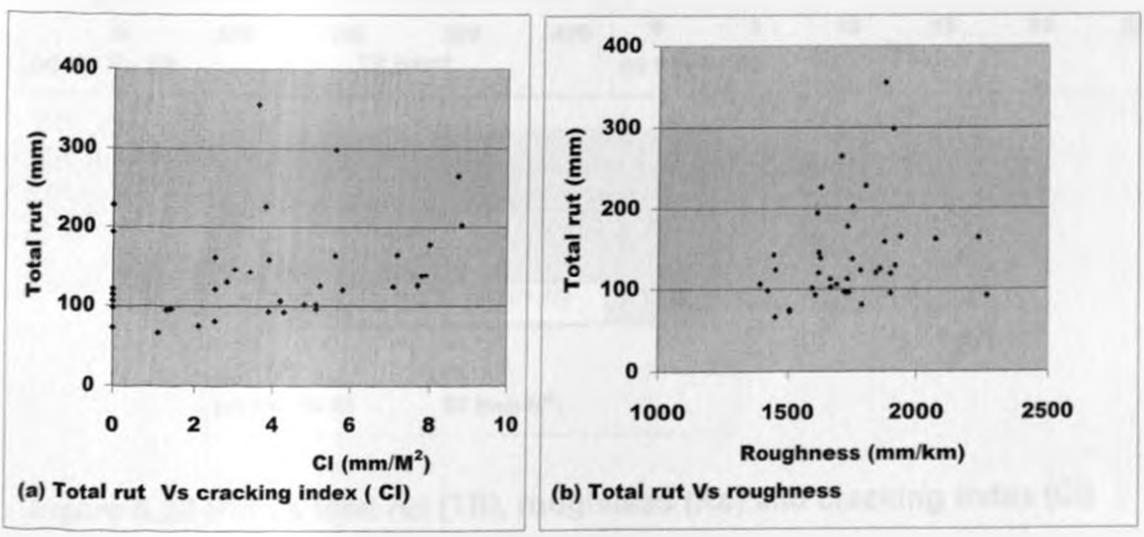


Figure 5.19 Total rut Vs cracking index and roughness

5.5.3 Present serviceability index

The present serviceability index (PSI) was obtained as the mean of the present serviceability rating (PSR) from the five raters. For each test road PSI was regressed with rutting, cracking and roughness. In general weak relationships were found to link PSI with roughness, rutting and cracking. These relationships were found to be weaker when data for the test roads was combined. This was in agreement with Wakyendo (1991) who produced regression Equations for the test roads separately. In order to obtain more rational relationships, which are independent of test roads, data for all the test roads was analysed and presented graphically on Figure 5.20.

It can be seen from the figure that the roughness of the road influences the PSI more than the other two surface conditions. The cracking of the surface marginally is related to PSI.

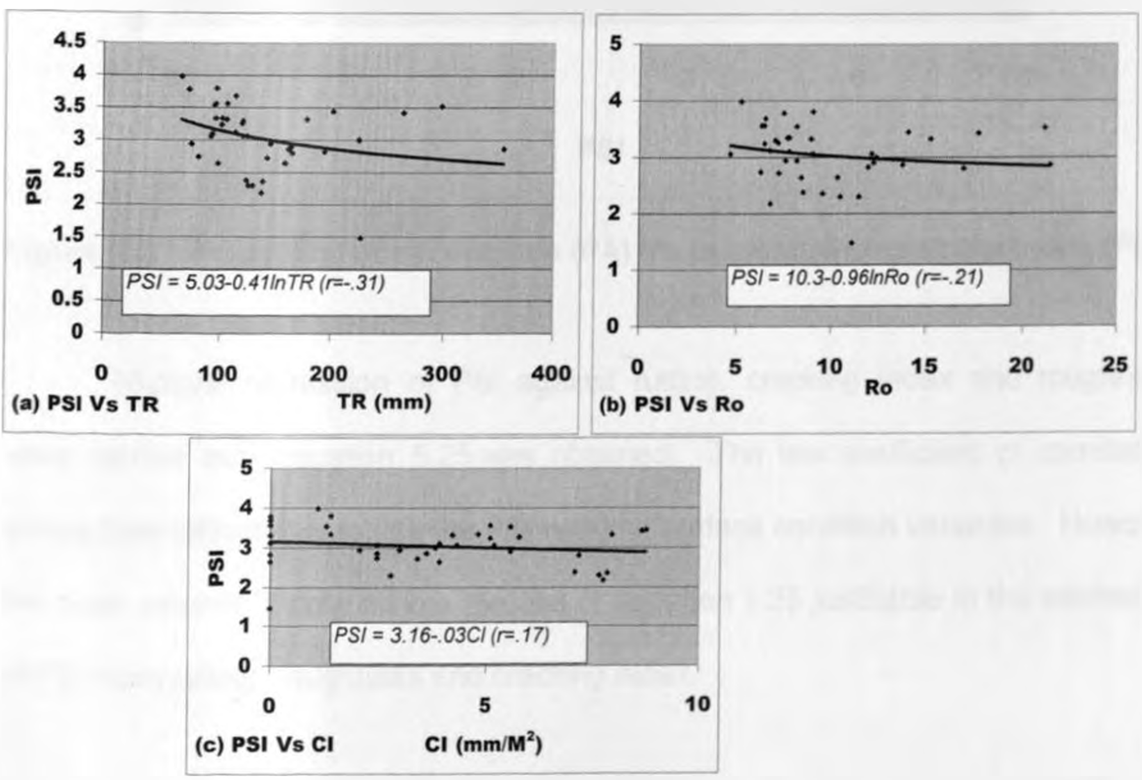


Figure 5.20 PSI Vs total rut (TR), roughness (Ro) and cracking index (CI)

For each of the sections rated, raters answered the question whether the section was acceptable or not. The probability of acceptance (PA) was obtained by calculating the ratio of those accepting a section to the number of raters. The relationship of PSI and PA (Equation 5.24) is lower bound, which has no meaning for values of PA greater than one. Figure 5.21 shows the developed power relationship of PSI and PA.

$$PA \quad = 0.14PSI^{1.57}\ (r{=}.85) \qquad [5.24]$$

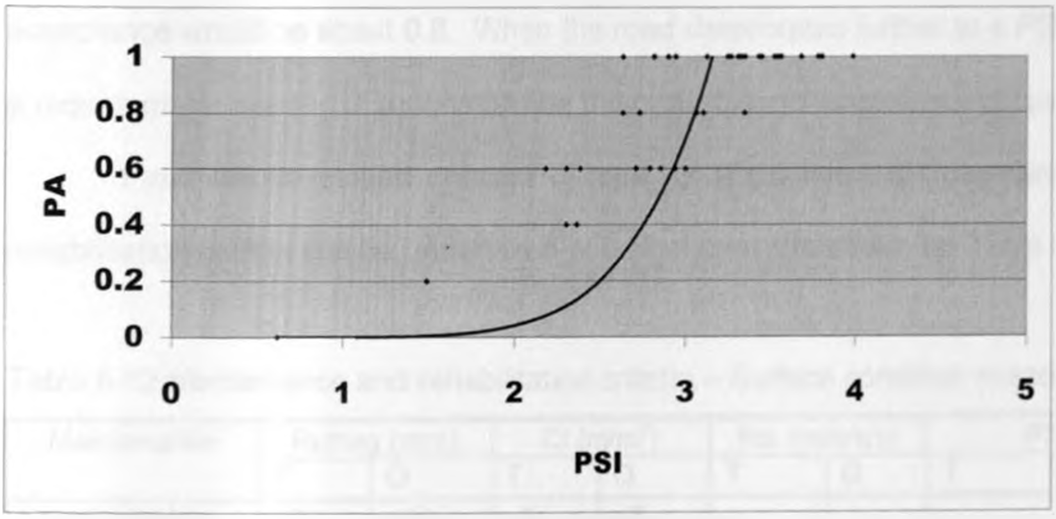


Figure 5.21 Probability of acceptance (PA) Vs present serviceability index (PSI)

Multiple regression of PSI against rutting, cracking Index and roughness were carried out. Equation 5.25.was obtained. The low coefficient of correlation shows how difficult it is to link the PSI with the surface condition variables. However the huge volume of data makes the use of equation 5.25 justifiable in the estimation of PSI from rutting, roughness and cracking index.

$$PSI \quad = 8.8-.338LnTR-.551LnRo-.0072CI \qquad (r{=}.33) \qquad [5.25]$$

5.6 The performance and remedial measures of the test roads

5.6.1 Performance

The performance of the test roads can be measured from the set targets for various surface conditions (Roads Department 1988a). The set targets are shown on Table 5.12. The table shows when various maintenance interventions should be carried out based on the various criteria.

Using this criteria it, can be seen that trunk roads require attention at a PSI of 3.0. From the developed relationship (Equation 5.24) at this PSI the probability of acceptance would be about 0.8. When the road deteriorates further to a PSI of 2.5 to a require major overlay or reconstruction the probability of acceptance drops to 0.6

From the developed concept of total rut (Equation 5.23) maintenance and rehabilitation criteria can be undertaken at critical total ruts shown on Table 5.13.

Table 5.12 Maintenance and rehabilitation criteria – Surface condition measurements

Maintenance	Rutting (mm)		CI (m/m^2)		Ro. (mm/km)		PSI	
	T	O	T	O	T	O	T	O
Local patching	>20	>20	>5	>5				
Surface dressing or slurry seal			>1	>1				
Minor overlay or resurfacing	>15	>15	>30% of WP	>50% of WP	2800	3100	3.0	2.5
Major overlay or reconstruction	>20	>25	-	-	3400	3750	2.5	2.0

Legend: - WP – wheel path, T – trunk roads, O – other roads

Source: Roads Department, 1988a

Table 5.13 Proposed total rut criteria for maintenance of flexible road pavements

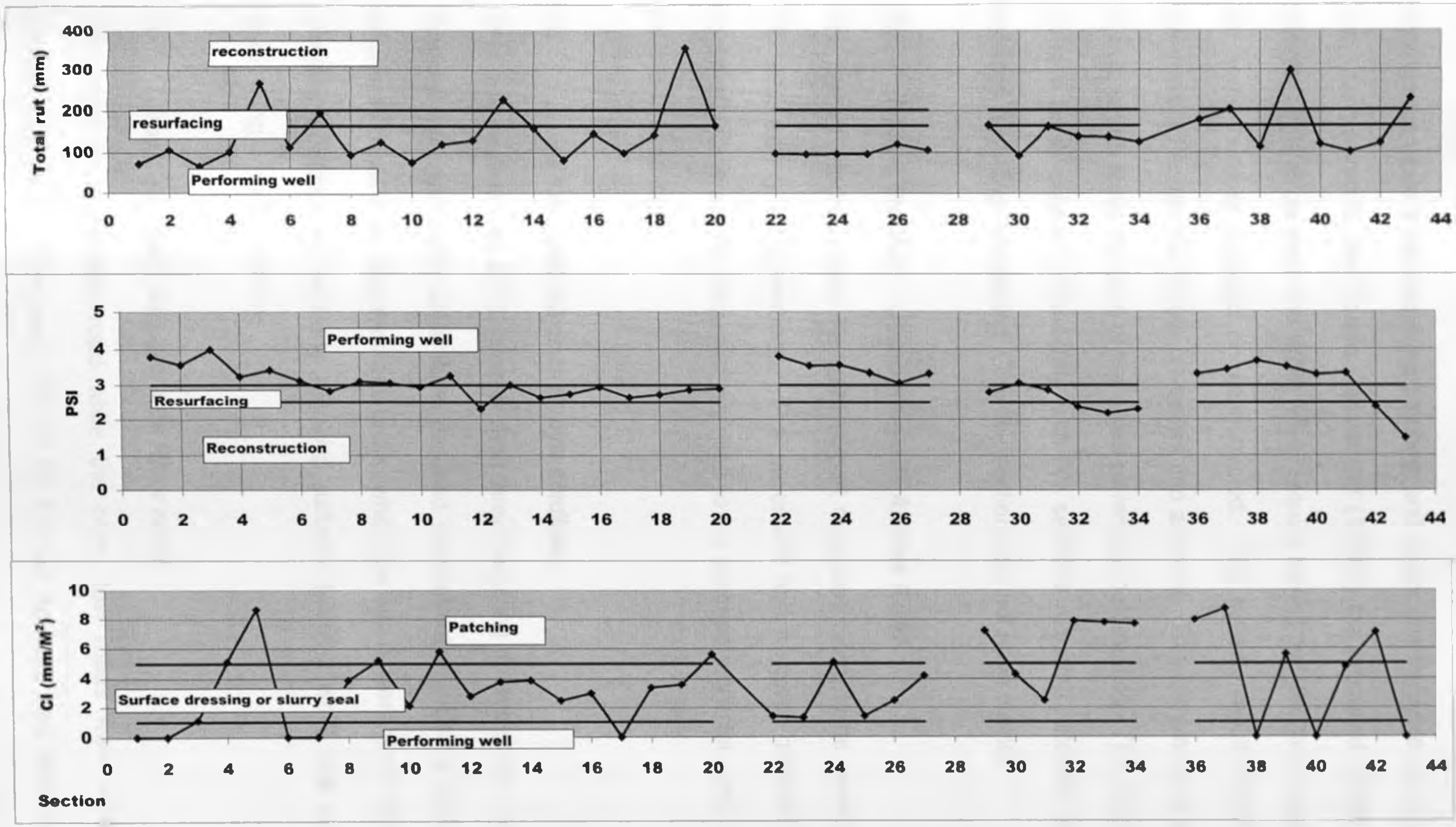
Type of maintenance	Minimum total rut	
	Trunk roads	Other roads
Local patching	201	201
Minor overlay or resurfacing	164	164
Major overlay or reconstructing	201	238

5.6.2 Proposed remedial measures

The roughness criteria passed all the roads. The proposed remedial measures were therefore based on rutting cracking and PSI. Figure 5.22 shows the proposed maintenance schedule based on the various criteria discussed below. For each section the more severe maintenance need is proposed where conflicting proposals are derived from the different criteria. The individual roads are now presented.

(a) *Nairobi-Thika Road*

The PSI for half of the road fell below three. Nine sections had PSI falling between two and half and three, while one section had a PSI of 2.30. The last section on the Thika side was completely damaged. From this criteria therefore nine sections of the road required major overlay or reconstruction.



Legend: Section 1-20 Nairobi – Thika road, 22-27 Nyayo stadium – Airport road, 29-34 Airport – Nyayo Stadium Road, 36- 43 Airport – Athi River road

Figure 5.22 Proposed maintenance schedule for the test roads

Using Table 5.13 of the proposed total rut criteria three sections require resurfacing while those requiring patching and major overlay or reconstruction were two. Compared to the Roads Department (1988a) the proposed criterion is more severe. This is because the latter would require having two sections resurfaced and the same number over-laid or reconstructed. The severity results from taking the deformation across the whole pavement into account. This compares to the concept of PSI, which takes account of the entire pavement deformation. The road was found to have been cracked extensively, with five sections requiring patching while eleven sections requiring resurfacing. Only five sections did not have cracks.

(b) Nyayo Stadium – Jomo Kenyatta Airport Road

The Nyayo Stadium-Jomo Kenyatta airport road was found to be serviceable from PSI and rutting considerations. The pavement had however cracked on all the sections. One section required the road to be patched while the rest of the road required resealing.

(c) Jomo Kenyatta Airport- Nyayo stadium

Only one section had a PSI greater than three requiring no attention. Two sections required no resurfacing while three required reconstruction. Using the total rutting criteria, two sections required resurfacing while the others were fine. By use of the cracking criteria, two sections required surface dressing while the rest required resurfacing or minor overlay.

(d) Jomo Kenyatta Airport- Athi River road

This road is a two-way road while the other test roads were one way dual carriageway roads. The mean PSI for the first six sections was above three, while the last two sections were below 2.5. These last two sections required major overlay or reconstruction while one section required major overlay or reconstruction. One section required resurfacing. The rest of the four sections were performing well. The cracking index showed that five sections required patching while the other sections did not have any cracks.

5.6.3 Proposed lower roughness criteria

It would appear from the observed roughness measurements that the maintenance criterion based on roughness is relaxed using the current Roads Department (1988a) criterion. In order to set a more realistic roughness criteria, regression analysis of roughness (Ro) and other surface condition measurements namely total rut (TR), the cracking index (CI) and present serviceability index (PSI) were done. Equations 5.26 through 5.28 were developed.

$$Ro = 1615 + 0.83\ TR \quad (r=0.3) \qquad [5.26]$$

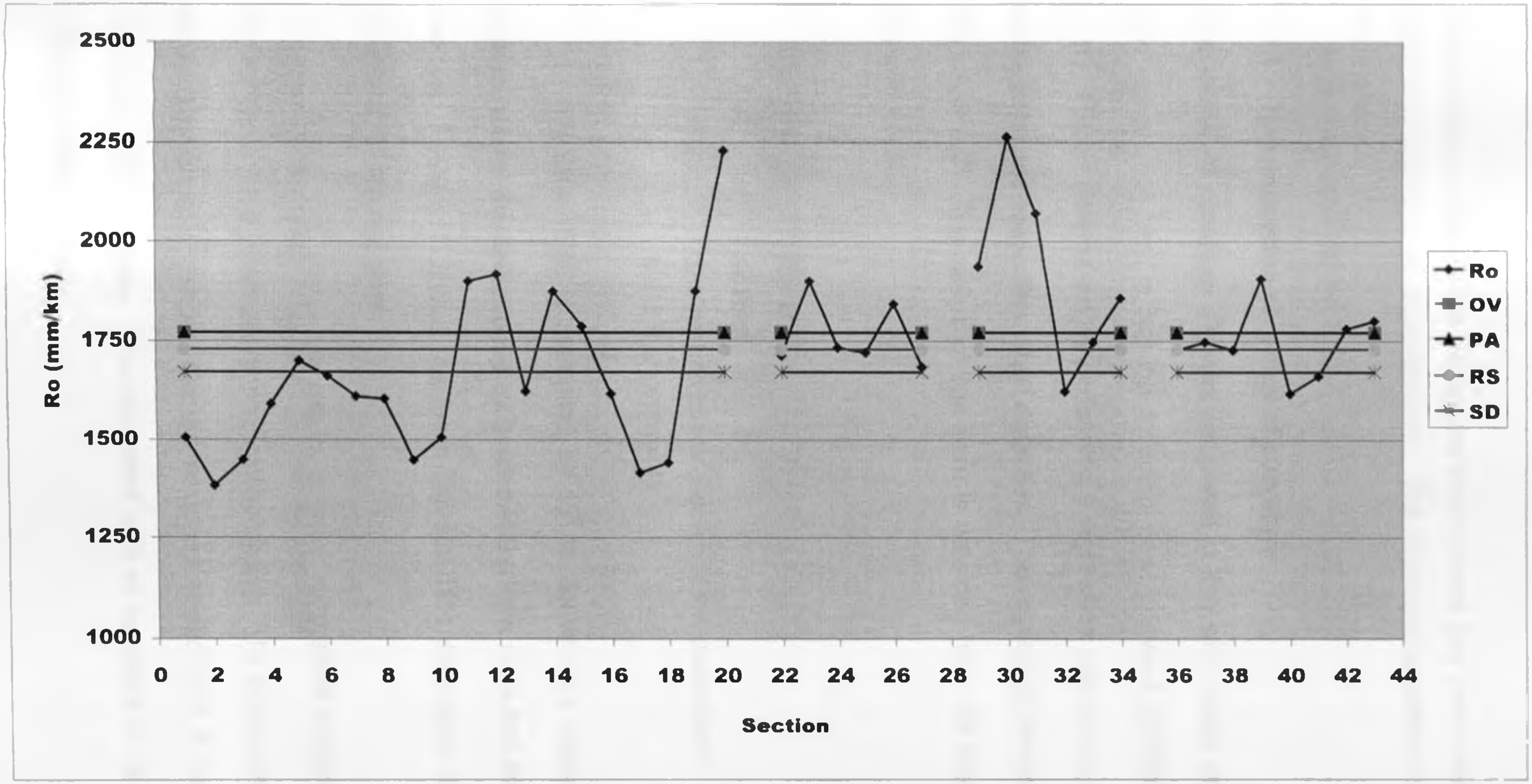
$$Ro = 1652 + 21\ CI \quad (r=0.3) \qquad [5.27]$$

$$\ln Ro = 7.60 - 0.048 PSI \quad (r=0.2) \qquad [5.28]$$

Using the relationships and converting the criteria in Table 5.12 into roughness values, the proposed maintenance and rehabilitation criteria based on roughness is shown on Table 5.14. Based on these proposed criteria the remedial measures for the test roads are shown on the Figure 5.23. The proposed measures compare favourably with those proposed on Figure 5.22 but are slightly more severe.

Table 5.14 Proposed maintenance and rehabilitation criteria based on roughness

Maintenance	Minimum roughness	
	Trunk roads	Others
Local patching	1770	1770
Surface dressing	1675	1675
Minor overlay or resurfacing	1730	1770
Major overlay or reconstruction	1770	1820



Legend: Ro - roughness, OV - overlay, PA – patching, RS – resurfacing, SD – surface dressing

Section 1-20 Nairobi – Thika road, 22-27 Nyayo stadium – Airport road, 29-34 Airport – Nyayo Stadium Road, 36- 43 Airport – Athi River road

Figure 5.23 Maintenance schedule based on the proposed new roughness criteria

5.7 Deflection data analysis

The deflection data picked during the field surveys has been used to model out the deflection bowl and its characteristics. The regressed equations have returned good correlations.

5.7.1 Determination of radius of curvature

The radius of curvature (R) was calculated in line with Roads department (1988a). The assumption here is that the deflection bowl follows a parabolic curve (Equation 2.15). R was determined by performing a regression analysis on the sets of inverse deflections and respective offset distances. The regression analysis is of the form of Equation 5.29. The radius of curvature is obtained from the relationship shown on Equation 5.30

$$1/y = A + Bx \qquad [5.29]$$

$$R = A/\,2Bd_o \qquad [5.30]$$

Where: d_o is the maximum deflection at the loaded point.
A and B are constants.

Equation 5.30 is multiplied by 10^5 to convert the radius of curvature into metres where the deflection is measured in 0.01mm units and offset is measured in metres. Table 5.15 shows the mean characteristic deflections at the various offset positions for the test sites.

Table 5.16 shows the results of linear regression analysis of the inverse of deflection and the offset positions (Equation 5.29). The high values of the coefficient of determination (r^2) show that the assumed relationship is good for analysis of rebound deflection data. The calculated radii of curvature R for the test sites were used for further analysis

Table 5.15 Mean characteristic deflection at various offset positions

Test site	Lane	Deflection (0.01mm) at offsets (m)					
		0.0	0.1	0.2	0.3	0.4	0.5
ES1	L	27	27	25	24	21	17
	R	15	15	14	13	13	11
ES2	L	46	43	39	37	34	29
	R	51	50	46	32	27	31
ES3	L	63	61	47	39	32	33
	R	47	46	39	29	24	17
ES4	L	45	43	39	33	28	24
	R	24	23	20	19	16	13
ES5	L	42	42	38	31	23	20
	R	56	54	43	33	24	16
ES6	L	11	11	11	8	8	6
	R	18	16	15	14	12	10
ES7	L	45	43	40	37	33	28
	R	74	69	62	53	47	43
ES9	L	52	49	44	31	23	14
	R	67	61	49	31	23	16
ES10	L	149	123	101	71	46	31
	R	65	62	52	38	26	16

Table 5.16 Regression constants for inverse deflections vs. offsets, maximum deflection and radius of curvature

Test Site	A	B	d_o (mm)	R (m)	r^2
ES1-L	0.034	0.041	27	1527	.83
R	0.064	0.045	15	4717	.86
ES2-L	0.021	0.024	46	954	.94
R	0.018	0.035	51	502	.79
ES3-L	0.015	0.035	63	339	.93
R	0.016	0.073	47	227	.89
ES4-L	0.020	0.040	45	554	.94
R	0.038	0.067	24	1164	.91
ES5-L	0.019	0.056	42	410	.90
R	0.011	0.086	56	114	.86
ES6-L	0.078	0.147	11	2216	.82
R	0.053	0.083	18	1765	.93
ES7-L	0.021	.026	45	892	.92
R	0.013	.021	74	421	.98
ES9-L	0.012	.021	52	105	.82
R	0.008	.095	67	63	.90
ES10-L	0.003	.049	149	21	.88
R	0.008	.088	65	66	.82

Legend:- L – Left lane, R – Right lane

5.7.2 Rutting at the experimental sites

The rutting results indicate that the heavily loaded section of the homogenous pavement structures tend to develop deeper rut than the lightly loaded section as expected. More importantly the effect of slope is felt, as the grades become steeper. Thus on ES5 and ES6 the traffic loading and pavement structures are the same. The difference here is in the grade. ES5 being on the steep climbing lane while ES6 is on the rolling section of the Airport-Athi River town test road. Despite the strengthening eight years into the service life the rut depth at ES5 was fifty-two millimetres twice that at ES6 which was not strengthened.

The use of high quality pavement accompanied by timely maintenance is inevitably useful. Thus on ES3 and ES4 along Nyayo Stadium-Airport road, the initial construction consisted of 100mm asphalt surfacing, graded crushed base and subbase placed on an improved subbase. This combined with an overlay of thirty-five millimetres after eight years of service gave rut depths of ten millimetres after twelve years of service.

On the test site ES2 the maintenance at the end of twenty-two years consisted of slurry seal while that of ES1 at the end of twenty-two years was a one hundred thirty five-millimetre thick overlay. The slurry seal did not improve the pavement resistance to rutting as the overlay did. Thus ES1 had a rut of ten millimetres against ES2 rut of twenty-nine millimetres. The difference in pavement structures makes a more elaborate comparison difficult

The ES7 was constructed in 1981 and after carrying four million standard axles, the ruts were deep (34.1mm). The test section was in urgent need of strengthening at the time of study, from rutting criterion.

5.7.3 Deflection variation with time

The deflection of a flexible pavement changes with time after construction. In the initial phase, various pavement layers undergo some consolidation due to the wheel loads. The amount of consolidation depends on the compaction received on the various layers during construction. When properly compacted, the deflection of a pavement during the consolidation has been found to be small (Roads Department, 1988a). The consolidation causes slight decrease in surface deflection.

After this initial phase the pavement passes on to the elastic phase. During this phase the wheel loads produce a deflection which is recovered upon passage of the vehicles. The elastic deflection remains constant during this stage. The stage can be regarded as the useful working period of the pavement.

Subsequently the pavement enters the plastic phase. In this phase the traffic loading and environment effects on the pavement are manifested. There is loss of strength of the pavement layers (Gichaga, 1982). The bitumen surface has cracked (Mwea and Bezabeh 1994). Each wheel passage produces small non-recoverable deformation, which manifest as rut (Wakyendo, 1991). The surface deflection increases and the pavement is usually in need of strengthening to preserve its structural integrity. After the plastic phase the pavement will usually fail if not strengthened. There is usually a sharp increase in deflection.

Against this background, deflection data has been assembled from past studies by Gichaga (1979), Murunga (1983), Atibu (1986) and Wakyendo (1991). Table A5.2 in appendix A shows the years of service, traffic loading and the mean deflection for the nine experimental sites that were also visited during this study. Figure 5.24a shows the variation of deflection with load for the high volume test sites while Figure 5.24b shows deflection verse time for the low volume sites. The main features from the deflection history for each site are discussed below:

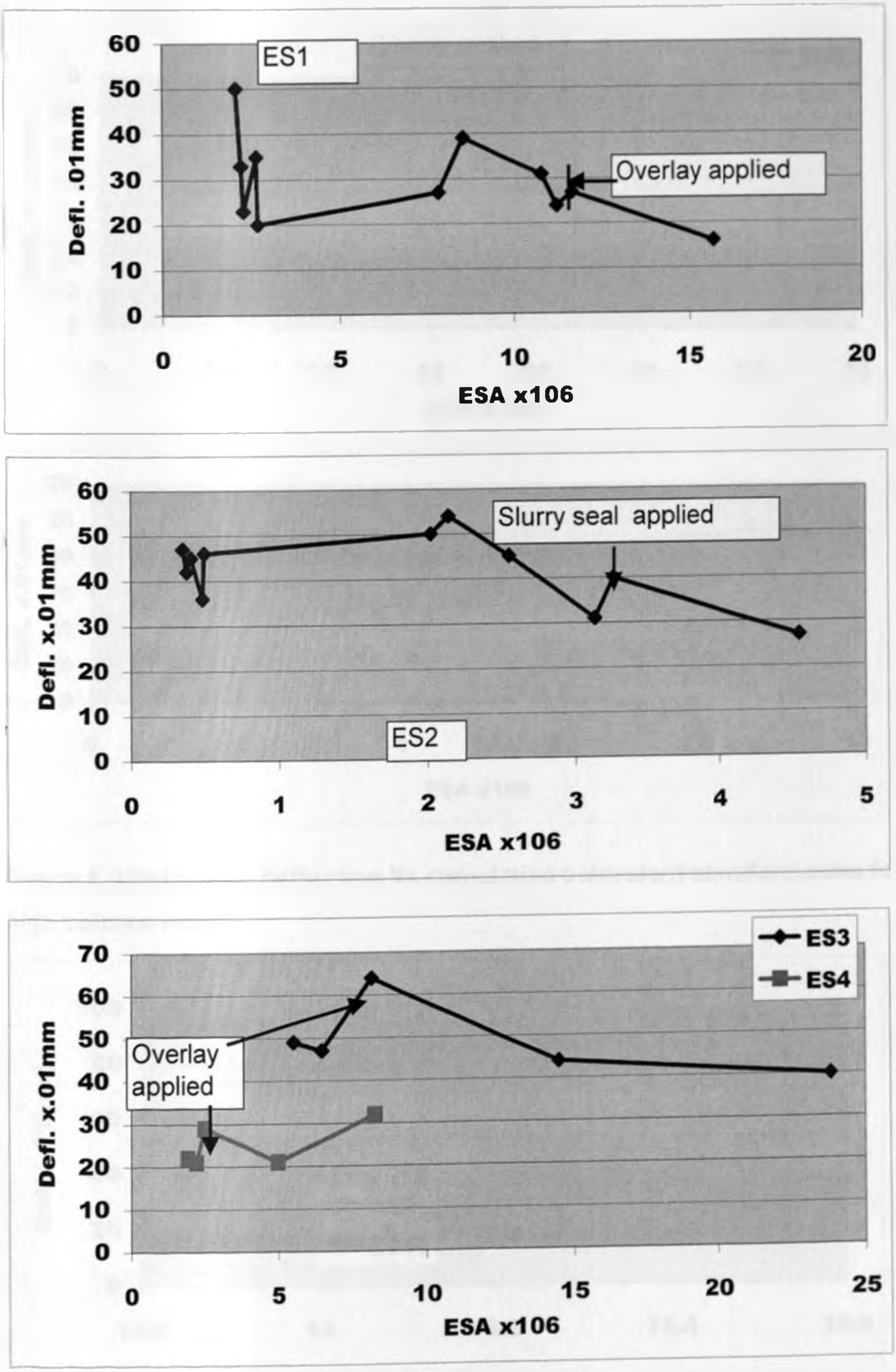
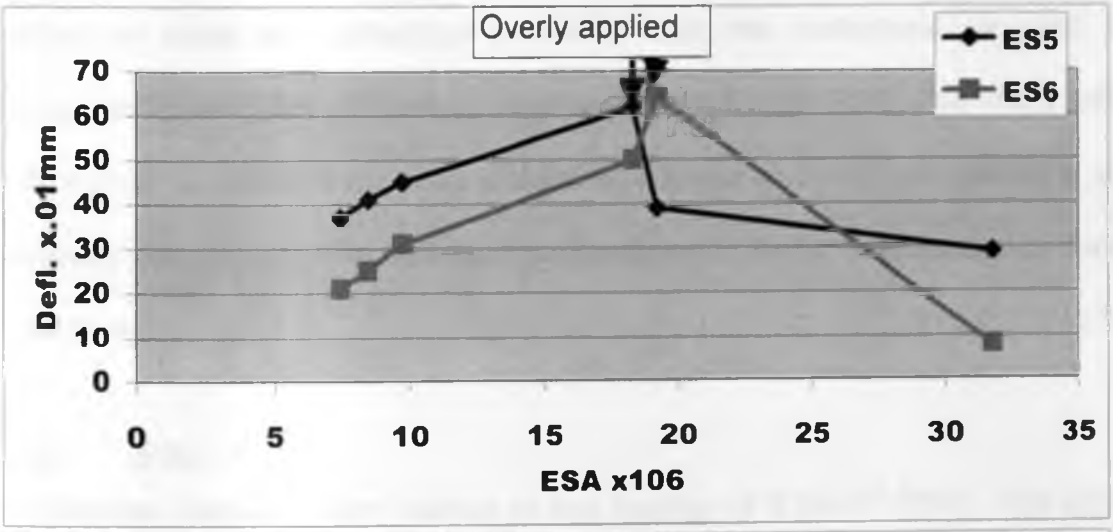


Figure 5.24a Deflection Vs cumulative equivalent standard axles for high volume roads



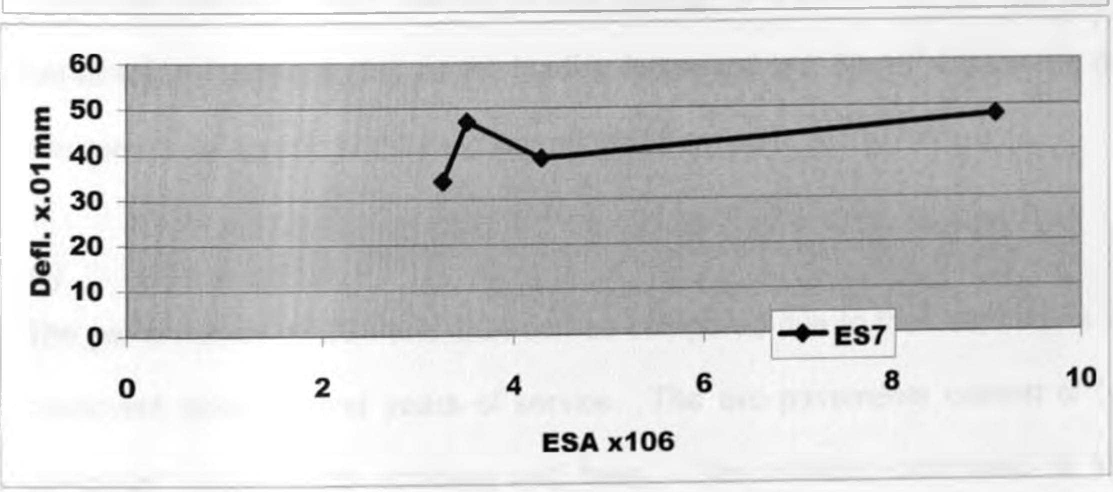


Figure 5.24a (contd.) Deflection Vs cumulative equivalent standard axles for the high volume roads

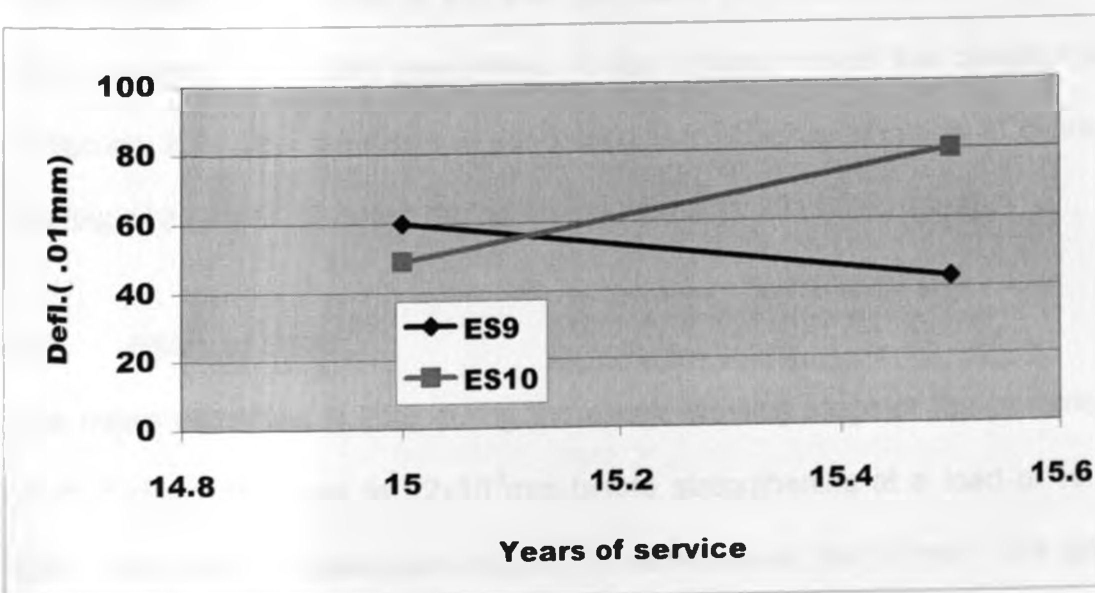


Figure 5.24b Deflection Vs years of service for the low volume roads

(a) ES1

After an initial high deflection of $50x10^{-2}$ mm, the deflections remained bound between 20 and 35 x 10^{-2} mm up-to a loading of $8.51x10^6$ ESA. The mean deflection then rose to $39x10^{-2}$mm. This deflection reduced to $31x10^{-2}$mm before a 135mm overlay was placed. The deflection then reduced to $24x10^{-2}$mm remaining low and at the final loading of $15.71x10^6$ ESA the deflection had reduced further to $16x10^{-2}$ mm.

(b) ES2:

Deflection measurements started at low loading of $0.34x10^6$ ESA. The deflection remained in the mid forties as the loading increased to $2.56x10^6$ ESA when a slurry seal was then applied and deflection reduced from 45 to $31x10^{-2}$mm.

(c) ES3 and ES4:

The performance of ES3 and ES4 can be compared due to their similarities in their pavement structure and years of service. The two pavements consist of 330mm combined cohesionless subbase and base. The surfacing consisted of 100mm-asphalt concrete. However, the low loading on ES4 kept the deflections consistently low over the years. Thus at five and half years the mean deflection for ES3 was 53mm, while that for ES4 was 24mm. It was however noted that despite the large deflection ES4 also remained in good condition. Upon application of overlay at a loading of $2.49x10^6$ ESA, the deflection reduced from 29 to $21x10^{-2}$mm.

(d) ES5 and ES56:

The mean deflection at ES5 during the elastic working stage of the pavement was $41x10^{-2}$ mm. This rose to $62x10^{-2}$mm before strengthening at a load of $18.31x10^6$ ESA. The overlay intervention reduced the deflection to $39x10^{-2}$mm. This deflection continued to decrease and at a loading of $31.70x10^6$ ESA after thirteen years of service the deflection reduced to $29x10^{-2}$mm.

Prior to getting loaded to $19x10^6$ ESA, the pavement at ES6 was performing well with deflection ranging from21 to $31x10^{-2}$ mm when the deflection rose to $62x10^{-2}$ mm. ES5 deflections were on the average 61% higher than those of ES6. The difference in deflections was however found to lessen with age. The higher deflections are attributable to the effect of low-speed heavy axle-loads on the steeper ES5 test site. This was also manifested in the rutting (plastic deformation) which was higher on the steep ES5 test site.

(e) ES7

This site was still in its elastic stage at $9.08x10^6$ ESA loading and eight and half year's service. The deflections fell between 34 and $48x10^{-2}$mm

(f) ES9 and ES10

ES9 and ES10 test sites are on low volume tea roads. The testing on these two sites was separated by seven months. Within that period the deflection at ES9 decreased from 60 to $43x10^{-2}$ mm while at ES10 it increased from 49 to $80x10^{-2}$ mm. The two sites have lower strength pavements compared to the high volume sites. Hence their deflections were higher. The sites did not manifest surface defects. This was despite their seventeen years of service. The low traffic combined with the ability of surfacing to remain intact preserved the pavement surface

5.7.4 Deflection bowls

Figure 5.25 shows half deflection bowls for the various test sites. The deflection bowls have been compared where the test sites share common features. The features of the sites are then discussed below.

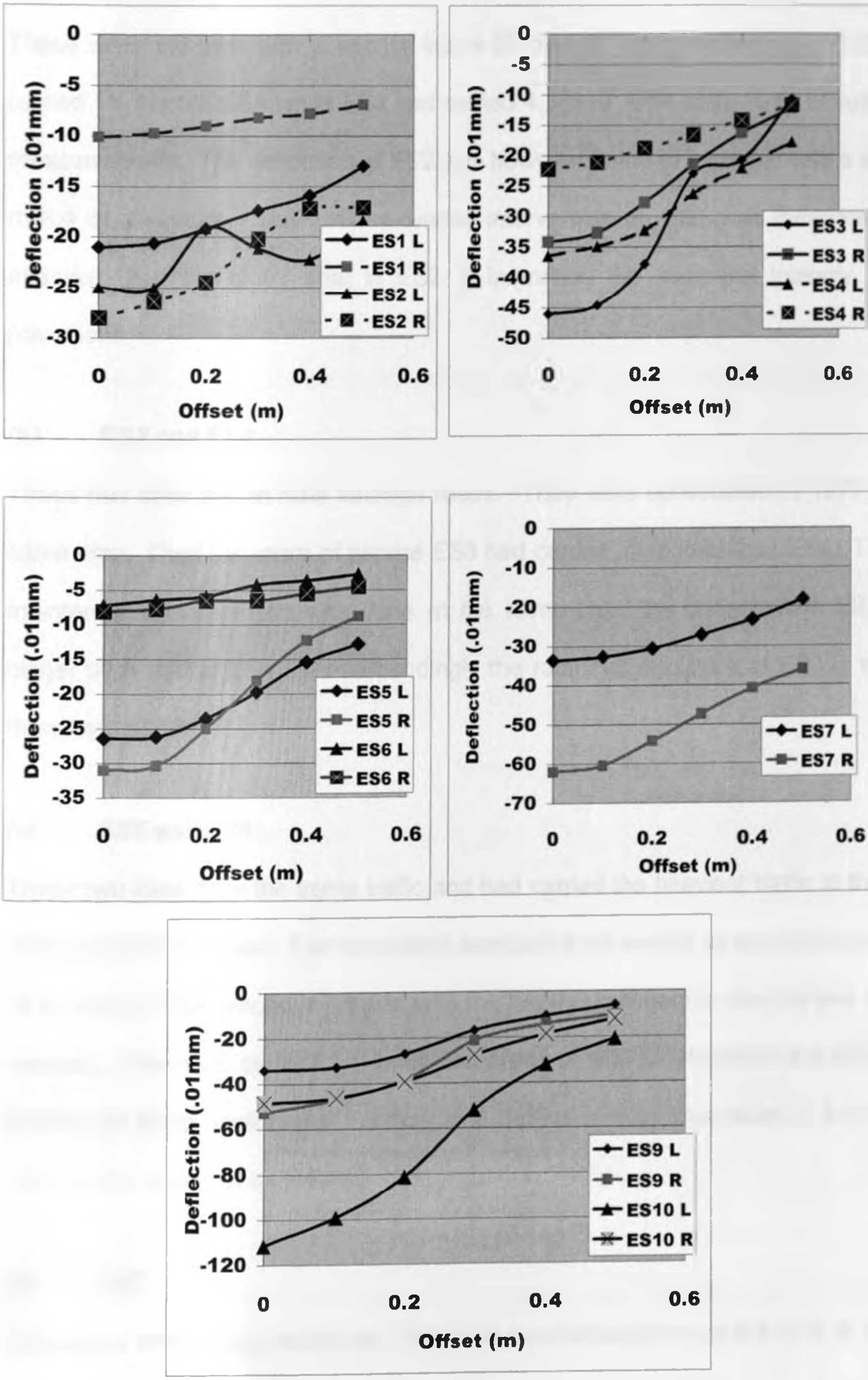


Figure 5.25 Deflection bowls for test sites

(a) ES1 and ES2:

These were old sites with a service life of 28 and 27 years respectively. ES1 had carried 15.71x106 ESA while ES2 had carried 4.53×10^6 ESA at the time of deflection measurements. The deflection at ES2 was however found to be larger with a shorter radius of curvature. The 135mm-overlay intervention on ES1 was therefore more effective than the slurry seal in ES2 in improving the structural integrity of the pavement.

(b) ES3 and ES4:

These two sites are on dual carriage roads. They were constructed in 1977 at the same time. Over the years of service ES3 had carried more load than ES4. Though maintenance intervention was done at the same time, the deflection at ES3 was bigger than that at ES4. Correspondingly the radius of curvature at ES3 is shorter than that at ES4.

(c) ES5 and ES6:

These two sites have the same traffic and had carried the heaviest traffic at the time of testing of all the sites. The equivalent standard axles carried at each of these sites of 31.70×10^6 ESA placed the traffic with the highest category in the Kenyan design manual. This was class T1 (Roads Department 1987). However the ES5 was located on steep section and the slow lane deflection bowl was found to be deeper with smaller radius of curvature.

(d) ES7

This site is on a natural subgrade. Historical data showed that at the time of testing the mean deflection was within the elastic phase magnitudes recorded at the beginning of monitoring. However the deflection bowl for the right hand lane was found to be generally deeper than that of the left lane.

(e) ES9 and ES10:

These two sites are on low volume tea roads. Their deflection bowls when compared to those of high volume roads would appear to be large and therefore the sites required a periodic maintenance intervention. However, visual examination of the site showed that the surface condition was good with no cracks and no permanent deformation. The deflection criteria would therefore appear to be unsuitable performance indicator for the low volume roads.

5.7.5 General observations

The interpretation and comparison of deflection data is complex due to a large number of variables that affect deflection. Thus, besides the above presentation of historical performance, deflection and by extension the structural strength of the pavement depends on routine maintenance. These include, minor resealing and patching, which seals off the cracks and the incidence of weakening the base and underlying pavement structures. The deflection is also affected by improvement of drainage, which again increases strength and reduces deflection.

An overall deflection comparison for all the sites is difficult. The difficulty in interpretation can be seen in Figure 5.26, which shows the relation of deflection with loading for all the sites. The Figure shows no noticeable trend. However comparison of deflections for individual sites immediately before resealing or overlaying or immediately after resealing or overlaying shows an immediate drop in deflection (Figure 5.27). The order of drop varies but an average of 35% was found.

A regression analyses of R vs. d_{90} was carried out for all the test sites (Equation 5.31). From the coefficient of determination (0.74) and the low ANOVA signf. F of .000 it can be seen that the R is significantly related to the d_{90}. This was despite the large variation in the pavement structures.

$$R = 596949 d_{90}^{-1.95} \quad (r^2=0.74) \qquad [5.31]$$

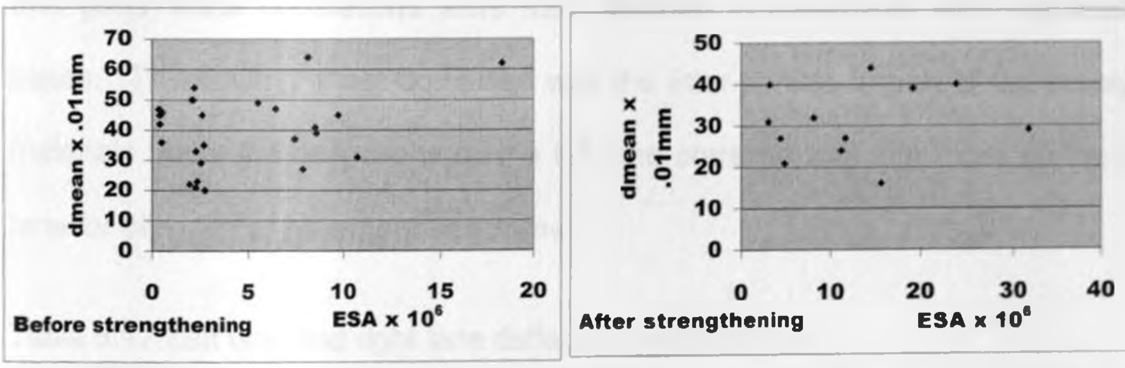
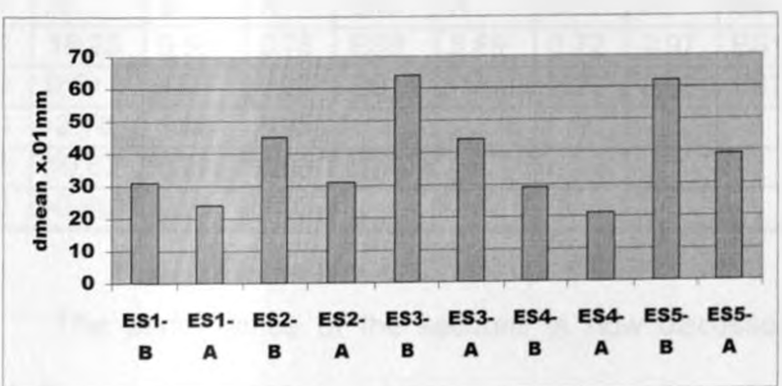


Figure 5.26 Deflection Vs load scatter for all the test sites



Legend: A – After strengthening, B – Before strengthening

Figure 5.27 Comparison of deflection before and after strengthening

5.7.6 Structural performance of the test sections

The deflections at the variations offset positions on the left lane were regressed with those of the right lane. The regression assumed that deflection for the left lane could be calculated from Equation 5.32. The values of signf F were below 0.05 showing a significant relationship.

$$d_l = A + B^* d_r \qquad [5.32]$$

Where: d_l is the deflection for the left lane

d_r is the deflection for the right lane

Statistical results for test sites are shown on Table 5.17. Gichaga (1979) found ES1 deflections for the left and right lane were well related. He likened the behaviour to that of a concrete slab. He attributed this to the cementing action of the stabilised murram, which was used in the base. Results from the Table 5.17 shows that good linear correlations were also obtained in pavements with cohesionless bases. The binding effect combined with the inter-particle friction of the pavement materials make the deflections on the left lane correlate well with those on the right lane for all types of pavement structures.

Table 5.17 Left lane and right lane deflection relationships

Cohesionless bases								Cohesive bases			
High volume sites				Low volume sites				High volume sites			
Site	A	B	R^2	Site	A	B	r^2	Site	A	B	r^2
ES2	18.23	0.50	0.76	ES9	5.89	0.72	0.97	ES1	-9.96	2.48	0.94
ES3	9.91	1.07	0.93	ES10	-11.0	2.27	0.97	ES7	8.44	0.50	0.95
ES4	-2.78	1.99	0.97								
ES5	10.62	0.59	0.98								
ES6	-0.49	0.68	0.83								

The performance of the sections is now discussed and compared to the Roads Department (1988a) guidelines.

The deflection data gives the product Rd_o that measures the strength condition of the pavement. A well performing pavement has typically high values of radius, which raise the product Rd_o. With a value of Rd_o above 10000 the pavement would be sound. Between 5500 and 10000 the pavement would be in transition requiring intervention in the form of surface dressing, patching or minor overlay to prevent further deterioration. When Rd_o falls below 5500 the pavement would then require a major overlay or reconstruction.

The concept of equivalent modulus (Eq) was discussed in section 2.5.5. Based on the deflection and pavement layer thickness for the various test sites their equivalent modulus has been calculated from data presented on Table 5.16. The total rut and cracking, which were measured during the deflection survey, are presented on Table 5.18 alongside Rd_o and Eq. The data is also presented in the form of a chart (Figure 5.28) to enable comparison of the performance and also check on the state of the section when compared to the Roads Department (1988a) thresholds for well performing pavements. The various test sites are discussed in detail using these test results:

Table 5.18 Rd_o, Equivalent modulus and surface condition for test sites

Test Site	Rd_o M*mm	Eq Kg/m^2	Rut (mm)	Total rut (mm)	CI (mm/m^2)
ES1-L	41222	5929	10.2	93	0
R	70756	12116			
ES2-L	43883	2902	29.0	277	0
R	25618	3989			
ES3-L	21400	2382	11	100	0
R	10668	2249			
ES4-L	24948	3394	10	106	.53
R	27941	6592			
ES5-L	6389	1438	52	382	3.03
R	17228	2969			
ES6-L	24378	8641	28	151	0
R	31780	12437			
ES7-L	40125	2371	34	299	36
R	31182	4359			
ES9-L	5488	987	55	15	0
R	4240	1448			
ES10-L	3172	381	66	16	0
R	4268	1025			

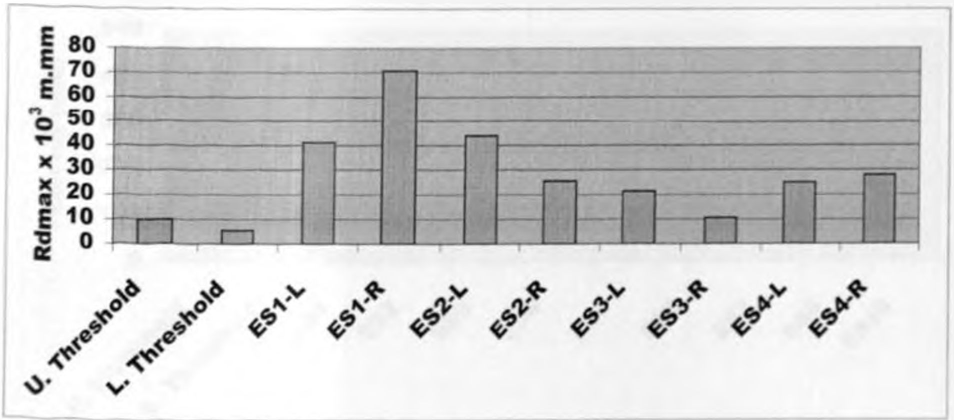


Figure 5.28 Performance of the experimental sites

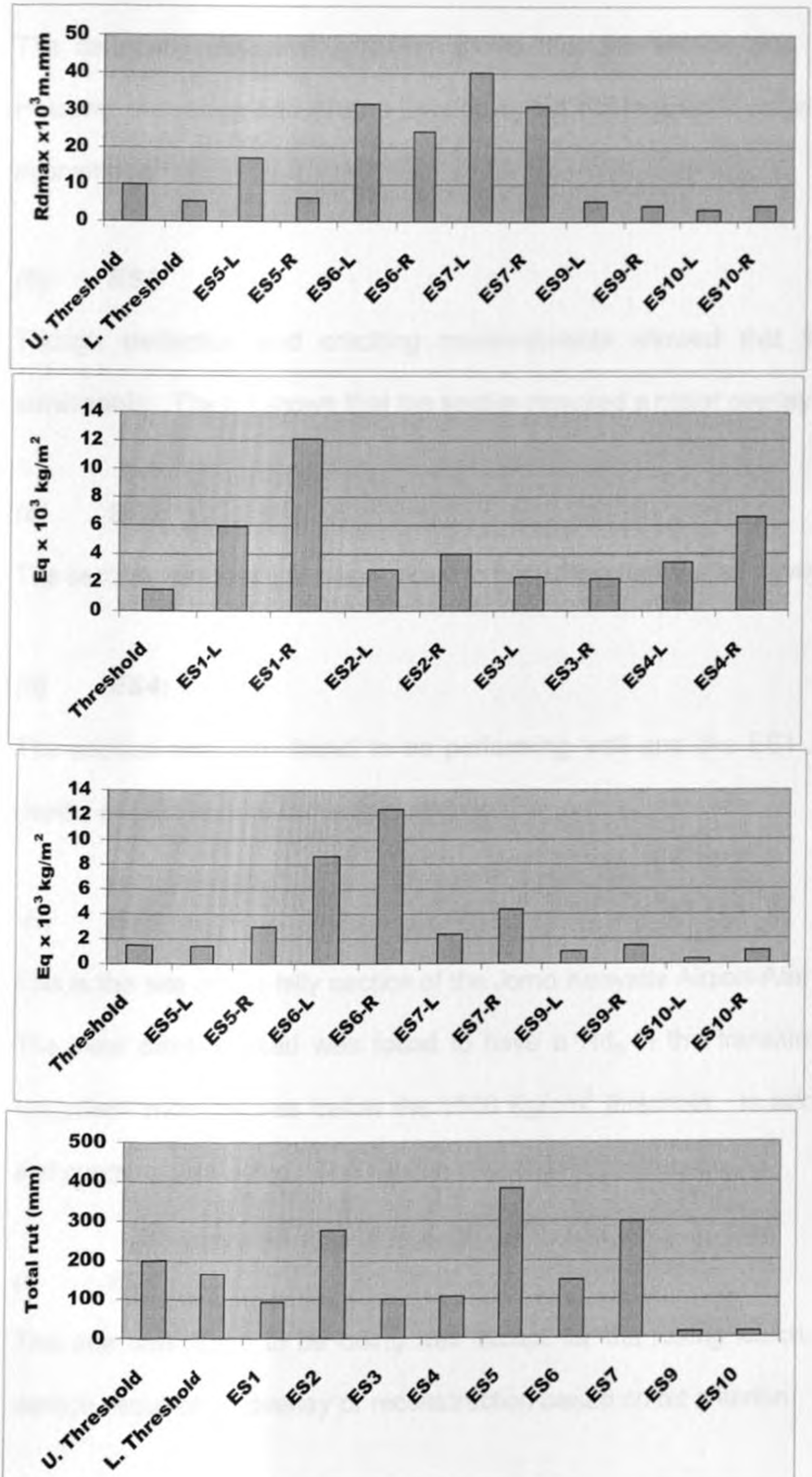


Figure 5.28 (contd): Performance of the experimental sites

(a) ***ES1***:

The deflection data and cracking shows that the section was performing well. However the rutting had started developing but not reached a stage for maintenance intervention.

(b) ***ES2***:

Though deflection and cracking measurements showed that the section was serviceable. The rut shows that the section required a major overlay intervention

(c) ***ES3:***

The section was found to be serviceable but rutting had started developing.

(d) ***ES4:***

The section was also found to be performing well and like ES1 and ES3 the rut depths did not require immediate attention.

(e) ***ES5***:

This is the site on the hilly section of the Jomo Kenyatta Airport-Athi River town road. The slow climbing road was found to have a Rd_0 in the transition range and the equivalent modulus was below the 1500 Kg/cm^2 threshold. In addition high rutting and cracking was noted. The section required reconstruction.

(f) ***ES6***:

This site was found to be doing well except for the rutting which was deep. The section required an overlay or reconstruction based on rut criterion.

(g) ***ES7***:

Though the deflection data appeared to suggest a sound pavement, the rutting and cracking were found to be high. The section therefore required reconstruction.

(h) ***ES9 & ES10***:

These two sections can be discussed together. The deflection data shows that the pavements are weak. However visual inspection showed that the sections were performing well. The low traffic on this road, estimated at below 500 average annual daily traffic (AADT) had not damaged the pavement structure over the years of service. Deflection as tool for pavement evaluation on these sections can therefore be misleading without an accompanying conditional survey. No maintenance intervention was required in these two sites as no heavy trucks were expected to ply the test sites.

5.8 Case airport flexible pavement investigations

5.8.1 Introduction

The analysis and discussion of the investigation of the case airport is now presented. Chapter four presented the methodology and data collection for the case study. This section presents analysis and discussions with respect to the field and laboratory test results. The presentation starts with findings of field and laboratory test results of the non-bituminous materials and drainage aspects for the pavements. The results of laboratory tests of bituminous materials, rutting and deflection surveys are then presented.

5.8.2 Materials below the bituminous layers and drainage considerations

Figure 5.29 shows the variation of the standard penetration blows (N) along the runway and taxiway. The blows were conducted in the improved subgrade horizons (depth 0-1m) and top fill materials, (depth 1-2 m). It can be seen that the improved subgrade material was generally softer than the top fill material. This scenario is contrary to design expectation. It resulted from ingress of water through the cracks, which were observed during the visual inspection.

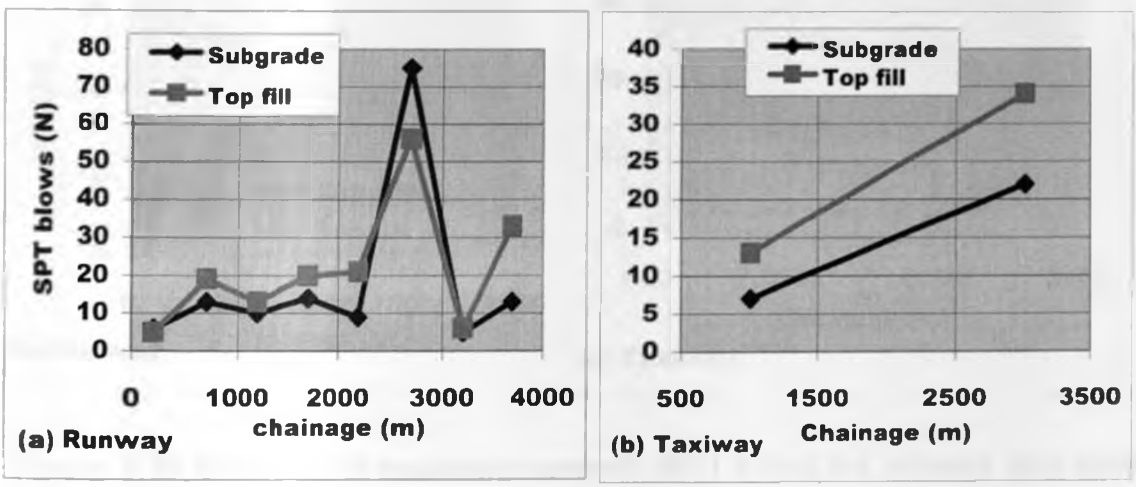


Figure 5.29 Variation of SPT along the runway and taxiway for subgrade and top fill

The blow count at km 2.7 defied the general trend of the count, which were 5-14 for the improved subgrade, and 5-33 for the upper fills. The isolated hardness was a result of brown stabilised murram, which was encountered below the graded crushed stone base. This stabilised layer protected the brown silty clay fill material for top fills and subsequently raised its N value. This isolated stabilised layer was not shown in the as built drawings or described in the materials report. The other non-bituminous layers encountered at the other drilled sections generally agreed with construction sections shown on Figures 4.1 and 4.2.

Figure 5.30 shows the variation of field moisture content of the improved subgrade and that of the top of the fill, along the runway and taxiway. In the case of the runway the moisture content for the improved subgrade was between 20 and 33%. This was higher than that of the top fill, which ranged from 18 to 32. This scenario was different at km 2.7 and 3.2 where the moisture content increased with depth. For the taxiway the moisture content at km 3.00 increased with depth.

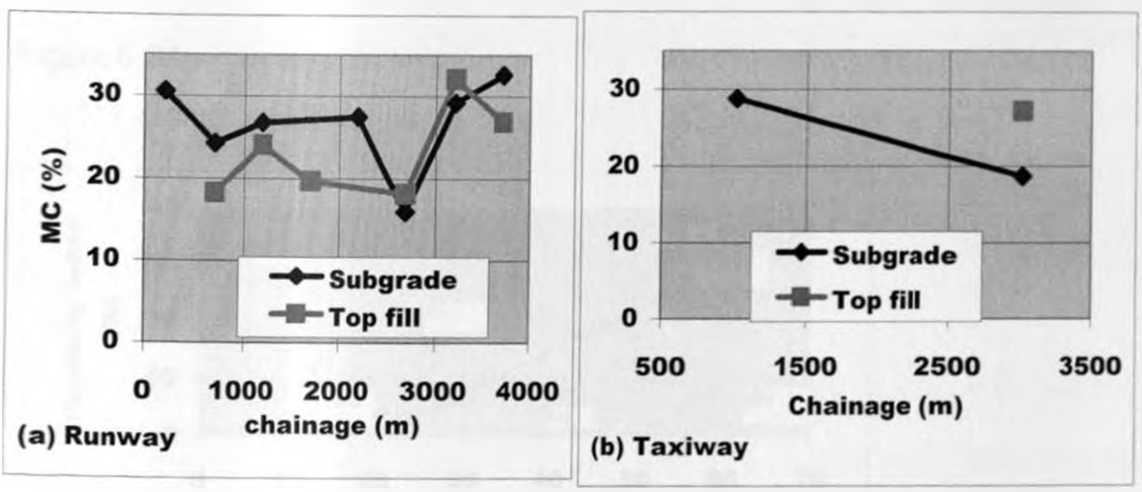


Figure 5.30 Variation of moisture content (MC) along the runway and taxiway for subgrade and top fill

The general increase of moisture content from the fill towards the pavement layers confirmed that the moisture content was due to precipitation percolation into pavement layers. When the moisture conditions of these materials was compared to that of the verges it was found out that the verges were drier than the material below the bituminous materials. Again this confirmed that water percolating into the pavement layers from the top was trapped raising the moisture content above the optimum moisture content of the materials.

CBR on samples remoulded at MDD and OMC showed that for both the improved subgrade and the top fill the CBR was 11 for boreholes up-to km 2.7. The CBR for fill materials at km 2.7 and 3.2 were higher at 16.0 while at km 3.7 it dropped to 3.0. Free swell measurements showed fill materials at km 1.2 and 3.7 had free swell of 17% and 15%. The other borehole materials did not have swell characteristics.

The fill materials were subjected to grading and atterberg limit tests for classification. The grading showed that the materials were fine grained while the atterberg limits plotted around the A line and can be classified as a mixture of silts of low compressibility (ML) and clays of low compressibility (CL) as can be seen on Figure 5.31

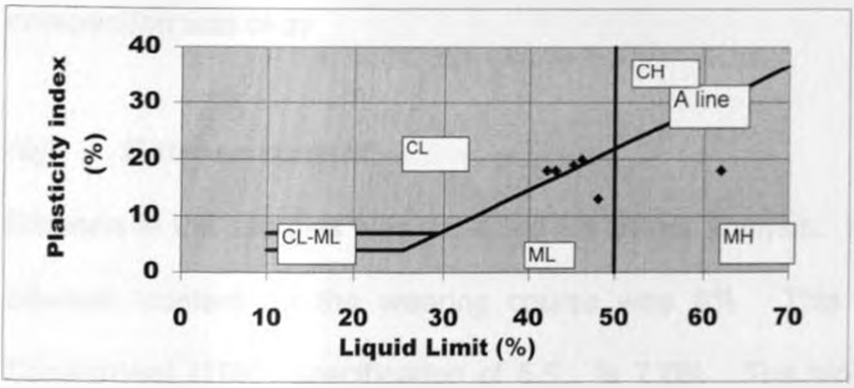


Figure 5.31 Classification of fill soils below the runway and the taxiway pavements

The State of California, Department of Transport (1992) recommends that a drainage layer should be incorporated in every flexible pavement design for the interception of the surface water that enters the pavement. Such a pavement layer extends outside the edge of the carriageway. The recommended permeability of a well-drained material is of the range of .01 cm/sec to .05 cm/sec. In the study case permeability tests on the improved subgrade and the top fill showed that these two materials had permeability in the order of 10^{-6} to 10^{-5}cm/sec. These low permeability values meant that the improved subgrade and the top fills were not well draining. Hence the subgrade and top fill materials retained the moisture, which they received from the top of the pavement. The material falling in the category of good drainage material is the sand-gravel mixture (Craig 1987).

5.8.3 *Bituminous materials*

(a) Density of samples:

Results of the test samples were shown on Table 4.2. The results showed that the densities of the cores recovered from the boreholes ranged from 2.06 to 2.29gm/cc. This range is within the densities obtained in the bituminous laboratory samples, described in chapter three. It would therefore, appear that the bituminous mixture compaction was okay.

(b) Bitumen content:

Bitumen in the samples was extracted for further analysis. It was observed that the bitumen content for the wearing course was 6%. This was within the Roads Department (1987) specification of 5.5. to 7.0%. The binder course and original wearing course had a bitumen content of 5.0- 5.9. This was within the specification range of 5.0 to 6.5%.

(c) Grading

The results of the grading of the aggregate were within the grading envelopes for the 0/14 high stability wearing and binder course.

(d) Penetration, softening point and air voids

The most significant test results with respect to the bituminous pavement materials were the results for the assessment of hardening of the bitumen binder. The hardening was assessed by the measurements of standard penetration and softening point. The penetration range of 11 to 19 achieved was much lower than the 60-70 range expected for 60/70 penetration grade bitumen used for the construction of the pavements. The hardened bitumen results in cracks and brittle surface (Mwea and Bezabeh 1994). The penetration (Pen) was found to be inversely proportional to the voids in total mix (VTM) as established for road asphalt concrete surfacing. The relationship obtained is of the form of power regression (Equation 5.33) which has a coefficient of correlation (r) of 0.82. The value of ANOVA signf F was below 0.05 showing a significant relationship, which compares well with the road pavements. The Equation compares well with Equation 5.18 developed for asphalt concrete surfacing for roads.

$$Pen = 112\,(VTM)^{-1} \qquad (r=.82) \qquad [5.33]$$

The softening point (Sp) ranged from 70^0c to 78^0c.This was well above the 48^0 -56^0c Roads Department (1987) specifications. The softening point was also power related to the voids in total mix as shown on the regressed Equation 5.34, which had a coefficient of correlation (r) of 0.80. The penetration decreases with the softening point as shown on Equation 5.35, which had a high coefficient of correlation of 0.98

$$Sp = 46\,(VTM)^{0.23} \qquad (r=.80) \qquad [5.34]$$

$$Pen = 1.543x\,10^9\,Sp^{-4.3} \qquad (r=.98) \qquad [5.35]$$

Equations 5.33 through to 5.35 are presented graphically on Figure 5.32. The Figure also compares the penetration/air-voids trends for the airport pavements with that the roads asphalt concrete surfacing. The trends were found to be fundamentally the same.

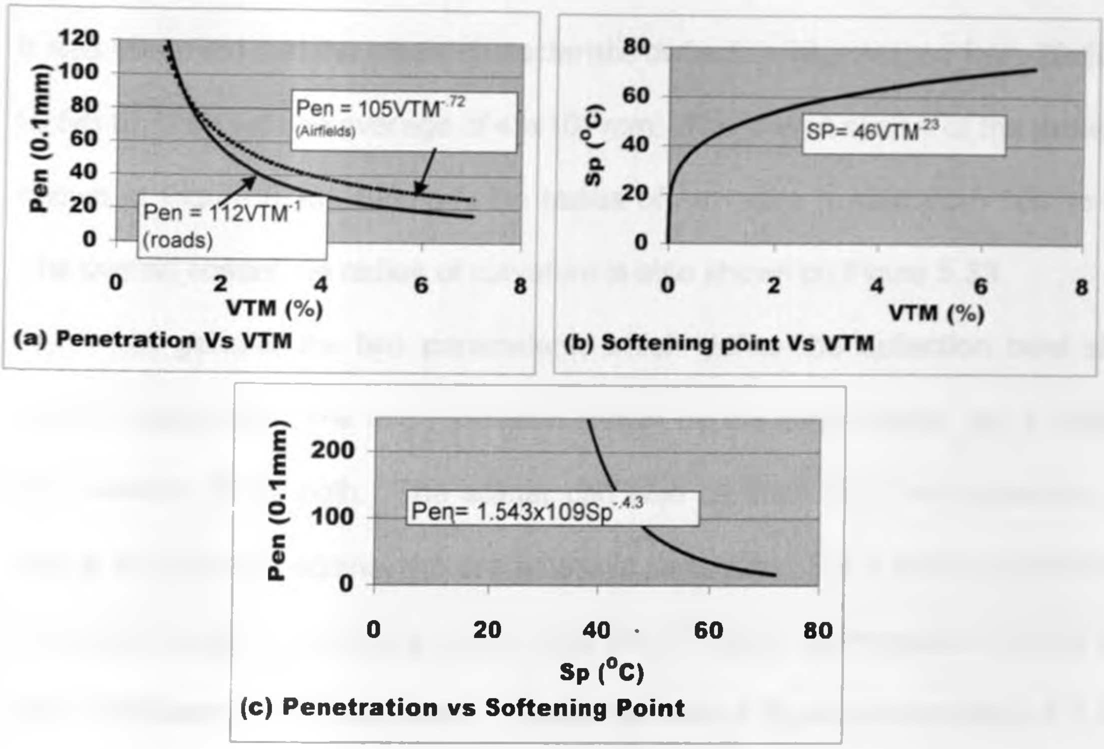


Figure 5.32 Penetration, softening point, and voids in total mix relationships for the case study airport flexible pavements

5.8.4 Roughness test results

The roughness measured was generally below 1000mm/km with only 2% of the sample in the case of taxiway having higher value and 10% in the case of runway. Roughness design criteria was not available for airport measurements. It was however considered that a roughness value of about 1000mm/km in the case study would represent critical conditions. This criterion is much more severe than that proposed by Roads Department (1988a) for highway pavements which requires trunk roads to be resurfaced and to receive major overlay at a roughness of 2800 and 3400 mm/km respectively. The 1000 mm/km is also more severe than the proposed criterion of trunk roads that requires resurfacing and major overlay at a roughness of 1675 and 1865 respectively. Even under these criterion the pavement was judged to be in reasonable condition of roughness.

5.8.5 Runway deflection test results

It was observed that the mean characteristic deflection (d_{90}) ranged from 20×10^{-2} mm to 58×10^{-2} mm with an average of 43×10^{-2} mm. The overall scatter of the deflection is shown in Figure 5.33. Similarly the radius of curvature ranged from 56m to 818m. The overall scatter the radius of curvature is also shown on Figure 5.33.

In general the two parameters, which define the deflection bowl show a normal distribution .The large variation shown by the large scatter, are indicative of the variation of strength. The scatter can also be seen from the regression of the radius of curvature against the characteristic deflection. For a uniform pavement the deflection would be basically similar and the R versus d_{90} regression would yield a high coefficient of determination. Instead the overall regression analysis of R versus d_{90} gave Equation 5.36 with a coefficient of determination (r^2) of 0.35 and a low ANOVA signf, F of .003.

$$R = 21.4 \times 10^3 d_{90}^{-1.20} \qquad [5.36]$$

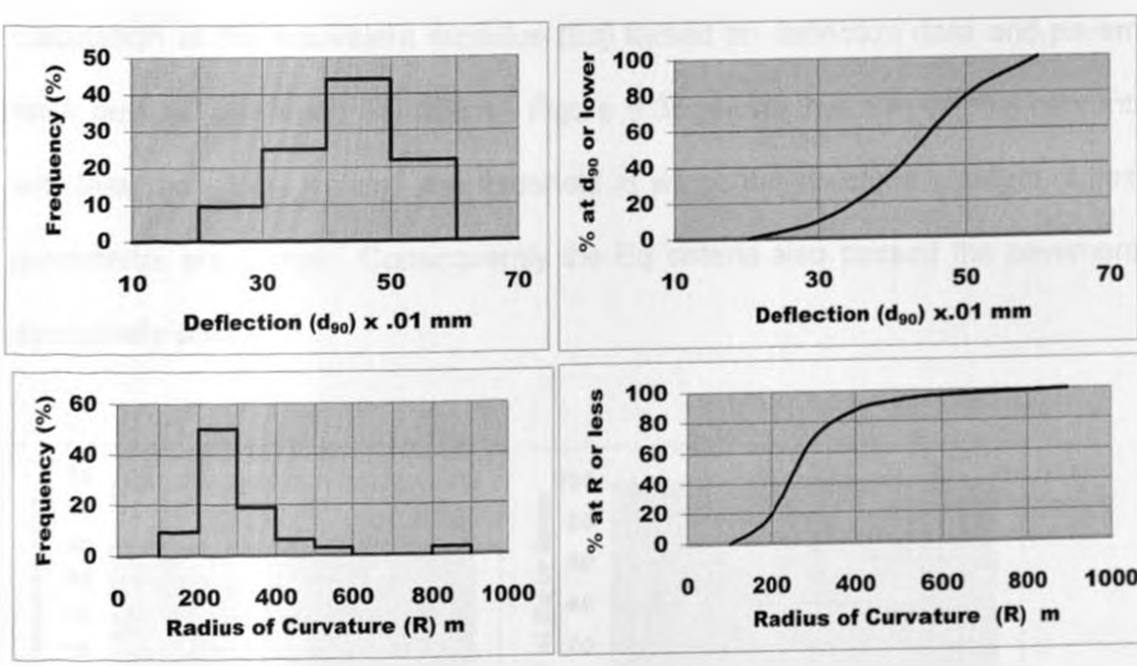


Figure 5.33 Characteristic deflection and radius of curvature scatter for the runway

The product of radius of curvature and deflection (Rd_o) was used to assess the structural condition of the runway pavement. The histogram and cumulative frequency of Rd_o is shown on Figure 5.34. The cumulative frequency curves show 85^{th} and 15^{th} percentile give Rd_o values of 23,000 and 8000 respectively. Only 3% of the test points gave Rd_o values of less than 5500. The values of Rd_o greater than 10,000 were 80%. This means that Rd_o criteria passed the runway pavement as structurally sound.

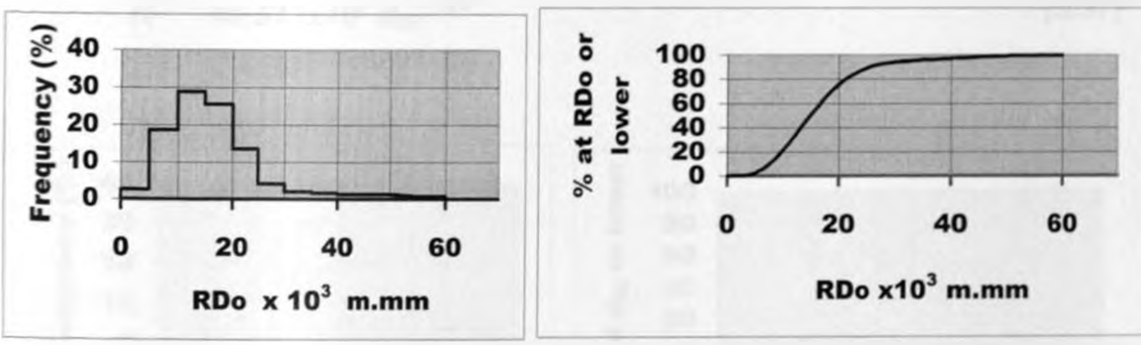


Figure 5.34 Product Rd_0 scatter for the runway

The final assessment of the structural integrity of the pavement was the calculation of the equivalent modulus (Eq) based on deflection data and pavement layer depths, confirmed by drilling. Figure 5.35 shows that a negligible percentage was less than 1500 Kg/cm^2, the threshold at which the structural strength of flexible pavements are critical. Consequently the Eq criteria also passed the pavement as structurally sound

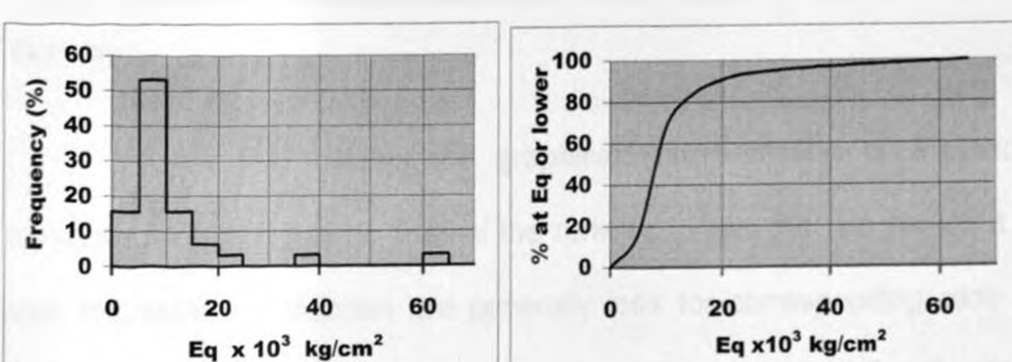


Figure 5.35 Equivalent moduli (Eq) scatter for the runway

5.8.6 Taxiway deflection test results

A similar analysis for the taxiway was carried out. The overall scatter of deflection and radius of curvature values is shown on Figure 5.36. A large scatter of the deflection bowl parameters can be seen. A regression of radius of curvature and characteristic deflection gave a coefficient of determination of 0.19 and a high ANOVA signf F of 0.091 (Equation 5.37)

$$R \quad = 9.573 x 10^3 \, d_{90}^{-1.02} \qquad [5.37]$$

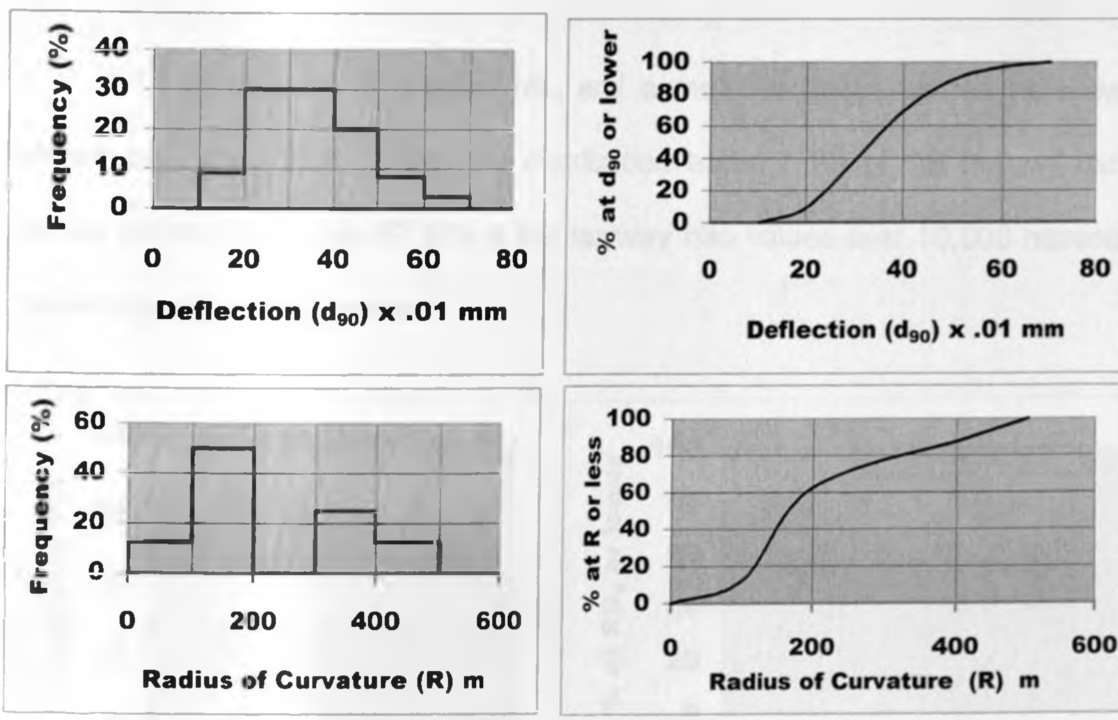


Figure 5.36 Characteristic deflection and radius of curvature scatter for the taxiway

Figure 5.37 shows the graphical representation of Equation 5.37 and comparison with Equation 5.36 of the runway. From the two curves it can be seen that the taxiway deflection are generally less for corresponding radii of curvature. Overall therefore Rd_0 for the runway is higher than that for the taxiway. This suggested that the runway pavement was structurally superior to that of the taxiway

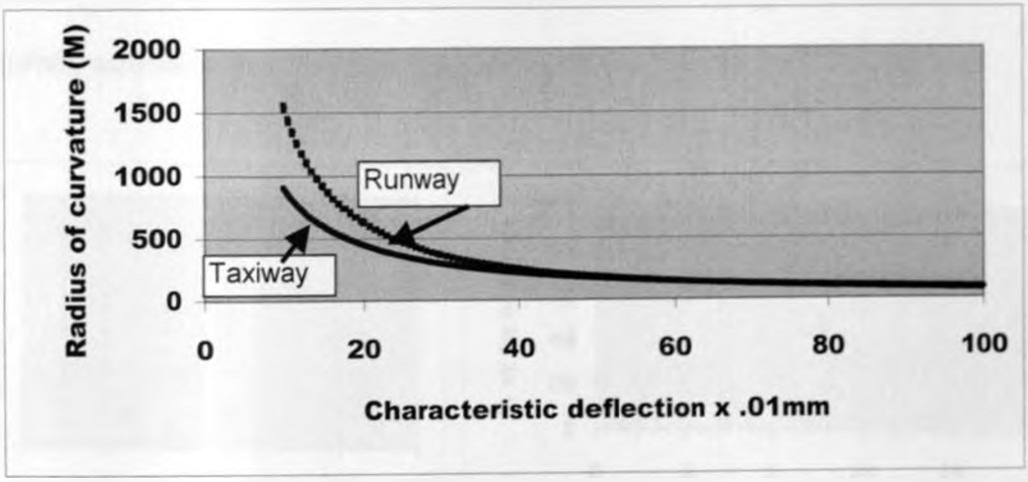


Figure 5.37 Radius of curvature Vs characteristic deflection for the runway and taxiway

The histogram of product Rd_0 and cumulative frequency for the taxiway is shown on Figure 5.38. From the distribution about 7.1% of the taxiway had Rd_0 values below 5500 while 67.8% of the taxiway had values over 10,000 representing the structurally sound portion.

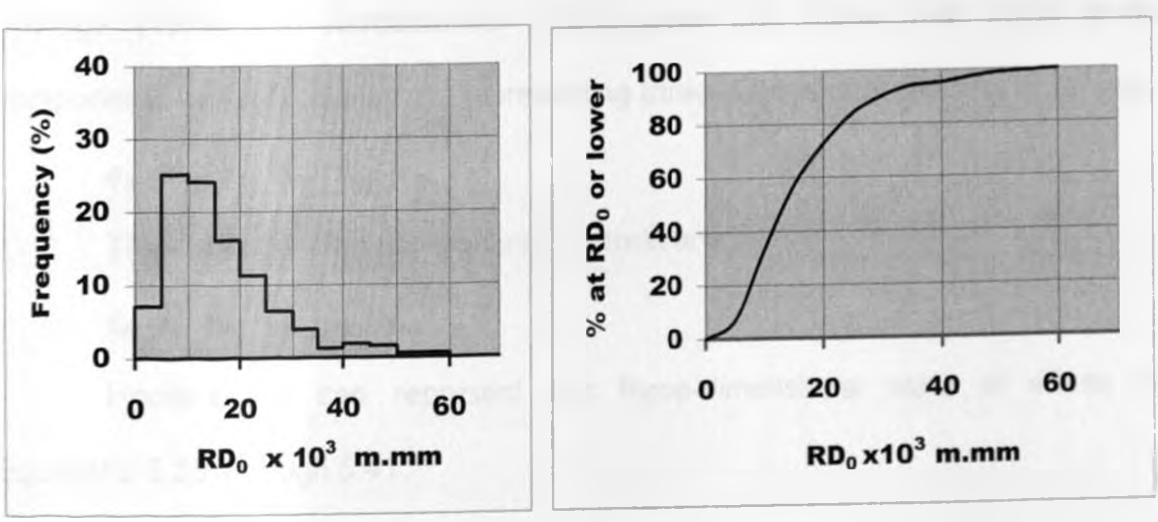


Figure 5.38 Product Rd_0 scatter for the taxiway

The equivalent modulus computation gave the distribution shown on Figure 5.39. This Figure read in conjunction with Table 4.3 shows that only about 12.5% of the taxiway pavement had an equivalent modulus less than the 1500kg/cm^2

threshold. The equivalent modulus criteria therefore passed the pavement as structurally sound.

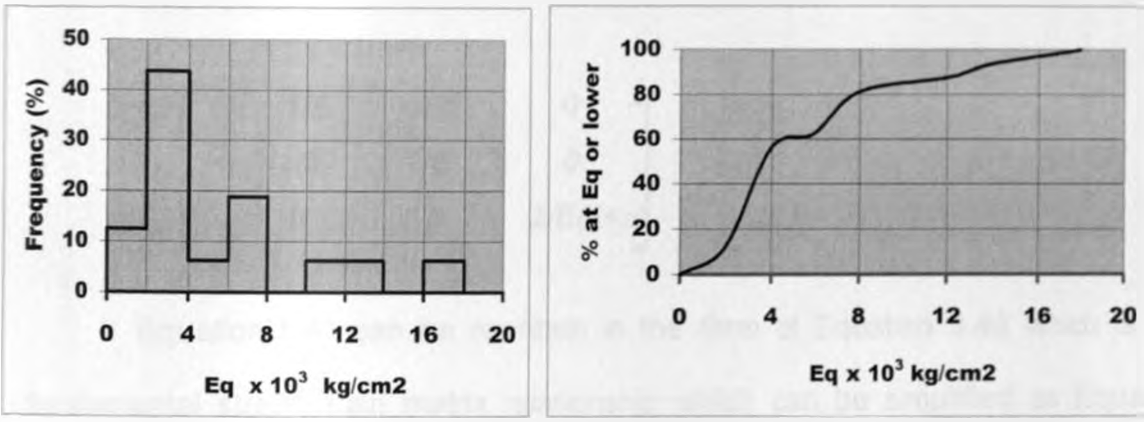


Figure 5.39 Equivalent modulus (Eq) scatter for the taxiway

5.9 Finite element analysis

5.9.1 Introduction

Gichaga (1979) and Kardestuncer (1987) gave the normal and shear stress components in a cubical element representing three-dimensional analysis as follows:

$\sigma_x, \sigma_y, \sigma_z, \tau_{xy}, \tau_{xz}, \tau_{yz}$

The corresponding components of strain are:

$\varepsilon_x, \varepsilon_y, \varepsilon_z, \gamma_{xy}, \gamma_{xz}, \gamma_{yz}$

Hooke's law can represent this three-dimensional state of stress by Equations 5.38 through 5.41.

$$\varepsilon_x = 1/E(\sigma_x - \mu(\sigma_y + \sigma_z) \quad [5.38]$$

$$\varepsilon_y = 1/E(\sigma_y - \mu(\sigma_x + \sigma_z) \quad [5.39]$$

$$\varepsilon_z = 1/E(\sigma_z - \mu(\sigma_y + \sigma_x) \quad [5.40]$$

$$\gamma = 2/E(1 + \mu)\tau \quad [5.41]$$

Where: E is the Young's modulus

μ is the Poisson's ratio

In the pavement evaluation by FEM analysis plane stress analysis gives sufficiently accurate results (Murunga et al 1985). Equations 5.38 through 5.41 are then reduced to matrix Equation 5.42.

$$\begin{bmatrix} \varepsilon_x, \\ \varepsilon_y, \\ \gamma_{xy} \end{bmatrix} = \begin{bmatrix} 1/E & -\mu/E & 0 \\ -\mu/E & 1/E & 0 \\ 0 & 0 & 2/E(1+\mu) \end{bmatrix} \begin{bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{bmatrix} \qquad [5.42]$$

Equation 5.41 can be rewritten in the form of Equation 5.43 which is the fundamental stress-strain matrix relationship which can be simplified as Equation 5.44 showing the stress matrix obtained by multiplication of the stiffness and strain matrixes.

$$\begin{bmatrix} \sigma_x, \\ \sigma_y, \\ \tau_{xy} \end{bmatrix} = \begin{bmatrix} E/(1-\mu^2) & \mu E/(1-\mu^2) & 0 \\ \mu E/(1-\mu^2) & E/(1-\mu^2) & 0 \\ 0 & 0 & E/2(1+\mu) \end{bmatrix} \begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \qquad [5.43]$$

$$\begin{bmatrix} \sigma \end{bmatrix} = \begin{bmatrix} D \end{bmatrix} \begin{bmatrix} \varepsilon \end{bmatrix} \qquad [5.44]$$

In the theoretical analysis by the finite element technique two-dimensional analysis of stress and deformation was assumed. The pavement structures were idealised in the X-Y plane. The loading was in the same plane.

Under this loading idealisation the pavement structure is under plane stress and plane strain. The direct stress in the Z plane and the shear stress in the Z-X and Z-Y planes are small and ignored in such an analysis. The element thickness in the Z direction was based on the Boussinesq theory (Smith and Smith, 1998). This shows that variation of stress was therefore in accordance with this theory.

The first step in finite element analysis is discretization and selection of the element configuration. The structure is usually divided into finite elements, which intercept at nodes. A large number of elements are made to increase the accuracy.

An approximation model or functions for the unknown parameter is then selected. The unknown quantity is the nodal displacement. The distribution function to represent the deformed shape is chosen. The choice here follows the laws, principles and constraints in the problem. The number of independent displacements that can occur in a node defines the degrees of freedom.

In the third step, stress-strain relationships are defined. For pavement structures the stress deformation linear elastic analysis (Hookes law) is used. Subsequently the vector nodal forces are obtained by multiplication of stiffness matrix and vector nodal displacement matrix.

As the element equations are assembled, the boundary conditions are introduced to ensure continuity of displacements. Finally the results of analysis yield displacements, stress and strains. The above in a nutshell represents the steps in a finite element computation suitable for flexible pavement analysis. With fast development of computer application, a package, LUSAS-S produced by Finite Element Analysis Ltd. of Surrey UK was used for this project.

The underlying assumption was that:

- The pavement structure was considered as an elastic multilayer system.
- The materials forming the various layers of the pavement structure had characteristic elastic modulus.
- The Poisson's ratio for all the layers was 0.3
- Layers had complete friction between them

5.9.2 Finite element formulation

The models for finite element analysis were selected from those sites where deflection was measured by use of the benkleman beam to allow for comparison of theoretical computation with the field deflection. The model geometry was adopted from known pavement structural configuration and physical measurements.

The pavement structure its geometry and materials composition including the wheel loading was symmetrical about the centreline of a two-lane single carriageway. Accordingly only half of the cross-section was modelled.

Boundary conditions were defined by allowing the symmetry line to deform in the vertical direction only. The bottom of the pavement was restrained in the X and Y directions.

The elastic modulus (E) values were based on those recorded by the Roads Department (1987). For the bituminous materials the modulus was determined considering the material as a visco-elastic with a load application equivalent to 60 kilometre per hour. The elastic moduli for the unbound non-visco-elastic materials were determined by static loading.

During analysis E values were varied between those for new materials and fractions of the said E values. This was an attempt to model the reduction in elasticity due to age and trafficking (Gichaga, 1982). A uniform Poisson's ratio of 0.3 was chosen. Changes in the value of Poisson's ratio have no major effect on pavement structural behaviour (Murunga, 1983).

The loading system was uniform as in the deflection field measurements. It consisted of two-twin wheel 1470 mm apart. Each twin wheel was 3175 kg to make up an axle load of 6350 equivalent to the field loading. The wheel load impression on the pavement surface was 180mmX200mm. The 180mm breadth was divided into two parts to form three nodal load positions 90mm apart. There were thus twelve loading nodes per cross section. Each nodal load was 5.19 kN. Figure 5.40 shows the loading system.

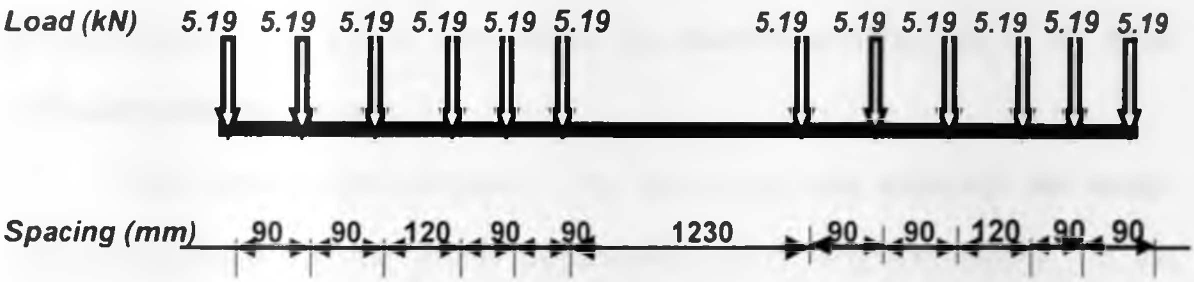


Figure 5.40 :Loading positions used during the finite element analysis

The FEM accurately determines the pavement condition of the idealised pavement under the estimated E and μ. The difference in the FEM and the benkleman beam measured deflections is as a result of the difficulty in the modelling out the field load and the pavement structure. For example the application of load in the FEM analyses is inevitably simplified as point loads which is different from the actual field loading. Additionally the boundary conditions in the field are difficult to model. When loading is assumed to be in the form of point loading the analyses leads to higher stresses and consequently higher deflections at the loading points than would be obtained by field measurement.

5.9.3 Details of pavement structural models

Of the nine sites where deflection was carried out four sites were chosen for analysis by the FEM. These sites included ES1, which represented the pavement structures, which had cement stabilised bases. ES3 On the other hand represented the pavements with cohesionless bases. ES7 was chosen as it was the only high volume site with a DBM as its base. The low volume test sites were represented with ES9. The details of the structural models are now presented.

(a) ES1:

The shoulders were surface dressed and 1500mm wide. The model consisted of 970 rectangle elements and 1040 nodes. Figure A5.4 in appendix A shows the element arrangements. The Figure also shows the element arrangement of the other analysed experimental sites.

Two models were analysed. The first model was analysed with elastic modulus equivalent to the Roads Department (1987) recommendations. In the second model the moduli was halved for all the pavement layers. The structural details for this site and other sites are presented on the Table 5.19.

(b) ES3:

The shoulders were also surface dressed. They were found to be 1300mm wide. The carriageway was 6500mm wide. As in ES1 two models were analysed. In the first model the elastic moduli of new materials was used while in the second one the moduli was halved. A longitudinal analysis of this site was analysed to compare the longitudinal profile with that obtained in the deflection field measurements.

Table 5.19 Finite element models

Layer	Material type	Elastic modulus kN/mm^2	Element Thickness mm
Test site ES1 model I (full modulus)			
Surfacing	135mm asphalt concrete	2.5	100
Base	300mm cement stabilised gravel	4	1040
Subbase	450mm Natural gravel	0.2	4300
subgrade	1000mm Natural Material	0.09	4300
Test site ES1 model I (half modulus values)			
Surfacing	135mm asphalt concrete	1.25	95
		1.25	380
Base	300mm cement stabilised gravel	2	1152
		2	3971
Subbase	450mm Natural gravel	0.1	8448
subgrade	1000mm Natural Material	0.045	8448

Table 5.19 (contd.) Finite element models

Layer	Material type	Elastic modulus kN/mm^2	Element Thickness mm
Test site ES3 model I (full modulus)			
Surfacing Base Subbase subgrade	135mm asphalt concrete 130mm GCS 200mm crushed stones 300mm improve subgrade	2.5 0.4 0.3 0.065	100 384 1166 1166
Test site ES3 model II (half modulus)			
Surfacing Base Subbase subgrade	135mm asphalt concrete 130mm GCS 200mm crushed stones 300mm improve subgrade	1.25 0.2 0.15 0.32	100 385 1166 1166
Test site ES7 model I (half modulus values)			
Surfacing Base Subbase subgrade	150mm asphalt concrete 150mm Dense bitumen macadam 250mm Natural gravel 500m subgrade of CBR= 20%	1.25 2.5 0.1 0.01	75 1190 5962 5962
Test site ES7 model ii (quarter modulus values)			
Surfacing Base Subbase subgrade	150mm asphalt concrete 150mm Dense bitumen macadam 250mm Natural gravel 500m subgrade of CBR= 20%	.625 1.25 0.05 0.005	75 1190 5962 5962
Test site ES9 model I (half modulus values)			
Surfacing Base Subbase subgrade	Surface dressing 130mm crushed stone 100mm Natural gravel 500m subgrade of CBR= 20%	- .15 .1 0.1	- 100 227 2273
Test site ES9 model II (quarter modulus values)			
Surfacing Base Subbase subgrade	Surface dressing 130mm crushed stone 100mm Natural gravel 500m subgrade of CBR= 20%	- 0.3 0.2 0.02	- 100 227 2273

(c) ES7:

The carriageway was found to be 7000m with1500mm wide shoulders. Two models were analysed with E values assigned half and a quarter values for new materials.

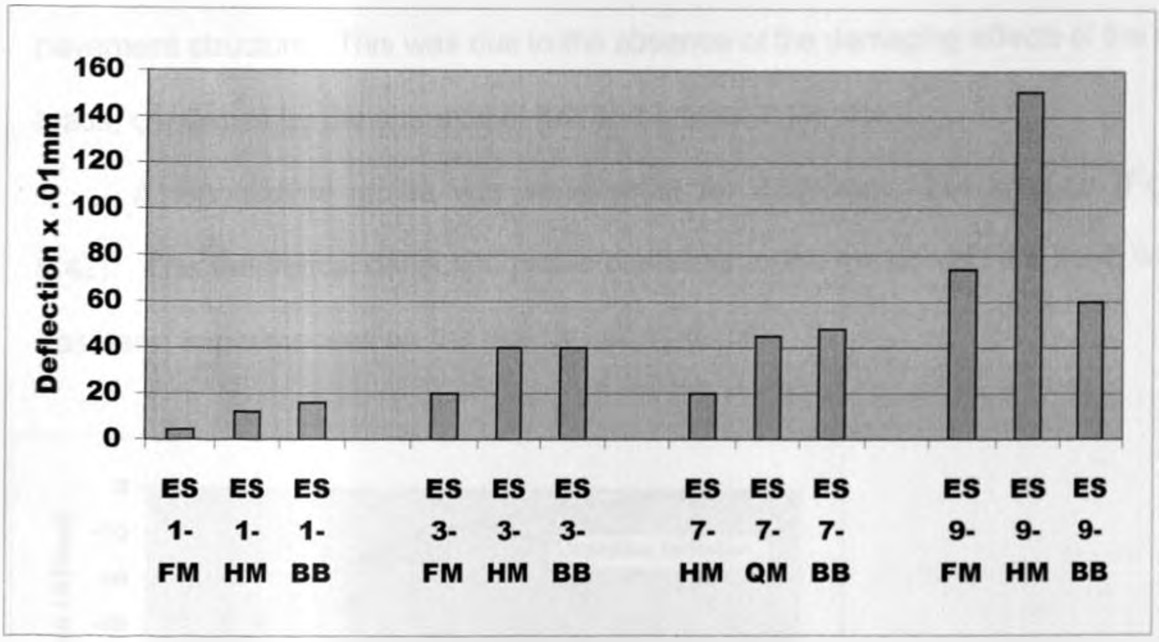
(d) ES9:

This site was on a low volume road. The width of the shoulders was 1000mm gravel. Two models were analysed. For the two models E values were assigned full and half values for new materials. It is to be noted that the other low volume site ES10 had the same service history and also the same environmental and traffic conditions.

5.9.4 Finite element analysis results

(a) Deflection

It was found that deflections decreased from the higher values at the pavement surface to the subgrade. In the high volume sites the deflection associated with elastic moduli for new materials were found to be low when compared to those in the field. Adjustment of the moduli to an appropriate fraction of the moduli resulted in deflections approaching the field results. Typical deflections under the wheel loads are shown on Figure A5.5 in appendix A. The summary of the surface deflections from analysis and field measurements is shown on Figure 5.41.



Legend: FM – Full modulus, HM – Half modulus, QM – Quarter modulus, BB – Benkleman beam

Figure 5.41 Comparison of FEM and BB deflections

Reduction of the modulus by half resulted in the surface deflection approaching field results for ES1 and ES3. The deflections in ES3 were generally more than those recorded in ES1. This is difficult to explain due to variation in environmental factors and loading history. However a major contributory factor is higher moduli and thicker pavement materials with respect to ES1.

For ES7 at half the elastic modulus the deflection was still quite low when compared to the field results. It was at the reduction of the moduli to a quarter that of new materials that the deflection was comparable to the field results. The extensive cracking observed during the field survey probably caused the very low moduli of the material, which resulted in the relatively high deflection. The cracks must have let in water into the underlying cracks, which weakened the pavement.

The theoretical deflection for ES9 was $74x10^{-2}$mm compared to the field value of $60x10^{-2}$mm. Further reduction of the modulus to half the value increased the theoretical deformation to $151x10^{-2}$mm. It would appear that despite the seventeen years of service, the surface dressing helped in preserving the quality of the pavement structure. This was due to the absence of the damaging effects of the axle loads, confirmed by the absence of ruts and cracks at the site.

A longitudinal profile was constructed for ES3 from FEM analysis (Figure 5.42). The theoretical deflection profile conforms to the measured field bowl, which has been superimposed on the plot for half modulus

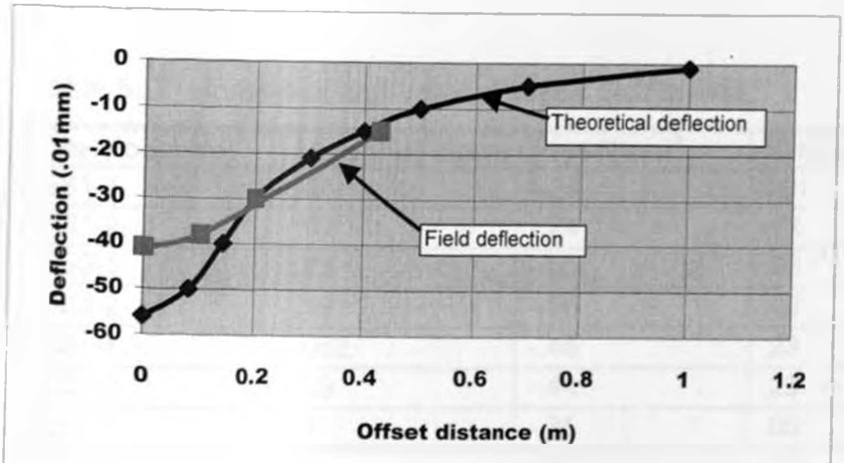


Figure 5.42 Comparison of longitudinal field deflection and FEM analyses deflection for ES3

This analysis showed that FEM techniques can be used for estimation of the material modulus at the time of testing. This is possible when FEM analysis is done together with field deflection data. The material modulus at the time of testing can be referred to as residue modulus. The residue modulus would have dropped from a high value at the time of construction due to the traffic loading and environmental factors.

(b) Stresses

FEM analysis showed substantial stresses being taken by surfacing. The reduction of moduli did not alter the stress distribution significantly. The pavement geometry is therefore key in stress distribution. Figure A5.6 in appendix A shows the detailed stress contours for the test sites. Table 5.20 shows the distribution of the vertical and horizontal stresses for the test sites.

The highest stresses were found on ES7. This site had a dense bitumen macadam base. The high tensile stress induced in the surfacing and base could be partially responsible for the cracking. The other main contributory factor is the environment effects discussed earlier. In ES9 where the surface dressing was ignored in the FEM analysis the base and subbase layers took the stresses.

Table 5.20 Horizontal and vertical stress distribution

Site/Model No	Horizontal stress range N/mm^2		Vertical stress range N/mm^2	
ES1-1	.05	-.24	-.01	-.61
ES1-2	.02	-.24	.02	-.64
ES3-1	.19	-.55	.06	-.63
ES3-2	.19	-.56	.06	-.63
ES7-1	.082	-.45	.23	-.92
ES7-2	.09	-.44	.23	-.92
ES9-1	.1	-.35	.06	-.63
ES9-2	.1	-.35	.06	-.63

The most significant result in these results is the presence of tensile stresses in a loaded pavement. Wambura et al (1999) showed that the pavements in Kenya also develop tensile stresses associated with temperature variations. It can therefore be seen that stresses induced by very heavy wheel loads in combination with the thermally induced stresses can induce stress capable of cracking of a brittle surface of an age hardened bitumen mix.

5.10 Conclusions of test results and analysis

5.10.1 Low volume roads

From the foregoing the performance of low volume flexible pavements is different from that of high volume pavements. Heavy axle loads were shown to have considerable damaging effects on a pavement. This has led to damage of the pavement structure even where high quality materials have been used. However the low volume flexible pavements surveyed during the environmental effect survey showed a tendency of performance inclined to environmental factors and ability of surfacing to remain intact. Against this background a proposed specification for low volume roads is presented. The trial sections for the environmental survey were located in different parts of the country in order to represent different climatic, geographical and subgrade conditions. Similar to normal road construction projects standard equipment was utilised to exploit, haul and place pavement layers. Subgrade soils in the areas covered consisted mainly of flyable clays in the wet areas and clayey sands in the dry areas. The CBR of the subgrade soils was in all cases above seven.

The materials considered for subbase and base layers were laterites, quartzites, coral gravels and weathered rocks of different properties. The aggregates for surface dressings were generally of low quality. Weathered stones and gravels were investigated. The Roads Department (1987) has outlined specification for materials going into construction of low volume roads. Table 5.21 shows the plasticity and strength specifications for these materials.

Table 5.21 Specifications for low volume road construction materials

Pavement Layers	PI	CBR	MAX ACV	MAX LAA
Surfacing aggregate	-	-	26	35
Base	20	50	-	
Subbase	25	25		

Source: Roads Department (1987)

The base and subbase requirements are already relaxed when compared to those of the high volume roads. However these requirements still call for high quality materials which are hardly available in Kenya (Ikindu 1997). Similarly the specifications for the surfacing aggregate, dictate the use of high quality stones, which are also scarce and not uniformly distributed in the country.

The construction details for the test road sections are shown on the Table 3.7. From the table it can be seen that the base and surfacing construction were of lower standard than the specifications. Notwithstanding this several sections performed well as can be seen on Table 3.8.

In general the performance of the sections depended on the ability of the surfacing remaining intact. As would have been expected the sections constructed in quality aggregates provided the best performance. The descending order of performance as measured by the number of potholes and visual assessment of the test sections was quartzites and coral stones, weathered rock and lastly lateritic nodules. The laterite in Kwale was however relatively stronger with LAA of 54 and ACV of 42. This section was found to be in good condition ten years after construction in 1995. The Marich pass Lodwar and Karakol were also inspected in 1997, thirteen years after construction and found to be doing well.

Arising out of the observed performance the specifications for low volume pavements may therefore be relaxed. Table 5.22 is proposed as a guide. The proposed PI/CBR values are slightly above the value for the well performing sections and rounded to five percentage points. Similarly the LAA and ACV values are slightly above the average values for the weak stones of the well performing sections and also rounded to five percentage points. The laterites were found unsuitable for use as surface dressing aggregates.

Table 5.22 Proposal specification for pavement materials for low volume roads

Pavement layers	PI %	CBR %	LAA Max %	ACV Max %
Surfacing	-	-	50	40
Base	20	35	60	40
Subbase	20	40		

5.10.2 High volume flexible pavements

The strength of a pavement and environmental factors are important in the consideration of flexible pavement performance for the high volume roads. Bituminous mixture investigation showed conclusively the weakening of the bituminous base with rising temperatures and increasing bitumen content beyond an optimum value. This implies that the correct design mixture is required to ensure satisfactory pavement performance. A huge drop in strength of unbound bases and subbase upon flooding occurs due to ingress of water. Hence the pavement structures require to be well drained all the time.

Timely intervention of maintenance operations was shown to have a positive effect. Continuos monitoring, evaluation and maintenance of pavements is therefore important for maintaining the high volume flexible pavements in sound conditions.

5.10.3 Case study conclusions

The scatter of the deflection data is related to the instability arising from poor subgrade and the pavement layers. These pavement structures were weakened by ingress of water through the cracked pavement-wearing course. Thus, drilled materials below the pavement layers showed sections with high moisture contents and on several occasions this moisture content was above the optimum moisture content. Notwithstanding this visual examination of the cores and the deflection data analysis suggested that the pavement was structurally sound but needed maintenance intervention to improve the riding quality of the surface and hence remove the dangers that the pilots pointed to. In addition the maintenance intervention would save the pavements from further deterioration.

5.10.4 Case study recommendations

(a) Improvement of drainage

The runway and taxiway drainage system required improvement. The base drain for the runway needed to be completely reconstructed with proper filter material taking account of the grading of the base materials. Additional manholes and inspection chambers were recommended for inclusion. Such manholes would then form added exit points for take off drains, which would drain, into the side drain.

Unlike the runway, the taxiway was not provided with a base drain. Instead the graded crushed stone base layer provided the internal drainage. River gravel then extended into the verges. The net effect observed here was that the permeability of the verges was very low and hence the pavement water did not drain freely. This scenario needed correction by extending the graded crushed stone into the verges. Alternatively a subsurface system including base drains together with manholes and inspection channels to act as take off points needed to be included in the rehabilitation program.

(b) Side drains

The existing side drains were found to have overgrown with weeds and grass. It is recommended that the side drains be concrete lined to ease maintenance and removal of blockages.

(c) Correcting defects

All the visible defects including cracks, crazing, stripping and surface disintegration should be corrected. The correction principally includes removal of weeds and debris from the pavement surface. Sealing of cracks then follows this activity.

(d) Application of levelling layer and seal

Areas with depression or where pavement shows signs of heave, a levelling course of bituminous material should be carefully applied to achieve a smooth surface. Subsequently a seal carpet should be applied to help in improving rideability and to ensure continuos waterproofing of the pavement layers.

(e) Future monitoring

With improved drainage consolidation of the existing layers of the pavement structure is expected. This would improve the bearing capacity of the base and subgrades. Subsequently further deflection measurements after six months intervals would give the state of pavement and its maintenance needs.

(f) Routine maintenance

The maintenance authority of the airport should adopt a more elaborate and strict maintenance schedule to ensure that defects are corrected as quickly as they appear to avoid the pavement from easily breaking down

Chapter Six

Conclusions and Recommendations

6.1 Conclusions

The worst condition of the flexible pavement structure is the soaked condition and is used in the CBR design of the pavement. This condition is an extreme condition, which is reached only in a few times during the life of the pavement. When this condition is reached, the result is fast deterioration of the pavement due to the loss of bearing capacity. Pavement structures should therefore be designed so that this worst condition is not reached during the design life of the structure.

The ability of the surfacing to remain intact is therefore the key element in the design of flexible pavement structures. Extreme rutting, distortion and subsequent pooling should be avoided since these lead the moisture into the pavement structure and its subgrade. The Roads Department (1987) currently relies on the integrity of pavement surfacing to shed away the rainwater. With the high incidence of cracking the pavement layers are therefore subjected to high moisture contents and subsequent lowering of the pavement strength in most cases before the expiry of the design period. The introduction of a drainage layer in all the pavement structures is recommended in order to shed away any moisture which penetrates the pavement structure through the cracks.

Some specific conclusions from the study can be made as follows:

(a) Non-bituminous materials

- The shape of the bulk density curve and that of the dry density are basically the same. The relationship of the bulk density versus added moisture content during testing can be used for the quick determination of maximum dry density and optimum moisture content.

- The strength of lateritic gravel increases with decrease in the fines content. When improved with lime the CBR of a gravel increases with lime content. The increase is in the form of power equation with diminishing effect at higher percentage of lime

(b) Bituminous materials

- Several parameters and relationships, which affect the performance of the bituminous materials, have been identified. The performance of the final mixture is dependent on all the ingredients going into the mixture. These are
    - Type of bitumen binder and content
    - Aggregate types and grading
    - The prevailing temperatures
- The drop in modulus of bituminous surfacing and bitumen bound bases with increasing temperature means that the traffic load is ultimately transferred to the bases and subbases with limited distribution. This leads to the deformation of the asphalt layer. The healing of the deformed structures has been shown. However, since healing does not restore the original characteristics, residual deformation does remain which manifests in rutting.

(c) Environmental factors

- Under the environmental investigation and confirmed in the case airport study relationships of penetration, ductility and softening point with ageing time and air voids has been developed. Age is significant on the surface dressings while air voids which are the conduits for ageing agents water and air are critical in the thicker asphalt concrete.

(d) Flexible pavement surface conditions

- The rutting of the slow lanes is inevitably higher than that of the fast lanes.
- A new concept for assessing the rutting of a pavement – Total rut, has been developed. Total rut criteria is more severe, but it takes into account the deformation of the entire cross section.
- The present serviceability index of 3 and 2.5 used for indicating the stages of resurfacing and major overlay translates into having 80% and 60% of road users accepting the condition of the pavement respectively. The acceptance drops fast as the road deteriorates.
- The roughness criterion for maintenance was found to be relaxed. A new criterion has been proposed.

(e) Pavement deflection

- The deflection of the pavement is more affected by the strength of the material while stress is more thickness dependent. The deflection measurement on low volume pavements is inadequate as a measure of pavement condition. Surface defects evaluation is more appropriate measure of the pavement condition.
- The deflections for the slow lanes are higher than those of the fast lanes. However the deflection of the lanes is well related for all types of pavement structures

(f) Finite element analyses

- The FEM analysis main draw back is in the assessment of the appropriate moduli of the pavement layers. This is because it has been shown that the moduli of pavement materials change with time, environment and loading.

However carried out simultaneously with deflection, the residual moduli and stress condition of the materials can be assessed.

6.2 Recommendations

A big proportion of the deterioration of the pavements can be attributed to administrative and basic organisation of the maintaining authorities. In the case airport study, details of routine maintenance investigations prior to overlay and details of the overlay were scanty. The authorities could not tell when the drains were last maintained. This basic and fundamental maintenance operations including, opening of drains, sealing cracks and potholes e.t.c should be done timely and also recorded to avoid expensive periodic investigations and maintenance.

6.3 Recommendations for further research

Further research is recommended in the areas of pavement materials, pavement conditions and performance, pavement management systems and geographical information systems. The specific topics recommended include: -

(a) The response of the pavement materials to repeated loading to model out traffic loads needs to be carried out for local materials.

(b) The deflection of the pavement inevitably continues all the way to the subbase and base. Tensile and compressive stresses are accordingly induced in lime and cement treated layers. There is therefore need for further research of these materials to understand their behaviour under stress reversal mechanisms.

(c) It has been shown that the low speed lanes, which carry higher axle loads, are prone to faster deterioration. There is need to carry out a comprehensive axle load survey and corresponding grades on a regular basis for the country network to establish the loading in the highways.

(d) In the case airport study cracking was also associated with different refractive properties of painted and unpainted surfaces. Further research is required in this area to minimise its incidence.

(e) Under the environmental study, it was shown that the softer bitumen, namely MB 5000 and 200/500 penetration grade tended to have higher penetration values after four years of service. There is therefore need for constructing more test sections on the heavier volume roads and on asphalt concrete surfaced sections to obtain more definitive behaviour.

(f) A comprehensive recommendation for relaxing standards for low volume roads has been presented and full-scale trial roads can be built.

(g) There is need for further research on bituminous materials with respect to their chemical characteristics under varying environmental, weather and loading conditions. This research should aim at characterising the chemical changes of the various compounds of the bitumen binder.

(h) There is need for application of pavement management systems to improve the management of our road network by showing the deterioration state for the different maintenance options.

(i) There is need to carry out studies on geographical information systems as a computer based tool for managing data on pavement road condition.

(j) There is need to carry out investigations on the locally available pavement materials with a view to establishing their pavement strength characteristics for use in prediction of pavement behaviour

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Appendix A

Data Tables, Test Sites and Analysed Data

Figure A2.1 Mean annual rainfall contours in Kenya (mm)

Source: Survey of Kenya, (1970)

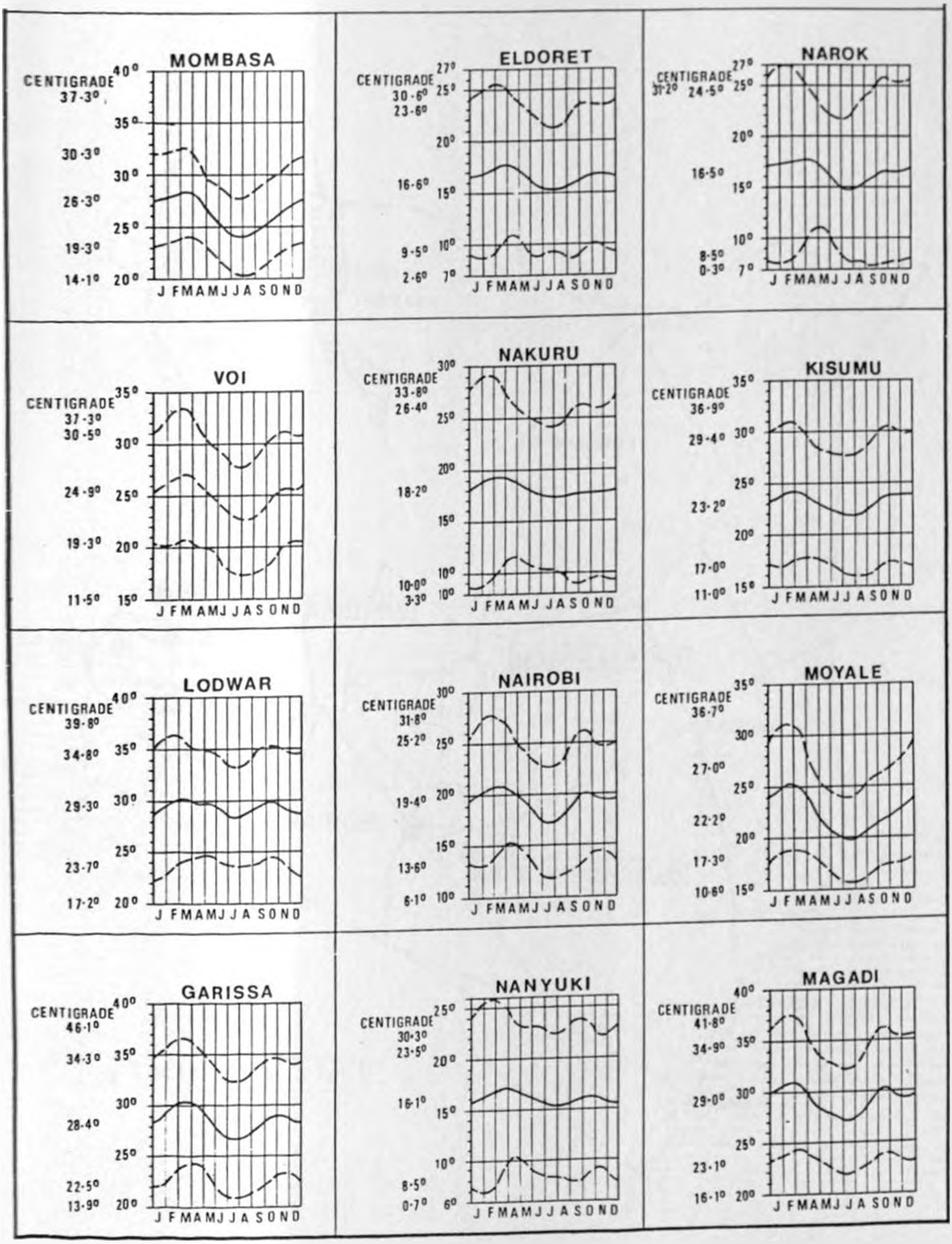


Figure A2.2 Air temperature charts for selected towns in Kenya

Source: Roads Department, (1987)

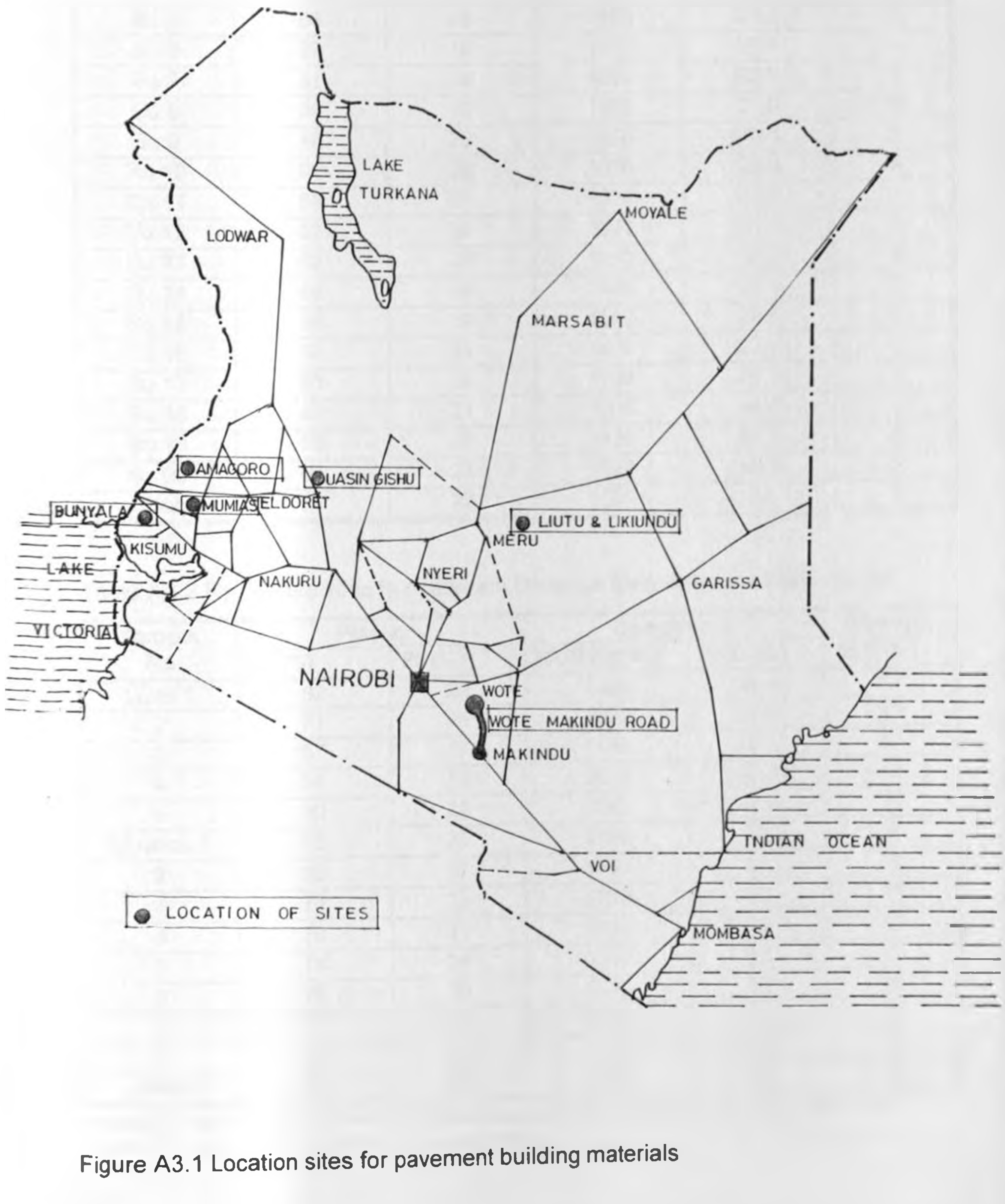


Figure A3.1 Location sites for pavement building materials

Table A3.1 Black cotton soils from Western Province, Bunyala Division - Test results)

Sample No	Plasticity		Compaction		Strength
	LL (%)	PI (%)	MDD (kg/m^3)	OMC (%)	CBR (%)
Bu 1	37	14	1600	21.6	6
Bu 2	60	20	1350	31.8	5
Bu 3	61	21	1380	31.0	5
Bu 4	59	19	1390	20.8	4
Bu 5	56	28	1450	25.1	2
Bu 6	37	15	1600	21.6	9
Bu 7	51	24	1550	22.1	3
Bu 8	39	13	1630	21.0	10
Bu 9	48	22	1620	20.5	3
Bu 10	50	29	1630	20.5	2
Bu 11	50	21	1600	21.5	3
Bu 12	52	32	1540	21.0	3
Bu 13	52	24	1560	22.8	4
Bu 14	58	29	1440	26.0	2
Bu 15	35	14	1580	22.0	5
Bu 16	58	31	1430	27.1	1
Bu 17	50	22	1530	22.5	3
Bu 18	49	21	1560	21.8	3
Bu 19	58	27	1430	27.0	2
Bu 20	58	21	1320	30.6	5
Mean	51	22	1509	24	4

Table A3.2 Red coffee soils from Eastern Province Meru District - Tests results.

Sample No	Plasticity		Compaction		Strength
	LL (%)	PI (%)	MDD (kg/m^3)	OMC (%)	CBR (%)
Liutu 1	62	29	1320	34.5	14
2	71	29	1330	33.0	10
3	57	17	1450	28.5	22
4	54	19	1900	15.3	49
5	60	23	1440	30.2	14
Likiundu 1	73	26	1240	40	9
2	80	37	1380	33	13
3	73	25	1250	40	16
4	70	31	1430	23	15
5	74	34	1390	33	25
6	70	30	1230	40	16
7	73	33	1380	31	28
8	64	25	1320	34	22
Mean	**68**	**28**	**1389**	**32**	**19**

Legend LL liquid limit, PI Plasticity index, MDD maximum dry density, OMC Optimum moisture content, CBR- California Bearing Ratio

Table A.3.3: Wote - Makindu road subgrade soils-test results

Location (km)	Plasticity		Grading			Compaction		Strength
	LL (%)	PI (%)	Gravel (%)	Sand (%)	Fines (%)	MDD (kg/m^3)	OMC (%)	CBR (%)
.5	39	14	1	55	44	1745	16.8	11
1.5	37	11	3	71	26	1705	18.0	11
2.5	39	15	12	42	46	1830	16.0	4
3.5	46	18	2	39	59	1580	21.8	9
4.5	NP	NP	1	78	21	2050	8.2	59
5.5	49	21	2	80	18	1890	9.4	28
6.5	42	20	63	26	11	1975	11.8	28
7.5	53	24	4	68	28	1820	17.2	9
8.5	21	6	53	40	7	2125	8.4	84
9.5	32	11	49	29	22	2090	9.0	41
10.5	25	3	1	80	19	2030	10.8	3
11.5	48	19	3	67	30	1615	20.6	11
12.5	41	16	1	79	20	1675	16.4	10
13.5	29	10	1	70	29	2010	10.6	3
14.5	38	14	33	37	30	1875	11	50
15.5	39	16	15	66	19	1630	21	6
16.5	38	15	25	42	33	2025	12.4	25
17.5	34	15	1	69	30	1925	11.6	8
18.5	26	11	50	37	13	2125	10.7	76
19.5	31	12	29	60	11	1965	12.2	10
20.5	41	15	11	77	12	1896	12	24
21.5	37	14	2	68	30	1765	16.8	28
22.5	36	11	57	33	10	1930	11.5	42
23.5	25	11	2	75	23	1900	11	7
24.5	36	12	7	76	17	1900	11	13
25.5	32	11	57	33	10	2015	12.2	18
26.5	34	10	1	65	34	1640	21.00	9
27.5	35	11	7	59	34	1680	19.00	8
28.5	39	18	2	59	39	1635	20.8	9
29.5	41	18	9	74	17	1590	19.5	10
30.5	37	13	5	52	43	1710	18.0	5
31.5	25	9	69	21	10	2175	7.1	16
32.5	31	13	7	65	28	1925	12.0	20
33.5	37	16	39	52	9	1985	13.4	7
34.5	29	12	12	75	13	2050	9.0	2
35.5	37	15	43	50	7	2185	12.0	8
36.5	30	11	3	78	19	1870	14.0	11
37.5	27	10	1	59	40	1890	13.4	9
38.5	33	11	44	47	9	2040	12.0	6
39.5	28	8	44	34	22	2032	12.0	7

Table A3.3 (continued): Wote - Makindu road subgrade soils-test results

Location (km)	Plasticity		Grading			Compaction		Strength
	LL (%)	PI (%)	Gravel (%)	Sand (%)	Fines (%)	MDD (kg/m^3)	OMC (%)	CBR (%)
40.5	32	14	37	37	26	2025	10.5	10
41.5	35	11	22	44	34	1830	14.0	14
42.5	41	18	11	61	28	1895	13.8	19
43.5	29	11	1	74	25	1920	13.4	12
44.5	29	20	34	49	17	1965	11.4	8
45.5	36	17	3	60	37	1885	15.0	7
46.5	39	16	17	57	26	1890	13.3	3
47.5	33	10	3	77	20	1710	17.4	12
48.5	34	13	28	46	26	1925	15.0	13
49.5	42	17	30	49	21	1935	14.2	14
50.5	50	22	0	21	79	1400	25.0	10
51.5	34	19	60	26	14	1935	12.2	11
52.5	33	10	5	84	11	1720	16.6	12
53.5	35	11	7	76	17	1700	16.8	11
54.5	32	21	50	38	12	2030	9.5	19
55.5	32	11	1	74	25	1795	13.6	11
56.5	31	11	25	55	20	1895	12.2	16
57.5	44	22	41	46	13	1930	12.2	3
58.5	33	10	41	66	30	1700	17.4	5
59.5	39	11	2	63	35	1670	18.80	4
60.5	30	20	1	75	24	1730	16.20	10
61.5	37	14	2	78	21	1680	18.0	4
62.5	36	11	9	73	18	1935	15.2	8
63.5	33	13	1	75	24	1755	16.0	10
64.5	32	13	2	64	34	1780	15.8	6
65.5	26	8	6	65	29	1755	16.4	11
Mean	**35**	**14**	**18**	**58**	**24**	**1861**	**14**	**15**

Table A3.4 Lateritic gravel from Western Province-Test results

Sample No.	PM	PI (%)	Grading			Compaction		Strength
			Gravel (%)	Sand (%)	Fines (%)	MDD (Kg/m3)	OMC (%)	CBR (%)
(a) Mumias area								
1	93	10	75	20	5	1850	18.0	154
2	156	13	80	15	5	1890	16.0	95
3	150	13	80	15	5	1900	16.0	97
4	114	11	80	15	5	1880	16.0	153
5	106	10	80	15	5	1960	15.4	97
6	179	14	80	25	5	1930	16.7	96
7	212	13	65	25	10	1920	12.0	35
8	193	11	65	25	10	1940	12.8	34
10	179	12	65	25	10	1930	16.0	71
11	151		80	15	5	1810	16.8	154
12	143	15	80	15	5	1970	15.8	97
13	260	14	60	35	5	1900	11.7	36
14	152	10	65	25	10	1930	12.6	35
15	189	13	75	15	10	1900	15.8	67
16	179	11	75	15	10	1940	16.4	67
17	231	15	75	15	10	1780	18.4	17
18	186	13	75	15	10	1770	18.0	15
19	588	13	35	30	35	1680	20.6	44
20	572	13	55	10	35	1640	22.0	42
21	510	12	55	10	35	1670	21.4	44
22	178	13	75	20	5	1820	18.0	44
23	235	14	75	15	10	1880	18.0	70
24	154	10	75	15	10	1880	16.5	11
25	204	13	75	15	10	1860	17.9	68
26	479	11	50	15	35	1690	20.4	49
Mean	**249**	**13**	**69**	**18**	**13**	**1839**	**17**	**63**
(b) Amagoro area								
1	224	11	70	18	12	1880	15	80
2	213	11	70	18	12	1900	14.7	73
3	192	10	70	18	12	1860	15.5	77
4	235	13	70	20	10	1980	14.9	76
5	83	7	80	10	10	1990	14.2	94
6	132	9	80	10	10	2000	14.5	93
7	92	11	65	20	15	1970	14.4	94
8	177	10	65	25	10	2000	14.2	95
9	246	10	85	10	5	1900	16.0	54
10	205	9	60	25	15	1950	15.8	54
11	216	10	60	25	15	1930	16.4	54
12	230	9	60	25	15	1950	16.4	54

Table A3.4 Continued: Lateritic grounds from Western Province-Test results

Sample No	PM	PI (%)	Grading			Compaction		Strength
			Gravel (%)	Sand (%)	Fines (%)	MDD (Kg/m3)	OMC (%)	CBR (%)
13	214	8	65	18	17	1920	15.0	94
14	24	10	65	18	17	1910	16.0	95
15	230	9	65	18	17	1930	14.90	93
16	331	12	65	15	20	1930	14.8	96
17	85	13	85	10	5	1850	16.5	91
18	59	11	85	10	5	1870	16.20	90
19	72	10	85	10	5	1840	15.8	91
20	245	12	75	10	15	1940	15.0	62
21	250	13	75	10	15	1960	15.4	59
22	250	12	75	10	15	1910	14.6	62
23	217	11	60	25	15	1940	15.7	59
24	278	12	60	25	15	1960	15.0	59
25	246	11	75	10	15	1970	15.2	59
26	289	13	60	30	10	1980	16.0	58
27	337	14	60	30	10	1950	15.4	58
28	123	11	80	10	10	1900	17.5	52
29	134	13	50	30	20	1910	16.9	51
30	118	10	50	13	7	1890	18.3	58
31	120	13	80	13	7	1910	17.2	57
32	276	11	80	13	7	1920	16.5	52
33	200	13	80	13	7	2020	14.5	113
34	152	9	80	15	5	1950	18.2	102
35	145	11	80	13	7	1980	17.1	121
36	124	12	75	15	10	2000	15.0	112
37	49	10	75	15	10	2000	15.0	86
38	206	9	65	20	15	1730	21.0	54
39	238	11	70	13	17	1750	20.6	64
40	311	13	80	10	5	1780	19.6	60
Mean	**189**	**11**	**72**	**16**	**11**	**1905**	**17**	**78**

Table 3.5 Lateritic gravels from Wote –Makindu road-Test Results

Quarry	PI (%)	PM	Gravel %	Sand %	Fines %	Compaction		Strength
						MDD (kg/m^3)	OMC (%)	CBR (%)
2B	14	534	46	35	19	2121	10	34
7	16	665	38	41	21	2095	10	27
8	14	595	37	42	21	2070	10	35
9	19	804	44	25	31	1944	12	34
10	13	345	54	33	13	2160	7	69
11	12	286	53	36	11	2219	9	47
12	16	418	54	34	12	2124	8	57
13	14	494	42	44	14	1996	10	23
14	16	336	63	23	14	2045	11	62
15	15	694	41	21	38	2105	9	16
16	13	400	55	28	17	2080	9	46
17	15	418	55	34	11	2048	10	45
3	13	328	59	28	13	2083	9	58
3B	19	633	51	30	19	2130	10	36
5	16	467	56	29	15	2074	11	42
6	14	538	45	35	20	2112	10	33
18	14	427	54	27	19	2080	11	47
19	13	358	62	24	14	1995	10	61
20	12	427	44	37	19	2100	9	53
21	15	322	63	26	11	2095	12	40
Mean	**15**	**474**	**50**	**32**	**18**	**2084**	**10**	**43**

Table A3.6 CBR for lime improved lateritic gravel from Wote-Makindu road

	% Lime content			
Quarry No	0	2	4	6
2B	34	129	159	193
3	58	147	158	166
3B	36	130	171	182
5	42	178	193	198
6	33	116	151	165
7	27	202	217	235
8	35	159	220	241
9	34	151	191	196
10	69	93	129	180
11	47	54	100	158
12	57	84	122	147
13	23	123	144	173
14	62	77	145	185
15	16	67	103	165
16	46	136	150	181
17	45	124	155	179
18	47	136	150	181
19	61	69	108	119
20	53	75	112	138
21	40	35	63	120
Mean	43	114	147	175

Legend LL liquid limit, PI Plasticity index, PM – Plasticity modulus, DD maximum dry density, OMC Optimum moisture content, CBR- California Bearing Ratio

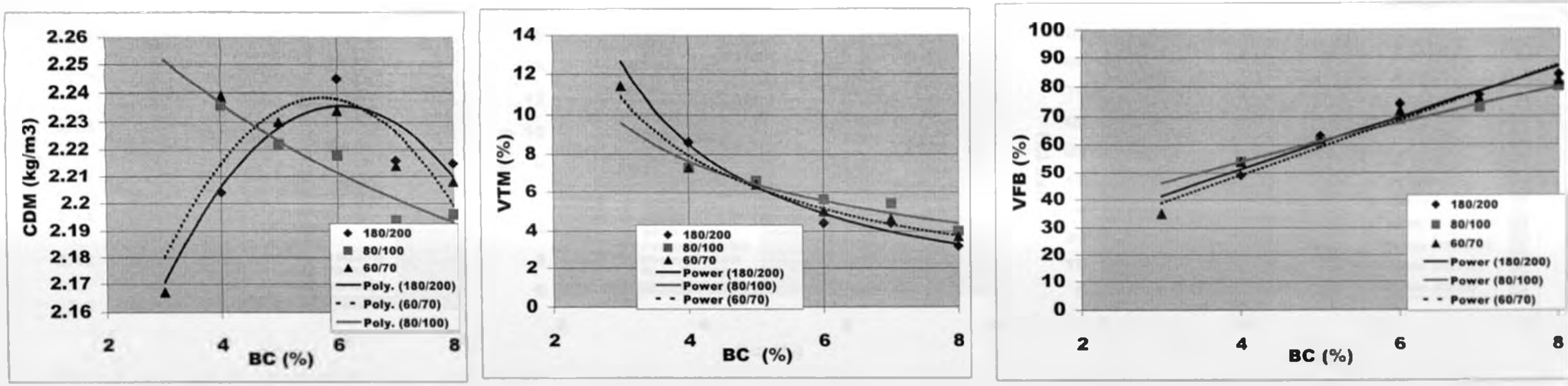
Table A 3.7 Marshal test results:

(a) Mixture properties for Bitumen Grades 180/200. 80/100 and 60/70

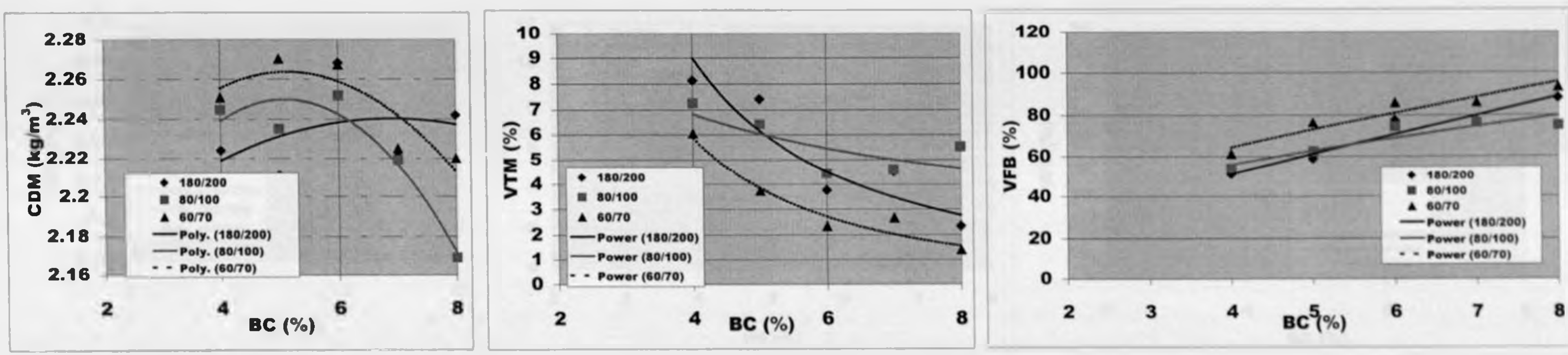
BC (%)	CDM kg/m³			VTM (%)			VFB (%)		
	180/200	80/100	60/70	180/200	80/100	60/70	180/200	80/100	60/70
Aggregate grading type A									
3									
4	2204	2236	2239	8.6	7.3	7.3	49	54	54
5	2229	2222	2230	6.4	6.6	6.4	63	61	62
6	2245	2218	2234	4.4	5.6	5.0	74	69	72
7	2216	2194	2214	4.4	5.4	4.6	77	73	76
8	2215	2196	2208	3.2	4.0	3.7	84	80	82
Aggregate grading type B									
4	2224	2245	2251	8.1	7.3	6.1	51	54	61
5	2211	2235	2270	7.4	6.4	3.7	59	62	76
6	2268	2252	2267	3.8	4.4	2.3	78	74	86
7	2221	2219	2225	4.5	4.6	2.7	77	76	86
8	2242	2169	2220	2.3	5.5	1.4	88	75	93
Aggregate grading type C									
3	2112			13.9			30		
4	2204	2199	2184	9.0	9.2	9.9	48	48	46
5	2198	2204	2206	8.0	7.7	7.7	57	58	57
6	2195	2179	2196	6.9	7.5	7.1	65	62	64
7	2177	2170	2195	6.4	6.7	5.8	69	68	72
8	2164	2153	2176	5.7	6.2	5.4	74	72	75
Aggregate grading type D									
3			2211			10.0			39
3.5	2238			8.2			48		
4	2268	2279	2274	6.3	5.9	6.1	58	60	58
5	2288	2288	2293	4.2	4.2	4.1	72	72	73
6	2245	2248	2262	4.6	4.6	4.1	73	74	76
7	2214	2227	2232	4.8	4.3	4.2	75	78	78
8	2196	2200	2225	4.4	4.2	3.3	79	80	84

Table A3.7 (b) Stability (MST) flow (f) and stiffness (MSE)

BC	180/200			80/100			60/70		
	MST	f	MSE	MST	f	MSE	MST	f	MSE
(%)	kN	mm	kN/mm	kN	mm	kN/mm	kN	mm	kN/mm
Grading A									
3	5						8.42	2.54	3.31
4	5.92	2.79	2.12	8.82	3.05	2.89	8.37	3.56	2.35
5	6.87	3.56	1.93	6.65	3.56	1.87	6.66	3.56	1.87
6	8.34	3.81	2.19	8.51	3.56	2.39	6.17	3.81	1.62
7	6.73	4.32	1.56	5.54	5.84	.95	5.93	5.08	1.17
8	5.52	4.83	1.14	6.20	6.35	.98	5.97	7.37	.81
Grading B									
4	9.70	3.81	2.55	11.69	3.56	3.28	13.62	3.56	3.83
5	8.41	3.30	2.55	9.91	3.81	2.57	10.81	3.81	2.78
6	8.34	4.83	1.73	9.69	4.32	2.24	9.26	4.32	2.14
7	5.18	6.35	.82	5.90	6.35	.93	5.24	7.37	.71
8	5.49	5.59	.98	4.33	6.60	.66	5.04	7.11	.71
Grading C									
3	5.01	2.79	1.80						
4	6.8	3.30	2.06	8.56	3.30	2.59	7.95	3.3	2.41
5	5.88	3.05	1.93	9.70	4.06	2.39	7.29	2.79	2.61
6	6.05	3.30	1.83	5.36	4.32	1.24	7.26	3.30	2.20
7	4.85	4.06	1.19	5.01	4.83	1.04	6.27	4.57	1.37
8	4.11	5.33	.77	3.96	5.08	.78	5.17	5.84	.89
Grading D									
3							13.05	3.81	3.43
3.5	9.48	3.30	2.87						
4	10.88	3.05	3.57	11.34	3.30	3.44	10.51	3.05	3.45
5	8.84	4.06	2.18	10.43	4.06	2.57	8.38	4.83	1.73
6	4.87	5.33	.91	6.62	5.84	1.13	5.55	5.08	1.09
7	4.20	10.16	.41	5.04	8.89	.57	4.80	9.65	.50
8	3.51	10.41	.34	4.44	9.91	.45	4.19	8.89	.47

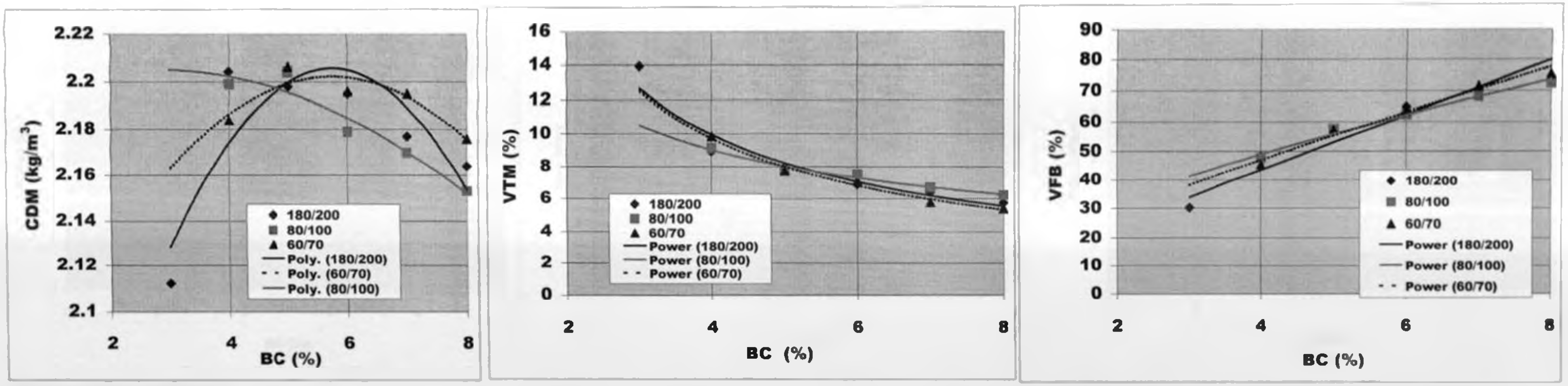


(a) Aggregate grading type A

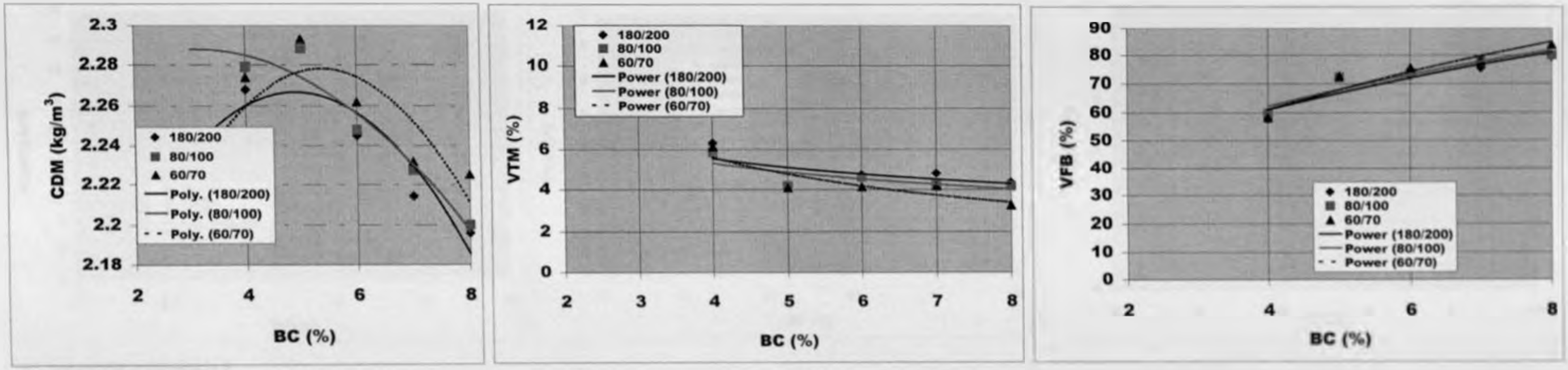


(b) Aggregate grading type B

Figure A3.2 Marshal test data: Effect of variables

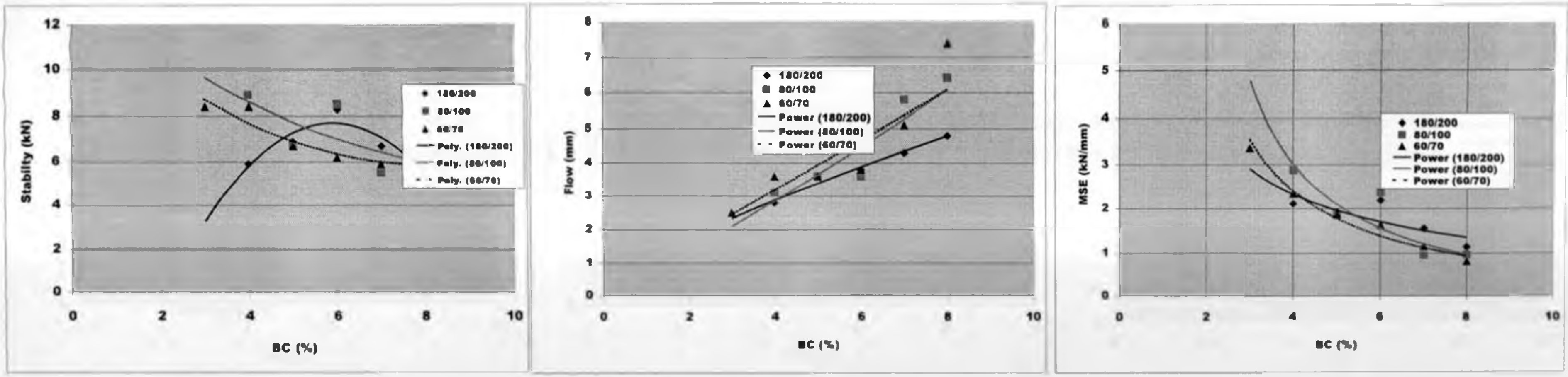


(c) Aggregate grading type C

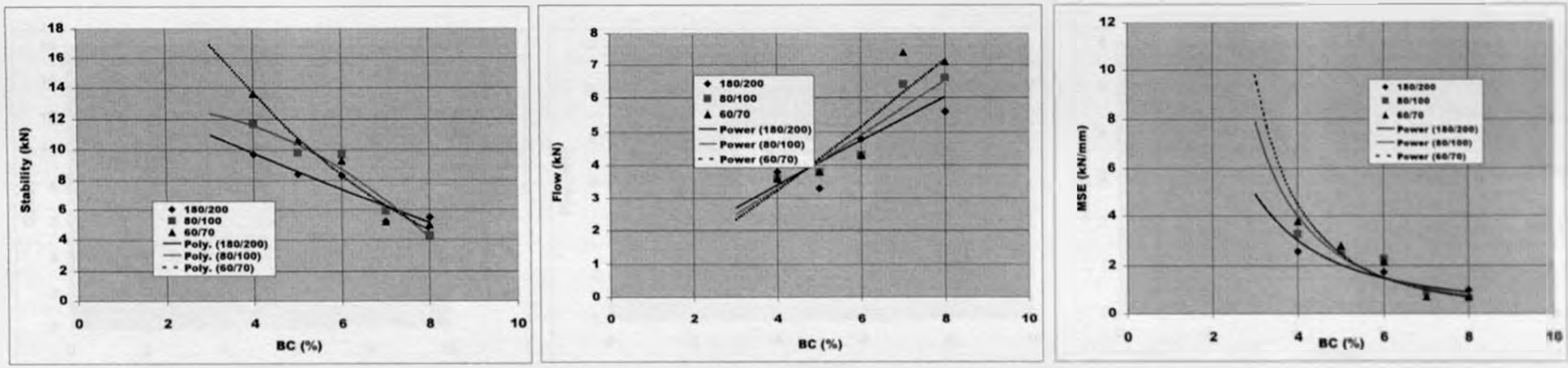


(d) Aggregate grading type D

Figure A3.2 (contd.) Marshal test data: Effect of variables

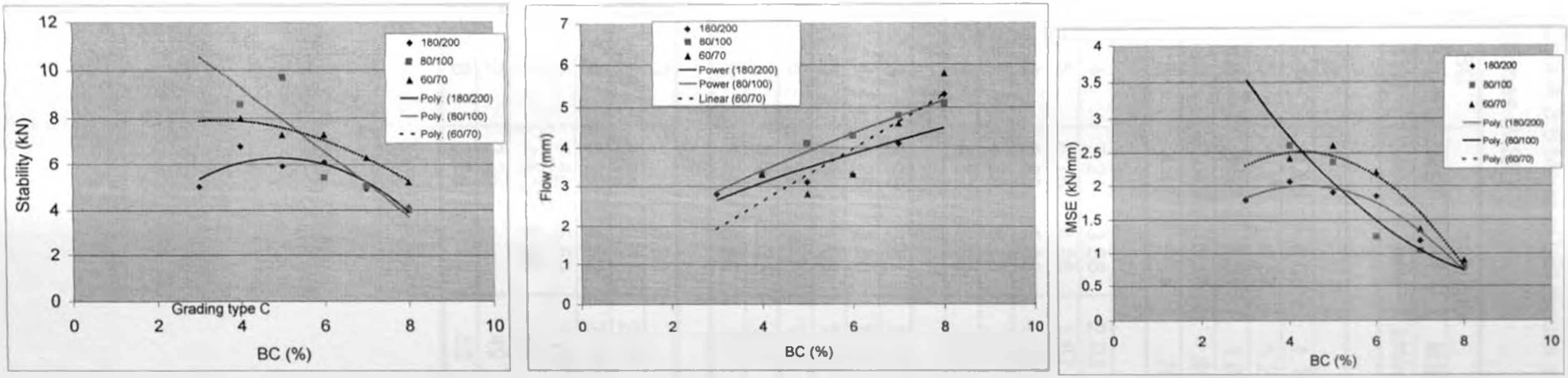


(a) Aggergate grading A

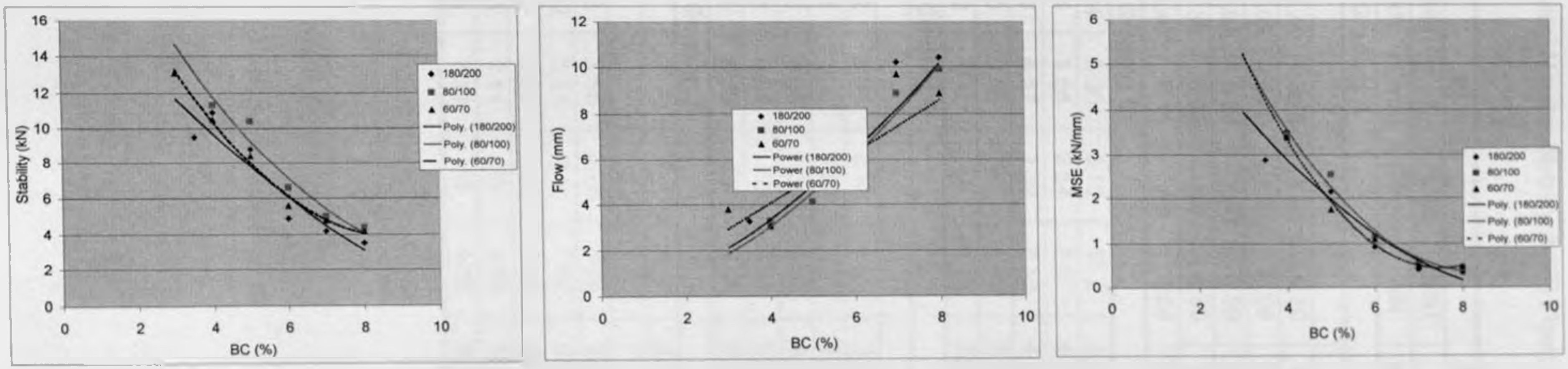


(b) Aggergate grading B

Figure A3.3 Marshal test data: Effect of binder content



(c) Aggregate grading C



(d) Aggregate grading D

Figure A3.3 (contd.) Marshal test data: Effect of binder content

Table A 3.8 Marshal test results after healing: Marshal Stability (MST) Flow (f) and Marshal Stiffness (MSE)

BC	180/200			80/100			60/70		
	MST	f	MSE	MST	f	MSE	MST	f	MSE
(%)	kN	mm	kN/mm	kN	mm	kN/mm	kN	mm	kN/mm
Grading A									
3									
4	4.85	4.06	1.19	6.24	3.56	1.75	6.30	4.83	1.30
5	5.71	4.57	1.25	6.03	4.83	1.25	6.05	4.32	1.40
6	6.97	3.81	1.83	6.94	5.33	1.30	6.09	4.83	1.26
7	5.41	6.10	.89	4.38	6.35	.69	4.99	6.86	.73
8	4.43	5.84	.76	4.45	7.11	.63	4.54	7.62	.60
Grading B									
4	7.21	3.56	2.03	10.24	3.30	3.10	10.2	3.50	2.87
5	6.78	4.06	1.67	9.04	3.56	2.54	8.57	4.32	1.98
6	6.89	5.08	1.36	9.23	4.57	2.02	8.10	4.06	2.00
7	4.29	8.13	.53	5.96	6.86	.87	4.83	8.38	.58
8	4.37	6.89	.63	4.13	6.6	.63	4.48	7.87	.57
Grading C									
3	3.95	3.81	1.04						
4	5.75	3.56	1.62	8.22	3.81	2.16	5.63	3.81	1.48
5	5.08	4.06	1.25	8.83	3.3	2.68	5.61	4.32	1.35
6	4.95	3.3	1.50	6.23	4.59	1.36	5.83	4.32	1.35
7	4.21	5.33	.79	4.55	6.35	.72	5.28	5.08	1.04
8	3.49	6.86	.51	3.56	6.35	.56	4.04	6.60	.61
Grading D									
3		3.30					10.60	4.57	2.32
3.5	8.66	3.81	2.62						
4	8.97	4.32	2.35	9.83	3.56	2.76	9.16	3.81	2.40
5	7.60	5.59	1.76	11.42	3.81	3.0	7.62	4.83	1.58
6	4.53	6.86	.81	6.72	5.59	1.20	4.96	5.59	.89
7	3.35		.49	6.03	7.11	.85	3.86	8.13	.47
8	3.58	7.11	.50	4.18	7.11	.59	4.80	8.38	.57

Table A 3.9 Variation of Marshal test results with temperature and grading for 180/200, 80/100 and 60/70 bitumen binders.

Temp °C	Stability (MST) kN			Flow (f) mm			Stiffness (MSE) kN/mm			Loading time (LT) Sec		
	180/ 200	80/ 100	60/7 0	180/ 200	80/ 100	60/ 70	180/ 200	80/ 100	60/ 70	180/ 200	80/ 100	60/ 70
Grading A												
20	23.5	24.3	41.3	3.30	3.81	5.08	7.12	6.38	8.13	17	17	26
30	15.2	14.9	24.2	6.35	4.32	5.08	2.39	3.45	4.76	19	15	18
40	10.4	13.6	13.4	4.06	4.57	4.06	2.56	2.98	3.30	10	15	14
50	7.6	8.3	9.6	5.59	5.08	4.57	1.36	1.63	2.10	14	15	13
60	7.5	8.9	8.2	3.30	3.56	4.32	2.27	2.50	1.90	10	12	14
Grading B												
20	27.3	28.2	45.1	3.05	4.57	2.79	8.95	6.17	16.1	17	17	21
30	15.7	21.5	23.3	3.81	4.32	3.30	4.12	4.98	7.06	14	15	20
40	12.0	17.4	15.3	3.05	4.57	3.81	3.93	3.81	4.02	12	14	15
50	9.9	11.1	10.3	4.06	4.32	3.81	2.44	2.57	2.70	12	13	13
60	6.8	9.9	10.0	3.56	3.56	4.06	1.91	2.78	2.46	9	13	12
Grading C												
20	26.3	19.6	41.8	4.06	3.56	4.06	6.48	5.51	10.3	14	14	19
30	15.1	14.6	20.8	4.57	3.56	4.06	3.30	4.10	5.12	14	11	16
40	10.7	10.1	11.7	4.06	2.79	3.30	2.64	3.62	3.55	13	9	11
50	7.4	7.3	10.0	4.06	3.30	3.30	1.82	2.21	3.03	11	9	11
60	6.4	7.6	7.3	3.81	3.30	2.79	1.68	2.30	2.62	10	10	10
Grading D												
20	32.3	36.6	48.9	4.83	4.57	2.03	6.69	8.01	24.0	20	18	20
30	19.8	28.6	38.6	3.30	4.83	6.10	6.00	5.92	6.33	15	18	24
40	13.6	20.5	26.3	4.83	4.06	5.33	2.82	5.05	4.93	17	16	20
50	10.4	15.5	15.7	3.56	4.06	4.83	2.92	3.82	3.25	12	15	19
60	10.0	12.9	14.2	4.32	5.08	4.57	2.31	2.54	3.11	14	14	14

Figure A3.4 Tests locations with respect to environmental effects on pavements

Plate A3.1 Kwale test site

Plate A3.2 Narok test site

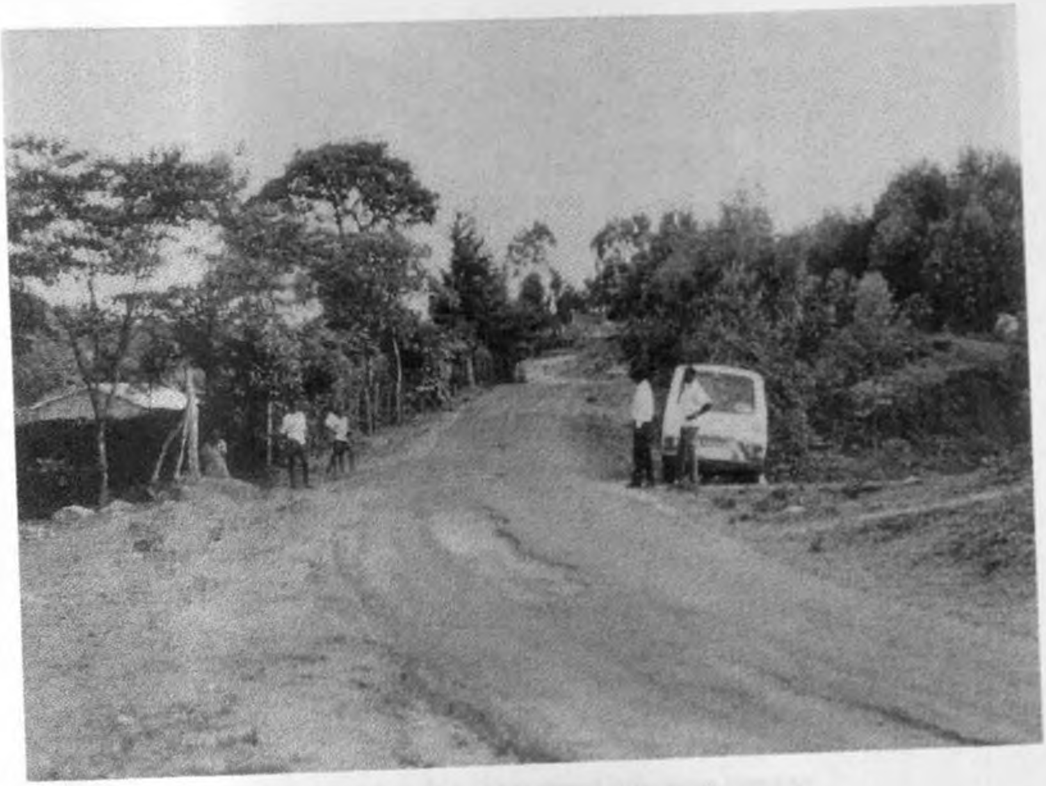


Plate A3.3 Kisii test site

Plate A3.4 Bitumen extraction from bituminous mixtures

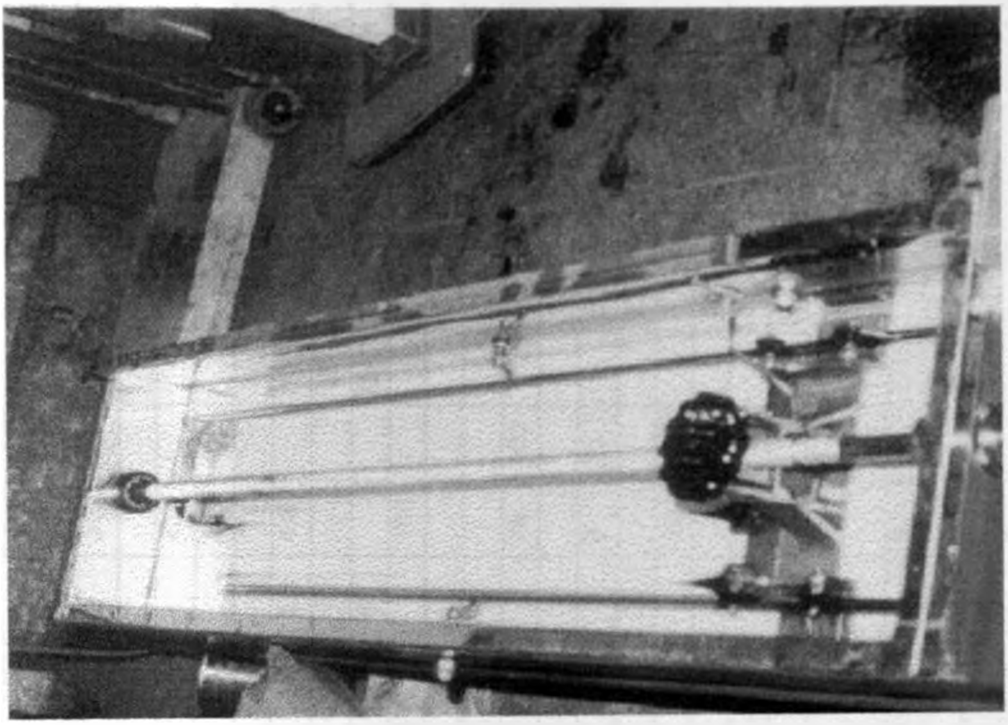


Plate A3.5 Ductility test apparatus for recovered bitumen binder

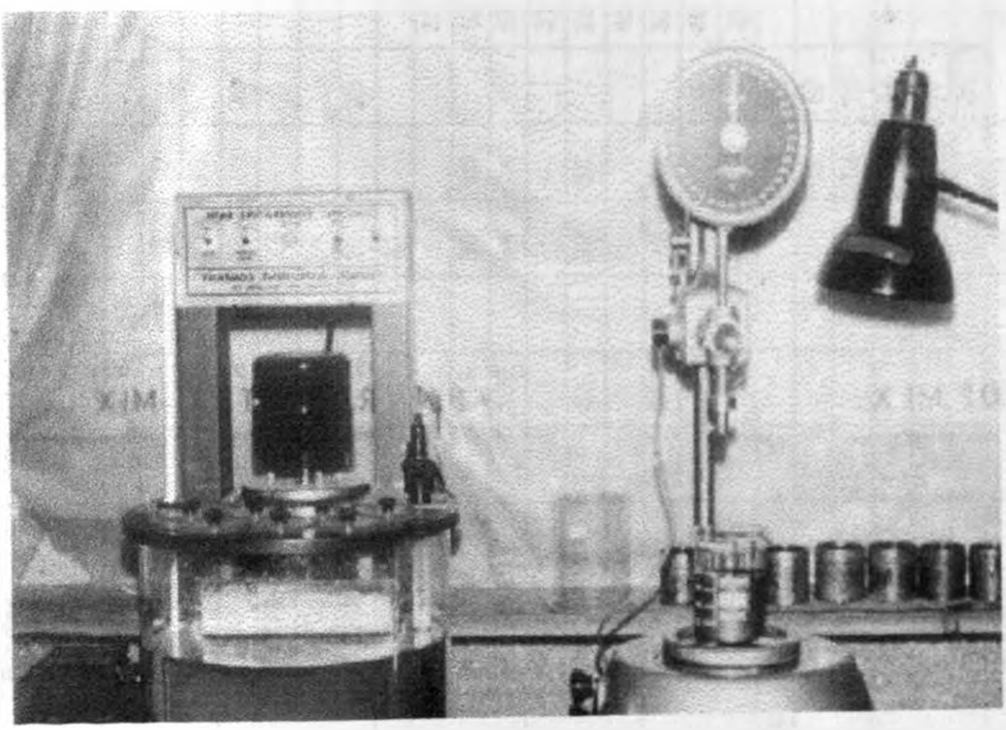


Plate A3.6 Viscosity and penetration testing equipments for recovered bitumen binder

Table A3.10 Properties of recovered binder (a) Low volume roads

Section	Binder	Penetration at 25°C									Ductility at 25 °C									Aggregate
		Years of service									Years of service									
		0	**1**	**2**	**3**	**4**	**6**	**7**	**9**	**10**	**0**	**1**	**2**	**3**	**4**	**6**	**7**	**9**	**10**	
Marich	MC 3000							13	11	11							5		7	Quartz
	MC 3000		53		19	20						89		4	4					Quartz
Pass	MB 5000		117		77	68						102		100	100					Quartz
Narok	K1-60			62	38		29									8				Trach
	80/100	82	74	27	23								5.2	4.6		15				Trach
	K1-70	46	19	23	24											18				Trach
	MC 3000	119		36	26		23						9			6				Trach
Lodwar	200/500	240		61	51						190			100						Quartz
Majengo	MC 3000							35									73			Quartz
	MC 3000		119		30	26		12				110					4			Laterite
	MC 3000		100		64	74		57				114		76	100		85			Phonolite
Oyugis	MC 3000							31									13			W-rock
	MC 3000		62		32	30		23				64		6	20		5			Laterite
	MC 3000							45									27			Basalt
Kisii	MC 3000		98		43	48		32				124		30	35		7			Basalt
	MC 3000							40									61			W-rock
	MC 3000							21									4			Laterite
Kisian Bondo	K1-60					18	24				9				4	6			4	
	80/100					19	12				17				4	4			4	
	200/300					18	15				15				3	4			4	
Kwale	MC 3000	114		30	26		21				110			6		6				Coral
	MC 3000						19									5				Laterite
	MC 3000						21									5				s-Stone
Ndumberi	MC 3000		40		19	21		12				25	5	5			4			Phonolite
	80/100							11									4			Phonolite

Table 3.10 (continued) Properties of recovered binders (a) Low volume roads

Section	Binder	Softening Point °C							Kinematic viscosity $mm^2/sec \times 10^3$							Aggregate
		Years of Service							Years of service							
		2	**3**	**4**	**6**	**7**	**9**	**10**	**2**	**3**	**4**	**6**	**7**	**9**	**10**	
Marich Pass	MC 3000						97	95						13.4	13.7	Quartz
	MC 3000		69	66						1.67	1.33					Quartz
	MB 5000		47	46						.237	.250					Quartz
Narok	K1-60		57		62											Trach
	80/100				61											Trach
	K1-70	7	69		69				2.89							Trach
	MC 3000	60	66	65					.455							Trach
Lodwar	200/500	49	49						.318	.251						Quartz
Majengo	MC 3000					58										Quartz
	MC 3000					71										Laterite
	MC 3000	52	45			56			.403							Phonolite
Oyugis	MC 3000					60										W-rock
	MC 3000	65	59			69				1.367						Laterite
	MC 3000					55										Basalt
Kisii	MC 3000	56	56			64				.784						Basalt
	MC 3000															W-rock
	MC 3000															Laterite
Kisian Bondo	K1-60		70			72		72								
	80/100		80			69		69			1.42					
	200/300		72			70		70			2.40					
Kwale	MC 3000	60	62	66					.674							Coral
	MC 3000			72												Laterite
	MC 3000			70												S-Stone
Ndumberi	MC 3000		70		61					3.77	3.73					Phonolite

Table A3.10 Properties of recovered bitumen

(b) Samples of asphalt concrete from high volume roads

Description	Ulu -Sultan Hamud Rd		Stadium-Airport Rd		Uplands-Longonot Rd		Nakuru Highway	
Age (yrs)	15		7		8		7	
Direction	1*	2**	1*	2**	1*	2**	1*	2**
Traffic CESAx10^6	36.3	13.2	18.6	11.8	8.8	6.0	10.1	15.2
BC (%)	4.9	4.9	6.0	5.7	5.6	5.7	6.6	6.6
Pen. (x0.1mm)	89	67	15	18	23	20	42	40
Aiv (%)	1.8	4.0	11.0	9.0	11.5	10.2	2.4	2.4
Duct. (cm)	100	55.5	3.8	3.8	11.4	7.1	21.1	18.1
Sp (^{o}C)	47.5	50.5	70.0	70.0	62.5	67.5	58	59.5

Legend: CESA – Cumulative equivalent standard axles, BC – Bitumen content, Pen - Penetration, Aiv – Air voids content, Duct – Ductility, Sp – Softening point

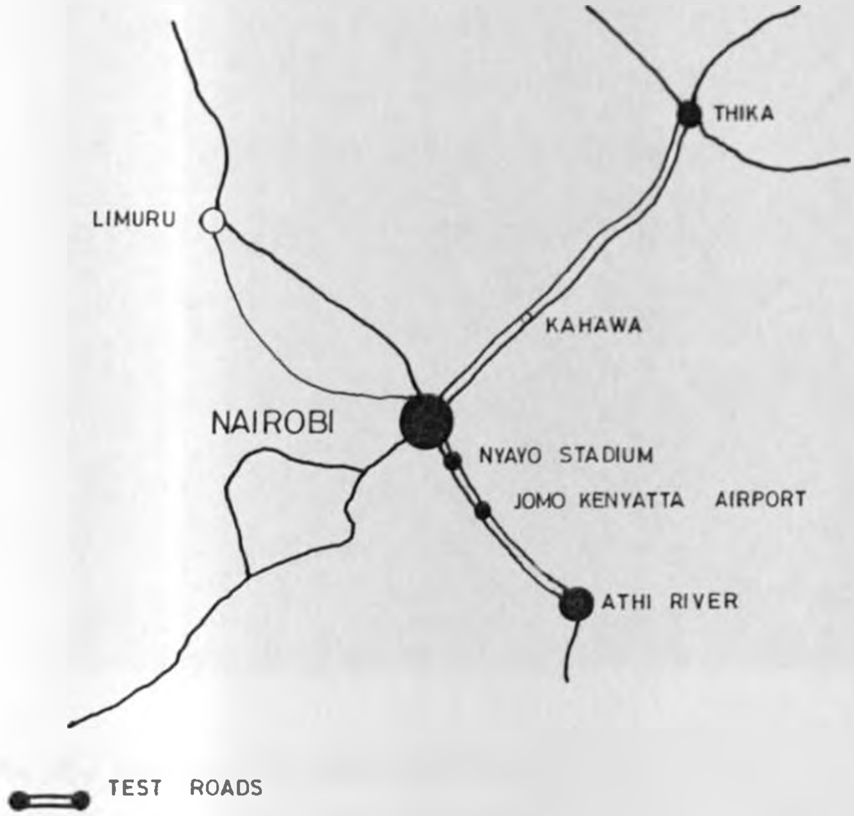


Figure A3.5 Location of roads and sites for non-destructive tests

Plate A3.7 Test site ES1 after an overlay

Plate A3.8 Test site ES5 with heavy axle loads, cracks and ruts

Plate A3.9 Test site ES6 showing ruts and cracks

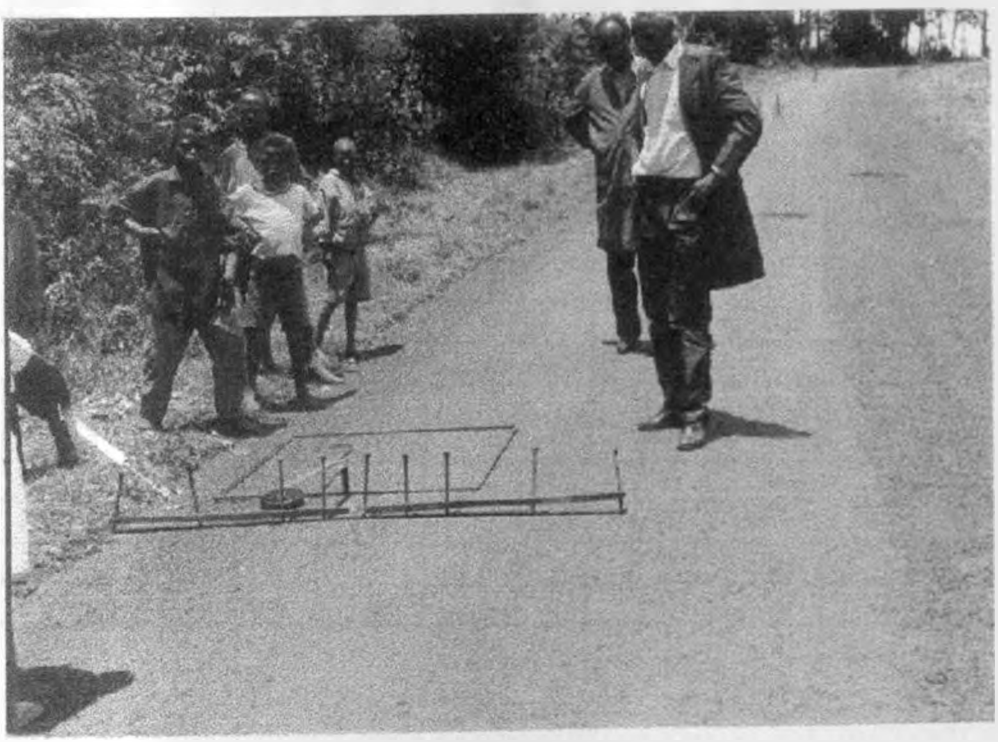


Plate A3.10 Test site ES9 showing signs of failure:
Potholes, cracks and ruts after twenty six years of service

Table A 3.11 Surface condition evaluation results:

Section	PSI	Rutting (mm)			Cracking m/m^2	Roughness (mm/km)		Probability. of Acceptance
		left lane	right lane	total rut		left lane	right lane	
Nairobi -Thika road								
1	3.78	8.5	6.5	72.5	0	1582	1430	1
2	3.56	10.3	2.6	106.3	0	1354	1417	1
3	3.98	7.7	2.7	66.4	1.1	1430	1468	1
4	3.24	6.6	6.3	101.3	5.1	1684	1506	1
5	3.42	27.4	7.9	264.8	8.7	1810	1595	1
6	3.12	8.3	5.2	113.6	0	1798	1531	1
7	2.82	17.8	6.1	194	0	1645	1582	1
8	3.10	7.7	7.9	93	3.9	1595	1620	0.8
9	3.06	11.4	12.9	125	5.2	1480	1417	0.8
10	2.94	10.3	4.9	75	2.1	1646	1366	1
11	3.26	6.8	7.6	120	5.8	1887	1912	1
12	2.30	14.1	4.8	130	2.8	2217	1620	0.4
13	3.00	18.1	6.7	226	3.8	1785	1468	0.6
14	2.64	13.2	7.4	159	3.9	2052	1696	0.8
15	2.74	5.1	7.4	81	2.5	1861	1709	0.8
16	2.94	9.9	6.5	146	3.0	1734	1506	0.6
17	2.64	11.4	5.5	98	0	1468	1366	1
18	2.72	7.6	6.9	143	3.4	1404	1480	0.6
19	2.86	38.1	4.9	355	3.6	2065	1684	0.6
20	2.90	8.1	16.4	164	5.6	2611	1849	1
21	0.62	COMPLETELY DAMAGED				6217	3982	0
Nyayo Stadium –Airport								
1	3.80	3.2	14.9	97	1.4	1722	1709	1
2	3.54	10.3	6.1	95	1.3	1900	1900	1
3	3.56	6.5	6.1	96	5.1	1709	1759	1
4	3.34	7.7	7.7	96	1.4	1811	1632	1
5	3.06	12.5	5.6	121	2.5	1950	1734	.8
6	3.32	6.5	11.4	106	4.2	1679	1696	1
Airport- Nyayo stadium								
1	2.80	12.7	21.1	165	7.2	2408	1468	.6
2	3.06	5.4	3.9	92	4.3	2941	1582	.8
3	2.86	17.3	10.3	162	2.5	2573	1569	.6
4	2.36	9.7	9.4	139	7.9	1836	1417	.4
5	2.18	8.7	4.9	138	7.8	1899	1595	.6
6	2.30	12.5	10.4	126	7.7	1916	1798	.6

Table A3.11 continued. Surface condition evaluation results

Section	PSI	Rutting (mm)			Cracking (m/m^2)	Roughness (mm/km)		Probability of acceptance
		Left Lane	Right Lane	Total Rut		Left	Right	
Airport – Athi river (two way)								
1	3.32	5.3	25.2	178	8.0	1646	1810	1
2	3.44	4.5	23.7	202	8.8	1455	2039	1
3	3.68	5.7	7.3	114	0	1544	1912	1
4	3.52	11.8	30.6	298	5.6	1582	2230	1
5	3.30	5.5	8.6	121	0	1519	1722	1
6	3.34	8.5	7.1	102	4.8	1607	1722	.8
7	2.38	7.4	9.7	124	7.1	1722	1836	.6
8	1.48	13.1	9.1	228	0	1747	1849	.2

Plate A3.11 Benkleman beam and wheel arrangement during deflection measurements

Table A 3.12 Rebound deflection data.

Test Site	Lane	Deflection (0.01 mm)					
		o m	0.1 m	0.2	0.3	0.4	0.5
ES1							
CS1	L	19	19	18	16.5	15	14.5
	R	6	5	4.5	4.5	4	3.5
CS2	L	29	29	27.5	26.5	23.5	16.5
	R	16	16	15	14	11.5	9.5
CS3	L	22	22	21	18.5	17	16.5
	R	12	12	11.5	11	13	12.0
CS4	L	15	14.5	13.5	11.5	11.0	7.50
	R	9.5	8.5	8.5	8.0	7.5	5.5
CS5	L	20	18.5	16.0	14.5	13.0	11.0
	R	7	7	5.5	3.5	3.0	4.0
ES2							
CS1	L	32	29	14.5	25	25.5	19.5
	R	60	60	55	37	29	37
CS2	L	4	11	-5	1	12	9.0
	R	30	23	20	17	17	16
CS3	L	19	18	31	32.0	30.5	25.5
	R	7	7	7	11.5	7	6
CS4	L	52	50	39	34.5	32.5	29.0
	R	16	16	15	14.5	14.0	13.0
CS5	L	18	17	16	13	10.5	9
	R	27	26	25.5	21.0	19.0	13.5
ES3							
CSI	L	34	31.5	30	3	18	0.5
	R	26	24	19	15.5	10	7
CS2	L	69	66	49	22	20.5	17
	R	47	47	40	29	22.5	15
CS3	L	36	32.5	30	19	8	16
	R	37	32.5	26	21.5	16.5	14.5
CS4	L	41	41	38	30	23	16
	R	41	40	36	26.5	22.5	16
CS5	L	50	52	42	41	34.5	38
	R	20	20	18	15	10	8.5

Table A3.12 (continued) Rebound deflection data

Test Site	Lane	Deflection (.01mm)					
		0m	.1m	.2m	.3m	.4m	.5m
ES4							
CSI	L	42	38.5	36	30	25.5	17.5
	R	21.5	21.5	19	17.5	15.5	13.5
CS2	L	39	39	34.5	29	25.5	23
	R	24.5	23.5	20.5	18.5	15.5	12.0
CS3	L	40	39.5	37	30	23.5	22
	R	24	22	19	17	14	11
CS4	L	24	22	23	17	14	11
	R	21	20	18	16	13	12
CS5	L	37	35.5	31	25.5	22	15.5
	R	20	18.5	17.5	14	13	10

ES5							
CSI	L	15.5	15.5	13.5	12.0	11.0	9.5
	R	-6	-2	-1	-3	-4	0
CS2	L	10	11	9.5	13	12	10
	R	35.5	37.5	34	28.5	24.0	17.5
CS3	L	38.5	39.5	38.5	31.5	21.5	19.5
	R	36.5	28.5	21.5	17.5	12.5	6.5
CS4	L	28	25	21	13	9	6.5
	R	51	52	38	25	14	10
CS5	L	40	40	35	29	23.5	18.5
	R	38	36	32.5	22.5	15.0	10.0

ES6							
CSI	L	9	9	11	8	7.5	6.5
	R	2	4	6	6	4	2
CS2	L	9	7.5	6	4.5	4	2.5
	R	5	6	5	3	4	3
CS3	L	2	0.5	0	-1.0	-1.0	-0.5
	R	23	20	18	16.5	14	12
CS4	L	5	4.5	3.5	2.5	1	1.5
	R	3	0.5	1	0.5	-1	0
CS5	L	11	10.5	9	7	6.5	3.5
	R	7.5	8.0	7.5	6.5	6.5	5.5

Table A3.12 continued: Rebound deflection data

Test Site	Lane	Deflection (.01mm)					
		0m	.1m	.2m	.3m	.4m	.5m
ES7							
CSI	L	30.5	31.5	30.0	27.5	25.5	23.0
	R	78	73	66	55	48.5	40.5
CS2	L	46	44.5	41	39	34.5	26.5
	R	58	58	51.5	43.5	42.5	42.5
CS3	L	20	19	18	14	13	11
	R	64.5	61.5	51.5	43.5	38.5	36.5
CS4	L	34	33	31.5	29	26	23.5
	R	57	57.5	51	45	34.5	26.0
CS5	L	37.5	35.5	32.5	26.0	17.0	6.5
	R	52.5	52.0	51	49.5	40.0	34.5

ES9							
CSI	L	16	18	14	8	0	0
	R	42	36	32	12	4	4
CS2	L	50	46	39	34	26	18
	R	64	52	40	34	24	16
CS3	L	36	34	31	14	8	6
	R	64	63	54	23	12	6
CS4	L	20	20	9	4	3	2
	R	56	50	32	16	15	10
CS5	L	48	46	42	24	16	4
	R	38	34	34	22	16	15

ES10							
CSI	L	90	80	78	60	40	28
	R	48	44	36	23	16	14
CS2	L	168	130	104	72	46	28
	R	66	58	44	32	24	10
CS3	L	90	88	76	52	32	14
	R	57	59	53	40	27	16
CS4	L	108	96	60	16	0	6
	R	44	44	40	28	14	12
CS5	L	102	102	88	60	42	28
	R	28	27	23	16	12	8

Plate A4.1 Pavement cracking on the runway

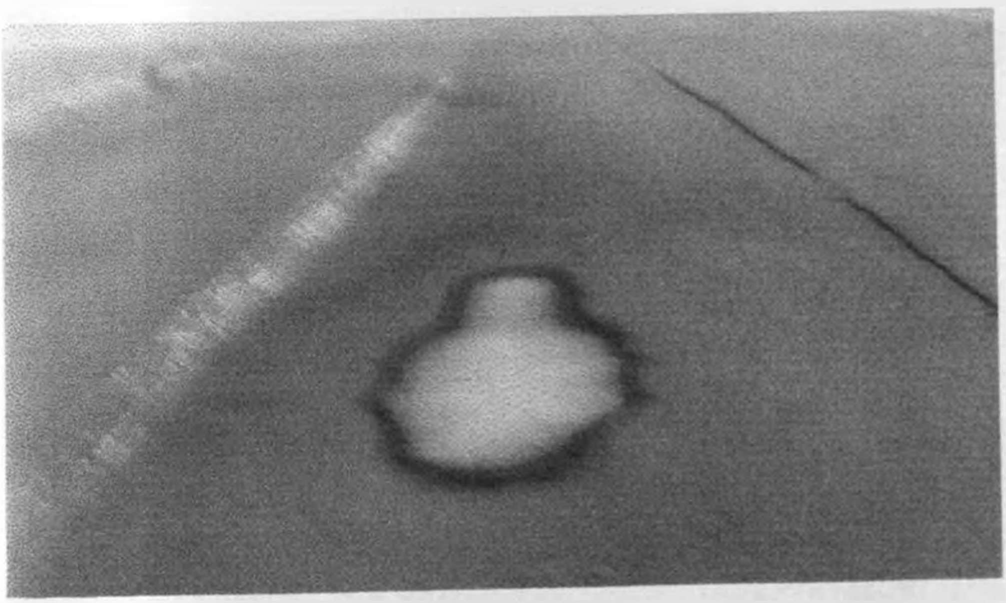


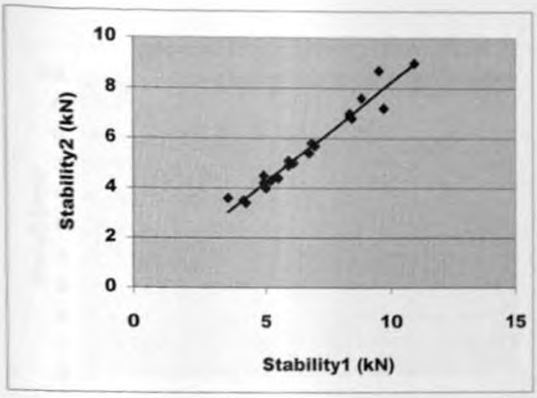
Plate A4.2 Ponding on the pavement

Plate A4.3 Low points of subsoil drain

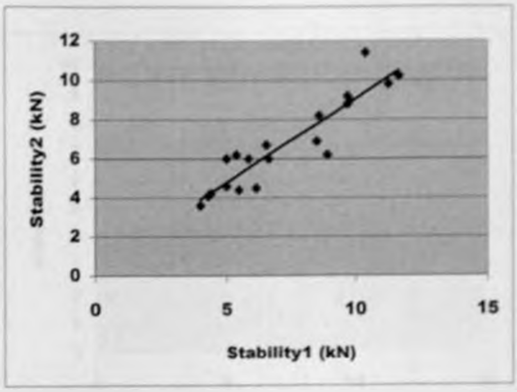
Plate A4.4 Low points for the open drains

Plate A4.5 Weed growth on the pavements

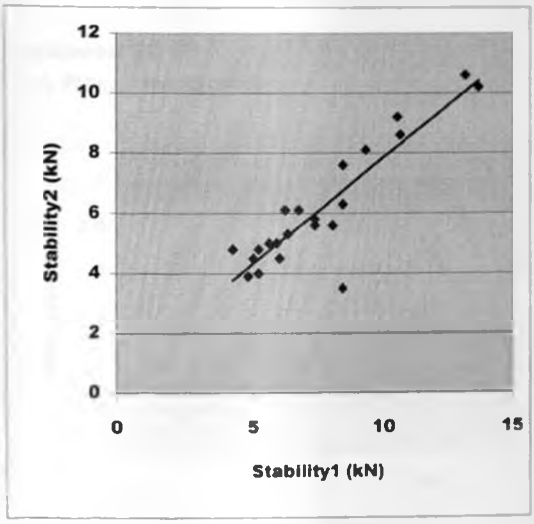
Plate A4.6 Bituminous material cores



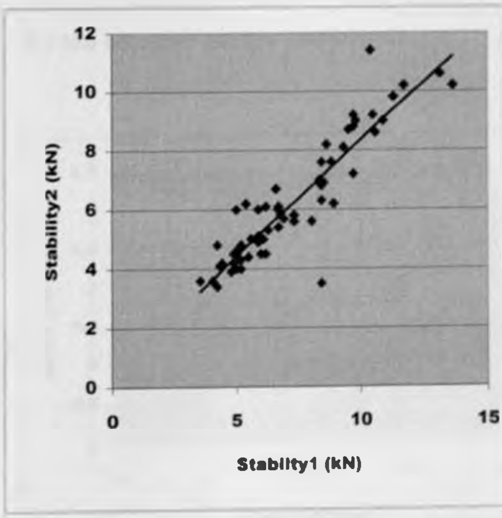
Bitumen 180/200



Bitumen 80/100

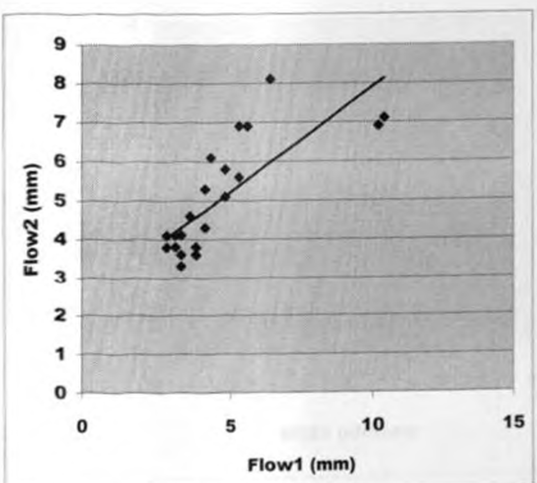


Bitumen 60/70

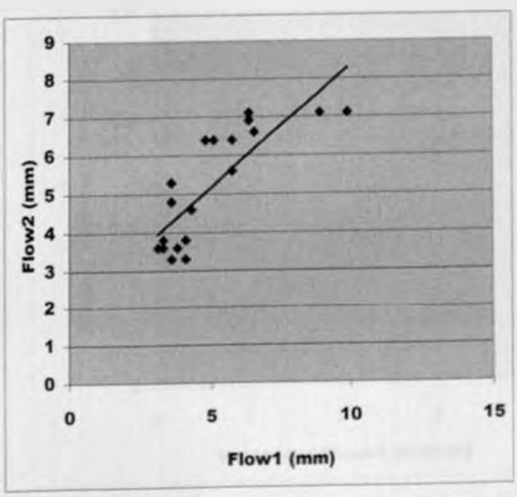


All the bitumen grades combined

a) Stability



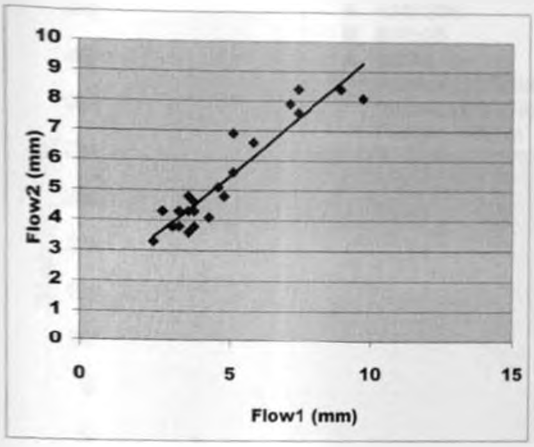
Bitumen 180/200



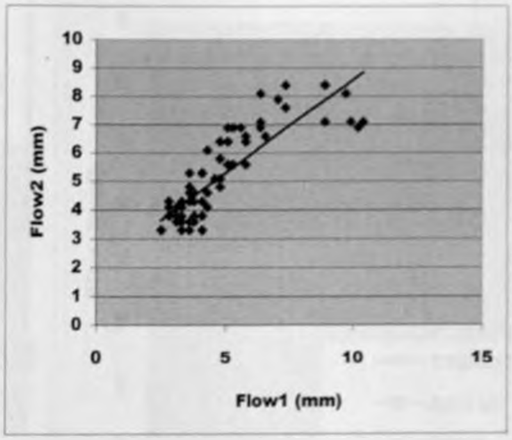
Bitumen 80/100

(b) Flow

Figure A 5.1 Healing behaviour for bituminous mixtures

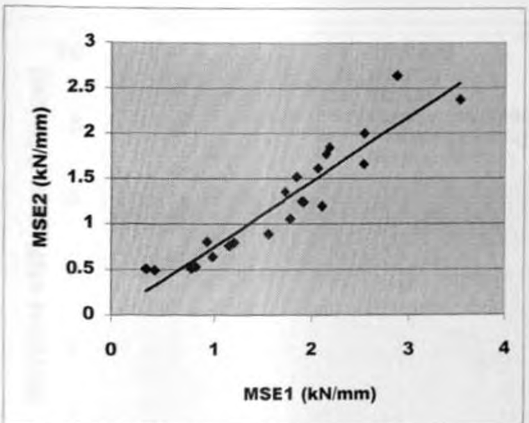


Bitumen 60/70

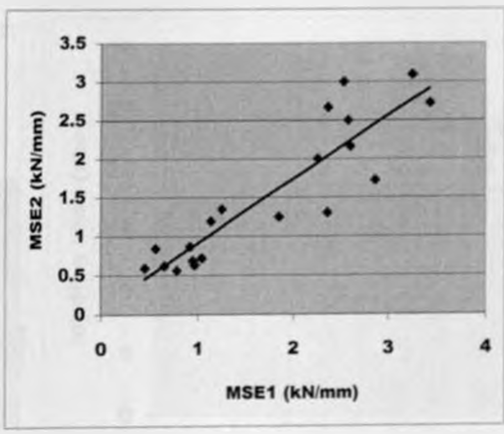


All the bitumen grades combined

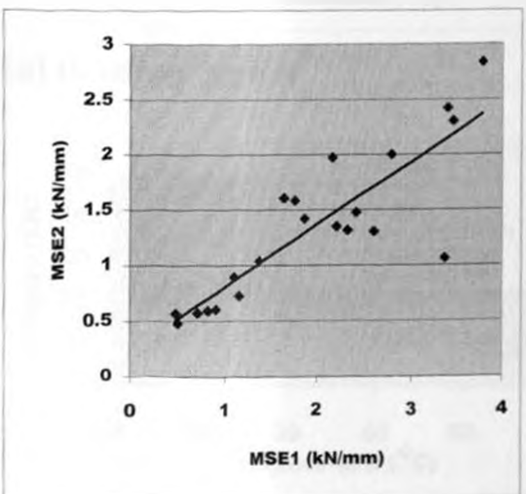
(b) Flow (continued)



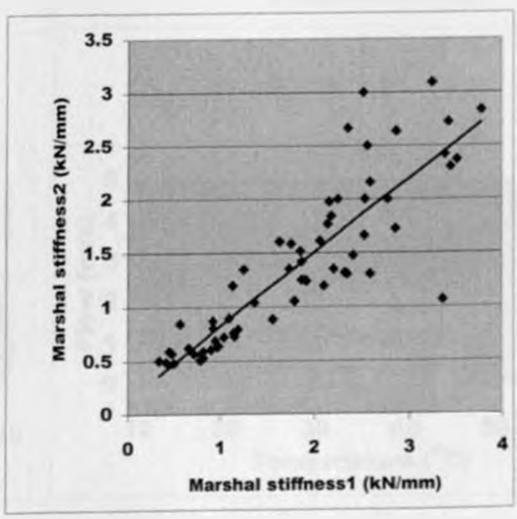
Bitumen 180/200



Bitumen 80/100



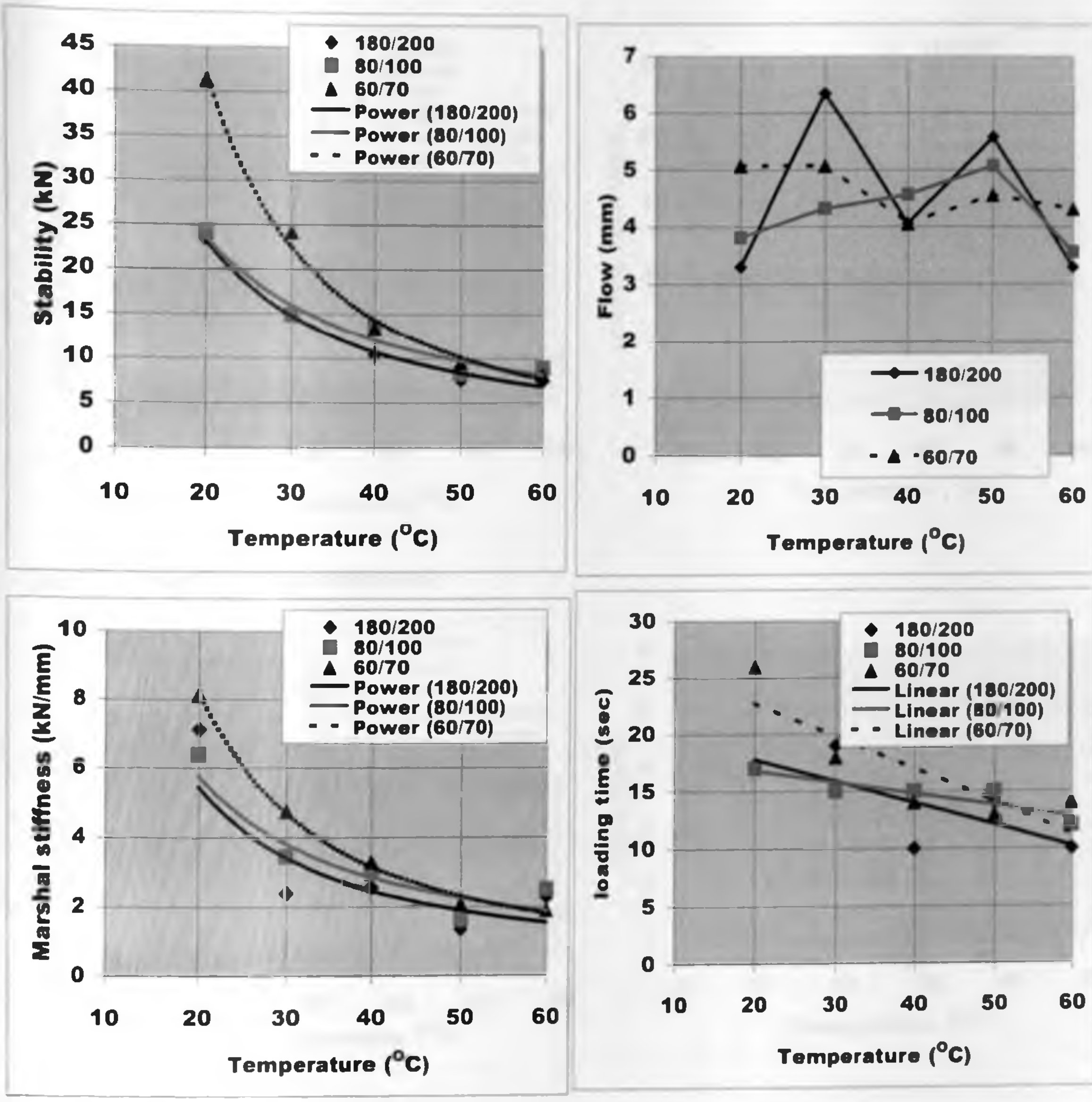
Bitumen 60/70



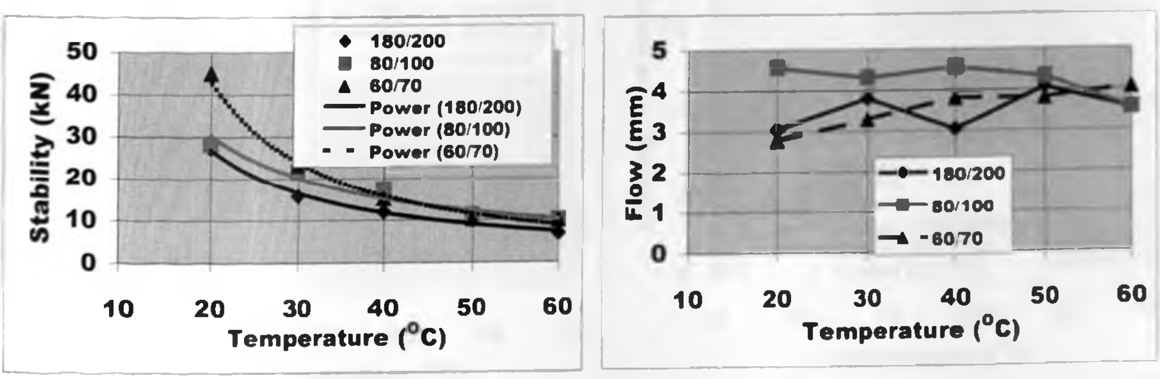
All the bitumen grades combined

(c) Marshal stiffness

Figure A 5.1 (contd.) Healing behaviour for bituminous mixtures

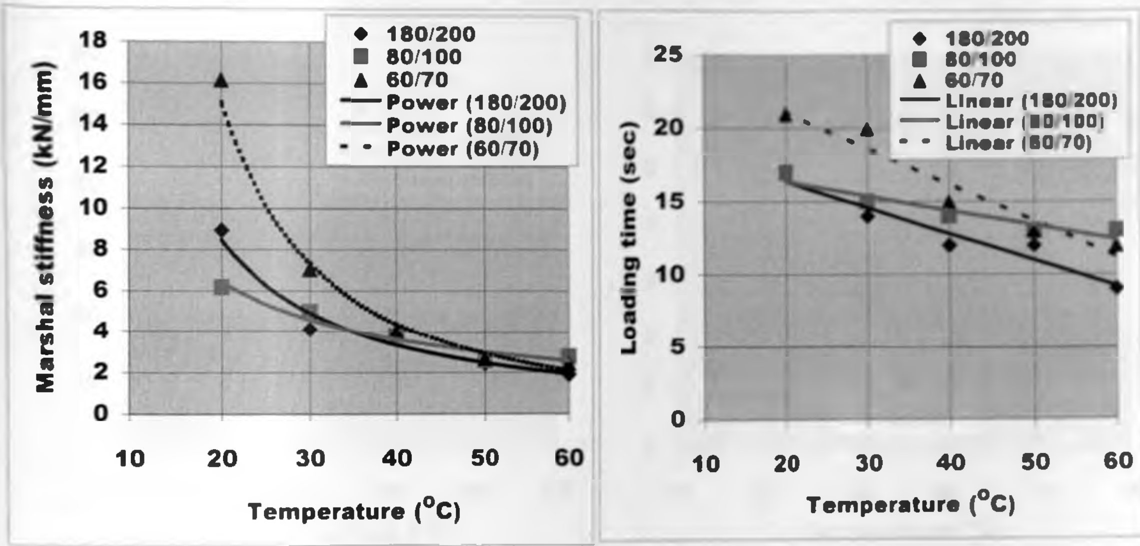


(a) Grading type A

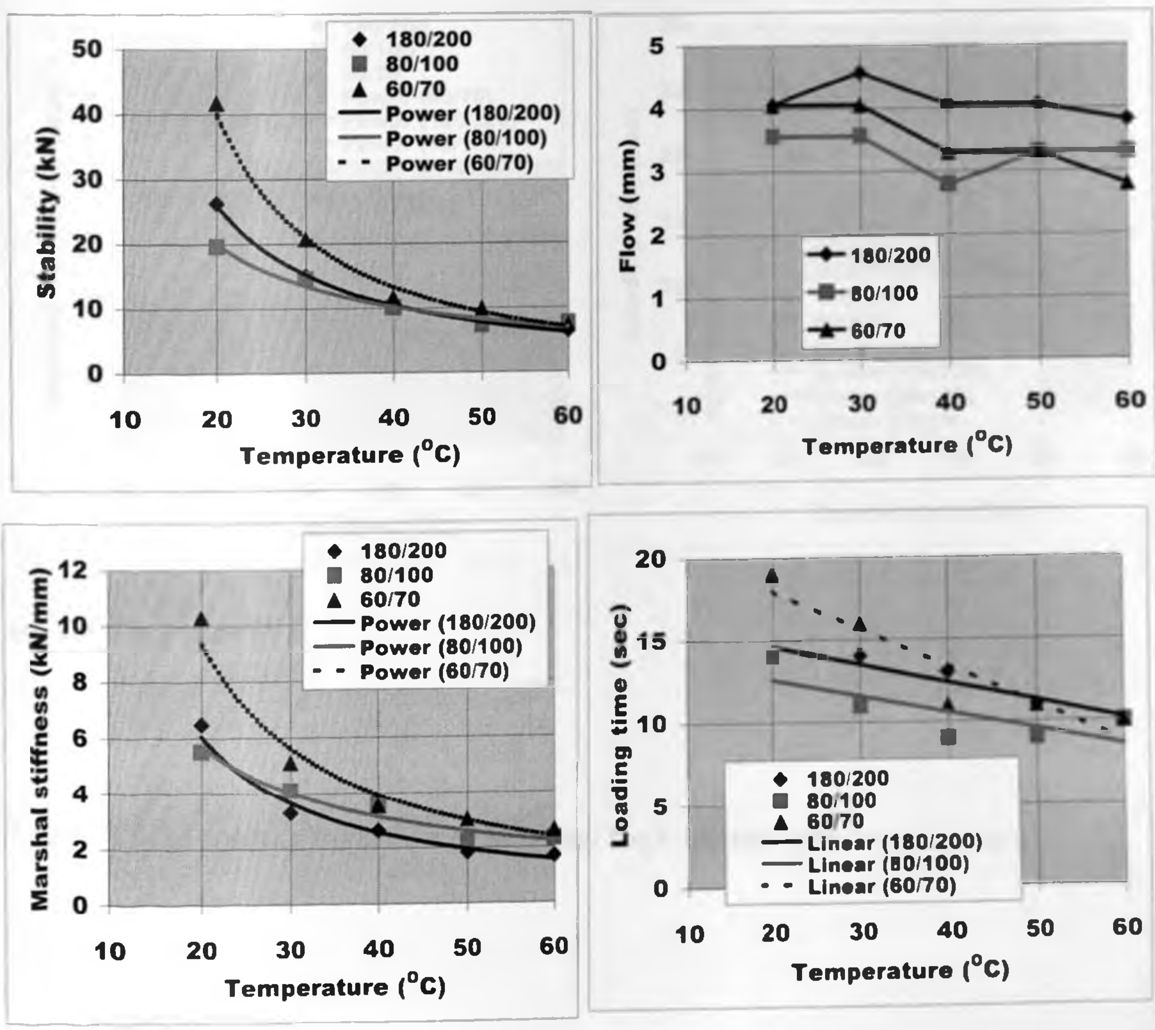


(b) Grading type B

Figure A 5.2 Variation of Marshal test results with temperature

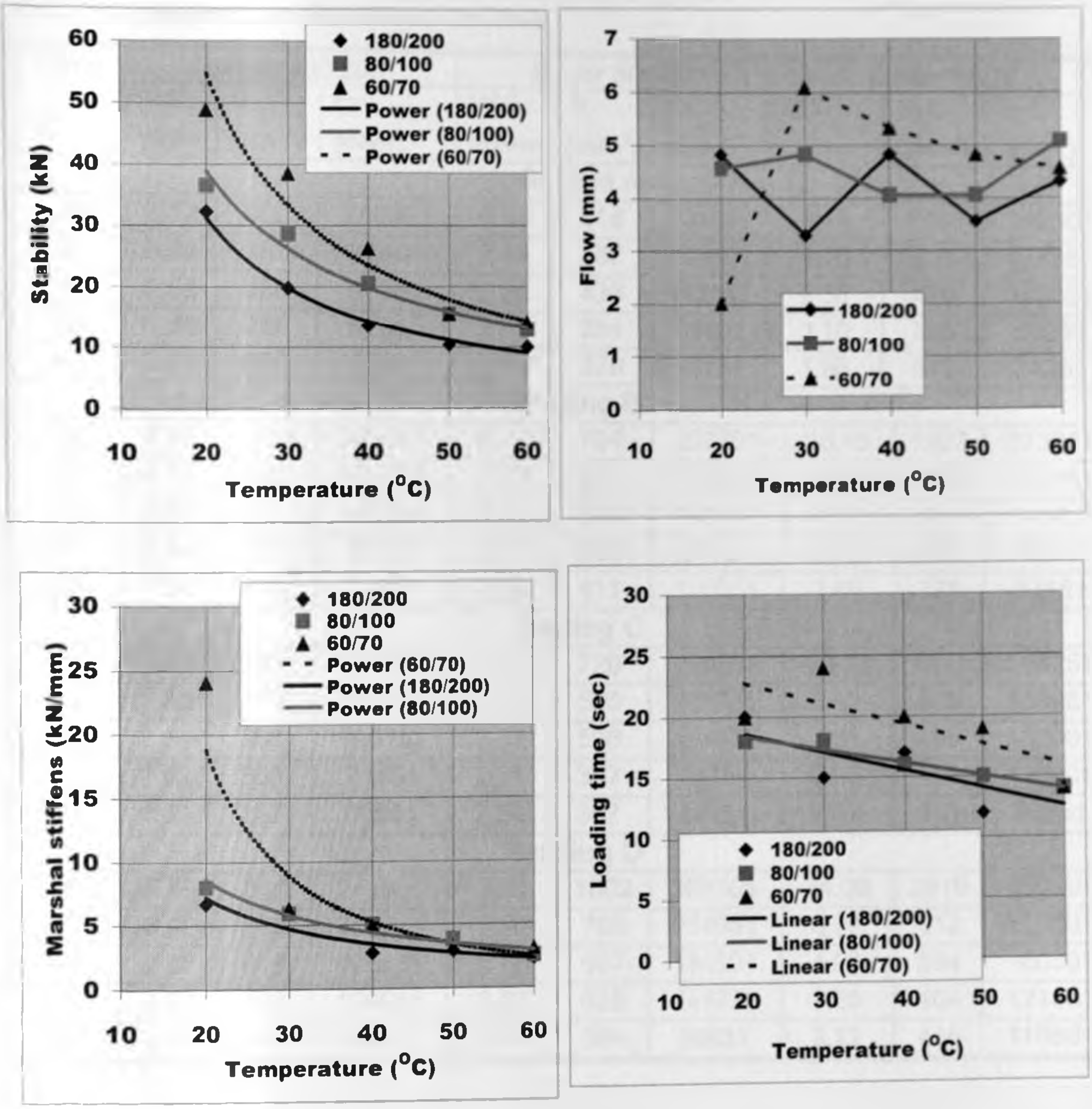


(b) Grading type B (continued)



(c) Grading type C

Figure A 5.2 (contd.) Variation of Marshal test results with temperature



(d) Grading type D

Figure A 5.2 (contd.) Variation of Marshal test results with temperature

Table A5.1 Variation of Marshal stiffness, tensile strength and compressive modulus with temperature for bituminous mixtures

Temp °C	Binder 180/200			Binder 80/100			Binder 60/70		
	MSE (kN/m	St (kN/m^2)	Ey (kN/m^2)	MSE (kN/mm	St (kN/m^2)	Ey (kN/m^2)	MSE (kN/mm)	St (kN/m^2)	Ey (kN/m^2)
Grading A									
20	7.12	902	256951	6.38	818	231051	8.13	1016	292301
30	2.39	367	91401	3.45	487	128501	4.76	635	174351
40	2.56	386	97351	2.98	433	112051	3.30	470	123251
50	1.36	250	55351	1.63	281	64801	2.10	334	81251
60	2.27	353	87201	2.50	379	95251	1.90	312	74251
Grading B									
20	8.95	1108	321001	6.17	794	223701	16.16	1923	573351
30	4.12	563	151951	4.98	660	182051	7.06	895	254851
40	3.93	541	145301	3.81	528	141101	4.02	551	148457
50	2.44	373	93151	2.57	387	97701	2.70	402	102251
60	1.91	313	74601	2.78	411	105051	2.46	375	93851
Grading C									
20	6.48	829	234551	5.51	720	200601	10.30	1261	368251
30	3.30	470	123251	4.10	560	151251	5.12	676	186951
40	2.64	395	100151	3.62	506	134451	3.55	495	132001
50	1.82	303	71451	2.21	347	85101	3.03	439	113801
60	1.68	287	66551	2.30	357	88251	2.62	393	99451
Grading D									
20	6.69	853	241901	8.01	1002	288101	24.09	2819	850901
30	6.00	775	217751	5.92	766	214951	6.33	812	229301
40	2.82	416	106451	5.05	667	184501	4.93	654	180301
50	2.92	427	109951	3.82	528	141451	3.25	464	121501
60	2.31	358	88601	2.54	384	96651	3.11	448	116601

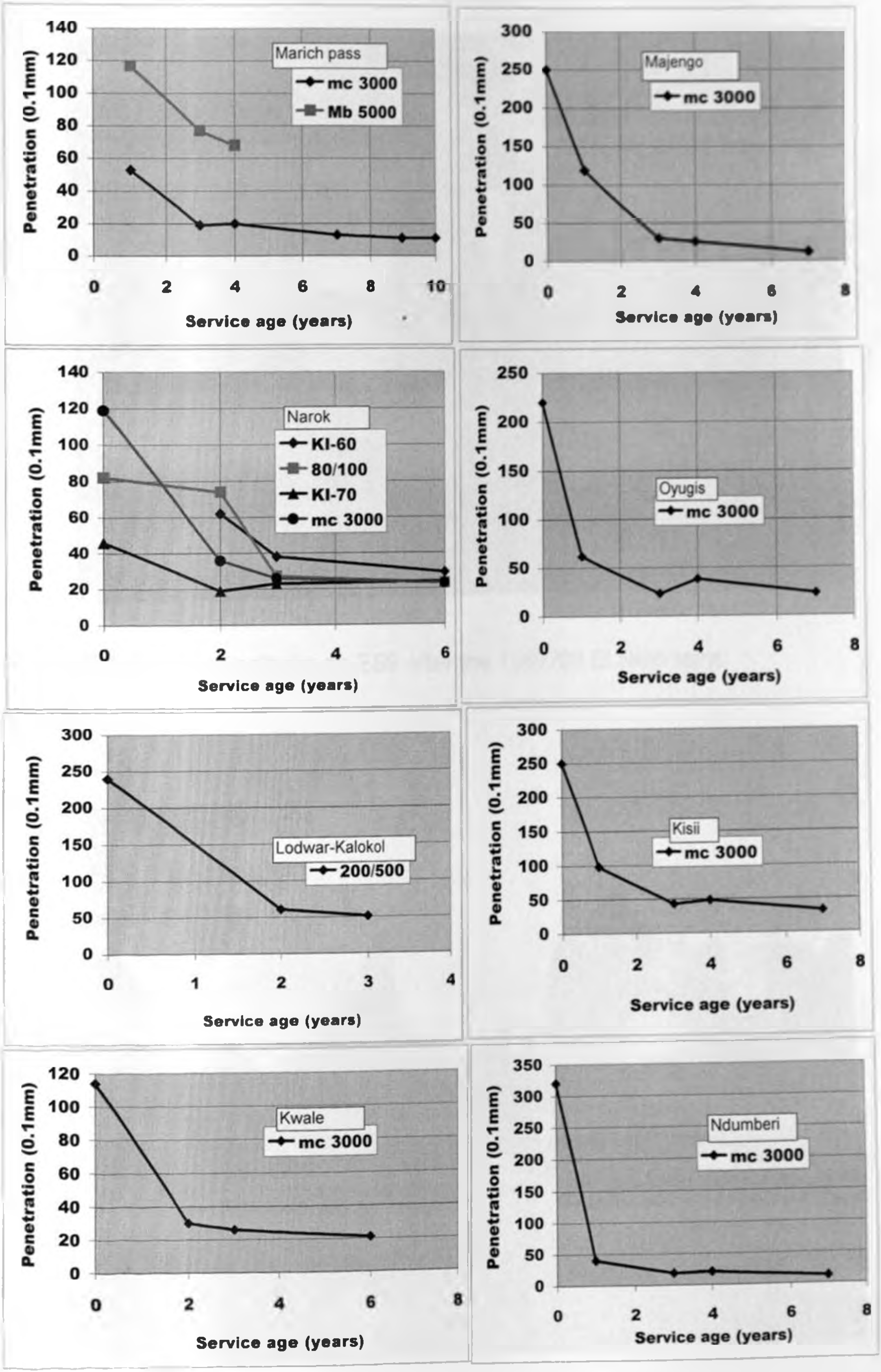


Figure A5.3 Penetration - Time curves for recovered bitumen binder from environmental sites

Plate A5.1 DCP at a pothole on ES9 after the 1997/98 El Nino rains

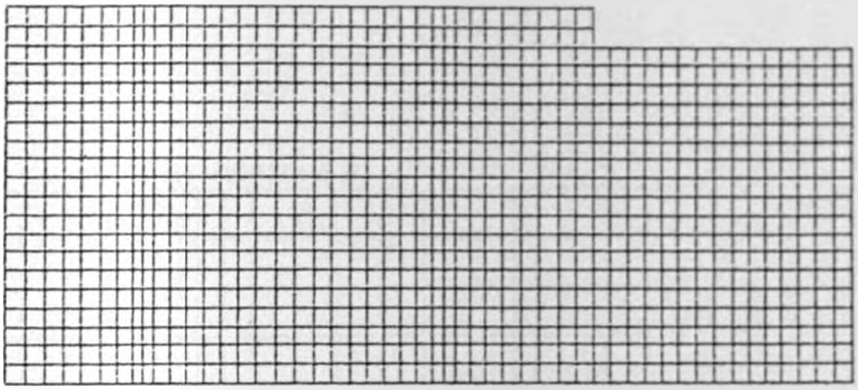
Plate A5.2 Surfacing worn out at ES10 after the 1997/98 El Nino rains

Table A5.2 Historical mean deflection for the experimental test sites.

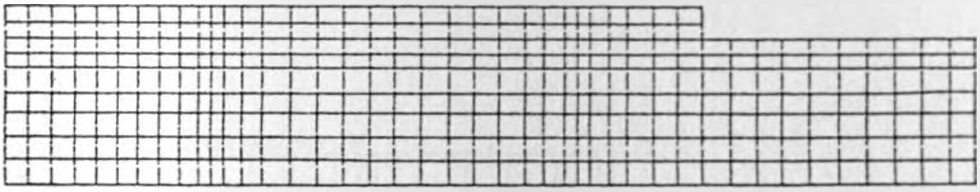
Years Of Service	ESA x 10^6	Deflection (x.01mm)
ES1-Opened to traffic in 1961.		
7.8	2.08	50
8.2	2.22	33
8.4	2.29	23
9.4	2.64	35
9.5	2.68	20
19.75	7.82	27
20.75	8.51	39
23.67	10.76	31
135mm thick overlay		
24.17	11.18	24
24.67	11.62	27
28.75	15.71	16
ES2-Opened to traffic in 1965.		
3.60	.34	47
3.85	.37	42
4.10	.40	45
4.85	.48	36
4.93	.49	46
14.85	2.02	50
15.43	2.14	54
17.35	2.56	45
Slurry seal application.		
19.60	3.13	31
20.10	3.26	40
24.18	4.53	27
ES3-Opened to traffic in 1977.		
4.00	5.51	49
4.58	6.42	47
5.58	8.07	64
35 mm overlay		
9.0	14.48	44
13.0	23.79	40
ES4 Opened to traffic in 1977		
4.00	1.90	22
4.58	2.21	21
5.08	2.49	29
35 mm overlay		
9.0	4.98	21
13.0	8.19	32

Table A5.2 (continued) Historical mean deflection for experimental sites

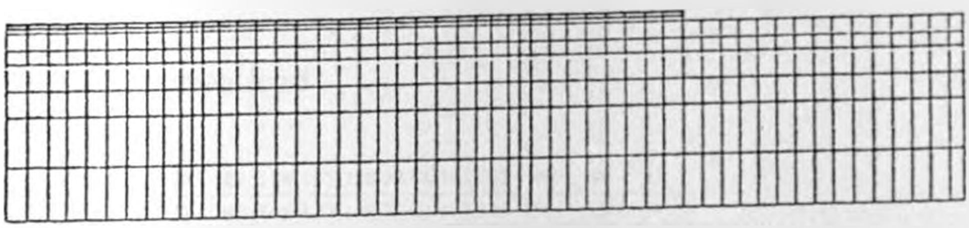
Years Of Service	ESA x 10^6	Deflection (x.01mm)
ES5-Opened to traffic in 1977		
4.00	7.41	.37
4.50	8.46	41
5.08	9.72	45
8.58	18.31	62
35 mm overlay		
8.92	19.24	39
12.92	31.70	29
ES6 – Opened to traffic in 1977		
4.00	7.41	21
4.50	8.46	25
4.92	9.37	31
8.58	18.31	50
8.92	19.24	64
12.92	31.70	8
ES7 – Opened to traffic in 1981		
3.50	3.25	34
3.75	3.50	47
4.50	4.30	39
8.42	9.08	48
ES9 – Opened to traffic in 1974		
15.0	-	60
15.50	-	43
ES10 – Opened to traffic in 1974		
15.0	-	49
15.50	-	80



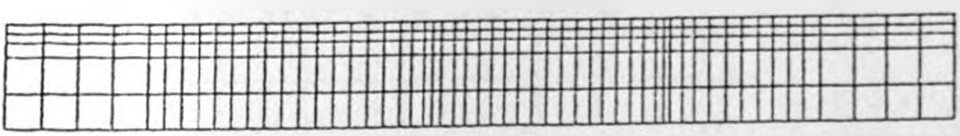
(a) ES1



(b) ES3

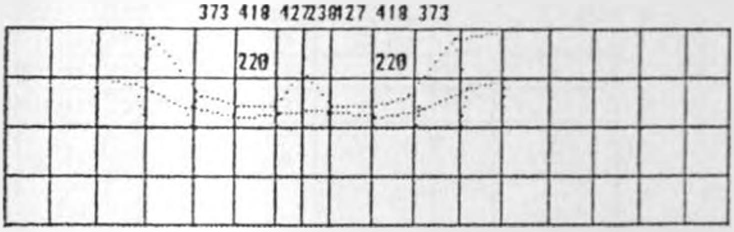


(c) ES7

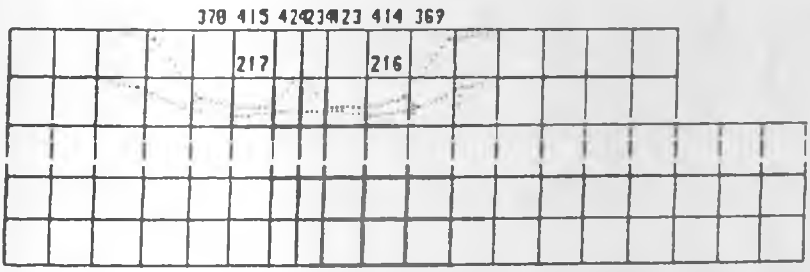


(d) ES9

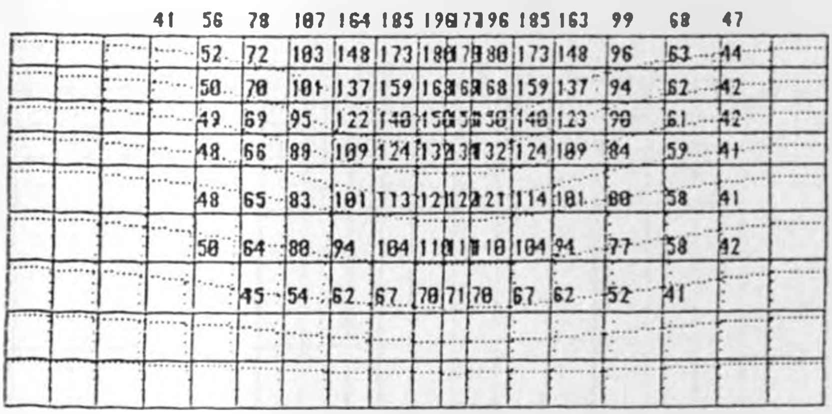
Figure A5.4 FEM analysis: Model geometry



(a) ES1 Inner wheel load

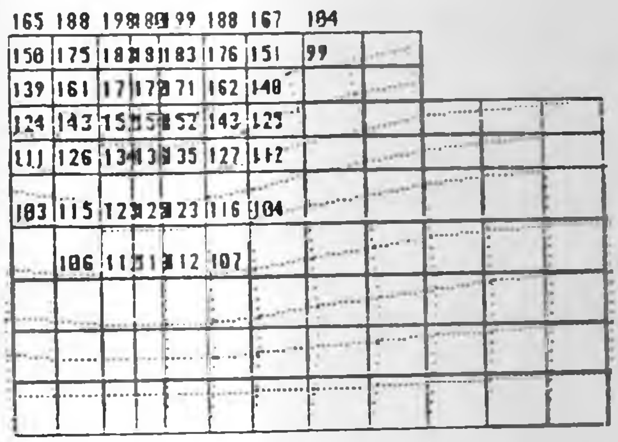


(b) ES1 Outer wheel load

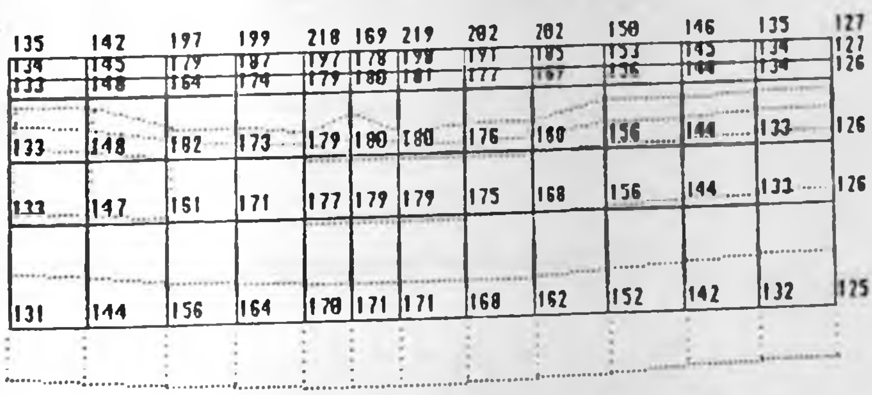


(c) ES3 Inner wheel load

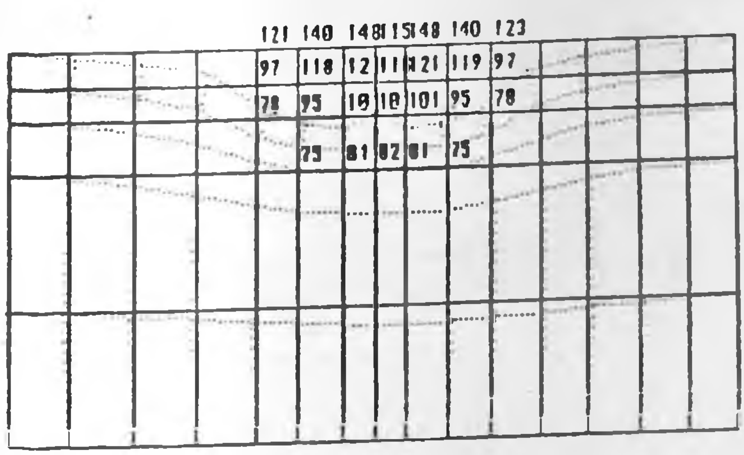
Figure A5.5 FEM analysis
Deflection under wheel loads x10^{-3} mm



(d) ES3 Outer wheel load



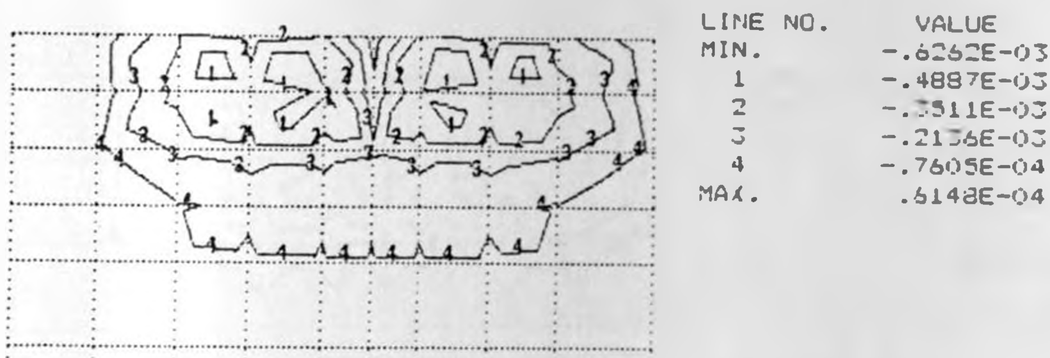
(e) ES7 Inner wheel load



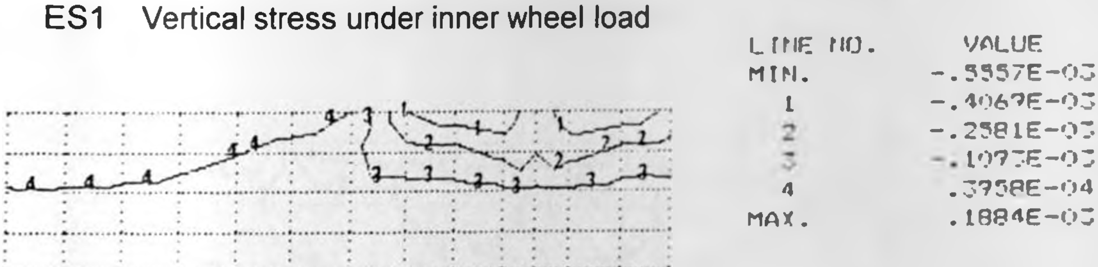
(f) ES9 Outer wheel load

Figure A5.5 (continued) FEM analysis:
Deflection under wheel loads x 10^{-3} mm

(a) ES1 Vertical stress under outer wheel load



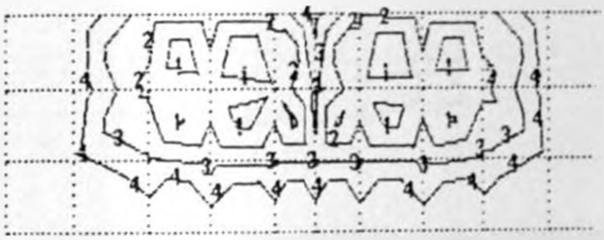
(b) ES1 Vertical stress under inner wheel load



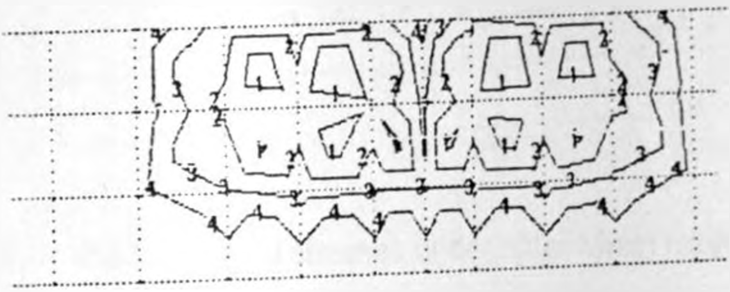
(c) ES1 Horizontal stress under outer wheel load

(d) ES1 Horizontal stress under inner wheel load

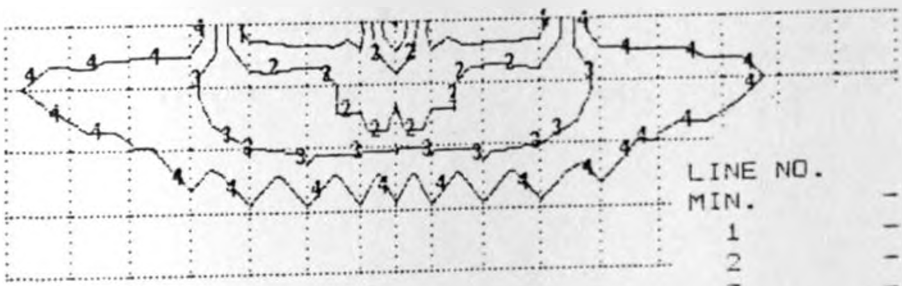
Figure A5.6 FEM analysis:
Vertical and horizontal stresses under wheel loads



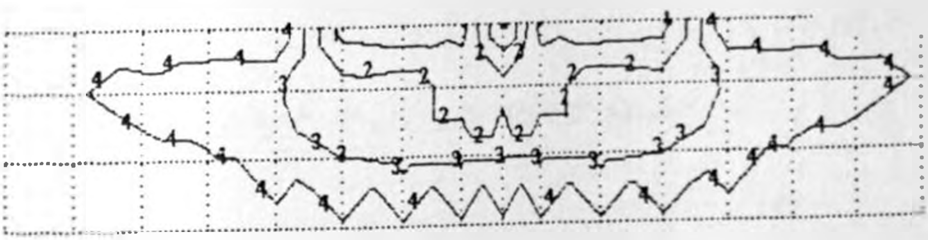
(e) ES3 Vertical stresses under outer wheel loads



(f) ES3 Vertical stresses under inner wheel loads



(g) ES3 Horizontal stresses under inner wheel loads

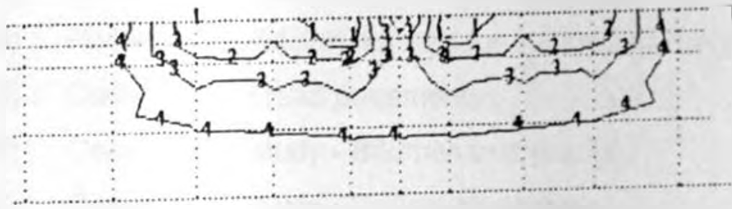


(h) ES7 Vertical stresses under inner wheel loads

Figure A5.6 (continued). FEM analysis
Vertical and horizontal stresses under wheel loads

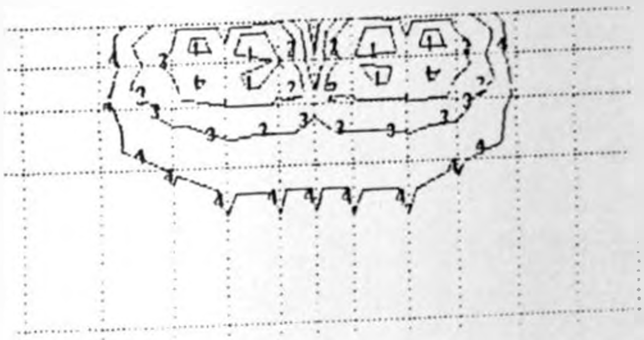
LINE NO.	VALUE
MIN.	-.4381E-03
1	-.3324E-03
2	-.2268E-03
3	-.1211E-03
4	-.1548E-04
MAX.	.9017E-04

(i) ES7 Horizontal stresses under inner wheel loads



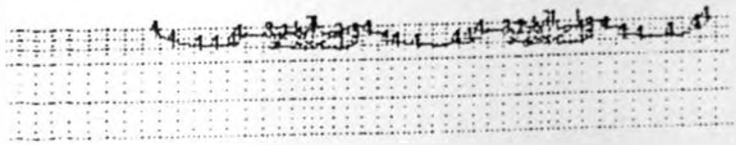
LINE NO.	VALUE
MIN.	-.4477E-03
1	-.3418E-03
2	-.2357E-03
3	-.1300E-03
4	-.2413E-04
MAX.	.8173E-04

(j) ES7 Vertical stresses under outer wheel loads



LINE NO.	VALUE
MIN.	-.6327E-03
1	-.4949E-03
2	-.3571E-03
3	-.2193E-03
4	-.8147E-04
MAX.	.5632E-04

(k) ES9 Vertical stresses under left wheel loads



LINE NO.	VALUE
MIN.	-.3580E-03
1	-.2664E-03
2	-.1748E-03
3	-.8325E-04
4	.8347E-05
MAX.	.9994E-04

(l) ES9 Horizontal stresses under left wheel loads

Figure A5.6 (continued). FEM analysis
Vertical and horizontal stresses under wheel loads

Appendix B

Statistical Analyses Results for the Developed Relationships

B1) Subgrade and non-bituminous materials

Correlation		Analysis of Variance Parameters					
r	r^2		DF	SS	MS	F	Signf. F
Equation 5.1- (Black cotton Soils) PI = -.5 0 + 0.45LL							
.64365	.41428	Regression	1	257.0820	257.0820	12.73146	.0022
		Residual	18	363.4679	20.19266		
Equation 5.2 - (Red coffee soils) PI = -15.6 + .64LL							
.84181	.70865	Regression	1	285.7497	285.7497	26.75537	.0003
		Residual	11	117.4810	10.68009		
Equation 5.3 – (Black cotton soils) OMC = 71 – MDD/32							
.84968	.72196	Regression	1	190.3990	190.3990	46.73870	.0000
		Residual	18	73.32645	4.07369		
Equation 5.3 – (Red coffee soils) OMC =84 – MDD/27							
.92303	.85199	Regression	1	495.5286	495.5286	63.32106	.0000
		Residual	11	86.08217	7.82565		
Equation 5.3 – (Clayey sands) OMC = 54 – MDD/47							
.92065	.84760	Regression	1	784.9698	784.9698	355.9449	.0000
		Residual	64	141.1400	2.20531		
Equation 5.3 – (Lateritic gravels) OMC = 66 – MDD/38							
.88755	.78774	Regression	1	751.8182	751.8182	308.0275	.0000
		Residual	83	202.5822	2.44075		
Equation 5.3 – (General equation) OMC = 66 – MDD/37							
.92915	.86332	Regression	1	6710.400	6710.400	1162.170	.0000
		Residual	184	1062.420	5.77402		
Equation 5.4 –CBR = 11 – 0.33PI							
.81892	.67063	Regression	1	67.06283	67.06283	38.68559	.0000
		Residual	19	32.93717	1.73354		
<u>Figure 5.5 (a) Western Kenya gravel soils</u> – MDD = 719.6 + .53MBD							
.51994	.27033	Regression	1	49984.18	49984.18	20.00641	.0000
		Residual	54	134914.0	2498.407		
<u>Figure 5.5 (b) Meru red coffee soils</u> – MDD = -1337 + 1.5MBD							
.96045	.92247	Regression	1	292544.3	292544.3	237.9618	.0000
		Residuals	20	24587.49	1229.374		
Figure 5.7 – CBR = 125LC$^{1/13}$							
.82556	.68155	Regression	1	22.44591	22.44591	166.9387	.0000
		Residuals	78	10.48756	.134456		

B2) Bituminous Mixtures

Correlation		Analysis of Variance Parameters					
r	r^2		DF	SS	MS	F	Signf F
Equation 5.5 (180/200 binder) - VTM = 36.74BC$^{-1.091}$							
.80047	.64076	Regression	1	2.1988075	2.1988075	35.67281	.0000
		Residual	20	1.2327640	.0616382		
Equation 5.5 (80/100 binder) - VTM = 15.74BC$^{-.57}$							
.58969	.34773	Regression	1	39690894	.39690894	9.59614	.0062
		Residuals	18	.74450380	.04136132		
Equation 5.5 (60/70) - VTM = 36.87BC$^{-1.18}$							
.66811	.44637	Regression	1	2.0531191	2.0531191	15.31883	.0009
		Residuals	19	2.5464904	.1340258		
Equation 5.6 (180/200 binder) - VFB = 17.08BC.^{774}							
.91116	.83022	Regression	1	1.0992641	1.0992641	97.79924	.0000
		Residuals	20	.2248001	.0112400		
Equation 5.6 (80/100 binder) - VFB = 27.22BC$^{.517}$							
.87058	.75791	Regression	1 .	31160262	.31160262	56.35397	.0000
		Residuals	18	09952888	.00552938		
Equation 5.6 (60/70 binder) - VFB = 21.42BC$^{.672}$							
.87260	.76143	Regression	1	73046628	.73046628	60.63986	.0000
		Residuals	19	.22887353	.01204598		
Equation 5.7 (180/200 binder) - F = .90BC$^{.91}$							
.73373	.53836	Regression	1	1.5196354	1.5196354	23.32395	.0001
		Residuals	20	1.3030687	.0651534		
Equation 5.7 (80/100 binder) - F = 1.1BC$^{.68}$							
.83062	.68993	Regression	1	1.4678530	1.4678530	40.05161	.0000
		Residuals	18	.6596827	.0366490		
Equation 5.7 (60/70 binder) - F = .95BC$^{.90}$							
.77955	.60769	Regression	1	1.8179122	1.8179122	30.98034	.0000
		Residuals	20	1.1735908	.0586795		
Equation 5.8 (180/200 binder) - MSE = 18.93BC$^{-1.49}$							
.72974	.53252	Regression	1	4.1044585	4.1044585	22.78274	.0001
		Residuals	20	3.6031304	.1801565		
Equation 5.8 (80/100 binder) - MSE = 79.65BC$^{-2.26}$							
.89983	.80970	Regression	1	6.1544521	6.1544521	76.58754	.0000
		Residuals	18	1.4464511	.0803584		

Correlation		Analysis of Variance Parameters					
r	r^2		DF	SS	MS	F	Signf F
Equation 5.8- (60/70 binder) - $MSE = 33.94BC^{1.8}$							
.86042	.74033	Regression	1	6.5039941	6.5039941	57.01964	.0000
		Residuals	20	2.2813173	.1140659		
Equation 5.9 (Marshal stability healing) - $MST2 = .45 + 0.80MST1$							
.94029	.88415	Regression	1	228.24720	228.24720	465.54235	.0000
		Residual	61	29.90722	.49028		
Equation 5.10 (Flow healing) - $F2 = .68 + 1.97F1$							
.84422	.71271	Regression	1	99.84536	99.84536	148.84462	.0000
		Residual	60	40.24816	.67080		
Equation 5.11 (Marshal stiffness healing) - $MSE2 = .09 + 0.72MSE1$							
.90533	.81963	Regression	1	27.69284	27.69284	277.19555	.0000
		Residual	61	6.09412	.09990		
Equation 5.15a - $MS = 506Temp^{-.97}$							
.82320	.67766	Regression	1	2.8602673	2.8602673	37.84224	.0000
		Residuals	18	1.3605118	.0755840		
Equation 5.15b - $LT = 18.1 - .102Temp$							
.54789	.30018	Regression	1	42.02500	42.02500	7.72085	.0124
		Residual	18	97.97500	5.44306		
Equation 5.15c - $MSE = 103.05Temp^{-.92}$							
.85885	.73763	Regression	1	2.5500644	2.5500644	50.60509	.0000
		Residuals	18	.9070463	.0503915		

B3) Environmental effects on pavement materials

Correlation		Analyses of variance					
r	r^2		DF	SS	MSS	F	Signf F
Equation 5.17 (Majengo) $T/\Delta pen = .0814 + .0054T$							
.978	.9564	Regression	1	.03935	.03935	90.01842	.0109
		Residual	2	.00087	2.00044		
Equation 5.17 (Oyugis) $T/\Delta pen = .0771 + .0119T$							
.9703	.9416	Regression	1 .	.06810	.06810	1380.4324	.0007
		Residual	2	.00010	.00005		

Correlation		Analyses of variance					
r	r^2		DF	SS	MS	F	Signf F
Equation 5.17 (Kisii) T/Δpen = .1043 + .0067T							
.9890	.9782	Regression	1	.04248	.04248	216.75000	.0046
		Residual	2	.00039	.00020		
Equation 5.17 (Ndumberi) T/Δpen = .0486 + .0094T							
1	1	Regression	1	.02484	.02484	1552.6875	.0006
		Residual	2	.00003	.00002		
Equation 5.17 (Kwale) T/Δpen = .0707 + .0094T							
1	1	Regression	1	.18286	.18286	1678.6885	.0006
		Residual	2	.00022	.00011		
Equation 5.17 (Narok) T/Δpen = .0707 + .0094T							
.9950	.9900	Regression	1	.18286	.18286	8533.3333	.0001
		Residual	2	.00004	.00002		
Equation 5.17 (Marich Pass) T/Δpen = .0707 + .0094T							
.9921	.9843	Regression	1	.47681	.47681	14.80885	.0614
		Residual	2	.06439	.03220		
Equation 5.18 Pen = 105(VTM)$^{-.72}$							
.87324	.76255	Regression	1	2.2489937	2.2489937	19.26846	.0046
		Residuals	6	.7003133	.1167189		
Equation 5.19 - Duct = 109(VTM)$^{-1.22}$							
.80565	.64907	Regression	1	6.4048508	6.4048508	11.09767	.0158
		Residuals	6	3.4628099	.5771350		
Figure 5.13C - SP = 50 + 1.6(VTM)							
.79681	.63491	Regression	1	323.46790	323.46790	10.43440	.0179
		Residual	6	186.00085	31.00014		
Figure 5.14 - Pen = 165 – 4.63Asc							
.80281	.64450	Regression	1	2956.9579	2956.9579	14.50341	.0052
		Residual	8	1631.0420	203.88025		
Equation 5.20 - FMC = -2.86 + 0,91OMC							
.85240	.72659	Regression	1	207.93600	207.93600	23.91724	.0009
		Residual	9	78.24582	8.69398		
Equation 5.21 – CBR_{soak} = .628 + .529CBR_{omc}							
.94250	.88830	Regression	1	311.71328	311.71328	71.57448	.0000
		Residual	9	39.19581	4.35509		

Correlation		Analyses of variance					
r	r^2		DF	SS	MS	F	Signf F
Equation 5.22 – $CBR_{1/2omc} = 17 + 1.5CBR_{soak}$							
.76452	.58448	Regression	1	819.55254	819.55254	12.65980	.0061
		Residual	9	582.62927	64.73659		

B4) Pavement condition surveys for the road pavements

Equation 5.23 – TR = 40.9 + 7.5CR							
.88423	.78186	Regression	1	119142.73	119142.73	136.20357	.0000
		Residual	38	33240.125	874.74015		
Figure 5.20a - PSI = 5.03 - .41LnTR							
.30809	.09492	Regression	1	.96238	.96238	3.98529	.0531
		Residual	38	9.17633	.24148		
Figure 5.20b - PSI = 10.3-.96LnRo							
.21470	.04610	Regression	1	.46737	.46737	1.83637	.1834
		Residual	38	9.67134	.25451		
Figure 5.20c - PSI = 3.16 - .03CI							
.17044	.02905	Regression	1	.29453	.29453	1.13693	.2930
		Residual	38	9.84418	.25906		
Equation 5.24 – $PA = .14PSI^{1.57}$							
.85066	.72362	Regression	1	3.37337	3.37337	99.4912	.0000
		Residual	38	1.28843	.033906		
Equation 5.25 – PSI = 8.82 -.338LnTR -.551LnRo - .0072CI							
.33432	.11177	Regression	3	1.13322	37774	1.51004	.2284
		Residual	36	9.00549	.25015		
Equation 5.26 – Ro = 1615 +.83TR							
.25796	.06655	Regression	1	103742.4	103742.4	2.70899	.1080
		Residual	38	1455231.	38295.56		
Equation 5.27 – Ro = 1653 + 21 CI							
.29527	.0871	Regression	1	135916.1	135916.1	3.62938	.0644
		Residual	38	1423057.	37448.89		
Equation 5.28 – LnRo = 7.60 - .048PSI							
.21470	.04610	Regression	1	.02322	.02322	1.83637	.1834
		Residual	38	.48047	.01264		

B5 Deflection for road pavements

Correlation		Analyses of variance					
r	r^2		DF	SS	MS	F	Signf F
Table 5.16 – (ES1-L) 1/y = .033530 + .040670X							
.90851	.82539	Regression	1	00029	.00029	18.9079	.0122
		Residual	4	.00006	4.00002		
Table 5.16 – (ES1R) - 1/y = .063671 + .044993X							
.92718	.85966	Regression.	1	.00035	00035	24.50212	.0078
		Residual	4	00006	.. 00001		
Table 5.16 – (ES2L) - 1/y = .020957 + .023878 X							
.97333	.94736	Regression.	1	.00010	00010	71.99492	.0011
		Residual	4	.00001	.00000		
Table 5.16 – (ES2R) - 1/y = .018 + .035 X							
.88835	.78917	Regression	1	.00022	.00022	14.97246	.0180
		Residual	4	.00006	.00001		
Table 5.16 – (ES3L) - 1/y = .014807 + .034595 X							
.96582	.93281	Regression	1	.00021	.00021	55.53624	.0017
		Residual	4	.00002	.00000		
Table 5.16 – (ES3R) - 1/y = .015627 + .073245X							
.94233	.88799	Regression	1	.00094	.00094	31.71125	.0049
		Residual	4	.00012	.00003		
Table 5.16 – (ES4L) - 1/y = .019853 + .039788X							
.97141	.94363	Regression	1	.00028	.00028	66.96250	.0012
		Residual	4	.00002	.00000		
Table 5.16 – (ES4R) - 1/y = .037678 + .067423X							
.95257	.90739	Regression	1	.00080	.00080	=39.19399	.0033
		Residual	4	.00008	.00002		
Table 5.16 – (ES5L) - 1/y = .019286 + .055972X							
.94610	.89511	Regression	1	.00055	.00055	34.13606	.0043
		Residual	4	.00006	.00002		
Table 5.16 – (ES5R) - 1/y = .010943 + .085630X							
.92986	.86465	Regression	1	.00128	.00128	25.55213	.0072
		Residual	4	.00020	.00005		
Table 5.16 – (ES6L) - 1/y = .078102 + .147186X							
.90680	.82229	Regression	1	.00379	.00379	18.50834	.0126
		Residual	4	.00082	.00020		

Correlation		Analysis of Variance Parameters					
r	r^2		DF	SS	MS	F	Signf. F
Table 5.16 – (ES6R) - 1/y = .052570 + .082710X							
.96239	.92620	Regression	1	.00120	.00120	50.19960	.0021
		Residual	4	.00010	.00002		
Table 5.16 – (ES7L) - 1/y = .020780 + .025894X							
.96004	.92167	Regression	1	.00012	.00012	47.06681	.0024
		Residual	4	.00001	.00000		
Table 5.16 – (ES7R) - 1/y = .012794 + .020515X							
.99092	.98191	Regression	1	.00007	.00007	217.15372	.0001
		Residual	4	.00000	.00000		
Table 5.16 – (ES9L) - 1/y = .012794 + .020515X							
.90377	.81680	Regression	1	.00165	.00165	17.83431	.0134
		Residual	4	.00037	.00009		
Table 5.16 – (ES9R) - 1/y = .008019 + .094565X							
.95087	.90415	Regression	1	.00156	.00156	37.73004	.0036
		Residual	4	.00017	.00004		
Table 5.16 – (ES10L) - 1/y = .003132 + .049355X							
.93785	.87957	Regression	1	.00043	.00043	29.21338	.0057
		Residual	4	.00006	.00001		
Table 5.16 – (ES10R) - 1/y = .007552 + .088474 X							
.90765	.82384	Regression	1	.00137	.00137	18.70630	.0124
		Residual	4	.00029	.00007		
Equation 5.31 – $R = 596949 d_{90}^{-1.95}$							
.86071	.74081	Regression	1	25.755825	25.755825	45.73160	.0000
		Residuals	16	9.011126	.563195		
Table 5.17 – (ES1) - $d_l = -9.956522 + 2.478261 d_r$							
.96721	.93550	Regression	1	70.63043	70.63043	58.01786	.0016
		Residual	4	4.86957	1.21739		
Table 5.17 – (ES2) - $d_l = 18.232660 + .500439\ d_r$							
.87100	.75864	Regression	1	142.62511	142.62511	12.57304	.0239
		Residual	4	45.37489	11.34372		
Table 5.17 – (ES3) - $d_l = 9.911269 + 1.066992 d_r$							
.95964	.92091	Regression	1	855.37193	855.37193	46.57531	.0024
		Residual	4	73.46140	18.36535		

Correlation		Analysis of Variance Parameters					
r	r2		DF	SS	MS	F	Signf. F
Table 5.17 – (ES4) - d_l = -2.779271 + 1.988484d_r							
.98576	.97173	Regression	1	343.34485	343.34485	137.49628	.0003
		Residual	4	9.98848	2.49712		
Table 5.17 – (ES5) - d_l = 10.620927 + .58528d_r							
.98816	97647	Regression	1	448.52351	448.52351	165.96880	.0002
		Residual	4	10.80983	2.70246		
Table 5.17 – (ES6) - d_l = -.489796 + .681633d_r							
.91153	.83090	Regression	1	18.97211	18.97211	19.65398	.0114
		Residual	4	3.86122	.96531		
Table 5.17 – (ES7) - d_l = 8.438918 + .503927d_r							
.97681	.95416	Regression	1	194.01178	194.01178	83.25298	.0008
		Residual	4	9.32155	2.33039		
Table 5.17 – (ES9) - d_l = 5.887459 + .719333d_r							
.98578	.97176	Regression	1	1132.5897	1132.5897	137.65810	.0003
		Residual	4	32.91023	8.22756		
Table 5.17 – (ES10) - d_l = -11.021586 + 2.266909d_r							
.98591	.97202	Regression	1	10117.594	10117.594	138.95926	.0003
		Residual	4	291.23914	72.80979		

B6) Case airport study Bitumen test results

Equation 5.33 – Pen = 112 $(VTM)^{-1.0}$							
.82317	.67760	Regression.	1	.21933108	.21933108	8.40711	.0441
		Residuals	4	.10435505	.02608876		
Equation 5.34 – Sp = 46. $(VTM)^{0.23}$							
.80533	.64856	Regression	1	.01086996	.01086996	7.38172	.0532
		Residuals	4	.00589021	.00147255		
Equation 5.35 – Pen = $1.543x10^9$ $(SP)^{-4.3}$							
.98115	.96266	Regression	1	.31159910	.31159910	103.11851	.0005
		Residuals	4	.01208703	.00302176		

B7 Case airport study : deflection analysis

Equation 5.36 – R = $21.359x10^3$ $(d_{90})^{-1.2}$							
.60087	.36104	Regression	1	3.0280201	3.0280201	16.95139	.0003
		Residuals	30	5.3588892	.1786296		
Equation 5.37 – R = $9.573x10^3$ $(d_{90})^{-1.0}$							
.43685	.19084	Regression	1	1.0385774	1.0385774	3.30179	.0907
		Residuals	14	4.4036945	.3145496		