ee in

The Study of Pavement Design of Nairobi-Thika Highway

By

Clive Kiage Temu,

B.Sc. in Civil Engineering (Hons.)

Department of Civil & Construction Engineering, University of Nairobi,

al as

A thesis submitted in partial/fulfillment for the Degree of Master of Science in Civil Engineering in the Department of Civil and Construction Engineering in the University of Nairobi.

October, 2012

Declaration

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

Clive Kiage Temu,

This thesis has been submitted for examination with my approval as University Supervisor.

Prof. F.J. Gichaga

Signature...

Department of Civil and Construction Engineering

University of Nairobi.

Dedication

This work is dedicated to my children, Barbara and Ethan.

Acknowledgement

I am heartily thankful to my supervisor, Prof. F.J Gichaga, whose encouragement, guidance and support from the initial to the final level enabled me to develop an understanding of the subject. Prof. F.J Gichaga , supported me throughout my thesis with his patience and knowledge whilst allowing me the room to work in my own way. I attribute the level of my Masters degree to his encouragement and effort and without him this thesis, too, would not have been completed or written. One simply could not wish for a better or friendlier supervisor.

I would also like to express my sincere gratitude to the Supervision Consultants of the Nairobi-Thika Road particularly APEC Consortium Limited for allowing me to sample and test materials in their site laboratories and also for sharing with me test results of materials tested earlier for sections where the road pavement had been built. I would also like to thank the Kenya National Highways Authority who were able to share with me recent valuable information regarding axle loads for the Nairobi-Thika Road.

Prof. S.K Mwea encouraged me to produce this piece of work and also inadvertently, and without fail, provided something much greater in all the years I've known him: a friendly smile and a hello every time we met. In my daily work I have been blessed with a friendly and cheerful group of fellow students. Franklin Kipyator, you were very helpful in ensuring that I got the right axle load data and within reasonable time.

Lastly, I offer my regards and blessings to all of those who supported me in any respect during the completion of the project.

Abstract

Highways (Roads) perform a very useful role in meeting the strategic and developmental requirements, accelerating all round development. Technological progress in road construction technology has kept pace with rapid changes in the field of infrastructural development. In Kenya, the projected (20 year design) traffic loads on most sections of Nairobi-Thika Road (A2) surpass the highest traffic classes stipulated in the Ministry of Roads (MoR) Road Design Manual (RDM) Part III, Materials and Pavement Design for New Roads.

Over the years, there has been a significant change in vehicle volumes and designs. Current heavy and medium goods vehicles are designed to carry more axle loads as compared to 40 years ago. There is also a significant change in the spectrum of loads carried by these vehicles. This has effectively changed the repetitions of axle loads and magnitude of damage caused by the vehicles on the roads. A lot of research has previously been carried out on the Nairobi-Thika Road and documented. Based on this research and the fact that this road is currently undergoing improvement, this provides a good case for study as current vehicle volumes, axle loads and design could be used to compare with the scenario that existed 40 years ago.

This research involved a study of both alignment soils and axle load data of the Nairobi-Thika (A2) Road. The objectives of the study were to establish the variation of engineering properties of soils with depth particularly at the deep cut sections, to establish axle load data for the Nairobi-Thika Road and to compare year 2011 axle loading with the 1971 axle load.

In this study, existing data from the design consultant was collected and in addition, laboratory tests of soils at selected sections along the study road were done.

An axle load study was carried out and it was established that there is a significant change in the current axle load as compared to 40 years ago. Also, the axle load spectrum of current heavy and medium goods vehicle was found to be narrower compared to 40 years ago.

From both the alignment soil studies and axle loads analysis, conclusions were drawn. It was established that soil properties vary both along the road alignment and with depth. It was established that it is not possible to establish the ruling CBR during construction at design stage. Therefore, at construction stage, detailed investigations should be carried out which would be used for design purposes.

TABLE OF CONTENTS

LIST OF TABLES

List of Figures

ACRONYMS/ABBREVIATIONS

AADT - Average Annual Daily Traffic

AASHO - American Association of State Highway Officials

AASHTO - American Association of State Highway and Transport Officials

ADT - Average Daily Traffic

CBR - California Bearing Ratio

CNSA - Cumulative Number of Standard Axles

EALF - Equivalence Axle Load Factors

ESAL - Equivalent Standard Axle Load

KENHA - Kenya National Highways Authority

MDD - Maximum Dry Density

MOR - Ministry of Roads

OMC - Optimum Moisture Content

RDM - Road Design Manual

RE - Resident Engineer

RN 29 - Road Note 29

RN 31 - Road Note 31

RRTP - Road Research Technical Paper

SN - Structural Number

1 Introduction

1.1 Background

In the recent past, research has been undertaken in different fields leading to innovations that have basically changed the way people live. In highway pavement design, a lot of research has also been undertaken leading to control of material behaviour through the use of synthetics and geobelts in the construction of retaining walls. However, most of the developing nations have for the better part been mere consumers of technology developed elsewhere (Murunga, 1983).

Pavement design in Kenya has undergone considerable development since rule-of-thumb design in the 1940s and 1950s. During the 1960s, most major roads were designed on the basis of the earlier editions for Road Note 31(RN 31) and RN 29. Then, a road design manual adopted in 1970 required the designer to determine traffic loading on the basis of the number of heavy vehicles expected per 24-hour day five years after the road was opened to traffic. The latest design procedure, adopted in 1981 requires the designer to determine the subgrade quality, in terms of the CBR and traffic loading, during the design life of the pavement, in terms of cumulative number of standard axles as determined by RN 29. The pavement structure is then selected from a catalogue of structures (Gichaga and Parker, 1988).

However, the projected traffic loads on some of the major highways in Kenya have surpassed the highest traffic class stipulated in the 1981 design procedure. As a result, traffic classes as stipulated in the 1981 design procedure cannot satisfactorily account for all traffic categories likely to be carried by the bitumen roads in Kenya. There have also been developments in terms of speeds of heavy commercial vehicles as they

are now much faster as compared to our transit goods trains. The rigidity of the train service to overhaul and adopt new technology to usher in new generation train system has contributed to the high traffic loads witnessed on our roads today. As a result, there is need to conduct research that will lead to revision of the current manual to accommodate the new developments.

Highway pavements are a conglomeration of materials. These materials, their associated properties, and their interactions determine the properties of the resultant pavement. Satisfactory pavement performance over its design life is the holistic consequence of the proper design and functioning of all of the key components of the pavement system. Thus a good understanding of these materials, how they are characterized, and how they perform is fundamental to understanding highway pavements. The materials which are used in the construction of a highway are of intense interest to the highway engineer. This requires not only a thorough understanding of the soil and aggregate properties which affect pavement stability and durability, but also the binding materials which may be added to improve their engineering properties.

The first issue to be confronted in any pavement design is whether the project involves new construction, rehabilitation, or reconstruction. New construction is the construction of a pavement system on a new alignment that has not been previously constructed. Rehabilitation is defined as the repair and upgrading of an existing in-service pavement. Typically, this involves repair/removal and construction of additional bound pavement layers (asphalt concrete overlays) and could include partial-depth or full-depth recycling or reclamation. Reconstruction is defined as the complete removal of an existing pavement system, typically down to and including the upper portions of the foundation soil, and the replacement with a new

2

are now much faster as compared to our transit goods trains. The rigidity of the train service to overhaul and adopt new technology to usher in new generation train system has contributed to the high traffic loads witnessed on our roads today. As a result, there is need to conduct research that will lead to revision of the current manual to accommodate the new developments.

Highway pavements are a conglomeration of materials. These materials, their associated properties, and their interactions determine the properties of the resultant pavement. Satisfactory pavement performance over its design life is the holistic consequence of the proper design and functioning of all of the key components of the pavement system. Thus a good understanding of these materials, how they are characterized, and how they perform is fundamental to understanding highway pavements. The materials which are used in the construction of a highway are of intense interest to the highway engineer. This requires not only a thorough understanding of the soil and aggregate properties which affect pavement stability and durability, but also the binding materials which may be added to improve their engineering properties.

The first issue to be confronted in any pavement design is whether the project involves new construction, rehabilitation, or reconstruction. New construction is the construction of a pavement system on a new alignment that has not been previously constructed. Rehabilitation is defined as the repair and upgrading of an existing in-service pavement. Typically, this involves repair/removal and construction of additional bound pavement layers (asphalt concrete overlays) and could include partial-depth or full-depth recycling or reclamation. Reconstruction is defined as the complete removal of an existing pavement system, typically down to and including the upper portions of the foundation soil, and the replacement with a new

pavement structure. New construction has been the traditional focus of most pavement design procedures in many developing countries, although this focus with time (on major highways) will shift to rehabilitation and reconstruction. The roadwork on the Nairobi-Thika Road (A2) involves reconstruction.

1.2 Problem Statement

In the last 10 years, Kenya has witnessed construction of a number of major trunk roads. Construction of some of these roads is complete while others are on-going. The trunk roads have been constructed to very high standards and are expected to carry relatively high traffic loads during their design life which ranges from 15-20 years. The Nairobi-Thika highway is one such example whose construction is underway. In this project, the Ministry of Roads (MoR) settled for different pavement designs in terms of pavement layer thickness and material characteristics. In construction of these roads, is it possible to tell the ruling CBR during construction at the design stage? How have axle loads of heavy and medium goods vehicles changed from the 1970s, through the time our current manuals were developed to the present?

This has prompted the need to address the above issues by carrying out soil tests and also by analysing available axle load data for the Nairobi-Thika Road.

1.3 Study Objectives

The objectives of this study were:

i.   To establish the variation of engineering properties of soils with depth at deep cut sections and also to see how the problem of

3

varying subgrade bearing capacities of alignment soils has been taken care of.

ii.   To establish axle load data for the Nairobi-Thika Road and project what is likely to happen to the pavement that has been designed.

iii. To compare current axle loads of vehicle with the axle loads of the 1970s.

iv.   To document challenges, solutions, guidelines and recommendations that can be used in future projects based on Nairobi-Thika Highway Improvement Project experience.

The choice of Nairobi-Thika Road was based on its complexity in terms varying alignment soils capacities, big concrete structures and also it being a huge investment in terms of project cost.

1.4   Justification of the Study

Knowledge of alignment soils is a critical element as far as design of roads is concerned. Equally important is axle load data. In this study, information regarding the two elements along the study road was collected and analysed. The conclusions drawn from the study shall be used as reference in future road design projects.

1.5   Research Hypothesis

i.   How do engineering properties of soils vary with depth?

ii.   Is it possible to tell the ruling CBR during construction at the design stage?

iii. How have axle loads of heavy and medium goods vehicles changed from the 1970s, through the time our current manuals were developed to the present?

4

iv. What percentage of heavy and medium goods vehicles is overloaded? What is the effect of overloading on the newly constructed pavement?

v. What are some of the challenges encountered when a road project of such magnitude as Nairobi-Thika road is implemented?

By carrying out the study, it is expected that the above questions will be addressed.

1.6   Selection of Test Sites

The test sites are located at various locations along the Nairobi-Thika Road (A2). The choice of test sites was guided by the ease of establishing the variation of engineering properties with depth and therefore deep cut areas were selected. Test site location map is given as Figure 1.1.
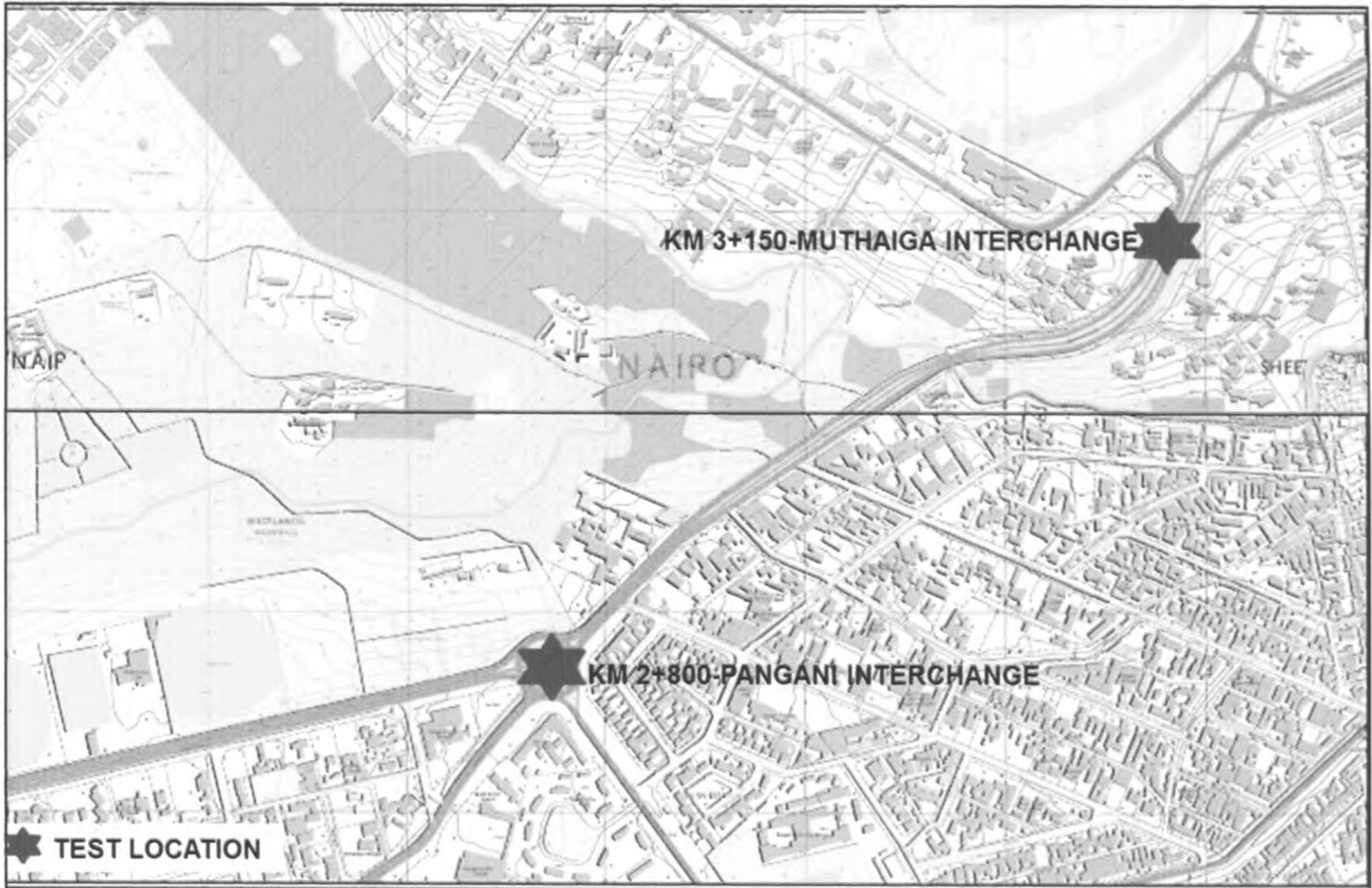
Figure 1-1: Map for Thika Road Test Sites (Source, Survey of Kenya)

1.7 Scope and Limitations

The scope of this research was limited to Nairobi-Thika Road Improvement Project whose alignment soils were tested, traffic data and factual materials report could be obtained for evaluation and analysis and design.

2 Literature Review

2.1 General

Roads originated as a primary means of communication and came into existence with the creation of mankind. Road building however, as a science was first started during the Roman Empire. Most of the Roman roads were of very high specifications using various sizes of broken stone and lime as a cementing material. The following people are credited with conducting early research that led to the development of the road designs that we know today:

i. Pierre Tresaguet

This was a French engineer who laid the foundation for the modern road construction. He introduced the practice of digging the ground to provide base course along with the concept of providing slope at subgrade stage to drain away water which got into the bottom of road formation through seepage. He also provided a camber for quick and efficient drainage for the top surface and tried to distribute the stresses transmitted to the foundation by spreading the stones over the entire foundation and ensuring that each foundation stone had at least one flat surface (O'Flaherty, 2002).

ii. Thomas Telford (1757-1834)

Telford perfected a method of broken stone which built upon but refined Tresaguet's. He used level foundations but raised the pavement above ground level to reduce drainage problems, shaped the foundation stones so that they fitted more closely together, and made the pavement as dense as possible to minimize moisture penetration. In his road

8

construction, in order to drain away water which percolated to the bottom, he provided cross drains at intervals of 30-50 metres (O'Flaherty, 2002).

iii.   John MacAdam

This was a Scottish road builder who influenced road construction so profoundly that the term "macadam" is frequently used in pavement specifications even to this day. His two important principles of good road construction were:

- It is the native soil that supports the traffic load ultimately, and when the soil is maintained in the dry state, it can carry heavy load without settlement.
- Stones which are broken to smaller angular pieces and compacted can interlock with each other and form a hard surface.

In Macadam type construction, formation at the dug up stage was kept cambered thereby providing normal drainage of the percolated water to the side. Since then, a lot of research has been done in road pavement design resulting in improved understanding of the practical and theoretical aspects of road pavement design.

The objective of this chapter is to review some of the notable contributions towards the characterization of subgrade materials, their improvement, construction and its overall effect on the determination of the pavement layer thicknesses.

2.2   Work done in Kenya

2.2.1 Structural Behaviour of Flexible Pavements

Long term studies on flexible pavements in Kenya by Gichaga (1979) suggest an increase in pavements strength (assessed on the basis of

deflections) with pavement age and a decrease in strength with cumulative traffic loading. Higher pavement deflections during periods of high rainfall and temperatures are also reported. The author recommends the regular monitoring of factors affecting performance in order to facilitate proper financial planning for pavement strengthening and routine maintenance (Gichaga, 1979).

In 1982, Gichaga published a research paper on laboratory study of deformation modulus-time relationship for various subgrade soils under road pavement structure. The object of this study was to evaluate the change of modulus of deformation with time for conditions under a typical road pavement. The three typical soils in Kenya were tested under almost fully saturated conditions and they included murram, red coffee soil and black cotton soil. Elastic theory was assumed in determining the stresses to be used during the experimentation. The behaviour of these soils under constant stresses showed creep behaviour.

2.2.2 Deflection Characteristics for Flexible Road and Airport Pavements in Kenya

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

Gichaga (1979), Murunga (1983) and Atibu (1986) established eight test sites on high volume roads for long term monitoring and evaluations. Two test sites along low volume road along (Gatura-Mataara Road) were later added to the testing program (Mwea 2001). The historical sites, namely ES1 through ES10 were visited with an exception of ES8, which was not visited during this research. Maintenance interventions during the study period were also recorded. Deflection variations with time were recorded and the information represented in graphs. Test sites ES9 and ES10 were sited on low volume roads.

The analysis of the results showed that "the equivalent modulus gave a good indication of pavement condition for high volume roads, while visual condition indices were found to be more appropriate for low volume roads" (Mwea and Gichaga, 2004). When the deflection profiles of low volume roads (ES9 and ES10) were compared to those of high volume roads, they appeared 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 low traffic on this road estimated at below 500 average annual daily traffic (AADT) had not damaged the pavement structure over the years of its service. The deflection criteria for pavement evaluation without accompanying conditional survey therefore appeared to be inadequate performance indicator for low volume roads.

2.2.3 Performance Study on Flexible Road Pavements in Kenya

In 1983, Murunga did a research on the performance of flexible road pavements in Kenya. The research set out to evaluate the performance of some six road test sections located on in-service bitumen standard roads in and around Nairobi, Kenya.

11

The study found out that pavement age, traffic and climate were some of the major factors affecting pavement performance. Evidence was found to suggest that for cracked pavement sections, rebound deflections provided reliable indications of pavement weaknesses. This relationship was however found not to hold in the case of rutting. One of the recommendations of this study was the need for further research on the compaction characteristics of subgrade soils in Kenya.

2.3 Work Done Outside Kenya

2.3.1 Pavement Design in the Tropics, Investigations of Subgrade Conditions under Roads in East Africa

An investigation was carried out in East Africa by the Road Research Laboratory of the United Kingdom, to determine the moisture conditions in the subgrade soils under sealed pavements in tropical countries.The investigation comprised the sampling of in-situ moisture conditions at seven sites covering a range of soil types, in Kenya, over a complete climatic cycle. This was followed by a comprehensive programme of tests in the laboratory on both disturbed and undisturbed samples from these sites. To check the general application of these findings to areas where the water-table was 10ft or more below the road surface, a further investigation covering 48 sites widely scattered throughout Kenya, Tanzania and Uganda was carried out.

The investigation indicated that where the water-table was at depths greater than 3 feet in sands to 20 feet in heavy clays below the road surface, the chief factors controlling the moisture conditions in the subgrade under bituminous surfaces were climate and soil permeability. Surface water infiltrating from the pavement edge will cause edge effects that are small in magnitude if reasonable engineering requirements of drainage design are applied. The moisture contents recorded showed that

12

in these circumstances moisture conditions at subgrade level are, in general, at or drier than the optimum moisture content given by the B.S. compaction test or the plastic limit. Low states of compaction were found at many sites. The attainment of higher states of compaction would reduce the permeability which, in a climate of alternating wet and dry seasons, would result in an increase in the equilibrium pore-water tension under the pavement.

2.3.2 Investigations on Flexible Pavements

In the year 1993, TRL carried out a research on the structural design of bitumen-surfaced roads in tropical and sub-tropical countries that led to the publication of Road Note 31, (Fourth Edition) 1993. This research led to the advancement of understanding of the behaviour of road building materials and their interaction in composite pavements and the development of knowledge that can be used with confidence in tropical and sub-tropical regions where conditions are often very different. In addition to differences associated with climate and types of materials, problems also arising from uncontrolled vehicle loading and unreliable road maintenance were also taken care of. At the same time, it was also argued that the level of technology available for construction and maintenance in these countries was relatively low (Road Note 31, 1993).

The pavement designs incorporated in the fourth edition of Road Note 31 are based primarily on:

i. The results of full-scale experiments where all factors affecting performance had been accurately measured and their variability quantified.

ii. Studies of the performance of as-built existing road networks.

Where direct empirical evidence was lacking, designs were interpolated or extrapolated from empirical studies using road performance models. In view of the statistical nature of pavement design caused by large uncertainties in traffic forecasting and the variability in material properties, climate and road behaviour, the design charts were presented as catalogues of structures, each surface being applicable over a small range of traffic and subgrade strength.

In this study, traffic was expressed in terms of the cumulative number of 8,200kg standard axles the pavement is expected to support over its design life, while subgrade strength was denoted by its CBR. The traffic loading and subgrade CBR are used to select appropriate layer thickness from design charts.

2.3.3 AASHTO

This is a design process recommended by the American Association of State Highway and Transport Officials (AASHTO) and is based on the results of extensive AASHO Road Test conducted in Ottawa ,Illinois, USA in the late 1950s and early 1960s (Huang, 2004). The AASHTO design guide is suitable for use in the USA. However, it has been widely used in tropical countries. Subgrade strength is defined in terms of the soil support value; while pavement thickness is expressed in terms of the structural number (SN) ranging from 1.0 to 6.0. Traffic loading is expressed in terms of cumulative standard axles during the design life of the pavement, or in terms of daily axle applications.

2.3.3.1 Design Variables

The following are the design variables when using the AASHTO method:

i. Time Constraints

To make the best use of the available funds, the AASHTO design guide encourages the use of a longer analysis period for high volume facilities, including at least one rehabilitation period. Thus, the analysis period should be equal to or greater than the performance period.

Performance Period

This refers to the time that an initial pavement structure will last before it needs rehabilitation or the performance time between rehabilitation operations. It is equivalent to the time elapsed as a new, reconstructed, or rehabilitated structure deteriorates from its initial serviceability to its terminal serviceability. The designer must select the performance period within the minimum and maximum allowable bounds that are established by agency experience and policy. The selection of performance period can be affected by such factors as the functional classification of the pavement, the type and level of maintenance applied, the funds available for initial construction, life cycle costs, and other engineering considerations.

Analysis Period

This is the period of time that any design strategy must cover. It may be identical to the selected performance period. However, realistic performance limitations may necessitate the considerations of staged construction or planned rehabilitation for the desired analysis period. In the past, pavements were typically designed and analysed for a 20-year performance period. It is now recommended that that consideration be given to longer analysis periods, because they can be better suited to the evaluation of alternative long-term strategies based on life cycle costs.

Table 2.1: Guidelines for Length of Analysis Period

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

Source: AASHTO (1986)

ii.   Traffic

The design procedures are based on cumulative expected 18-kip (80kN) equivalent standard axles.

AASHTO Equivalent Factors

The following regression equations based on results of road tests can be used for determining equivalence axle load factors (EALF):

$$log\left(\frac{W_{tx}}{W_{t18}}\right) = 4.79\,log(18+1) - 4.79\,log(log\,L_x + L_2)$$

$$+4.33\,log\,L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}} \qquad \text{(Equation 2.2)}$$

$$G_t = log\left(\frac{4.2 - P_t}{4.2 - 1.5}\right) \qquad \text{(Equation 2.3)}$$

$$\beta_x = 0.4 + \frac{0.08(L_x + L_2)^{3.23}}{(SN+1)^{5.19}\,L_2^{3.23}} \qquad \text{(Equation 2.3)}$$

In these equations, W_{tx} is the number of x-axle load applications at the end of time t; W_{t18} is the number of 18-kip (80kN) single-axle load applications to time *t*; L_x is the load in kip on one single axle, one set of tandem axles, or one set of tridem axles; L_2 is the axle code (1 for single axle, 2 for tandem axles, and 3 for tridem axles); SN is the structural number, which is a function of the thickness and modulus of each layer and the drainage conditions of base and subbase; p_t is the terminal serviceability ,which indicates the pavement conditions to be considered

16

as failures; G_t is a function of P_t; and β_{18} is the value of β_x when L_x is equal to 18 and L_2 is equal to one (Huang, Y.H.).

Note that:

$$EALF = \frac{W_{18}}{W_{tx}} \quad \text{(Equation 2.4)}$$

The effect of p_t and SN on EALF is erratic and is not completely consistent with theory. However, under heavy axle loads with an equivalent factor greater than unity, the EALF increases as p_t or SN decreases. This is as expected, because heavy axle loads are more destructive to poor and weaker pavements than to good and stronger ones. A disadvantage of using the above equations is that the EALF varies with the structural number, which is a function of layer thickness (Huang, Y.H.).

Theoretical Analysis

In the mechanistic method, the EALF can be determined from the failure criteria. Deacon (1969) conducted a theoretical analysis of EALF by layered theory based on an assumed f_2 of 4,

$$EALF = \frac{W_{18}}{W_{tx}} = \left(\frac{\epsilon_x}{\epsilon_{18}}\right)^4 \quad \text{(Equation 2.5)}$$

In which ϵ_x is the tensile strain at the bottom of asphalt layer due to an x-axle load and ϵ_{18} is the tensile strain at the bottom of asphalt layer due to an 18-kip (80kN) axle load. If W_{tx} is also a single axle, it is reasonable to assume that tensile strains are directly proportional to axle loads, or

$$EALF = \left(\frac{L_x}{18}\right)^4 \quad \text{(Equation 2.6)}$$

In which L_x is load in kip on a single axle Equation 2.6 is valid only when L_x is on a single axle. For tandem or tridem axles, a more general equation is

17

EALF= $\left(\frac{L_x}{L_s}\right)^4$ (Equation 2.7)

In which L_s is the load in kip on standard axles which have the same number of axles as L_x. If the EALF for one set of tandem or tridem axles is known, that for the other axles can be determined from Eq. 2.7.

If a pavement is designed for the analysis period without any rehabilitation or resurfacing, all that is required is the total ESAL over the analysis period. However, if stage construction is considered and rehabilitation or resurfacing is anticipated a graph or equation of cumulative ESAL versus time is needed so that the ESAL traffic during any given stages can be obtained.

iii. Reliability

There are two methods of pavement design: deterministic and probabilistic. In the deterministic method, each design factor has a fixed value based on the factor of safety assigned by the designer. Using judgement, the designer usually assigns a higher factor of safety to those factors that are less certain or that have a greater effect on the final design. Application of this traditional approach based on the safety factors applied and the sensitivity of the design procedures. A more realistic approach is the probabilistic method, in which each design factor is assigned a mean and variance. The factor of safety assigned to each design factor and its sensitivity to the final design are automatically taken care of, and the reliability of the design can be evaluated.

Reliability is defined as the probability that the design will perform its intended function over its design life. Reliability is a means of incorporating some degree of certainty into the design process to ensure that the various design alternatives will last the analysis period. The level

of reliability to be used for design should increase as the volume of traffic, difficulty of diverting traffic, and public expectation of availability increase.

Traffic, or the number of load repetitions, is one of the most important factors in pavement design. There are two types of load repetitions: the predicted number of load repetitions n and the allowable number of load repetitions N. In the deterministic method, both n and N have fixed values, whereas in the probabilistic method, each has a mean and a variance. If the design is based on a single value of equivalent single-axle load (ESAL), n and N can be compared directly to evaluate the adequacy of the design. If the design is based on a variety of loads or on ESAL applications over two or more seasons, the concept of damage ratio is used. The damage ratio (D_r), is the ratio between the predicted and allowable number of repetitions, computed for each load group in each period and summed over the year.

The allowable ESAL during the design period is designated W_t and can be determined from design equations or computer programs. The AASHTO design equation can be used as an example (section 2.3.3.2, equations 2.11 and 2.12).

When the mean and variance of log W_T and log W_t are known, the reliability of the design can be determined. Reliability is the probability that log W_T - log W_t, <0, or

Reliability = Probability (log D_r = log W_T - log W_t, < 0)

In which $D_r = W_T/W_t$ is the damage factor. The Variance of log D_r can be obtained by

$$V [(\log D_r] = V [\log W_T] + V [\log W_t]$$

By assuming D_r to have a log normal distribution, the reliability of the design as indicated by the shaded area in Figure 2.1 can be determined.
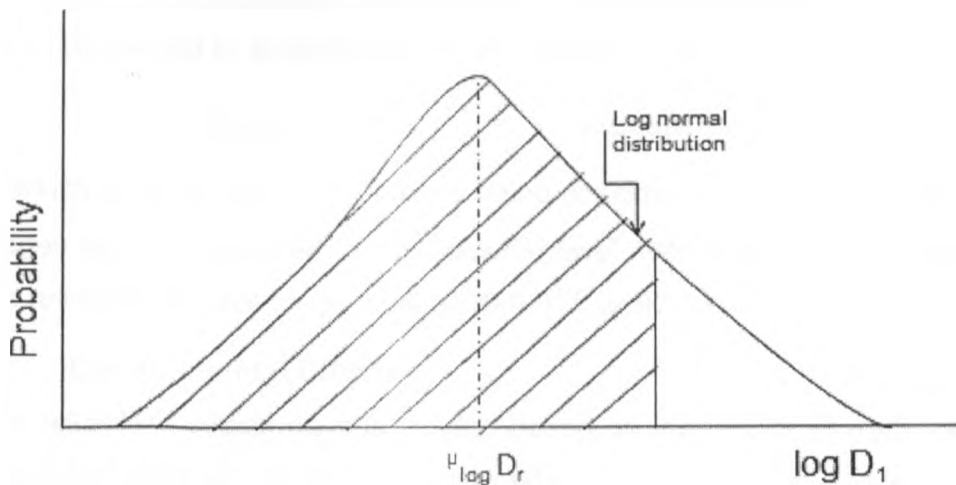
Figure 2-1:Reliability based on damage ratio
Source: AASHTO (1986)

Table 2.2 presents recommended levels of reliability for various functional classifications.

Table 2.2: Suggested Levels of Reliability for Various Functional Classifications

Functional Classification	Recommended level of reliability	
	Urban	Rural
Interstate and other freeways	85-99.9	80-99.9
Principal arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

Source: AASHTO (1986)

Application of the reliability concept requires the selection of a standard deviation that is representative of local conditions. It is suggested that

20

standard deviations of 0.49 be used for flexible pavements and 0.39 for rigid pavements. These correspond to variances of 0.2401 and 0.1521 (Huang, Y.H.).

When stage construction is considered, the reliability of each stage must be compounded to achieve the overall reliability; that is,

$$R_{Stage} = (R_{overall})^{\frac{1}{n}} \qquad \text{(Equation 2.8)}$$

in which n is the number of stages being considered. For example, if two stages are contemplated and the desired level overall of reliability is 95%, the reliability of each stage must be $(0.95)^{1/2}$, or 97.5%.

iv. Environmental Effects

The AASHTO design equations were based on the results of traffic tests over a two year period. The long-term effects of temperature and moisture content on the reduction of serviceability were not included. If problems of swell clay and frost heave are significant in a given region and have not been properly corrected, the loss of serviceability over the analysis period should be estimated and added to that due to cumulative traffic loads.

The serviceability loss due to roadbed swelling depends on the swell rate constant, potential vertical rise and the swell probability.

v. Serviceability

Initial and terminal serviceability indexes must be established to compute the change in serviceability to be used in design equations. The initial serviceability index is a function of pavement type and construction quality. Typical values from the AASHO Road Test were 4.2 for flexible pavements and 4.5 for rigid pavements. The terminal serviceability index is the lowest index that will be tolerated before rehabilitation, resurfacing and reconstruction become necessary. An index of 2.5 or higher is suggested for design of major highways and 2.0 for highways with lower traffic. For relatively minor highways where economics dictate a minimum

21

initial capital outlay, it is suggested that this can be accomplished by reducing the design period or total traffic volume, rather than designing a terminal serviceability index less than 2.0.

2.3.3.2 Design Equations

The original equations were based purely on the results of AASHO Road Test but were modified later by theory and experience to take care of subgrade and climatic conditions other than those encountered in the Road Test.

Original Equations

The following are the basic equations developed from the AASHO Road Test for flexible pavements (Huang, 2004):

$$G_t = \beta(logW_t - log\rho) \qquad \text{(Equation 2.9)}$$

$$\beta = 0.4 + \frac{0.081(L_1+L_2)^{3.23}}{(SN+1)^{5.19} L_2{}^{3.23}} \qquad \text{(Equation 2.10)}$$

$$\log\rho = 5.93 + 9.36 \log(SN + 1) - 4.79 \log(\log L_1 + L_2) +$$
$$4.33 \log L_2 \qquad \text{(Equation 2.11)}$$

Where,

G_t= logarithm of the ratio of loss in serviceability at time t to the potential loss taken   at a point where p_t = 1.5 or G_t = [(log 4.2 p_t)/ (4.2-1.5)], noting that 4.2 is the initial serviceability for flexible pavements;

β= a function of design and load variables, as shown by Eq. 2.9, that influences the shape of ρ versus W_t curve;

ρ= a function of design and load variables, as shown by Eq. 2.11 ,that denotes the expected number of load applications to a pt of 1.5 as can be seen from Eq. 2.9,where ρ=W_t when pt = 1.5;

W_t=axle load application at end of time t;

pt =serviceability at end of time t;

L_1=load on one single axle or a set of tandem axles, in kip;

L_2=axle code-1 for single axle, 2 for tandem axle;

SN= structural number of pavement, which was computed as

$$SN = a_1D_1 + a_2D_2 + a_3D_3 \quad \text{(Equation 2.12)}$$

in which a_1, a_2 and a_3 are layer coefficients for the surface, base and subbase, respectively and D_1 D_2 and D_3 are the thicknesses of the surface, base and subbase respectively.

The procedure is greatly simplified if an equivalent 18-kip (80-kN) single axle load is used. By combining Eqns. 2.9, 2.10 and 2.21 and setting L_1 = 18 and L_2 = 1,we obtain the equation

$$\log W_{t18} = 9.36 \log(SN + 1) - 0.20 + \frac{\log[\frac{(4.2-pt)}{(4.2-1.5)}]}{0.4+1094/(SN+1)^{5.19}}$$

(Equation 2.13)

in which W_{t18} is the number of 18-kip (80kN) single axle load applications to time t and p_t is the terminal serviceability index. Equation 2.12 is applicable only to the flexible pavements in the AASHO Road Test with an effective subgrade resilient modulus of 3000 psi (20.7Mpa) (Huang, Y.H.).

Modified Equations

For other subgrade and environmental conditions, Eq. 2.13 is modified to

$$\log W_{t18} = 9.36 \log(SN + 1) - 0.20 + \frac{\log[\frac{(4.2-pt)}{(4.2-1.5)}]}{0.4+1094/(SN+1)^{5.19}} +$$

$2.32 log M_R - 8.07$ Equation 2.14

in which M_R is the effective roadbed soil resilient modulus. It is to be noted that when M_R =3000psi (20.7Mpa), Eq.3.4 is identical to Eq.2.13.To take local precipitation and drainage conditions into account the, Eq. 2.12 was modified to

SN = $a_1D_1 + a_2D_2m_2 + a_3D_3m_3$ (Equation 2.15)

in which m_2 is the drainage coefficient of base course and m_3 is the drainage coefficient of subbase course.

Equation 2.14 is the performance equation that gives the allowable number of 18-kip (80kN) single axle load applications W_{t18} to cause the reduction of ΔPSI to pt. If the predicted number of applications W_{18} is equal to W_{t18}, the reliability of the design is only 50%, because all variables in Equation 2.14 are based on mean values. To achieve a higher level of reliability W_{18} must be smaller than W_{t18} by a normal deviate Z_R.

$$z_R = \frac{\log W_{18} - \log W_{t18}}{S_0} \qquad \text{(Equation 2.16)}$$

Here, Z_R is the normal deviate for a given reliability R and S_O is the standard deviation. Z_R can be determined from Table 2.3. Combining equations 2.14 and 2.16 and replacing (4.2-pt) by ΔPSI yields

$$\log W_{18} = z_R S_0 + 9.36 \log(SN + 1) - 0.20 + \frac{\log\left[\frac{\Delta PSI}{(4.2-1.5)}\right]}{0.4 + \frac{1094}{(SN+1)^{5.19}}} +$$

$2.32 \log M_R - 8.07$ (Equation 2.17)

Equation 2.17 is the final design equation for flexible pavements. Figure 2.1 is a nomograph for solving Equation 2.17. A procedure for using the design chart for flexible pavements is given under section 2.3.3.5- selection of layer thickness.

Table 2.3: Standard Normal Deviates for Various Levels of Reliability

Reliability (%)	Standard Normal Deviate (Z_R)	Reliability (%)	Standard Normal Deviate (Z_R)
50	0.000	93	-1.476
60	-0.253	94	-1.555
70	-0.524	95	-1.645
75	-0.674	96	-1.751
80	-0.841	97	-1.881
85	-1.037	98	-2.054
90	-1.282	99	-2.327
91	-1.340	99.9	-3.090
92	-1.405	99.99	-3.750

Source: AASHTO, 1986
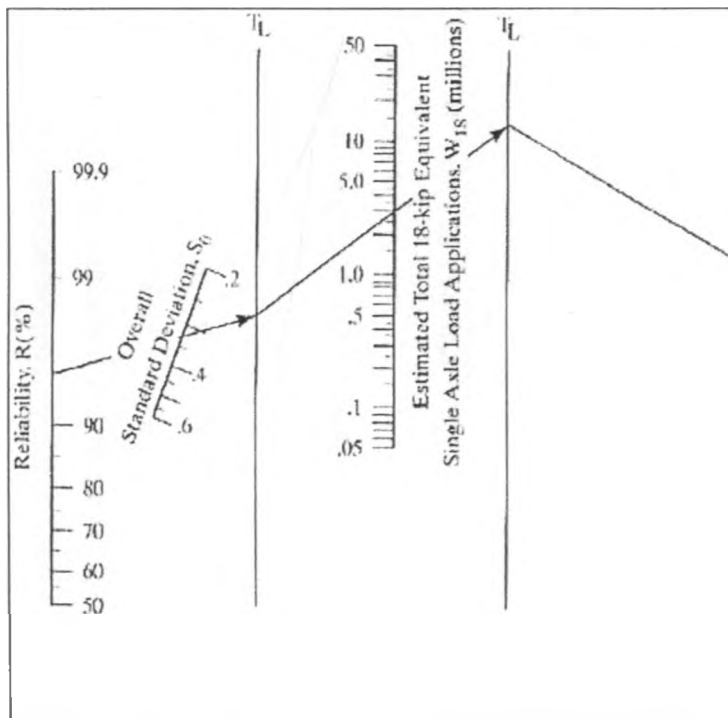
Figure 2-2: Design Chart for Flexible Pavements
(Source:AASHTO,1986)

26
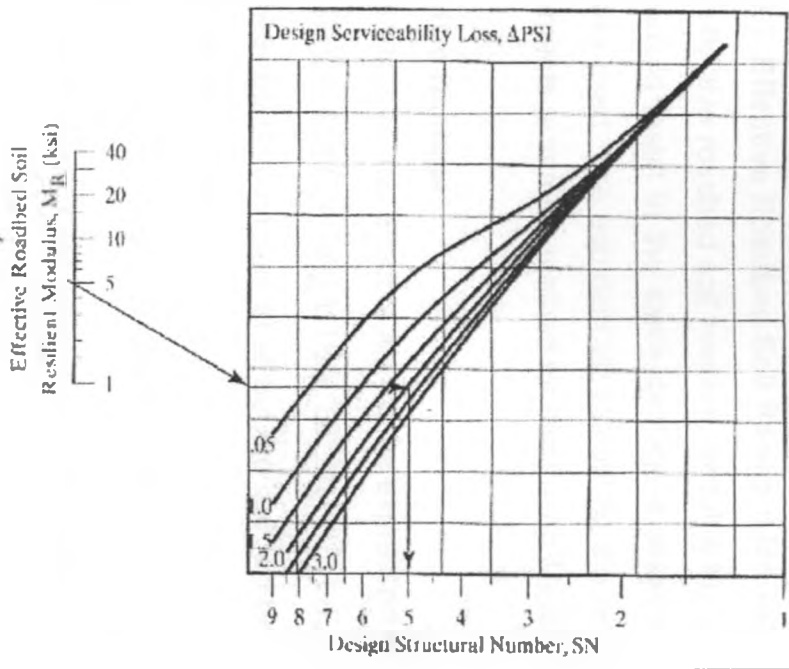
2.3.3.3 Effective Roadbed Soil Resilient Modulus

The effective roadbed soil resilient modulus M_R is an equivalent modulus that would result in the same damage if seasonal modulus values were actually used. The equation for evaluating the relative damage to flexible pavements u_f and the method for computing M_R is discussed below.

Relative Damage

From Eq. 2.17, the effect of M_R can be expressed as

$$log\ W_{18} = \log C - \log(1.18\ X\ 10^8 M_R^{-2.32}) \quad \text{Equation 2.18}$$

in which log C is the sum of all but the last two terms in Eq. 2.17. Equation 2.18 can be written as

$$W_{18} = \frac{C}{1.18\ X\ 10^8 M_R^{-2.32}} \quad \text{Equation 2.19}$$

If W_T is the predicted total traffic, the damage ratio, which is a ratio between predicted and allowable number of load repetitions, can be expressed as

$$D_R = \frac{W_T}{C/(1.18\ X\ 10^8 M_R^{-2.32})} = \frac{W_T}{C}(1.18\ X\ 10^8 M_R^{-2.32} \quad \text{Equation 2.20}$$

If W_T is uniformly distributed over n periods, the cumulative damage ratio is

$$D_R = \sum_{i=1}^{n} \frac{W_T/n}{C/(1.18\ X\ 10^8 M_{Ri}^{-2.32})} \qquad = \frac{W_T}{C}\frac{1}{n}\sum_{i=1}^{n}(1.18\ X\ 10^8 M_{Ri}^{-2.32}$$

Equation 2.21

Equating Eq.2.20 to Eq.2.21 gives

$$1.18\ X\ 10^8 M_R^{-2.32} = \frac{1}{n}\sum_{i=1}^{n}(1.18\ X\ 10^8 M_{Ri}^{-2.32} \quad \text{Equation 2.22}$$

Equation 2.22 can be used to determine the effective roadbed soil resilient modulus M_R in terms of a seasonal moduli M_{Ri}. Although the coefficient 1.18×10^8 can be cancelled out to simplify the equation, AASHTO design guide keeps the coefficient and defines the relative damage u_f as;

27

$$u_f = 1.18 \text{ X } 10^8 M_{Ri}^{-2.32} \quad \text{Equation 2.23}$$

Computation of Effective Roadbed Soil Resilient Modulus

Figure 2.3 is a worksheet for estimating effective roadbed soil resilient modulus, in which Eq. 2.23, together with a vertical scale for graphical solution of u_f, is also shown. A year is divided into a number of periods during which different roadbed soil resilient moduli are specified. The shortest period is half a month. The seasonal moduli can be determined from correlations with soil moisture and temperature conditions or from non-destructive deflection testing.

In the worksheet, 12 monthly subgrade moduli and a normal modulus of 4,500 psi (31MPa), are used as an example. The relative damage during each month can be obtained from the vertical scale or computed from Eq. 2.23; the sum, 25.30 is shown at the bottom. The average relative damage = 25.30/12 = 2.11, which corresponds to an effective roadbed resilient modulus of 2200 psi (15.2MPa).

In the preceding example, there is a variation in the monthly resilient modulus. The maximum and minimum values are outside the range of the vertical scale and must be computed from Eq. 2.23. About 65% of the damage is done in May alone. This is the reason that a very low effective modulus, 2,200 psi (15.2MPa), is obtained, one much lower than the normal modulus, 4,500 psi (31.1 MPa).

Month	Roadbed Soil Modulus M_R (psi)	Relative Damage, u_f
Jan.	15,900	0.02
Feb.	27,300	0.01
Mar.	38,700	0.00
Apr.	50,000	0.00
May	900	16.52
June	1,620	4.22
July	2,340	1.80
Aug	3,060	0.97
Sept.	3,780	0.59
Oct.	4,500	0.39
Nov.	4,500	0.39
Dec.	4,500	0.39
Summation $\Sigma u_f =$		25.30

Average: $\bar{u}_f = \dfrac{\Sigma u_f}{n} = \underline{2.11}$

Effective Roadbed Soil Resilient Modulus, M_R (psi) = $\underline{2,200}$ (corresponds to $\bar{u}_f$)
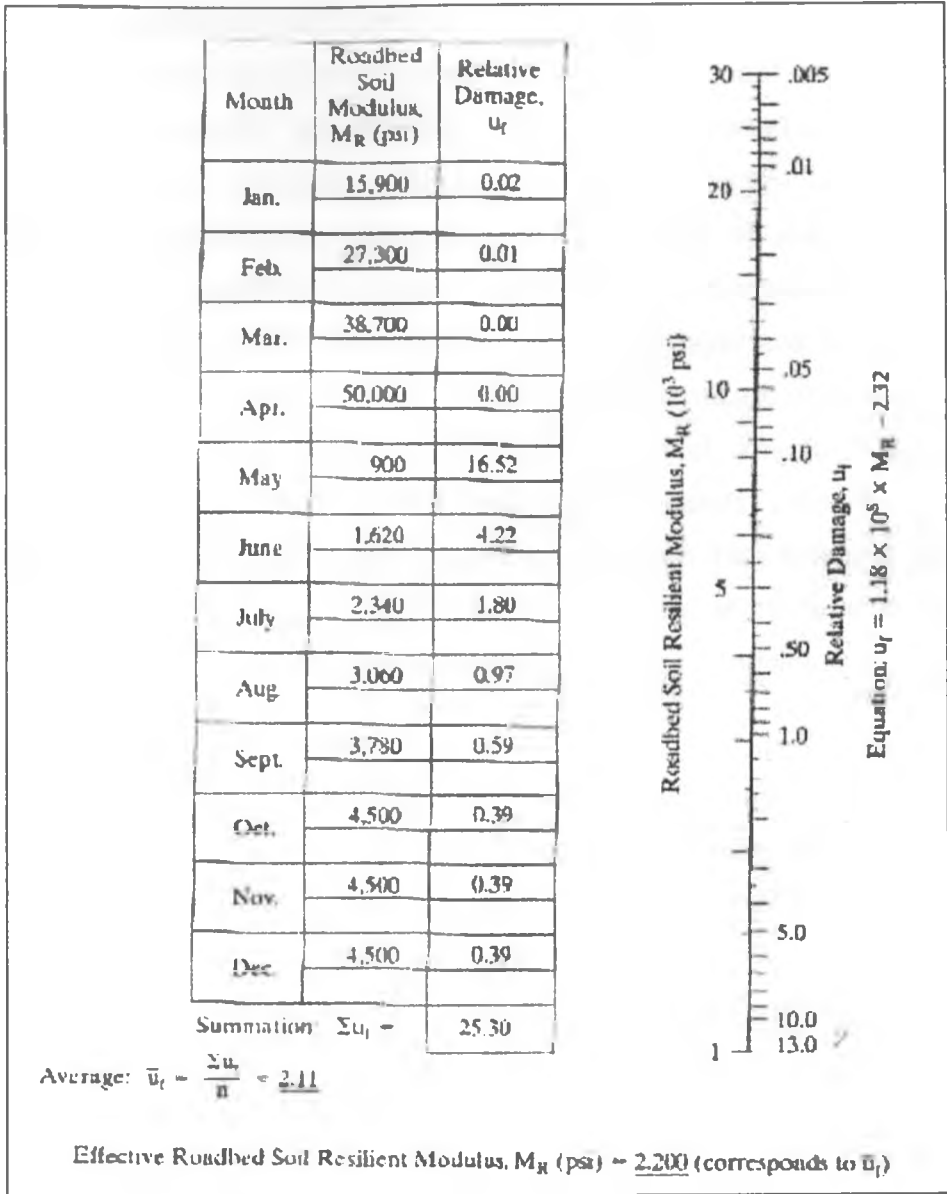
Figure 2-3: Worksheet for estimating effective roadbed soil resilient modulus

(1 psi=6.9kpa)

(Source:Huang,2004)

The structural number is a function of layer thickness, layer coefficients and drainage coefficients and can be computed from Equation 2.15.

29

Layer coefficient a_i is a measure of the relative ability of a unit thickness of a given material to function as a structural component of the pavement. It is recommended that the layer coefficient be based on the resilient modulus, which is a more fundamental material property.

Drainage Coefficient: Depending on the quality of drainage and the availability of moisture, drainage coefficients m_2 and m_3 should be applied to granular bases and subbases to modify the layer coefficients, At the AASHTO Road Test site, these drainage coefficients are all equal to 1.The quality of drainage is measured by the length of time for water to be removed from bases and subbases and depends primarily on their permeability. The percentage of time during which the pavement structure is exposed to moisture levels approaching saturation depends on the average yearly rainfall and the prevailing drainage conditions.

2.3.3.4 Selection of Layer Thickness

Once the design structural number SN for an initial pavement structure is determined, it is necessary to select a set of thickness so that the provided SN, as computed from Equation 2.15, will not be greater than the required SN. It is to be noted that Equation 2.15 does not have single unique solution. Many combinations of layer thicknesses are acceptable, so their cost effectiveness along with the construction and maintenance constraints must be considered to avoid the possibility of producing an impractical design.

Minimum Thickness

It is generally impractical and uneconomical to use layers of material that are less than some minimum thickness. Furthermore, traffic considerations may dictate the use of a certain minimum thickness for stability. Table 2.4 shows the minimum thickness of asphalt surface and

30

aggregate base. Because such minimums depend somewhat on local practices and conditions, they may be changed if needed.

Table 2.4: Minimum Thickness of Asphalt Surface and Aggregate Base

Traffic (ESAL)	Asphalt	Aggregate
Less than 50,000	1.0	4
50,001 - 150,000	2.0	4
150,001- 500,000	2.5	4
500,001 - 2,000,000	3.0	6
2,000,001 – 7,000,000	3.5	6
Greater than 7,000,000	4.0	6

Note: Minimum thickness is in inches; 1 inch = 25.4 mm

Source: AASHTO, 1986

General Procedure for Thickness Design

The procedure of thickness design is usually started from the top as shown in Figure 2.2 and described as follows:

i. Using E_2 as M_R, determine from Figure 2.4 the structural number SN_1 required to protect the base, and compute the thickness of layer 1 from

$$D_1 \geq \frac{SN_1}{a_1} \qquad \text{(Equation 2.18)}$$
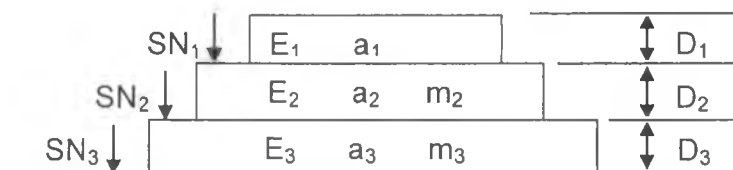
Figure 2-4: Selection of thicknesses

ii. Using E_3 as M_R, determine form figure 2.1 the structural number SN_2 required to protect the subbase, and compute the thickness of layer 2 from

$$D_2 \geq \frac{SN_2 - a_1 D_1}{a_2 m_2} \qquad \text{(Equation 2.19)}$$

31

iii. Based on the roadbed resilient modulus M_R, determine from Figure 2.1 the total structural number SM_3 required, and compute the thickness of layer 3 from;

$$D_3 \geq \frac{SN_3 - a_1 D_1 - a_2 D_2 m_2}{a_2 m_2} \qquad \text{(Equation 2.20)}$$

Example 2.1: Sample Calculation

Figure 2.5 is a pavement system with the resilient moduli, layer coefficients, and drainage coefficients as shown. If predicted ESAL = 18.6 x 10^6, R = 95%, S_0 = 0.35, and ΔPSI = 2.21, select thickness D_1, D_2 and D_3.

E_1 = 400.000 psi	a_1 = 0.42	
E_2 = 30,000 psi	a_2 = 0.14	m_2 = 1.2
E_3 = 11,000 psi	a_3 = 0.08	m_3 = 1.2

Figure 2-5: Sample Calculation

Solution

With $M_R = E_2$ = 30,000 psi (207 MPa), from figure 2.2, SN_1 = 3; from equation 2.18;

$D_1 \geq$ 3.2/0.42 = 7.6 in. (193 mm); use D_1 = 8 in. (203 mm).

With $M_R = E_3$ = 11,000 psi (76 MPa), from figure 2.2, SN_2 = 4.5; from equation 2.19, $D_2 \geq$ (4.5 - 0.42 x 8)/ (0.14 x 1.2) = 6.8 in. (173 mm); use D_2 = 7 in. (178 mm) to meet the minimum thickness shown in table 2.4.

With M_R = 5,700 psi (39.3 MPa), from figure 2.2, SN_3 = 5.6; from equation 2.20, $D_3 \geq$ (5.6 − 0.42 x 8 − 0.14 x 7 x 1.20) / (0.08 x 1.2) = 11.1 in. (282 mm); use D_3 =11.5 in. (292 mm).

2.4   Nairobi-Thika Road

Nairobi – Thika Road is part of international trunk road connecting Nairobi City with Ethiopia to the north and is located in Nairobi and Central Provinces of Kenya. It starts in Nairobi on Uhuru Highway at three points namely Haile Selassie Avenue, University Way and Museum Hill Roundabout and converges at Pangani Roundabout on Thika Road. It then proceeds to Thika via Muthaiga, GSU, Kasarani, Githurai Roundabouts, Kenyatta University, Ruiru Town, Juja Town and ends at the bridge near Blue Post Hotel. The total project length is 50.4 km.

The traffic flow along Nairobi – Thika Road has been marred by traffic jams, hence the need for expansion of the road. The road is being improved from the current 4 (four) lanes up to 8 (eight) lanes including provision of service roads, cycle tracks and footpaths. The overall objectives of the project are to:

- Contribute to the improved performance of the economic sectors and the delivery of social services in Kenya and the neighbouring regional partner's states such as Ethiopia and therefore improving the transport network.
- Improve land transport communication between Kenya and Ethiopia thereby contributes to regional integration (CES/ APEC, 2007).

From the Design, the traffic capacity of the Nairobi – Thika will be increased by expanding the roads as follows;

i.   University Way Roundabout- Muthaiga.

This section starts at University Way Roundabout and ends at Muthaiga Roundabout. It includes the construction of dispersal roads such as Forest Road and Kariakor Road. This section is approximately 12 km and most of it falls within the City of Nairobi. Expansion details involve:

- Widening of Kariakor Road to standard 4 lanes with median and footpath.
- Improvements of junctions en route which include Juja Road roundabout Ring Road Ngara Road/Racecourse Road Roundabout.
- Interchange at Museum Hill roundabout on Uhuru Highway, flyover at Museum, flyover at Limuru Road Junction, 1 underpass at Pangani and 1 flyover at Muthaiga
- Flyover Globe Cinema roundabout to Pangani
- University Way-Globe Cinema-Pangani
- 4+4 (4 No. lanes both directions on the main carriageway) at-grade road

ii.   Muthaiga -Kenyatta University.

Expansion details involve:

- Widening of lanes Muthaiga-Kasarani to 4+4 and Kasarani-Kenyatta University to 3+3 lanes.
- Parallel 2 lane service roads for full length
- Cycle track and footpath
- Fly overs, 2 No. (Kasarani, Githurai) Ruiru Bypass, Overpass-3 No., GSU, Nakumatt, Underpass-2 No. (Muthaiga, Survey and Kahawa).
- Railway Bridges-1 No.
- River Bridges-2 No.
- Pedestrian grade separations (FOBs)-4 No.

iii.   Kenyatta University-Thika.

Expansion details involve:

- Widening of project roads to 3+3 lanes from Kenyatta University-Juja and standard 2+2 lanes with paved shoulders from Juja to Thika.
- Parallel service roads (discontinuous at some locations)
- Cycle track and footpath
- Junction improvements.
- Fly overs-1 Ruiru Bypass, Overpass-3No., Gatundu, Mangu and Kimbo.
- Railway Bridges-3.
- River Bridges-7.
- Pedestrian grade separations(FOBs)-3

PROJECT: NAIROBI – THIKA ROAD (A2)
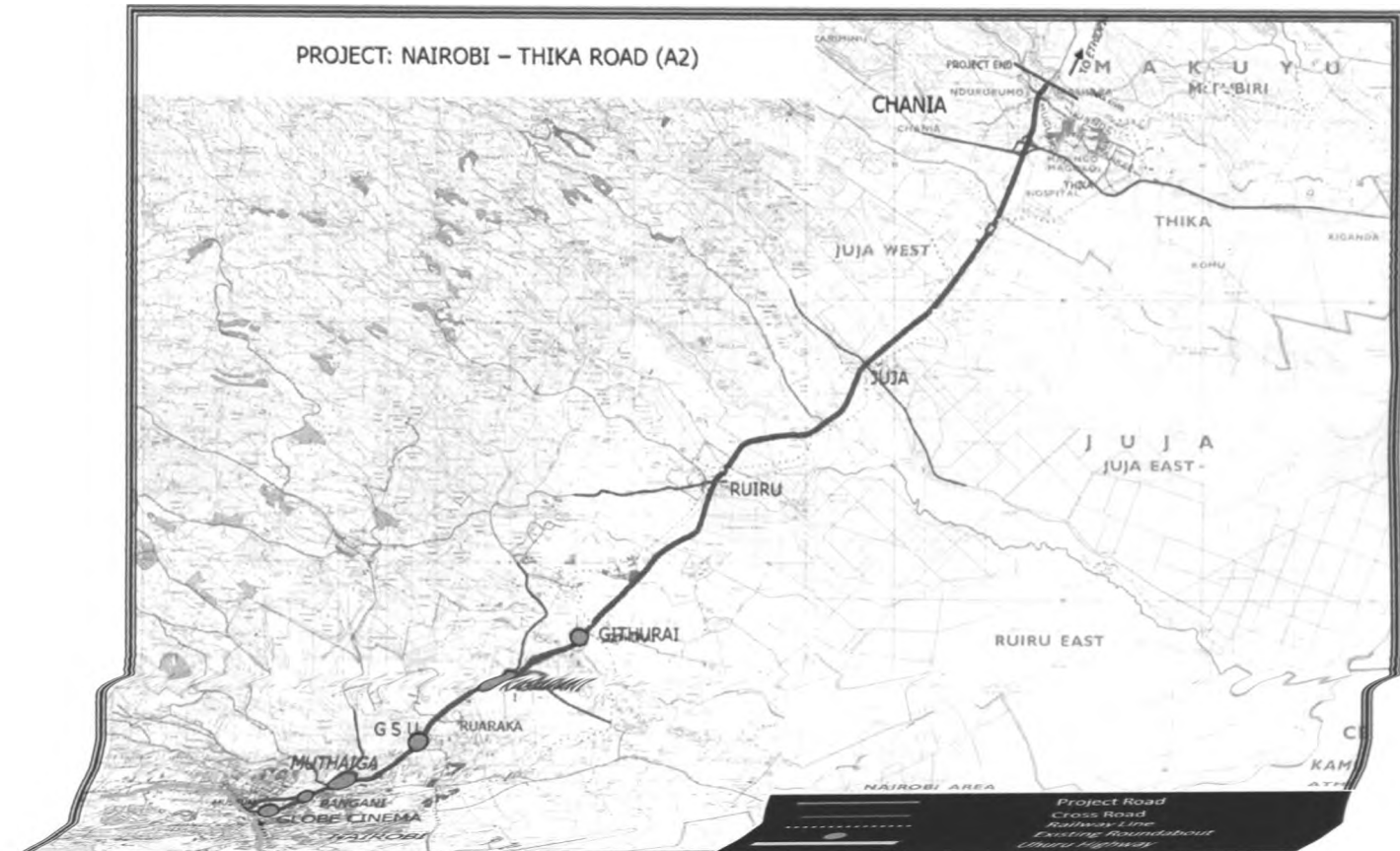
Figure 2-6: Nairobi-Thika Road Project Map
(Source: CES/APEC. 2007)

36
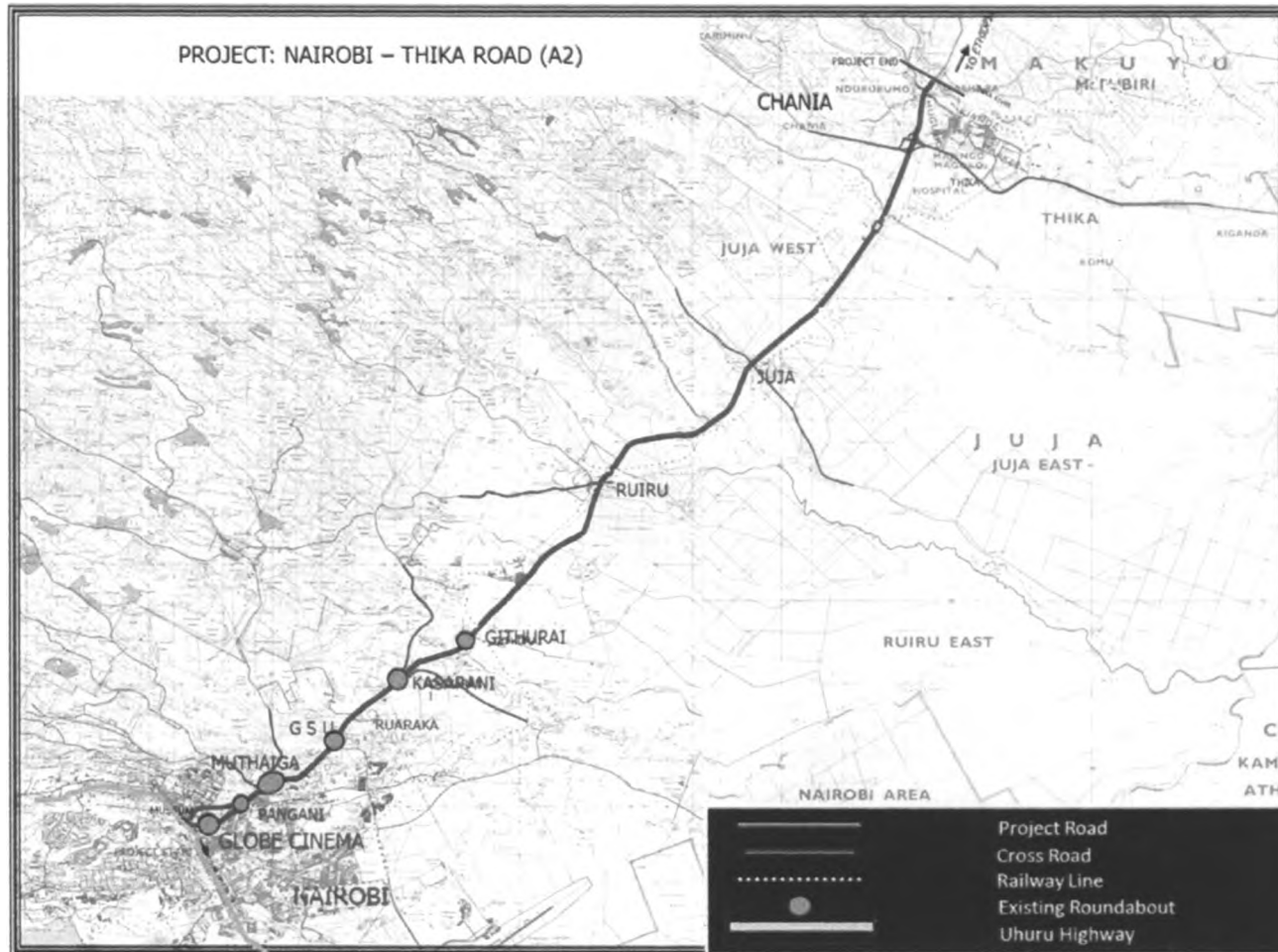
Figure 2-6: Nairobi-Thika Road Project Map

(Source: CES/APEC, 2007)

2.4.1 Traffic Survey, Analysis and Forecast

Traffic forecasts for the above projects were conducted for year 10 and 20 under the following two options:

   i.   Without Bypasses

   ii.   With Bypasses

The final traffic survey, analysis and forecast obtained are summarized below:

Table 2.5: Option 1: Without Bypass

Section	Direction	AADT in No. of Vehicles		AADT in PCU		CSA Loading mill ESA		Traffic Class for Y20
		Y10 (2020)	Y20 (2030)	Y10 (2020)	Y20 (2030)	Y10 (2020)	Y20 (2030)	
University Way	IN	36,544	49,369	42,581	56,730	11.40	26.13	T1
	OUT	32,947	44,782	39,009	52,224	12.95	30.08	T1
Globe	IN	59,448	77,705	83,046	106,340	62.72	141.96	T1
Jamhuri	OUT	30,839	40,709	41,432	53,602	20.72	47.01	T1
Jamhuri	IN	31,529	41,753	43,151	55,930	36.08	81.80	T1
Parkroad	OUT	27,347	36,182	34,864	45,335	11.45	26.19	T1
Parkroad	IN	19,960	25,911	25,757	32,895	4.61	10.53	T2
Pangani	OUT	30,078	38,361	45,603	57,547	37.20	87.24	Over T1
Pangani	IN	53,865	71,387	73,492	95,852	60.31	140.23	Over T1
Muthaiga	OUT	50,751	67,377	67,417	88,030	42.74	99.69	Over T1
Muthaiga	IN	53,676	71,272	72,816	95,159	59.52	138.41	Over T1
GSU	OUT	47,383	62,990	63,184	82,648	42.72	99.99	Over T1
GSU - Ruaraka	IN	45,927	61,035	68,640	90,008	89.36	210.25	Over T1
	OUT	44,344	59,070	63,824	83,899	77.91	184.22	Over T1
Ruaraka	IN	45,164	60,024	67,504	88,524	87.91	206.85	Over T1
USIU	OUT	39,529	52,652	56,922	74,819	69.53	164.47	Over T1
USIU	IN	46,139	60,921	74,173	96,475	113.56	264.57	Over T1
Kasarani	OUT	40,267	53,367	56,356	73,488	50.95	119.36	Over T1
Kasarani	IN	47,993	63,503	69,710	91,246	79.25	187.59	Over T1
Githurai	OUT	34,369	45,434	52,790	68,930	69.08	163.23	Over T1
Githurai	IN	22,329	29,516	42,053	55,075	90.96	214.36	Over T1
Kahawa	OUT	25,288	33,388	40,095	52,258	62.48	147.44	Over T1
Kahawa - Ruiru	IN	26,590	35,087	40,681	53,234	55.53	131.94	Over T1
	OUT	28,666	37,724	43,528	56,800	54.38	129.40	Over T1
Ruiru - Juja	IN	20,327	26,950	36,068	47,432	72.38	171.74	Over T1
	OUT	22,583	29,909	36,266	47,676	64.89	154.75	Over T1
Juja - Mangu	IN	21,538	28,642	37,822	49,873	74.57	177.01	Over T1
	OUT	26,192	33,905	41,312	53,246	59.89	142.82	Over T1
Mangu - Thika	IN	16,359	21,326	28,430	36,902	50.35	119.40	Over T1
	OUT	18,124	23,762	29,522	38,503	52.12	124.25	Over T1

Source:CES/APEC, 2007

38

Table 2.6: Option 2: With Bypass

Section	Direction	AADT in No. of Vehicles		AADT in PCU		CSA Loading - mill ESA		Traffic Class for Y20
		Y10 (2020)	Y20 (2030)	Y10 (2020)	Y20 (2030)	Y10 (2020)	Y20 (2030)	
University Way	IN	33,377	45,045	38,768	51,565	9.09	20.57	T1
	OUT	30,095	40,867	35,200	47,063	7.48	16.94	TI
Globe - Jamhuri	IN	55,012	71,772	76,887	98,235	56.27	126.92	Over T1
	OUT	28,367	37,381	38,068	49,138	17.80	40.20	TI
Jamhuri - Parkroad	IN	28,950	38,277	39,524	51,134	31.64	71.61	Over T1
	OUT	25,112	33,166	31,806	41,260	8.67	19.65	TI
Parkroad - Pangani	IN	18,482	23,947	23,702	30,196	2.69	5.99	TI
	OUT	27,550	35,036	40,111	50,322	19.86	45.91	TI
Pangani - Muthaiga	IN	48,391	64,010	62,696	81,426	32.36	73.68	Over T1
	OUT	45,906	60,826	58,656	76,296	20.02	45.36	TI
Muthaiga - GSU	IN	48,204	63,882	62,173	80,904	32.50	74.02	Over T1
	OUT	42,742	56,706	54,557	71,052	19.52	44.25	TI
GSU - Ruaraka	IN	40,346	53,508	55,089	71,842	45.18	104.66	Over T1
	OUT	39,245	52,183	51,877	67,875	33.93	79.17	TI
Ruaraka - USIU	IN	39,676	52,619	54,174	70,651	44.43	102.94	Over T1
	OUT	34,975	46,506	46,230	60,485	30.16	70.37	Over T1
USIU - Kasarani	IN	40,123	52,831	59,947	77,459	69.87	160.40	Over T1
	OUT	36,289	47,994	48,501	62,962	27.09	62.33	Over T1
Kasarani - Githurai	IN	42,264	55,792	56,028	72,914	35.40	82.66	Over T1
	OUT	30,159	39,768	42,305	54,884	32.48	75.76	Over T1
Githurai - Kahawa	IN	18,258	24,045	29,815	38,667	46.13	107.15	Over T1
	OUT	22,107	29,114	31,790	41,150	28.52	66.31	Over T1
Kahawa -	IN	23,130	30,434	31,287	40,643	22.09	51.92	T1
	OUT	25,010	32,817	33,999	44,045	21.11	49.65	T1
Ruiru - Juja	IN	20,518	27,216	36,298	47,747	72.52	172.05	Over T1
	OUT	22,809	30,219	36,532	48,039	64.99	154.99	Over T1
Juja - Mangu	IN	21,755	28,945	38,096	50,258	74.76	177.54	Over T1
	OUT	26,416	34,202	41,584	53,602	59.98	143.02	Over T1
Mangu - Thika	IN	16,494	21,504	28,597	37,120	50.44	119.62	Over T1
	OUT	18,296	24,001	29,736	38,794	52.23	124.53	Over T1

Source:CES/APEC, 2007

From the two tables, it was clear that at design year 10, traffic loading expressed in CNSA, will have exceeded the upper limit stipulated for Traffic Class 1 in the current road design manual. It was therefore clear that a research is necessary that will provide guidelines/specifications on how such loading can be adequately incorporated in pavement designs. The current traffic classes in Kenya are shown in Table 2.7.

Table 2.7: Traffic Classes

Traffic Class	Cumulative No. of Std. Axles
T1	$25\text{-}60 \times 10^6$
T2	$10\text{-}25 \times 10^6$
T3	$3\text{-}10 \times 10^6$
T4	$1\text{-}3 \times 10^6$
T5	$0.25\text{-}1 \times 10^6$

Source: RDM, Part 3, 1987

Since these projects traverse through vast areas comprising different geological/subgrade soils, climate, traffic loading and maintenance interventions, they present a good case to be studied.

2.4.2 Engineering Surveys and Investigations

Field tests on the existing subgrade were conducted at all pit locations to assess in-situ CBR of various layers of pavement and subgrade using the Dynamic Cone Penetration Test (DCP, TRRL Method).

The results of the field and laboratory testing on subgrade soils are given in Table 2.8.

Table 2.8: Summary of Subgrade Soil Laboratory Test Results

Chainage (Km)	Sample Side	NMC	Plastic limit	liquid limit	plasticity index	plasticity modulus	linear shrinkage	passing 420mic	passing 10mm	passing 5mm	passing 2mm	passing 75mc	OMC	MDD	CBR (4 days soak)	Swell	Specific gravity
		%	%	%	%		%	%	%	%	%	%	%	Kg/m3	%	%	%
0+100	RHS	38	28.7	54	25.3	835	12.9	33	69	54	43	32	22	1574	13	0.8	
1+100	LHS	36	15.9	34	18.1	507	9.3	28	98	96	64	16	17	1674	32	0.3	
2+100	RHS	31.8	15.1	34	18.9	586	9.3	31	99	97	86	22	19.	1645	30	0.3	
3+100	LHS	28.9	30.1	51.5	21.4	835	10.7	39	94	78	57	36	21.	1611	29	0.3	
4+000	RHS	36.1	35.1	50	14.9	448	7.1	32	94	71	46	27	18.	1786	45	0.2	
5+000	LHS	28.1	32.6	50.2	17.6	581	7.9	33	98	85	56	26	19.	1682	33	0.3	
6+000	RHS	25.8	29.0	52.9	23.9	155	12.1	65	99	92	75	62	22.	1533	15	0.9	
7+100	LHS	67.5	27.4	55.8	28.4	232	14.3	82	98	94	87	80	24.	1252	9	0.9	
8+000	RHS	69.8	21.9	33.2	11.3	452	5.7	40	83	66	48	38	22.	1701	57	0.1	
9+000	LHS	58.7	35.1	58.2	23.1	131	11.4	57	95	83	67	55	23.	1532	18	0.5	
10+000	RHS	69.1	15.4	29.7	14.3	265	7.1	19	91	67	32	15	22.	1755	48	0.2	
11+000	LHS	35.4	17.9	36.3	18.5	666	9.3	36	96	82	54	31	24.	1607	19	0.5	
12+000	RHS	23.5	22.3	34.2	20.1	884	10	44	93	81	60	40	22.	1546	18	0.4	
13+000	LHS	21.4	20.1	46.5	26.4	502	12.9	19	93	77	45	13	21.	1465	20	0.4	
14+000	RHS	20.9	23.9	40.5	16.6	365	8.6	22	100	96	85	66	17.	1762	51	0.1	
15+000	LHS	24.3	21.0	47.1	26.1	120	12.9	46	97	83	64	40	18.	1467	13.0	0.8	
16+000	RHS	23.9	23.9	47.7	28.8	121	12.1	51	99	91	70	44	24.	1550	21	0.5	
17+000	LHS	22.1	28.8	44.7	15.9	419	7.9	53	97	93	79	45	16.	1628	23	0.4	

Chainage (Km)	Sample Side	NMC	ATTERBERG PARAMETERS					GRADING					COMPACTION		CBR (4 days soak)	Swell	Specific gravity
			Plastic limit	liquid limit	plasticity index	plasticity modulus	linear shrinkage	passing 420mic	passing 10mm	passing 5mm	passing 2mm	passing 75mc	OMC	MDD			
18+000	RHS	21	27.0	51.1	24.1	106	12.1	44	99	90	68	37	27	1498	17	0.4	
19+000	LHS	19	22.3	40.1	17.8	801	8.6	45	99	89	67	39	25	1586	34	0.3	
20+000	RHS	18.9	17.6	35.9	18.3	677	9.3	37	97	85	60	31	22	1644	29	0.5	
21+000	LHS	19.9	21.1	40.3	19.2	749	10.0	39	96	90	71	23	22	1647	25	0.5	
22+000	RHS	20.8	23.0	33.2	10.2	571	5.0	56	99	94	77	50	18	1836	59	0.1	
23+000	LHS	21.4	13.3	33.7	20.4	408	10.0	20	93	68	38	13	18.	1526	20	0.5	
24+000	RHS	21.4	26.7	39.0	12.3	664	6.0	54	97	91	74	48	18.	1789	48	0.1	
25+000	LHS	19.5	29.2	45.0	15.8	822	7.9	52	99	91	72	46	19.	1688	40	0.1	
26+000	RHS	20.2	28.0	46.9	18.9	737	9.0	39	98	80	54	34	21.	1483	16	0.4	
27+000	LHS	22.5	31.4	54.2	22.8	107	11.4	47	99	90	66	42	22.	1424	15	0.4	
28+000	RHS	15.6	33.8	42.4	8.6	335	4.3	39	96	85	59	35	17	1965	69	0.0	
29+000	LHS	18.4	31.3	46.0	15.0	315	7.0	21	94	70	34	18	12.	1637	33	0.3	
30+000	RHS	17.8	26.1	52.2	26.1	114	13.0	44.0	97	89	65	38	19.	1771	27	0.3	
31+000	LHS	14.5	20.7	44.9	24.2	799	12.1	33.0	99	87	54	29	16.	1719	38	0.3	
32+000	RHS	21.8	24.1	47.5	23.4	655	11.4	28.0	95	77	46	23	16.	1720	37	0.3	
33+000	LHS	20.3	24.3	53.3	29.0	113	14.3	39.0	91	75	55	35	18.	1460	17	0.4	
34+000	RHS	20.4	25.7	42.1	16.4	607	8.0	37.0	97	86	59	32	15.	1400	15	0.5	
35+000	LHS	20.8	30.2	41.5	11.3	497	6.0	44.0	99	89	61	37	17.	1726	37	0.3	
36+000	Inne	RH		11.3	31.7	35	20.4	10	714	96	91	81	23	19.9	15.6	31	0.4
37+000	Out	LHS															

Chainage (Km)	Sample Side	NMC	ATTERBERG PARAMETERS					GRADING					COMPACTION				
			Plastic limit	liquid limit	plasticity index	plasticity modulus	linear shrinkage	passing 420mic	passing 10mm	passing 5mm	passing 2mm	passing 75mc	OMC	MDD	CBR (4 days soak)	Swell	Specific gravity
38+000	Inne	RH	16.8	34.3	54.2	43	19.9	10	856	97	86	61	36	18.6	16.1	27	0.4
39+000	Out	LHS	22.1	22.7	40.5	43	17.8	8.6	765	96	81	56	39	17.2	16.7	33	0.3
40+000	Inne	RH	20.5	21.8	35.4	31	13.6	7.1	422	92	70	45	26	18.4	17.4	45	0.2
41+000	Out	LHS	18.6	19.4	37.8	33	18.4	9.3	607	94	78	49	27	15.9	16.6	30	0.4
41+800	Inne	RH	19.8	17.6	33.8	47	16.2	15.8	761	95	85	65	45	16.6	17.6	54	0.2

Source: CES/APEC, 2007

2.4.3 Pavement Design

The design of Nairobi –Thika road was based on the AASHTO guidelines which comprise of two parts:

i. Calculation of Structural Number requirement for different layers depending upon the type of material used for construction.
ii. Calculation of pavement layer thickness is based on interlayer compatibility.

Basic Design Equation AASHTO

The basic design equation used for flexible pavements in AASHTO guide is as under:

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN + 1) - 0.20$$
$$+ \frac{\log_{10}\left[\dfrac{\Delta PSI}{4.2 - 1.5}\right]}{0.40 + \dfrac{1094}{(SN + 1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

Where,

$W_{18} =$ predicted number of 18-kip equivalent single axle load applications,

$Z_R =$ standard normal deviate,

$S_o =$ combined standard error of the traffic prediction and performance prediction,

$\Delta PSI =$ difference between the initial design serviceability index, p_o, and the design terminal serviceability index, p_t, and

$M_R =$ resilient modulus (psi)

$SN =$ Structural Number

Z_R . The value of the normal deviate, Z_R, corresponding to 95 percent reliability was taken as −1.645 (AASHTO, 1993)

S_o - The range of S_o value suggested by AASHTO guide is identified as:

0.40 to 0.50 for flexible pavement. A value of 0.44 was considered as design input for the project.

44

The values of initial serviceability index, p_i and terminal serviceability index, p_t were assumed as under:

- The p_i value of 4.5 was adopted on the basis of the consideration that the implementation of the project would be done with strict adherence of quality control norms for materials and works.
- Being a heavily trafficked road, the value of the terminal serviceability index, p_t, i.e. the lowest acceptable level of serviceability before the resurfacing would be necessary was taken as 2.5.

Detailed process of thickness calculations is outlined in the following steps:

1. Calculate SN_3 (SN subgrade) based upon M_R value for subgrade soil

2. Calculate SN_2 (sub-base) based upon M_R value for sub-base

3. Calculate SN_1 (base) based upon M_R value for base course

4. Calculate thickness of bituminous material:

$$D_1 = \frac{SN_1}{a_1}$$

a_1 = Structural layer coefficient of bituminous concrete = 0.42

Provide bituminous thickness $D_1^{\cdot}$ as per constructability criteria.

$$SN_1^{\cdot} = D_1^{\cdot} \times a_1$$

5. Calculate thickness of base material:

$$D_2 = \frac{SN_2 - SN_1^{*}}{a_2}$$

a_2 = structural layer coefficient for base material = 0.14 for Graded Crushed Stone (GCS)

45

Provide D_2^* as per constructability requirement.

$$SN_2^* = D_2^* . a_2$$

6. Calculate thickness of sub-base material:

$$D_3 = \frac{SN_3 - SN_2^*}{a_3}$$

a_3 = structural layer coefficient for sub-base material = 0.11 for GSB material

Provide D_3^* as per constructability requirement.

7. Calculate overall structural number provided:

$$SN_{Provided} = \square\, a_i\, D_i\, m_i$$

8. Compare $SN_{Provided} > SN_3$

Based on the method of design described above, the Structural Number calculations and thickness required for all homogeneous sections are as below.

Table 2.9: Traffic Loading on various Homogenous Sections

Direction	Homogeneous Section (HS)	Traffic Loading (million standard axles)
Right Carriageway	HS -1	92.274
	HS -2	136.66
	HS-3	171.97
	HS-4	139.33
	HS-5	115.05
Left Carriageway	HS -1	64.79
	HS -2	119.74
	HS-3	77.58
	HS-4	106.09
	HS-5	100.58

Source: CES/APEC, 2007

Table 2.10: Structural Number Calculations for Homogenous Sections

Homogenous Sec.	Structural Number Right Carriageway	Structural Number Left Carriageway
HS – 1	4.83	4.63
HS – 2	5.06	4.98
HS – 3	5.21	4.73
HS – 4	5.08	4.91
HS - 5	4.96	4.88

Source: CES/APEC, 2007

Pavement Composition

Based on the above analysis, the following pavements structure was provided:

Asphalt Concrete	-	40 mm
Dense Bituminous Macadam	-	120 mm
Cement Treated Base Course	-	250 mm
Sub-base Course	-	250 mm
Subgrade	-	300 mm

NAIROBI THIKA HIGHWAY IMPROVEMENT PROJECT, LOT-3
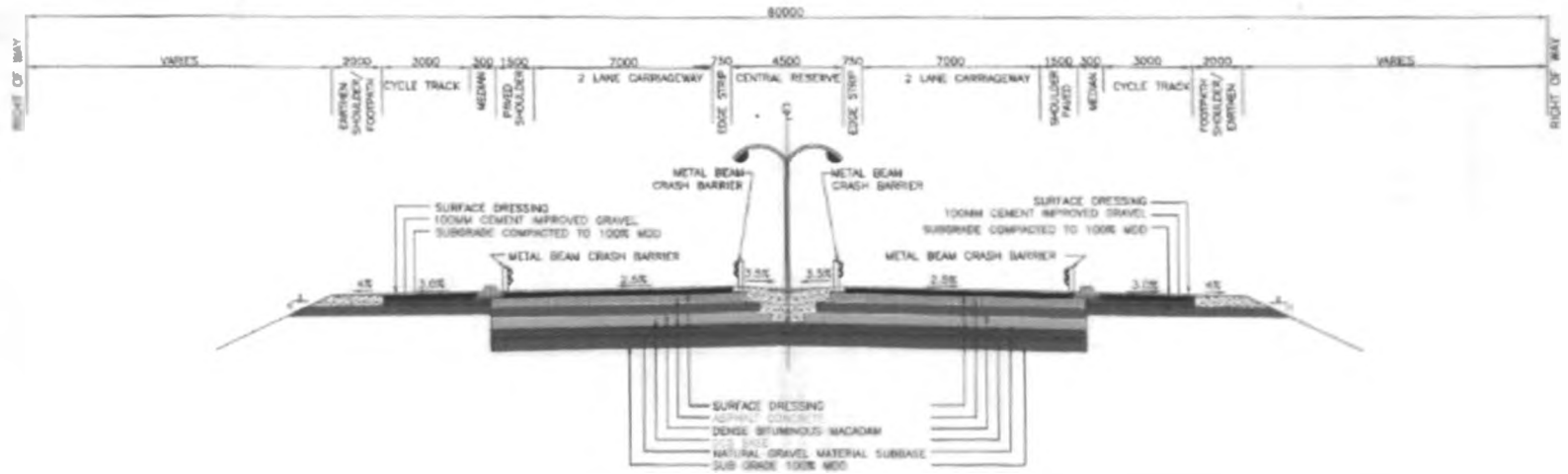
TYPICAL CROSS-SECTION
JUJA TO THIKA

Figure 2-7: Typical Cross-section

Source: CES/APEC, 2007

3 Theoretical Background

3.1 Design of Paved Roads

Highway pavement engineering may be defined as the process of designing, constructing and maintaining highway pavements in order to provide a desired level of service. Pavement design is aimed at achieving a pavement structure which is economical and comfortable to the motorist and which minimises development of pavement distress features such as rutting, cracking, pot-holes, ravelling, fretting, crazing distortions among others during the design life of the pavement. The design should take account of environmental factors and must also aim at desirable balance between construction, road-user and maintenance costs (Gichaga and Parker,1988). Various design methods have been developed to suit different climatic and traffic-loading conditions. However, most of the design methods used in the tropical countries were adapted from those developed for the European temperate climate. While there were substantial differences, a considerable knowledge existed that enabled experienced designers to incorporate reasonably satisfactory provisions within their designs for all but the most extreme effects of tropical climates. (Pell,1978).

Equally important differences between pavement engineering in developing countries and industrialized countries are the greater variability of construction materials, quality of consruction and the larger fluctuations in the volume and weight of traffic load that are encountered in developing countries.

Flexible pavement design methods can be divided broadly into empirical and and analytical methods. Due to the accumulated experience in the use of empirical and semi-empirical methods, empirical and semi-empirical methods continue to be used more than analytical methods.

3.1.1 Empirical and Semi-Empirical Pavement Design Methods

Empirical and semi-empirical design methods have been developed on the basis of long-term pavement performance for specific traffic loading and environment conditions. This means that for as long as conditions for which these methods were developed prevail, the performance of the pavement should be satisfactory.

3.1.2 Analytical Design Methods

Analytical pavement design involves the assumption of a pavement structure system. The strength characteristics (modulus and Poisson's ratio) may be established or assumed. Traffic loading is then introduced and structural analysis carried out to determine the stresses and strains at critical points in the structure. The values of stresses and strains obtained from the analysis are compared with the maximum allowable values, to determine whether the design is satisfactory. If the design is unsatisfactory, another system is tried.

Analytical design methods have not yet gained wide acceptance by road pavement designers mainly because of the complexity of the mathematical models involved. Other reasons for the lack of acceptance relate to inadequate material characterisation which prevents a designer from carrying out a theoretical analysis with confidence.

3.2 Principles of Pavement Design
3.2.1 Pavement Type, Wheel Loads and Design Factors
General

The field of pavement design is dynamic in that concepts are continually changing as new data become available.There are many methods of design available,since opinions regarding suitability of designs vary from one location to another. In particular, materials that are available for construction of pavements have a major influence on

50

design. There are, however, principles of design that are common to all problems irrespective of other extenuating circumstances.

The design of highway pavements involves a study of soils and paving materials, their behaviour under load, and the design of a pavement to carry that load under all climatic conditions. All pavements derive their ultimate support from the undelying subgrade: therefore a knowledge of basic soil mechanics is essential.

In the early stages of development, design consisted of rule-of –thumb procedures based on past experiences. During the period 1920 to 1940, engineers made a concerted effort to evaluate the structural properties of soil, principally for foundations for buildings and bridges. During this time, a vast amount of basic data was accummulated, which enabled the engineer to design foundations on a rational basis. Highway engineers were aware that performance of pavements was dependent to a large extent upon the types of soils over which the highway was constructed. As a result, correlations of pavement performance with subgrade types were established. In general, the studies showed that highways constructed over plastic soils showed higher degrees of distress than those constructed over granular deposits. Frost action and adverse drainage conditions were recognized early as two of the primary causes of pavement failure (Pell, 1978).

Nevertheless, many highway departments utilized standard cross sections for most highways. This meant that a road, even though it crossed several soil types, was constructed using constant thickness. This practice was often justified on the basis of economics.

Types of pavements
Pavements are broadly divided into two categories:

i.   Flexible pavements

ii. Rigid pavements

i.   **Flexible Pavements**

The flexible pavement consists of a relatively thin wearing surface built over a base course and subbase course, and they rest upon the compacted subgrade. The strength of a flexible pavement is derived from the composite effect of the various layers of the pavement. These layers are arranged in such a way that the layer strength increases from the subgrade upwards, with the strongest material being placed on the surface. (Gichaga and Parker,1988).

A flexible pavement comprises of the following layers:

- Surfacing layer
- Base
- Sub-base
- Subgrade

Surfacing Layer

The main function of the surfacing layer is to provide a running surface capable of carrying wheel loads without undue discomfort to motorists. It also protects the underlying layers from adverse weather conditions and provides the necessary skid resistance for ensuring road safety characteristics when braking becomes necessary.

Base

This is the main load carrying structural component in a flexible pavement. It should be designed to resist and distribute stresses induced by vehicles to the underlying layers.

Sub-base

The role of the sub-base is to help in distributing induced stresses into the subgrade as well as protecting the base from adverse soil conditions that may prevail in the subgrade.

Subgrade

This is the natural or improved ground on which the pavement structure is constructed. The design of the pavement structure should minimise the possibility of the subgrade undergoing excessive deformation.

ii.   Rigid pavements

Rigid pavements are made up of portland cement concrete and may or may not have a base course between the pavement and the subgrade. The essential difference between the two types of pavements, flexible and rigid, is the manner in which they distribute the load over the subgrade. The rigid pavement, because of its rigidity and high modulus of elasticity, tends to distribute the load over a relatively wide area of soil, thus a major portion of the structure capacity is supplied by the slab itself.

Design Factors

Pavement design consists of two broad categories:

i.   Design of paving mixtures and

ii.   Structural design of the pavement components

These two steps go hand in hand.

The structural design of pavements is greatly influenced by environmental factors. Likewise, a highway will cross many different soil deposits and it becomes necessary for the design engineer to select in a rational manner a design value representative of the area under question.The strength of soil is affected by many factors including density, moisture content, soil texture, rate of load application and degree of confinement.

3.3   Subgrade Construction in Kenya

Subgrade in a road pavement structure represents the natural or improved ground on which the pavement structure is constructed. (Gichaga and Parker, 1988). Subgrade can also be defined as a

53

pavement structure consisting of all the material below the sub base, including in-situ material, fill and improved subgrade. (In AASHTO subgrade has the same meaning as roadbed: another name for the subgrade is foundation) (DRDM, Part 3). It can also be defined as the layer of material on top of the roadbed, or fill and below the sub base.

The thickness of pavements is governed by the traffic intensity and the strength of the subgrade soil. Subgrade strength is greatly influenced by the moisture conditions prevailing within the subgrade soil. In the design of pavements therefore, it is important to make an assessment of the strength of subgrade soils at the most critical conditions likely to occur in the subgrade soil during the life of the pavement (RRTP No.80, 1968). In most cases in Kenya, roads are constructed in the dry season and the most adverse moisture conditions will occur some years later. Research carried out by the Road Research Laboratory indicated that the optimum moisture content in the British Standard compaction test represents the highest value to be considered for subgrade soils provided that:

i.   Drainage arrangements for the pavement layers are adequate
ii.   The water-table does not influence conditions under the pavement
iii. The subgrade soils are adequately compacted (RRTP No.80, 1968).

It is important that subgrade materials, whether in the form of parent material or borrowed material, are subjected to tests to establish their engineering properties. It is also important that the necessary preparation and compaction are carried out before the pavement structure starts to be laid (Gichaga & Parker, 1988).

In pavement design, the aim of the design process is to protect the bearing capacity of the in situ subgrade material in order that the road

pavement will be able to fulfil its service objective over the design period. The bearing capacity and quality of the subgrade (or roadbed or fill) is of prime importance in the selection of pavement type and is improved by overlaying it with layers of material to achieve an integrated and structurally balanced system (DRDM, Part 3). Classification of subgrade is based on CBR test as shown in table 3.1.

Table 3.1: Subgrade Classifications in Kenya

Soil Class	CBR Range (%)
S1	2-5
S2	5-10
S3	7-13
S4	10-18
S5	15-30
S6	>30

Source: RDM, Part III, 1987

According to the RDM, Part III, 1987, material of subgrade CBR value less than 2% cannot be used on a road directly. It will have to be improved.

In pavement design, the overall thickness of the pavement structure as well as of the individual layers will depend on the traffic load to be carried, the quality of the subgrade, the mechanical properties of construction materials constituting the pavement layers and prevailing climate. Thus the occurrence of cracks, distortions and other distress features on road pavements indicate inadequacy in pavement structure.

Even after the optimum road alignment has been carefully selected, the road engineer will meet varying types of alignment soils in tropical areas. These soils will vary in strength, particle size distribution, plasticity, moisture content among other engineering properties. Those soils that meet the specifications for road construction in their natural form would be processed and compacted forming the subgrade layer.

However, there are those soils that barely meet the specifications in their neat state. The engineering properties of these soils can be improved through physical or chemical processes. Those that will not meet the specifications even after improvement are normally scooped out and replaced by better materials.

During construction, it is important to check that the specifications for preparing the subgrade are met. Common methods of checking subgrade preparation will involve checking the levels of the formation, checking the horizontal alignment and the degree of compaction (Gichaga and Parker, 1988). It is also important to check the level of the water table so that, if necessary, the formation level could be raised to reduce the effects of the water table on the pavement structure.

3.3.1 Types of Subgrade Soils in Kenya

Different types of subgrade soils are found in Kenya. Soil characteristics are greatly influenced by mineralogy, climatic conditions, drainage conditions, relief and climate (Gichaga & Parker,1988). Table 3.2 represents Subgrade soils commonly found in Kenya.

Table 3.2:Subgrade Soils Commonly found in Kenya

No.	Material Type	C.B.R (%)	
		Soaked	Unsoaked
1	Black cotton soils	2-5	15-30
2	Micaceous silts	2-5	7-13
3	Other alluvial silts	5-10	10-18
4	Red friable clays	7-13	15-30
5	Sand clays on volcanics	7-18	15-30
6	Ash and pumice soils	7-18	15-30
7	Silty loams on gneiss and granite	10-18	15-30
8	Calcareous sandy soils	10-30	15-30
9	Sandy clays on basement	10-18	>15
10	Clay sands on basement	10-18	10-30
11	Dune sands	>10	15-30
12	Coastal sands	>10	>15
13	Weathered lava	>10	>15
14	Quartizitic gravels	>10	>15
15	Soft (weathered) tuffs	>10	>15
16	Calcareous gravels	>15	>30
17	Lateritic gravels	>15	>30
18	Coral gravels	>15	>30

Source: Murunga, 1983

The above soils can broadly be classified as:

i.   Lateritic gravel soils

ii. Red coffee soils

iii. Expansive soils (such as clays)

i.   Lateritic Gravel Soil

Lateritic gravel soil is a product of typical alteration suffered by alkali rocks such as trachyte, basic igneous rocks like diorites and basalts, gneiss rocks rich in feldspar and sedimentary rocks including shales and impure limestones. Lateritic gravel soil is a product of weathering of the parent rock.

57

Lateritic gravel soils have good drainage and high bearing strength properties and are commonly used in the construction of sub-base and base layers

ii.   Red Coffee Soil

Red coffee soil is formed by the waethering of acid granites and gneisses, quartizitic and felsphatic rocks rich in iron and magnesium bearing. The colour of the soil is generally red. Red soils generally exhibit collapse characteristics because of the mineral halloysite. This type of soil requires special attention, particularly with regard to water content during compaction. It has been found, for example, that red soils are susceptible to cracking when compacted at moisture contents below the optimum (Gichaga & Parker, 1988).

iii.   Expansive Soils (Such as Black Cotton Soil)

The name black cotton soil is derived from its black colour appearance and immense fertility for growing cotton. The main property of expansive soil is the significant volume changes it undergoes when wetted and dried.

Black cotton soils have CBRs ranging from 2-5 (soaked) and 15-30 (unsoaked). The shrinking and swelling property is caused by the presence of the clay mineral montmorillonite. Four possible treatments are possible to overcome the problem of expansive soils:
- Avoid by re-alignment
- Excavate and replace with non-expansive materials
- Stabilise with lime, or
- Minimise moisture changes by engineering measures such as provision of a free draining layer in the form of a cushion of granular material.

3.3.2 Subgrade Requirements for Pavement Design

Materials forming the direct support of the pavement shall normally comply with the following requirements:

- CBR at 100% MDD (Standard Compaction) and 4 days soak: more than 5
- Swell at 100% MDD (Standard Compaction) and 4 days soak: less than 2%
- Organic matter (percentage by weight): less than3%

This means that no pavement should be placed directly on Class S1 and that an improved subgrade is required on such soil.

3.3.3 Improved Subgrade

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

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

It may prove technically and economically advantageous to lay an improved subgrade not only on S1, but also on S2 and S3 Class soils. The decision will generally depend on the respective costs of subbase and improved subgrade materials. An improved subgrade would generally not be economically justified on Class S4 soils.

An improved subgrade placed on soils of any particular class must obviously be made of a material of higher class (up to Class S5, since Class S6 is subbase quality).

3.3.4 Influence of Improved Subgrade on Subgrade Bearing Strength

Where a sufficient thickness of improved subgrade is placed, the overall subgrade bearing strength is increased to that of a higher class and the subbase thickness may be reduced accordingly.

Table 3.3 shows the minimum thickness of each class of improved subgrade required on each class of natural soil to obtain a higher class of subgrade bearing strength. These minimum thicknesses have been calculated taking into account the respective elastic moduli of each class of soil.

Table 3.3: Minimum Thickness of Improved Subgrade Required

Strength class of native subgrade soils	Improved subgrade		New class of subgrade bearing strength
	Material Strength Class	Minimum thickness required (mm)	
S1*	S2	400	S2
	S3	350	S2
		425	S3
	S4	275	S2
		325	S3
		450	S4
S2*	S3	300	S3
	S4	200	S3
		350	S4
S3	S4	300	S4
	S5	150	S4
		350	S5

3.3.5 Selection of appropriate design using RDM Part III Catalogue

Suppose average subgrade CBR along a road was established as 8% and traffic loading found to be 12,858,865.73 CSA, according to tables 2.7 and 3.1, this corresponds to traffic class T2 and subgrade class S3 respectively. According to Figure 3.1, this gives pavement structure type 5 which comprises of:

- 75 mm thick Asphalt Concrete Type 1
- 150 mm thick cement stabilized gravel
- 225 mm thick cement or lime improved material (base quality)

STANDARD PAVEMENT STRUCTURE TYPE 5

BASE : Cement stabilised gravel
SUBBASE: Cement or lime improved material (Base quality)

	T 5	T 4	T 3	T 2	T 1
S 1				75 / 150 / 350	100 / 150 / 350
S 2				75 / 150 / 250	100 / 150 / 250
S 3		ECONOMICALLY UNJUSTIFIED		75 / 150 / 225	100 / 150 / 225
S 4				75 / 150 / 200	100 / 150 / 200
S 5				75 / 150 / 175	100 / 150 / 175
S 6				SEE TYPE 4	

SUBGRADE		TRAFFIC	
CLASS	CBR (%)	CLASS	ESA x 10⁶
S 1	2 - 5	T 1	25 - 60
S 2	5 - 10	T 2	10 - 25
S 3	7 - 13	T 3	3 - 10
S 4	10 - 18	T 4	1 - 3
S 5	15 - 30	T 5	0-25 - 1
S 6	> 30		

IMROVED SUBGRADE (Reproduced from Table 8.3.1)												
Native Subgrade Class		S1						S2		S3		
Improved Material	S2	S3		S4			S3	S4		S4	S5	
Subgrade Thickness (mm)	400	350	425	275	325	450	300	200	350	300	150	350
New Class	S2	S2	S3	S2	S3	S4	S3	S3	S4	S4	S4	S5

Figure 37: Sample RDM Part III Catalogue Relating Subgrade CBR and Traffic Loading.

3.4   Control of Material Behaviour in Road Construction

3.4.1   Background Information on the use of Geotextiles

In the road industry there are four primary uses for geotextiles namely:

i.   Separation,

ii.   Drainage,

iii.   Filtration and

iv.   Reinforcement.

 In separation, inserting a properly designed geotextile will keep layers of different sized particles separated from one another. In drainage, water is allowed to pass either downward through the geotextile into the subsoil, or laterally within the geotextile which functions as a drain. How it is used depends on the drainage requirements of the application. In filtration, the fabric allows water to move through the soil while restricting the movement of soil particles. In reinforcement, the geotextile can actually strengthen the earth or it can increase apparent soil support. For example, when placed on sand it distributes the load evenly to reduce rutting.

Geotextiles now are most widely used for stabilizing roads through separation and drainage. When the native soil beneath a road is very silty, or constantly wet and murky, for example, its natural strength may be too low to support common traffic loads, and it has a tendency to shift under those loads. Although the subgrade may be reinforced with a base course of gravel, water moving upward carries soil fines or silt particles into the gravel, reducing its strength.

Geotextiles keep the layers of subgrade and base materials separate and manage water movement through or off the roadbed. In separation functions geotextiles keep fines in the subgrade from migrating into the base course. Tests show that it takes only about 20% by weight of subgrade soil mixed into the base course to reduce its bearing capacity to that of the subgrade. This problem usually is due to the movement of large amounts of water. When large loads cross the surface of the

roadway they set up a pumping action which accelerates this water movement and soil particle migration, and speeds up the failure of the road.

In the construction of the Nairobi-Thika Road, Geotextiles (geobelts) have been used to provide reinforcement in the construction of reinforced earth walls by providing tensional resistance. Reinforced earth walls have been used on grade separated intersections. This technology is particularly useful in roads passing through urban areas where the cost of land acquisition is very high.

4 Methodology and Data Collection

Introduction

This study employed:

  i. A review of the Consultant's Draft Final Feasibility Report for the Nairobi-Thika Highway Improvement Project.

  ii. Laboratory testing of alignment soils at deep cut sections.

  iii. Collection of test results from the Resident Engineer's Laboratory for the deep cut sections where the road had already been built.

  iv. Collection of Juja Weighbridge axle load data for the Nairobi-Thika Road. This data was obtained from the Kenya National Highways Authority.

4.1 Review of the Consultant's Draft Final Feasibility Report.

The Traffic Report, Engineering Surveys and Investigations Report and the Soil Investigation Report for the sections where structures are located were obtained. Data from the Traffic Report and Engineering Surveys and Investigations Report is important in that it details the traffic loading and the engineering properties of the alignment soils which are important variables in pavement design. The soil investigation report was used to identify homogenous sections which together with the corresponding traffic loads was used to design homogenous sections. This report was also used in analysis and subsequent soil classification. The design year traffic was compared with earlier traffic loading data to establish whether there are any trends in traffic loading changes.

4.2 Laboratory Tests of Alignment Soils at Deep Cut Sections

Laboratory tests were also carried out at areas where there are deep cuts to establish the variation of engineering properties of soil with depth.

The information collected was presented in graphs and charts. These were clearly labelled and presented so that the reader can rapidly make sense of the information contained in them.

4.2.1 Sampling

Soil sampling was done in accordance with *BS 5930: The Code of Practice for Site Investigations*. Sampling was done at different levels of the embankment, the guiding principles being:

i. Avoidance of sampling of the loose top material
ii. Avoidance of sampling of the borrowed material
iii. Sampling at one (1) metre intervals from top downwards or
iv. Sampling at each layer where the layers were observed to substantially vary in terms of engineering properties (sizes of soil particles, colour, cleavage among others).

Note: Where the embankment was found to be too high and therefore dangerous for manual sampling, an excavator was used as shown in Plate 4.

The undisturbed samples were obtained in accordance with BS 1377 Part 19, 1990.

Material sampling was done at the following locations:

a) Km 2+800 (Pangani interchange). Here, sampling was done at 3 m from the top of the old pavement surface. The top 3m was found to comprise of fill material overlain by the pavement layers. This constitutes borrowed material and was therefore avoided.

66

b) Km 3+150 (Muthaiga interchange). Here, samples were taken at four levels. The top 150 mm was found to be the overburden material and was therefore not sampled.

These samples were then tagged and labelled based on location and depth and thereafter taken to the Engineer's site laboratory for testing. The following tests were carried out:

 i.   Particle Size Distribution
 ii. Atterberg Limits
 iii. Compaction test
 iv. California Bearing Ratio (CBR).

The tests were carried out according to British Standards (BS 1377). The sae mpling and testing processes are captured in Plates 1-8.

Plate 1: Material sampling @ the Pangani interchange

Plate 2: Excavation of material at Pangani Interchange

Plate 3: Section of embankment @ km 3+100 before sampling

68

Plate 4: Scooping of soil sample @ km 3+100 using a track excavator

Plate 5:Mixing of soil sample with water-Proctor Test

Plate 6:Placing of soil sample into the mould

Plate 7: Compaction of soil sample-Proctor Test

Plate 8: Weighing of soil sample at the RE's Laboratory

Compaction Test

This is a standard test which enables the response of a soil to variations in moisture content and compactive effort to be studied. The test equipment consists of a standard cylindrical mould into which the soil to be tested is introduced in layers, and a falling weight hammer which is used to impact a standard compactive effort to each layer of the soil. Moisture content is varied from sample to sample and measured, and the weight and hence the density of the known volume of soil in the standard mould is recorded for each sample. It is thus possible to find how the dry density (the weight of soil solids in unit volume) of the soil varies with its moisture content, given a constant compactive effort.

At low moisture contents, the soil is stiff, unworkable and difficult to compress but as the moisture content is increased so the water acts as a lubricant between the soil particles, easing relative movement, and

the soil becomes easier to compress. As further water is added to the mix, the voids between the soil particles become almost completely filled with water whose effect is to keep the soil particles apart-and so the density falls. Increases in moisture content increases the proportion of voids present in the soil and further reduces its density.

The moisture content at which the maximum density is obtained is reported as the optimum moisture content.

California Bearing Ratio (CBR)
When in service beneath a pavement, soil is required to resist deformation which locally applied forces tend to cause. For example, a wheel load acting on top of a thin layer of unbound granular material overlying a stiff soil may cause a small displacement in that soil; the same wheel load applied to a similar layer of granular material which overlies a soft soil will tend to cause a much greater displacement in the soil. In the design of the lower layers of the pavement, knowledge of the subgrade of such loading is available.

The California Bearing Ratio (CBR) test represents an attempt to quantify the behaviour of a soil in such circumstances. The test was developed in the 1930s and now forms the basis of the most widespread empirical pavement design method.

The test consists of driving a standard cylindrical plunger into the soil sample at a standard rate of penetration and measuring the resistance to penetration offered by the soil. This resistance is then compared with certain standard results, the ratio of the result for the soil to the standard result being reported as the CBR.

There are two main variables which can affect the test results, and the properties of the soil which the test quantifies. The moisture content of

the soil can be very significant, particularly (but not exclusively) in the case of cohesive soils. It is for this particular reason that care must be taken in the control of moisture content during laboratory testing and more particularly to ensure that the test conditions adequately represent those which will be experienced in the field.

The ability of a soil to resist deformation is also influenced by any surcharge acting on the soil. Provision is made in the test for this phenomenon by applying annular weights around the plunger to impose a local surcharge on the soil, the intention being that this surcharge should equal that which will be imposed by the weight of the pavement which is expected to overlie the soil in service. As the surcharge increases, so will the effective CBR.

Deep Cut Sections after Road Construction

Different slope stability measures were undertaken at various deep cut sections. The determination of appropriate stability measure at each location was determined by either experience or 'rule of thumb' or analysed individually using appropriate geotechnical methods. In order to appreciate the slope stability measures undertaken at the deep cut sections road after construction, the interventions at such locations were captured in photographs. These slope stability measures are captured in Plates 9-12.

At the Pangani interchange, reinforced concrete retaining walls were used. At the Muthaiga Flyover, the side slopes were cut to an angle in accordance to the recommendations set out in RDM Part III.

Plate 9: Pangani Underpass after construction (junction comprises of three levels)

Plate 10: Muthaiga Flyover

Plate 11: Showing embankment @ km7+100

Plate 12: Deep cut section of the Nairobi-Thika Road (Km 6+550-km 7+730)

4.3 Axle Load Data

4.3.1 Introduction

Traffic is the most important factor in pavement design. The consideration of traffic should include both the loading magnitude and configuration and the number of load repetitions.

Axle load data is an important element in the design of a new or rehabilitation/ strengthening an existing pavement. The loads imposed by private cars and light goods vehicles with axle weights < 1.5tonnes do not contribute significantly to the structural damage of a paved road and thus, for design purposes, can be ignored. However, for economic and congestion forecasting, the total traffic is determined and routine traffic counts are carried out annually at a number of census points. They distinguish between cars, light goods, buses, medium goods and heavy goods vehicles. Where such results are available, the initial daily traffic can be estimated by extrapolation.

From axle load data, the road pavement designer is able to establish the damaging factor of different classes of traffic and therefore come up with an appropriate design. This is obtained by use of an equivalent axle load factor (EALF). EALF defines the damage per pass to a pavement by the axle in question relative to the damage per pass of a standard axle load.

4.3.2 Axle Loads Legal Limits for Vehicles in Kenya

The axle load limits for the various axle/wheel configurations, maximum gross vehicle weight, maximum vehicle dimensions and the minimum fines for excess axle load or gross vehicle weight are set in the Traffic Act (Cap. 403). Tables 4.1 and 4.2 give the permissible maximum axle loads and maximum gross vehicle weights respectively. The wheel configurations are given in Figure 4.1.

Table 4.1: Permissible Maximum Axle Loads

	Axle Group (with Pneumatic Tyres)	Maximum Axle Load (Kg)
A	Single Steering Axle (2 wheels –single tyres)	8,000
B	Single Axle (4 wheels –dual tyres)	10,000
C	Tandem Axle Group (8 wheels- dual tyres)	16,000
D	Triple Axle Group (12 wheels-dual tyres)	24,000

Source: KENHA, 2012

Table 4.2: Maximum Gross Vehicle Weights

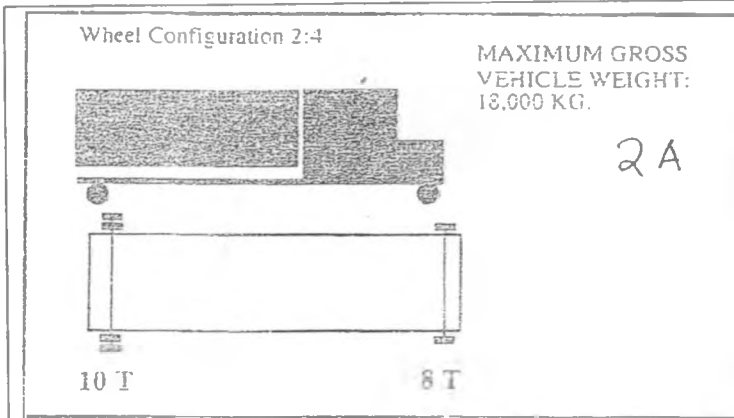|	Vehicle and Axle Configuration (Pneumatic Tyres)	Maximum Gross Vehicle Weight
A	Vehicle with 2 axles	18,000
B	Vehicle with 3 axles	24,000
C	Vehicle and semi-trailer with total of 3 axles	28,000
D	Vehicle with 4 axles	28,000
E	Vehicle and semi-trailer with total of 4 axles	34,000
F	Vehicle and drawbar trailer with total of 4 axles	36,000
G	Vehicle and semi-trailer with total of 5 axles	42,000
H	Vehicle and drawbar trailer with total of 5 axles	42,000
I	Vehicle and semi-trailer with total of 6 axles	48,000
J	Vehicle and drawbar trailer with total of 6 axles	48,000

Source: KENHA, 2012
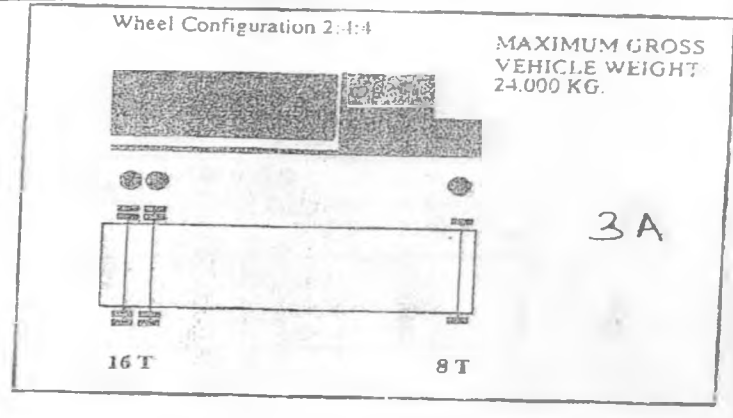
Diagram 1: Rigid Body Vehicle-2 Axles

Wheel Configuration 2:4

MAXIMUM GROSS VEHICLE WEIGHT: 18,000 KG.

2 A

10 T   8 T

Diagram 2: Rigid Body Vehicle-3 Axles

Wheel Configuration 2:4:4

MAXIMUM GROSS VEHICLE WEIGHT 24,000 KG.

3 A

16 T   8 T
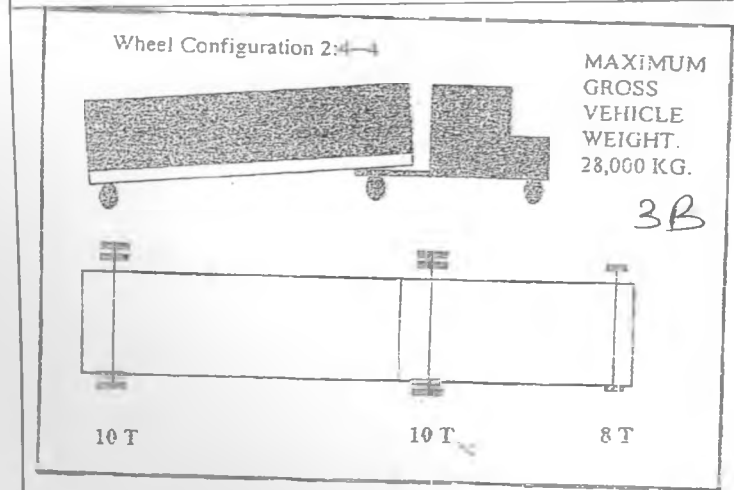
Diagram 3:Vehicle & Semi- Trailer-3 Axles

Wheel Configuration 2:4—4

MAXIMUM GROSS VEHICLE WEIGHT. 28,000 KG.

3 B

10 T   10 T   8 T
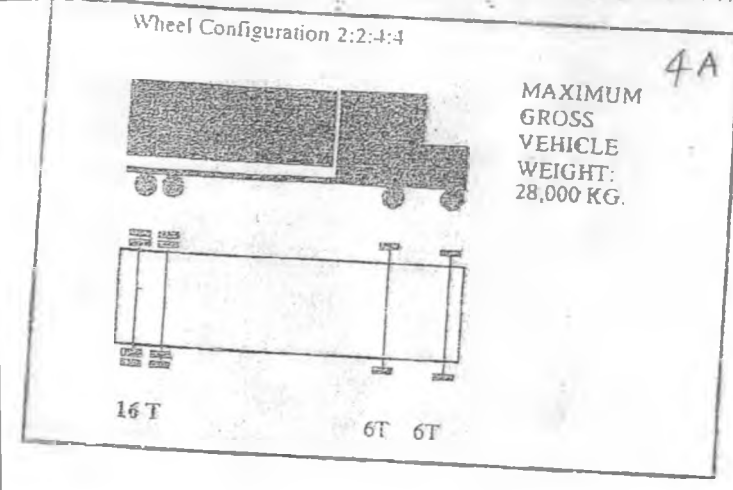
Diagram 4:Rigid Body Vehicle-4 Axles

Wheel Configuration 2:2:4:4

MAXIMUM GROSS VEHICLE WEIGHT: 28,000 KG.

4 A

16 T   6T   6T
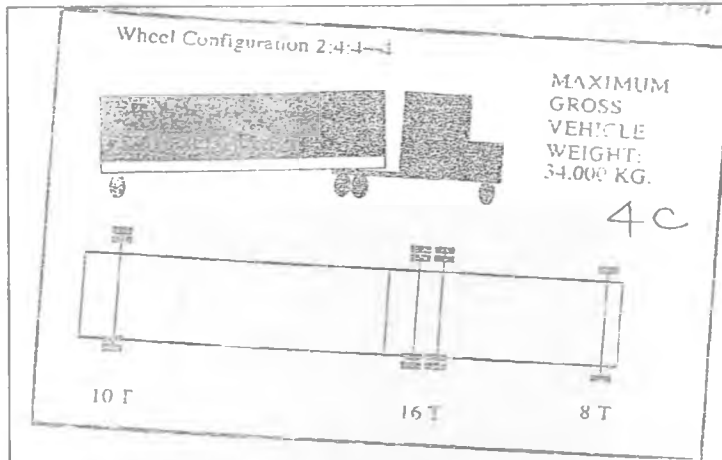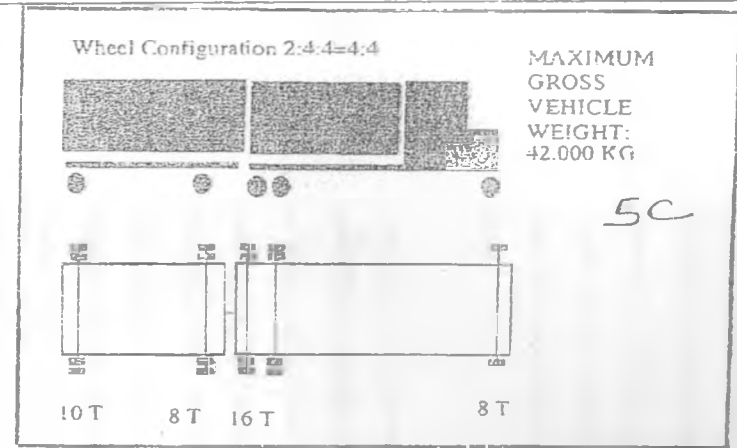
Diagram 5: Vehicle & Semi-trailer-4 Axles

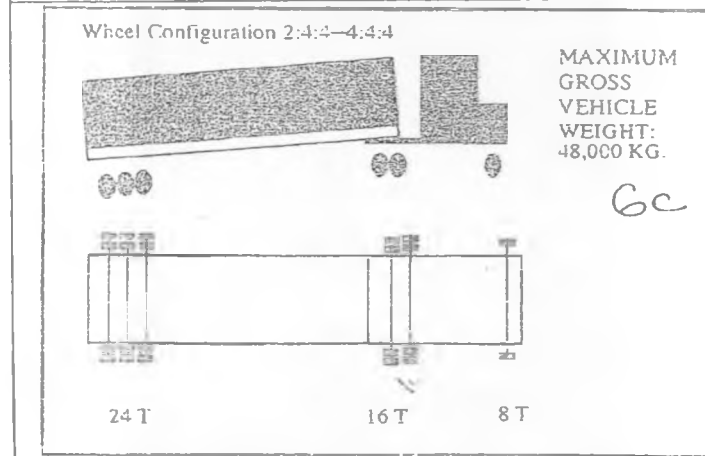Diagram 6: Vehicle & Drawbar Trailer-5 Axles
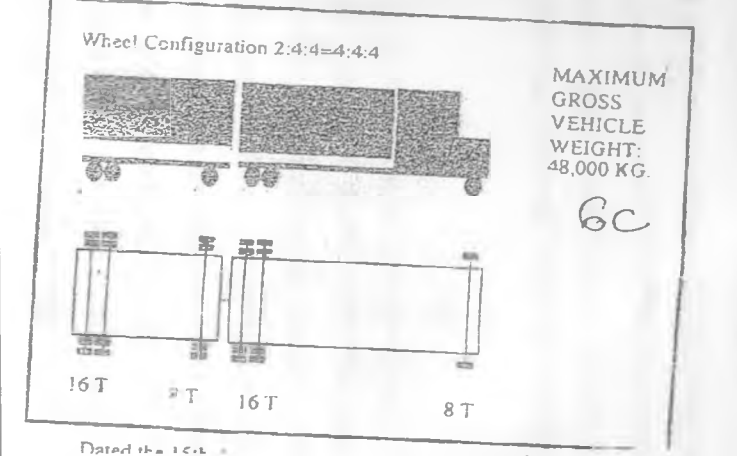
Diagram 7: Vehicle & Semi-trailer-6 Axles

Diagram 8: Vehicle & Drawbar-Trailer-6 Axles

Figure 4-1: Showing different wheel configurations

Source:Kenya Gazette Supplement No. 65. Legal Notice No. 118, The Traffic Act, 12th September, 2008

4.3.3 Methodology for Collection and Analysis of Axle Data

KeNHA collects axle load data at various points along its expansive network of roads throughout the country which is then stored in a data base. In the case of Nairobi-Thika Road, this information is collected by use of a portable weighbridge stationed at few kilometres before Juja town.

The Juja weighbridge comprises of a pit mounted truck scale with the weighing equipment and platform in a pit so that the weighing surface is level with the road. The weighbridge is typically built from steel resting on a robust concrete platform. The weighbridge device uses multiple load cells that connect to an electronic equipment to totalize the sensor inputs. The weight readings are typically recorded in a nearby office. The weighbridge is linked to a personal computer which runs truck scale software capable of printing tickets and providing reporting features.

This weighing process is captured in plates 13-16.

Plate 13: The Juja Weighbridge

Note: The steel truck scale resting on a concrete platform

Plate 14: Axle load weighing at the Juja Weighbridge

Plate 15: Weighing of a trailer at the weighbridge

Plate 16: Weighing of a rigid body vehicle at the weighbridge

Two sets of axle load data were collected. The first set of data obtained covered seven (7) days (i.e from 2nd to 9th May, 2011) while the second set of data covered nineteen (19) days (i.e from 1st to 19th November 2011.The choice of two sets of data was for comparison purposes and also to check whether there is any appreciable change in traffic as road construction progressed. The month of November and not December was taken as an ideal month (it is expected that seasonal traffic variations are minimal).

5 Analyses and Discussion

5.1 General

This chapter details the analysis and discussion of the data gathered in chapter 4.The analysis and discussion has been divided into various sub-sections.

5.2 Laboratory Tests

Full laboratory test results are given in Appendix A. These results were summarised in a tabular form and are given as Table 5.1.

5.2.1 Grading

Km 2+800-Pangani Interchange

The test results indicate the presence of fine sand, medium sand, coarse sand, fine gravel, medium gravel and some bit of coarse gravel.

Km 3+150-Muthaiga Interchange

The test results indicate the presence of fine sand, medium sand, coarse sand, fine gravel, medium gravel and some bit of coarse gravel.

5.2.2 Compaction

Km 2+800-Pangani Interchange

At km 2+800, two samples were taken. Sample tested at a depth of 2.2 m was found to have an OMC of 13.0% and an MDD of 1385 kg/m^3 while that from 3.3 m depth was found to have an OMC of 20.6% and an MDD of 1470 kg/m^3. At this location, the degree of compaction increased with depth.

Km 3+150-Muthaiga Interchange

At km 3+150, four samples were taken at depths of 1.15, 2.15, 3.15 and 4.15. At this location, based on the test results, there is no definite pattern in variation of compaction with depth. Sample taken at 4.15 m depth was found to have the lowest OMC and MDD values.

5.2.3 Atterberg Limits

Km 2+800-Pangani Interchange

Soil samples taken at this location were found to be non-plastic. This means that these soils were found to have little or no silt or clay. Their respective liquid limits at depths 2.2 m and 3.3 m were found to be 41.6% and 44.8% respectively. Therefore, at this location, the water content at which a soil changes from plastic to liquid behaviour increases with depth.

Km 3+150-Muthaiga Interchange

Samples taken at depths of 1.15 m, 2.15 m and 3.15 m were found to have PI's of 37.9%, 33.4% and 26.6% respectively. At a depth of 4.15, the soil was found to be non-plastic. Here, plastic limit was found to decease with depth. For Liquid Limit, there was no definite pattern.

5.2.4 CBR

Km 2+800-Pangani Interchange

The soaked CBR of soil samples taken at depths of 2.2 m and 3.3 m was 16% and 9% respectively. According to RDM Part III, (1987), this corresponds to subgrade classes S5 and S3.

Km 3+150-Muthaiga Interchange

Samples taken at depths of 1.15 m, 2.15 m, 3.15 m and 4.15 m were found to have CBR's of 5%, 11%, 9% and 6% respectively. The CBR of soils at this location increases with depth up to approximately 2.15 m depth after which it decreases with depth.

84

Table 5.1: Summary of Alignment Soils Laboratory Test Results (University Way-Muthaiga)

Chainage (Km+m)	Sample Depth (m)	NMC	Plastic limit	liquid limit	plasticity index	Passing 37.5 mm	passing 28 mm	passing 20 mm	passing 14 mm	passing 10 mm	passing 6.3 mm	passing 5.0 mm	passing 2.0 mm	passing 1.0 mm	passing 0.425 mm	passing 0.30 mm	passing 0.15 mm	passing 0.075 mm	OMC	MDD	4 days soak	Description
		%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	Kg/m3	%	
2+800	2.2		Non plastic	41.6		100	91.1	80	69.2	61.0	52.1	48.4	37.4	30.5	24.6	22.4	18.9	16.2	13.0	1385	16	Brownish material mixed with soft rock
2+800	3.1		Non plastic	44.8		100	87.8	77.4	69.0	63.4	57.9	55.3	47.8	42.2	36.7	34.1	30.6	27.7	20.6	1470	9	Brownish material mixed with soft rock
3+150	1.15		37.9	56.2	18.3		100	99.5	96.6	94.4	85.4	75.2	43.0	35.1	31.7	30.3	28.0	26.3	20.8	1570	5	Reddish lateritic gravel
3+150	2.15		33.4	52.8	19.4		100	97.5	91.5	82.6	65.8	58.5	41.1	34.2	30.5	29.4	27.6	26.3	22.0	1600	11	Dark lateritic gravel material
3+150	3.15		26.6	42.4	15.8		100	98.2	97.3	94.0	89.2	83.6	64.6	56.7	50.1	44.0	40.5	37.9	20.6	1735	9	Lateritic gravel mixed with whitish soft rock
3+150	4.15		Non plastic	65.5			100	98.3	93.4	83.7	66.3	59.3	44.0	36.6	31.9	29.7	26.6	24.2	27.6	1182	6	

Km 2+800-Pangani Flyover

Km 3+150-Muthaiga Flyover

Full laboratory results are presented in Appendix A.

5.3 Test Results from the Resident Engineers Laboratory

Table 5.2 presents the test results obtained from the Resident Engineer's materials laboratory. Here, it was not possible to sample materials at various depths at cut areas as the road had already been built and any effort to do that could have resulted to pavement damage. Therefore, data collected by the RE's office was used for analysis.

Analysis of this data indicates that along the GSU section (km 6+850-km 7+200), there is significant variation in the CBR of materials sampled along the road. However, for materials sampled at the same chainage reference, there is minimal variation in CBR of materials across the road.

Along km 8+100 to km 8+500, material properties varied both longitudinally and transversely.

Table 5.2: Summary of Alignment Soils Laboratory Test Results (Muthaiga-Kasarani)

LOCATION				ATTERBERG PARAMETERS				GRADING													COMPACTION		CBR	
Chainage (Km+m)	Sample Depth (m)	GRAVEL %	SAND %	SILT,CLAY %	Plastic Limit	liquid limit	plasticity index	Linear Shrinkage	Passing 37.5 mm	passing 28 mm	passing 20 mm	passing 14 mm	passing 10 mm	passing 6.3 mm	passing 5.0 mm	passing 2.0 mm	passing 1.0 mm	passing 0.425 mm	passing 0.30 mm	passing 0.15 mm	passing 0.075 mm	OMC	MDD	4 days soak	Remarks
		%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	Kg/m3	%	
6+850 LHS		39	30	31	33.9	48.5	14.6	7.1			100		94.3		77.4	61.2	53.8	45.3			31.1			8.9	Brownish gravel
6+850RHS		39	30	31	34.1	48.8	14.7	7.0																9.0	
7+000 LHS		59	14	27	34.2	48.9	14.7	7.2		100	97.9		88.4		62.5	41.3	37.1	34.1			27.3			17	Brownish gravel
7+000 RHS		59	14	27	33.9	48.5	14.6	7.2																15	
7+200 LHS		61	14	25	32.2	46.2	14.0	7.3	100	99.2	94.6		73.2		53.9	39.1	34.7	31.4			25.1			12	Brownish gravel
7+200 LHS		61	14	25	32.1	6.3	14.2	7.2																12	
8+100 LHS		58	12	30	29.4	48.9	19.5	9.3														18.1	1692	17.4	Reddish Brown Gravel
8+100 RHS		58	12	30	29.5	48.7	19.2	9.1			100		97.6		67.9	42.1	38.5	36.0			30.3	18.1	1692	15.2	Reddish Brown Gravel
8+500 LHS		7	48	45	32.9	49.1	16.5	8.0														27.5	1450	6.1	Reddish clay material
8+500 RHS		7	48	45	32.4	48.8	16.4	7.8			100		98.4		97.3	93.2	88.3	76.4			45.1	27.5	1450	10.1	Reddish clay material

87

5.4 Axle Load Data

The two sets of data were then analysed as follows:

i. Data Covering the Period between 2nd and 9th May, 2011

The collected axle load data was entered into the computer and checked for inconsistencies, and other logical errors. The checked and corrected data was then used in final analysis. The axle loads were first converted into pounds (lbs) and thereafter converted into wheel loads for ease of comparison with earlier studies (Gichaga 1971). Wheel loads were then classified into 1000 lbs intervals and respective percentage wheel loads obtained. This information was then presented in Table 5.3.

Table 5-3: Wheel load distribution for all trucks in 1971 and 2011

Wheel load x 1000 lbs	% of wheel loads weighed (Gichaga 1971)	% of wheel loads weighed (KeNHA 2011)
0 (0.0)	0	0
1(0.5)	0	0
2(2.5)	2.5	0
3(1.4)	15.5	0
4(1.8)	17.5	1
5(2.3)	14	0
6(2.7)	12	3
7(3.2)	9	13
8(3.6)	10	26
9(4.1)	5	42
10(4.5)	5	9
11(5.0)	4	2
12(5.4)	2.5	0
13(5.9)	2	1
14(6.4)	0	2
15(6.8)	1	1
16(7.3)	0	0
17(7.7)	0	0
18(8.2)	0	0
Total	100	100

(X)=Equivalent wheel load x 1000 kilograms

From table 5.3 above, a graph showing percentage wheel loads versus wheel loads (x1000) was then plotted and is presented as Figure 5.1.
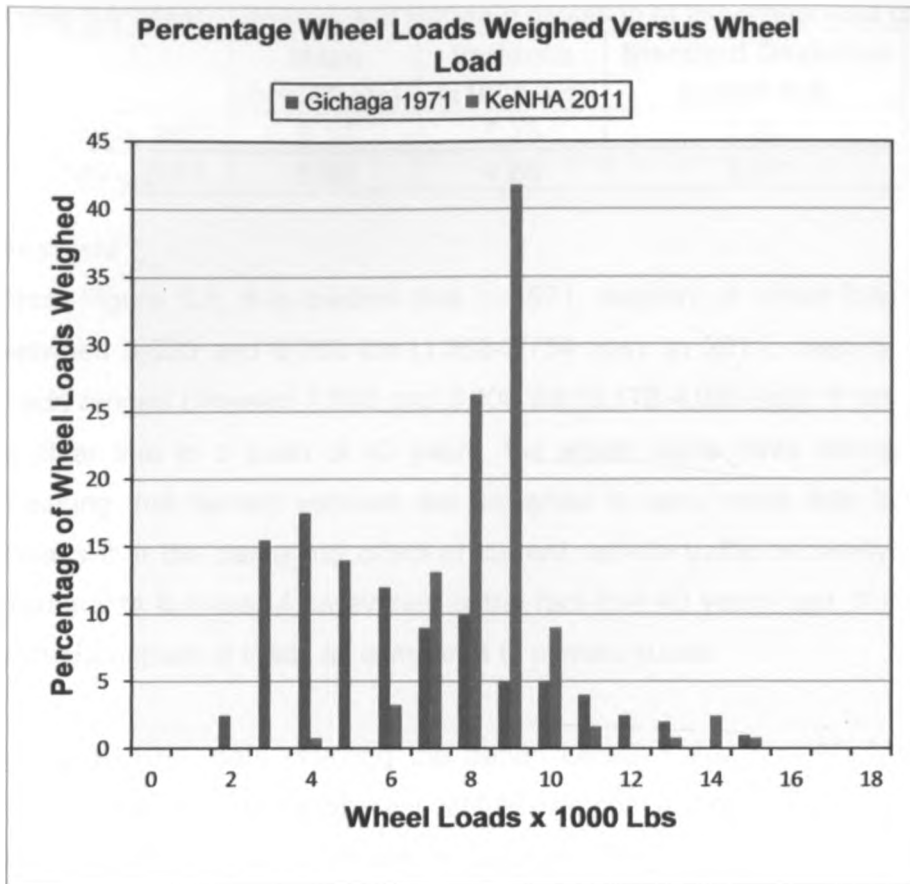
Figure 5.1: Wheel load distribution for all trucks in 1971 and 2012
Source, Gichaga 1971 and KeNHA 2011

No. of vehicle sampled, n, 1971=218

$$2011=122$$

In this study, a procedure involving the conversion of the maximum gross weights into axle loads was adopted.

The following measures of dispersion were sought to help capture the changes in wheel loads:

- Mean
- Variance and
- Standard deviation

The two wheel load cases are summarised in table 5.4.

Table 5.4: Mean, variance and standard deviation of two wheel load cases

	Mean (x1000 lbs)	Variance (x1000 lbs)	Standard Deviation (x1000 lbs)
Gichaga, 1971	6.17	7.23	2.69
KENHA, 2011	8.66	4.69	2.17

Analysis

From Figure 5.1, it is evident that in 1971, majority of wheel loads ranged between 3,000 and 6,000 lbs (1,362-2,734 kgs). In 2011, majority of wheel loads ranged between 7,000 and 9,000 lbs (3,178-4,086 kgs). From above, it is clear that in a span of 40 years, the wheel loads have almost doubled meaning that current vehicles are designed to carry more axle loads. This means that the damaging effect of current vehicle traffic on newly designed pavements is more. Also evident is the fact that 40 years ago, trucks had a wider spectrum of loads as compared to current trucks.

In the KENHA data covering the period between 2nd and 9th May, 2011, individual axle loads for different vehicle axle configurations were not captured and therefore average vehicle equivalence factors for different axle configurations could not be derived. It was therefore not possible to compute the cumulative number of standard ales.

From Table 5.4,the mean wheel loads for the Nairobi-Thika Road in 1971 and 2011 was established to be 6,170 lbs (2,798 kgs) and 8,660 lbs (3,828 kgs) respectively. In terms of axle loads, these correspond to axle loads of 12,340 lbs (5,596 kgs) and 17,320 lbs (7,256 kgs) respectively. This indicates that in a span of 40 years, the average wheel load has increased by 36.8%.The variation in wheel loads was higher in 1971(variance of 7,230 lbs) as compared to 2011(variance of 4,690 lbs). The standard deviation of wheel loads for 1971 was obtained as 2,690 lbs (1,220 kgs) while that of 2011 was 2,170 lbs (984 kgs). This indicates that those in the transport industry nowadays go for the biggest trucks so that they derive maximum benefit.

The damaging effect as a result of axle loads for the two years expressed in terms of equivalence factor using Liddle's formula is summarised in Table 5.5.

Table 5.5: Damaging effect of average axle loads in 1971 and 2011

Year	Average wheel load (kgs)	Average axle load (kgs)	Equivalence Factor
1971	2798	5596	0.18
2011	3828	7656	0.75

From table 5.5 above, the damaging effect as a result of the average axle loads of year 2011 vehicles is 4.1 times higher than that of year 1971.

ii.   Data Covering the Period between 1st and 19th November, 2011

The collected axle load data for 3,755 vehicles was entered into the computer and checked manually for inconsistencies, and other logical errors. This data was then classified according to the number of axles, vehicle with the same number of axles were classified together. Equivalence factors for each axle were obtained by applying Liddle's Formula and average vehicle equivalence derived. Average Daily Traffic (ADT) for each axle class was obtained. Average Vehicle Equivalence Factors for each axle class were then multiplied with the respective ADT   values to obtain Cumulative Standard Axles. This information was then used to project the Cumulative Number of Standard Axles (CNSA) for the road.

In traffic projection, the design period was taken as 20 years at a growth rate of 3.25%. (The same design period and rate of traffic growth adopted by the Consultant for comparison purposes).

The CNSA, T, for the chosen design period, N (in years), was then obtained by applying the following formula:

$$T = 365t_1 \frac{(1+i)^N - 1}{i}$$

Where:

t_1 is the average daily number of standard axles in the first year after opening, and

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

Design life traffic, T, was then obtained as 12,858,865.73 CNSA. The design life traffic obtained by the Consultant for the road section around Juja is presented in (see pages 40 and 41, Tables 2.5 and 2.6) for both with and without by-passes respectively. The lowest value obtained by the design Consultant was 142.02 million CNSA representing the projected traffic along the Mangu-Juja (Right Hand Side) carriageway.

When the design traffic obtained from the weighbridge is compared with the Design Consultants', it is found to be very low representing approximately 8% of what was obtained by the Design Consultant. This can be attributed to the fact that only loaded medium and heavy goods vehicles are weighed at the Juja weighbridge. In the Consultants estimation of Design traffic, all vehicles with the exception of private cars (all passenger motor vehicles seating not more than 9 persons, including the driver) are considered. Vehicle weighed at the Juja weighbridge mainly comprise of medium goods vehicles carrying construction stone from Thika to Nairobi.

Table 5.6: Showing summary of average VEF, ADT and CSA of various axles

No. of Axles	Average VEF	ADT	% of ADT	CSA	% of CSA
1	0.44	1	0.51	0.44	0.07
2	2.92	16.84	8.52	49.12	7.88
3	2.38	113.32	57.35	269.66	43.25
4	2.25	3.68	1.86	8.3	1.33
5	5.26	5.37	2.72	28.24	4.53
6	4.7	57.37	29.04	267.8	42.95
	Total	197.58	100	623.56	100.00

From table 5.6, 57.35% of vehicles weighed comprise of 3 axles followed by 6 axle vehicles at 29.04% that is the number of vehicles with 3 axles is almost twice that with 6 axles. The proportion of vehicles with 1, 2, 4 and 5 axles is very small. In terms of proportion of CSA, 3 axle vehicles represent 43.25 per cent while 6 axles represent 42.95 per cent. This indicates that the 6 Axle vehicles produce twice the damaging effect as the 3 axle vehicles.

Out of 3755 vehicles weighed, 1130 vehicles weighed were found either to have excess axle weight or excess vehicle weight. This represents 30%.

5.5   Unique Challenges Encountered in Project Implementation
5.5.1   Keeping Slopes of Deep Cut Sections Stable

5.5.1.1 Soil Nailing
Introduction

Soil nailing consists of the passive reinforcement (no post-tensioning) of existing ground by installing closely spaced steel bars (nails), which are subsequently encased in grout. As construction proceeds from top-to-bottom, shotcrete or concrete is also applied on the excavation face to provide continuity (Carlos *et al,* 2003). This process creates a reinforced section that is in itself stable and able to retain the ground behind it. The reinforcements are passive and develop their reinforcing action through nail-ground interactions as the ground deforms during and following construction.

Soil nailing is typically used to stabilize existing slopes or excavations where top-to-bottom construction is advantageous compared to other retaining wall systems. For certain conditions, soil nailing offers a viable alternative for the viewpoint of technical feasibility, construction costs, and construction duration when compared to ground anchor walls.

One of the challenges of road construction at deep cut sections was how to keep the natural soil side slopes stable. In the Nairobi-Thika Road, This problem was experienced between km 6+550-km 7+730. This called for embankment slope tests that yielded the following results:

Table 5.7: Soil Nailing Test Results

Test	Result
Moisture content (%)	20.4
M.D.D (Kg/m^3)	1647
O.M.C (%)	200
Friction angle (Ø)	36
Cohesion C (KN/m^2)	40

Soil nailing technique was used to stabilize the embankment. Soil nailing is an efficient, effective and economical method of earth reinforcement, which allows a controlled improvement of the natural stability of the soil. This geotechnical engineering process comprises the insitu reinforcement of soils and has a wide range of applications for stabilizing excavations and slopes.

The technique involves the insertion of relatively slender reinforcing elements into the slope – often general purpose reinforcing bars (rebar) although proprietary solid or hollow-system bars are also available. Solid bars are usually installed into pre-drilled holes and then grouted into place using a separate grout line, whereas hollow bars may be drilled and grouted simultaneously by the use of a sacrificial drill bit and by pumping grout down the hollow bar as drilling progresses.

Basic Elements of a Soil Nail Wall

The most common practice for soil nailing consists of drilled soil nails, in which a steel bar is placed in a pre-drilled and then grouted. Figure 5.2 shows a cross-section of a typical soil nail wall where the following components are illustrated:
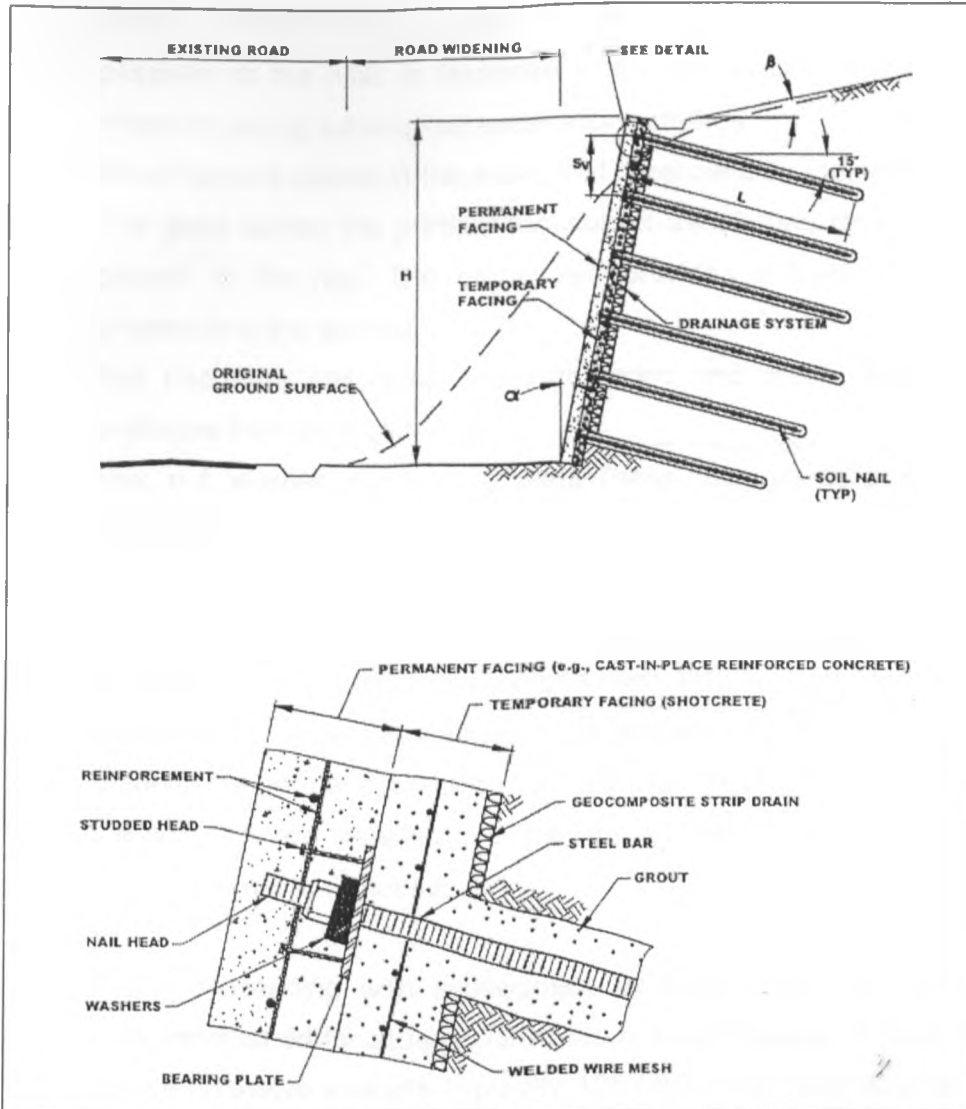
Figure 5.2: Typical Cross-Section of a Soil Nail Wall (Carlos et al, 2003)

where:

L = length of soil nail

Sv = distance between successive soil nails

α = wall face batter angle (from vertical)

β = slope angle

L, Sv, α and β are determined by carrying out slope stability analysis.

1. Steel reinforcing bars-The soil steel reinforcing bars are the main component of the nail wall system. These elements are placed in pre-

drilled drill holes and grouted in place. Tensile stress is applied passively to the nails in response to the deformation of the retained materials during subsequent excavation activities.

2. Grout-Grout is placed in the pre-drilled borehole after the nail is placed. The grout serves the primary function of transferring stress from the ground to the nail. The grout also provides a level of corrosion protection to the soil nail.

3. Nail head-The nail head is the threaded end of the soil nail that protrudes from the wall facing.

4. Hex, nut, washer and bearing plate-These components attach to the nail head and are used to connect the soil nail to the facing.

5. Temporary and permanent facing-The facing provides structural connectivity. The temporary facing serves as a bearing surface for the bearing plate and support the exposed soil. This facing is placed on the unsupported excavation prior to advancement of the excavation grades. The permanent facing is placed over the temporary facing after the soil nails are installed and the hex nut has been tightened. Both facings are never removed.

Before proceeding with subsequent excavation lifts, the shortcrete must have cured for at least 72 hours or have attained at least the 3-day compressive strength [typically 105 MPa (1500 psi)] (Carlos et al, 2003).

Soil Nailing Benefits and Limitations

Soil nailing has been used regularly over the last few years over traditional cut retaining walls due to several reasons:

Construction

- It is ideal for tight spaces.
- It can be used in irregular shapes.

- Less disruptive to traffic and causes less environmental impact compared to other construction techniques.
- Provides a less congested bottom of excavation, particularly when compared to braced excavations.
- Minimum shoring is required.
- Lower load requirements than tieback anchors systems.
- Eliminates the time and expense of placing steel piles.
- Can be used to repair other existing wall systems.
- It can be used on new constructions, as temporary structures or on remodelling process.
- Wall height is not restricted.
- Reduced right-of-way requirements.

Performance

- Soil nail walls are relatively flexible and can accommodate relatively large total differential settlements
- Measured total deflections of soil nail walls are within tolerable limits.
- Soil nails have performed well during seismic events owing to overall system flexibility

Cost

- Soil nail walls are more economical than conventional concrete gravity walls when conventional soil nailing construction procedures are used;
- Soil nail walls are typically less costly equivalent in cost or more effective than ground anchor walls when conventional soil nailing construction procedures are used;
- Shotcrete facing is typically less costly than the structural facing required for other wall systems.

Some of the limitations of soil nailing could be:

- Soil nail walls may not be appropriate for applications where very strict deformation control is required for structures and utilities located

behind the proposed wall, as the system requires some soil deformation to mobilize resistance; post tensioning of soil nails can overcome this shortcoming in most cases, but this step increases the project cost.

- In some instances soil might be overexposed prior to the installation of the nail;
- Granular soils might not be compatible with soil nailing;
- Soil nail walls are not well-suited where large amounts of groundwater seeps into the excavation because of the requirement to maintain a temporary unsupported excavation face;
- Soil nailing in very low shear strength soil may require a very high soil nail density;
- Soil nailing in sensitive soils and expansive soils for permanent long-term applications is not recommended;
- Construction of soil nail walls requires specialized and experienced contractors

5.5.2 Use of Natural Gravel as Sub Base Material

The unavailability of natural gravel in sufficient quantities for construction of this road was a key challenge. Some of the probable sources of natural gravel were found not to be within economic distances. Therefore, there is urgent need to research on alternatives to natural gravel as a construction material.

6 Conclusions and Recommendations

6.1 Conclusions

6.1.1 Alignment Soils Studies

On alignment soils studies, the research hypotheses sought to establish:

i. How engineering properties of soils vary with depth;

ii. Whether it is possible to establish the ruling CBR during construction at the design stage.

From the alignment soils studies, the following conclusions are made:

i. Soil properties vary both along the road and with depth. It was found that the ground varies continuously beneath a site, and it is not often possible to find sharp transitions from one type of material to another.

ii. It was established that it is not possible to establish the ruling CBR during construction at the design stage. Therefore, at construction stage, detailed investigations should be carried out which would be used for design purposes.

6.1.2 Axle loads

On axle loads studies, the research hypotheses sought to establish:

i. Whether there are any significant changes in axle loads of heavy and medium goods vehicles over the last 40 years;

ii. The effect of overloading on newly constructed pavements and what percentage of heavy and medium goods vehicles are overloaded;

iii. The challenges encountered when a road project of such magnitude as Nairobi-Thika road is implemented.

From the axle loads studies, the following conclusions are made:

i. Current vehicles are designed to carry more axle loads compared with vehicles that were in use 40 years ago. This means that the damaging effect of current vehicle traffic on newly designed pavements is more.

99

ii. Also evident is the fact that 40 years ago, trucks had a wider spectrum of loads as compared to current trucks.

iii. Design life traffic expressed in CNSA was found to be 12,858,865.73.This corresponds to traffic class T2. The design life traffic obtained by the Consultant for the road section around Juja is presented in (see pages 40 and 41, Tables 2.5 and 2.6) for both with and without by-passes respectively. The lowest value obtained by the design Consultant was 142.02 million CNSA representing the projected traffic along the Mangu-Juja (Right Hand Side) carriageway.

iv. The damaging effect due to average axle loads in year 2011 was found to be 4.1 times higher than that of year 1971.

v. 30% of all trucks weighed were found to have either excess axle load or excess vehicle gross weight.

6.2 Recommendations

The following recommendations are made:

i. There is need to carry out a review of the current road design manuals to accommodate the ever increasing axle loads due to advancements in new vehicle designs.

ii. Do research on alternatives to natural gravel as a construction material as the natural sources of gravel are getting depleted very fast given the high volumes of construction in the country. If this is not done, future road construction will be very high due to scarcity of gravel and availability within uneconomical leads.

7 References

- AASHTO, (1986). Guide *for Design of Pavement Structures;* American Association of State Highway and Transportation Officials.

- BELL F.G., (2004). *Engineering Geology and Construction.* Taylor and Francis Group, London and New York.

- CARLOS A., Victor E., David E. and Paul J. *Geotechnical Engineering Circular No 7*, Soil Nail Walls. Technical Report No.FHWAO-IF-03-017

- CES/APEC, (2007). *Feasibility Study and Detailed Engineering Design of Nairobi-Thika Road,* Draft Final Feasibility Report, June 2007.

- DEACON J.A, (1969). **Load Equivalency in Flexible Pavements**, *Proceedings Association of Asphalt Paving Technologists, Vol. 38.*

- DRAFT DESIGN MANUAL FOR ROADS AND BRIDGES, (2009). *Part III: Materials and Pavement Design for New and Reconstructed Bituminous, Gravel and Concrete Roads.* Ministry of Roads, Republic of Kenya.

- GICHAGA F.J., (1971). *A Study of Deflection Characteristics and Elastic Response of Flexible Pavements in Kenya.* M.Sc Thesis, University of Nairobi.

- GICHAGA F.J., (1979). *Structural Behaviour of Flexible Pavements in Kenya.* PhD Thesis, Department of Civil Engineering, University of Nairobi.

- GICHAGA F.J., (1982). *Laboratory Study of Deformation Modulus-Time Relationship for Various Subgrade Soils under Road Pavement Structure.* Kenya Journal of Science and Technoloy, Series A,Physical and Chemical Sciences. The Kenya National Academy for Advancement of Arts and Sciences, Vol 3,No. 2,1982.

- GICHAGA, F.J. and PARKER, N.A., (1988). *Essentials of Highway Engineering.* Macmillan Publishers.

- Huang, Y. H (2004). *Pavement Analysis and Design*. Pearson Prentice Hall.

- Kenya Gazette Supplement No. 65. Legal Notice No. 118, The Traffic Act, 12th September, 2008.

- MWEA S.K. and GICHAGA F.J., (2004). *Deflection Characteristics for Flexible Road and Airport Pavements in Kenya.*Presentation in the 8th *Conference on Asphalt Pavements for Soutern Africa,*Sun City South Africa,12th to 16th September 2004.

- MURUNGA P.A., (1983). *A Performance Study on Flexible Road Pavements in Kenya*. Msc Thesis,Department of Civil Engineering,University of Nairobi.

- O'FLAHERTY C.A., (2002). *Highways : The Location, Design, Construction & Maintenance of Pavements*. Butterwood Heinneman.

- ROAD DESIGN MANUAL, (1987).*Part 3: Materials and Pavement Design for New Roads.* Roads Department, Ministry of Transport and Communication, Republic of Kenya.

- Road Research Laboratory, (1968). Road Research Technical Paper No.80. *Pavement Design in the Tropics, Investigations of Subgrade Conditions under Roads in East Africa*.

- TRANSPORT RESEARCH LABORATORY, (1993). *Road Note 31, (Fourth Edition), A Guide to the Structural Design of Bitumen Surfaced Roads in Tropical and Sub-Tropical Countries.*

- YODER E.J. & WITZACK M.W., 1975. **Principles of Pavement Design.**

- PELL S. PETER, (1978). *Developments in Highway Pavement Engineering*. Applied Science Publishers Limited.