MODELING MECHANISTIC-EMPIRICAL ROAD DESIGN: THE CASE FOR KENYAN ROADS

BY

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Abstract

Regular revision of road design manuals is a recommended practice in pavement management, and Kenya has recently completed a review of hers. The outcome of the process is an enhanced but empirical manual with rigid models. The reviewers (BCEOM *egis*, a geotechnical consultancy) however strongly recommended the use of interactive mathematical models of a mechanistic-empirical (ME) nature in road design for the country in future, since ME models more accurately predict pavement performance than the rigid ones currently in empirical manuals. Using ME models in road design in Kenya now is a challenge since the climatic models in ME design require inputs of hourly intervals, yet most weather stations keep only daily interval records. This study investigated the possibility of adapting the existing ME mathematical models for use in road design in Kenya. We achieved this by rigorously analyzing the differential equations and other equations contained in the ME mathematical models. Simulating hourly weather data from daily data was applied for some weather elements, while adaptation of the governing equations in the models to take inputs of daily intervals was done for the others. Secondary climatic data obtained from the Kenya Meteorological Department and the Weatherbase websites were used as inputs into the models. The study established that with suitable adaptation, the existing climatic data can be used as inputs into the models with little loss in the reliability of their predictions, hence Kenya can successfully embrace the ME pavement approach. The findings of this study could serve as an initial step towards the realization of the reviewers' recommendations on use of ME mathematical models over empirical manuals.

Chapter 1

Introduction

1.1 Overview

Mathematical models find use as tools to simulate real life situations and to forecast future behavior in the physical and biological sciences, business and other fields of study. In engineering, specifically in road design and construction, such models fall under two broad categories; the purely empirical and the mechanistic-empirical (ME) types. Whatever type of model employed by a pavement designer, it is often necessary to regularly refine the model to cope with changes in variables such as traffic loading, geometric designs, environmental factors and construction materials.

While they appear on the surface as simple, pavements are complicated civil engineering structures, more so from a design perspective. This is drawn from the understanding that a pavement designer has to take into consideration that the pavement structure will slowly and gradually deteriorate towards a sure final failure lasting several years. It can thus be said that 'pavements are designed to fail'. This is unlike other structures that either last forever or fail suddenly at some point [38]. Further, a number of variables have to be considered together. One of these variables is the uncontrollable climatic one, chiefly temperature and precipitation.

Research in pavement engineering has shown that the use of purely empirical mathematical models for pavement response and performance prediction is fraught with large margins of unreliability [2]. This is so because the models are static, rigidly relying on the statistical correlations for their implementation and do not fully take into account the unique site-specific micro-climatic conditions through which the roads span. Further, they do not integrate all the variables involved, thus do not fully appreciate the complexity involved in pavement modeling.

A more promising approach to pavement design formulated in recent years is the mechanistic-empirical (ME) method whose development has been made easier by rapid advances in computing technology [38]. Heavily reliant on interactive mathematical models for its functioning, the most recent (hence most reliable) ME methodology uses as inputs into the models hourly climatic data, traffic loadings, construction material types and hydro-geological records, such information being site-specific. Its implementation involves two stages. The first step (mechanistic) formulates the mathematical models to calculate the responses (stresses, strains and deflections) on the pavement as a result of the traffic loading, design features and weather conditions. This step relies on mathematical procedures such as finite element and finite difference analyzes depending on the material characteristics of the pavement layer analyzed. The final stage (empirical), the more rigorous of the two, predicts the performance (rutting, cracking, roughness and permanent deformations), whose evaluation is achieved by empirically relating the responses earlier determined to the rates of deterioration over a prescribed time period.

To accommodate for changes in traffic, construction materials and geometric designs, Kenya has recently updated her road design manuals, a regular pavement management procedure. The review process has produced an enhanced empirical manual. The reviewers in addition strongly recommended that in future, the country adopts the more reliable ME method for its road design. However, one of the limitations to employing ME mathematical models in Kenya presently is the requirement by the models that the weather inputs into them be of hourly intervals. Further, the inputs comprising temperature, precipitation, wind speed, humidity, ground water table depth and percentage sunshine need to be site-specific for accurate analysis and prediction of pavement performance. While some hourly records are available, most weather stations in the country keep daily-interval records.

This study made the first attempt at embracing the mechanistic-empirical approach in road design in Kenya. It explored the feasibility of using the available climatic records as inputs into the ME models. We analyzed the existing weather related mathematical models in road design and made recommendations on how best they can be adapted to suit the Kenyan situation. It is hoped that this effort will help serve as a first step towards realization of the reviewers' recommendations on using ME mathematical models in designing, constructing, and maintaining roads in the country.

1.2 Statement of the problem

In a bid to attract investment and spur economic growth, Kenya has over the years been increasing her budgetary allocations to road network building and expansion. Such massive public investment in infrastructure should be accompanied by prudent management practices to inform future expenditure on maintenance, and hence ensure a return on investment.

A standard procedure in road network management involves regular review of the design manuals, and Kenya has recently updated hers. The outcome of this process is an enhanced traditional empirical design manual, but the reviewers also recommended that the country, in the future, moves towards the use of interactive mechanistic-empirical(ME) mathematical models that empirically relate the performance of roads to their response to traffic loading and climatic influences. This is because the latter more accurately predict the performance of roads than those in the empirical manuals. The ME models rely on detailed hourly interval weather records that stretch back several years as their inputs to ensure reliable predictions. However, such climatic data are not currently easily available in Kenya. This study analyses the feasibility of adapting the mathematical models in ME design to accommodate the current weather records in Kenya for improved design.

1.3 Objective of the study

The main objective of this research is to explore the possibility of adapting the existing mathematical models for use in the mechanistic-empirical design of pavements in Kenya from a climatic perspective. We analyse the existing climate-related models, particularly those used in road design, and investigated the feasibility of using such models in Kenya, applying the available climatic data as inputs.

1.4 Significance of the study

The most recent review of the country's road design manuals recommended a transition from the current empirical design methods to the mechanistic-empirical approach in future designs, due to the many benefits derived from using interactive mathematical models over the traditional manuals in predicting pavement response and performance. The study is a pioneering endeavour in this design shift, since it laid a firm foundation for employing the ME design paradigm in Kenya and hopes to contribute towards equipping the country's pavement designers with a more reliable design tool.

1.5 Research methodology

This research involves a comparative analysis of the mechanistic-empirical design approach and the current empirical design method used in Kenya, more so from a climatic perspective. Climatic data for Kenya obtained from secondary sources, chiefly the Weatherbase website, and road weather records from the Long Term Pavement Performance Program (LTTP) sites in the USA are used. A rigorous literature survey was conducted to assess the feasibility of adopting the ME methodology under prevailing circumstances in Kenya. Finally, the study formulates climate-related models that would enhance the transition from the empirical procedures to the mechanistic empirical method for Kenya.

1.6 Organization of the study

Chapter 1 of this thesis provides an overview of the study, setting out the methodology, objectives, significance of study and the statement of the problem.

Chapter 2 is an exploration of the literature pertinent to the study, while in Chapter 3, we lay down the fundamental concepts relating to pavements, including the pavement types, philosophy of pavement design and the twin procedures of response and performance evaluation. Considered here are the concepts of structural and functional conditions of roads and the various design approaches. The chapter concludes by giving a numerical example of a design using the most widespread approach, the empirical method.

Chapter 4 considers the mechanistic empirical design of roads, the new system that the study hinges on. Its evolution and adoption are evaluated, with the role played by the environmental factors in it highlighted.

Of significance, the mathematical models that define the approach are considered, with the chapter concluding by giving an example of a design using the method.

In Chapter 5, the pavement situation as it prevails in Kenya presently is analyzed. The administration, classification and financing of roads countrywide is considered with the challenges facing the sector highlighted.

The main work done in this research is contained in Chapter 6, setting out the various climatic factors in ME road design, and the best possible alternatives of accommodating them within the Kenyan context. Mathematical models applicable to Kenya are proposed.

In Chapter 7 we summarize the study by matching the findings to the objective and make, recommendations that should be adopted to meet the original research objectives and for further studies into the subject.

Chapter 2

Literature review

Our study borrows from a wide variety of areas in Mathematics, Engineering, Physics and Climatology. These include differential equations, mechanics, thermodynamics, materials science, fluid mechanics and soil mechanics among others.

Compared to other civil engineering design problems, pavement design has been slow in moving away from purely empirical methods [38]. For portland cement concrete (PCC) pavements, the use of analytical methods to estimate their response dates back to 1913 when Bradbury [5] published his paper on reinforced concrete pavements, yet similar methods are in use to date in many parts of the world. This lag in transition to more current methods of design may be partly attributed to the complex interaction between different variables involved in designing roads Mamlouk[26]. Such would include traffic loading, geometric design, construction materials and weather factors. Whatever the approach employed, different mathematical models have been used by pavement designers.

In the past, several countries have used the 'Catalogues' as a design tool. The more famous one, the French version "Catalogue de Structures types de Chaussees" [27], is a tabular representation relating the traffic class T_i, with the soil class S_i. The latter depends on the type of soil and the drainage conditions. The catalogues in their basic form set out the type of materials required for each layer (subgrade, subbase, base and surfacing), the thickness of the layers, and match this with the weight of the expected traffic. As a tool for road design, catalogues are confined to construction of new roads, do not dynamically take into consideration potential increase in traffic over the years and cannot predict the functional and structural state of a pavement structure at any time during its design life. Further, rigidity exist in using them since they spell out the required materials for the layers and do not allow for adjustments for better or poor materials (Ullidtz [38]).

The California Bearing Ratio (CBR) was one of the first empirical attempts at pavement design [45], and is based on the strength of the soil. Developed between 1928 and 1929, its application is simple. Each of the layers to be used in the pavement has its CBR determined, and the thickness required for the material in the layer above it is read from a chart or calculated from the equation below;

$$t = P\{\frac{1}{8.1CBR} - \frac{1}{\pi p}\} \quad (2.0.1)$$

where;

t= thickness in inches

P= single wheel load in pounds

p= tire pressure in pounds per square inch (psi)

The initial curves relating the quantities above were obtained from testing soil from failed and satisfactory pavements, an either/or situation. Unlike the catalogues, it explicitly defined pavement failure as lateral displacement of subgrade material due to pavement absorbing moisture, differential settlement of materials underneath the pavement and excessive deflection of materials under it [45]. For all its superiority to the catalogues, the CBR as a method fails to predict the pavement performance, or the economic consequences of different actions (Ullidtz [38]).

Road Notes (RN) 29 and 31 [36] were attempts by the British road engineers at filling the gaps in the CBR and the Catalogues approaches. RN 29, first published in 1960 was a guide for the structural design of roads under the British conditions of climate, materials and traffic loading. It was based on full scale experiments carried out in the U.K, the CBR method and the AASHO Road Test. While Road Note 29 defined the design standards for new roads, RN 31 dealt with the structural design of bituminous pavements in tropical and sub-tropical countries. A major attempt by RN 31 was to quantify the equivalent number of standard 8200 kg axles, obtained from the AASHO Road Test, whose terminal condition is determined by the subjective index, Present Serviceability Rating (PSR) of 2.0 (Ullidtz[38]). Both Notes however failed to quantitatively and objectively define the terminal condition of a pavement structure, instead opting for three subjective road-state classifications; sound (no cracking ,with ruts between 5mm and 9mm), critical (with no major cracks and rutting to depths of between 10mm and 19mm) and failed (with major cracks, and ruts greater than 20mm) [2].

In the late 1950's the American Association of State and Highway Officials (AASHO) commissioned a number of road tests (hereafter referred to as Road Test) near Ottawa, Illinois [2]. A guide was published based on the limited empirical performance equations developed from the Road Test. Later referred to as the 'old' AASHTO method, it gained much use from 1960 till 1986 when the pavement design community noted among its shortcomings the failure to account for changes in traffic loadings, materials, and design features as well as the direct consideration of site-specific climatic effects, since the equations were developed from one geographical location (Ullidtz [38]). At the heart of this was the realization that roads were built for the benefit of road users, thus the acceptability of a pavement condition depended on its rating by the users. A rating method was thus devised called the Present Serviceability Index (PSI), gauging a road on a scale of 0(impassable at normal speed) to 5(perfect), and based on a panel of road users taking a ride on a road section (Ullidtz [38]).

In spite of the vague and subjective nature of the Road Test, the following equations were developed from the Road Test;
For flexible pavements,

$$PSI = 5.03 - 1.9\log(1 + SV) - 1.38RD^2 - 0.01\sqrt{C + P} \qquad (2.0.2)$$

and for rigid pavements,

$$PSI = 5.41 - 1.8 \log(1 + SV) - 0.09\sqrt{C + P} \qquad (2.0.3)$$

where;

SV= slope variance (slope measured over 1 foot)

RD= rut depth in inches measured with a 4-foot straightedge

C= lineal/straight feet of major cracking per 1000 square feet

P= bituminous patching in square feet per 1000 square feet area

Improvement on equations 2.0.2 and 2.0.3 were made obtaining good correlations taking into account the slope variance, or roughness only. Yoder and Witczak [45] used correlation coefficients of 0.88 and 0.92 respectively to obtain the equations;
For flexible pavements,

$$PSI = 4.29 - 0.40\sqrt{SV} \qquad (2.0.4)$$

and for rigid pavements,

$$PSI = 4.81 - 0.47\sqrt{SV} \qquad (2.0.5)$$

Attempts at overcoming the failures of the 'old' AASHTO method gave rise to three major revisions to it in 1972, 1986 and 1993. The latter two reviews had refinements in material input parameters, design reliability and empirical procedures for rehabilitation design [2]. The said reviews however hinge on the Road Test, thus rely solely on limited empirically based performance equations [2].

In recognition of the limitations of the AASHTO Guides, the pavement engineering community drawn from academia, industry and public and private agencies in 1996 formulated a framework for the development of a design process based as fully as possible on both mechanistic and empirical principles [2], two approaches hitherto employed independently. The outcome of this is the Mechanistic-Empirical Pavement Design Guide (MEPDG) procedure. The core components of MEPDG approach are the traffic loading, foundations, materials characterization and environment, all used to predict the instantaneous performance and response of a pavement structure during its design life [2].

The MEPDG [2] adopts an iterative approach, taking in the core components stated above as the inputs, to develop a trial design based on mechanistic response models (for the environment, critical stresses, strains and deflections) and empirical performance models (relating to damage, distresses and smoothness). A reliability level is then defined and a performance criteria used to evaluate the achievement (or lack of it) of the initial set goal. In the event of failure to meet the defined requirements, a new trial design is devised iteratively until an acceptable level is attained. In this situation, a final design is developed. Of particular note is the complete integration of climatic factors into the whole procedure.

Long recognized as a key component of the road design process, climate has had its own share of models, mostly precipitation and temperature related. The Collaborative Historical African Rainfall Model (CHARM) is one such model and it overcomes the lack of reliable gauge observation in the continent and other similar regions. The CHARM equations blend global numerical weather prediction (NWP) model reanalysis fields, interpolated station data, and output from an orographic model to generate 36 years of daily precipitation grids with wall-to-wall coverage of Africa [10]. Though mostly used in crop modeling, its output can equally be applicable to road design as it provides reliable precipitation estimates.

The mathematical model for pavement performance (MMOPP), a computer based program first developed in 1976 [38], has a constitutive model for temperature (mean weekly) prediction given by the cosine function;

$$T = \frac{T_1 + T_2}{2} + \frac{T_1 - T_2}{2^{\frac{\cos(U-U_0)}{26\pi}}} \tag{2.0.6}$$

where;

T_1= maximum weekly air temperature during the year

T_2= minimum weekly air temperature during the year

U= week number (counted from new year)

U_o= week number corresponding to the maximum temperature, T_1.

According to Barker et al. [4], the temperature of the asphalt (upper layer), T_{asp} (in ^{0}C), is then obtained from the equation above as;

$$T_{asp} = 1.2T + 3.2 \tag{2.0.7}$$

One of the key strengths of the MEPDG is its ability to fully integrate weather models into the road design process with high levels of reliability in making predictions, since all the models are calibrated and validated using actual climatic records. For this it relies on weather data obtained at over 800 locations (in North America) as part of a wider program called long term pavement performance (LTPP) study, a 20-year effort which began in 1986 [2]. In this, accurate predictions on pavement conditions are made based on the previous 10 years' hourly weather records. Owing to the massive quantity of data involved, an analysis tool called the Integrated Climatic Model (ICM) was developed in 1989 (with later revisions) to simulate the temporal variations in temperature, moisture and freeze-thaw conditions in the pavement structure and their effects on the material properties.

In MEPDG, an improved version of ICM called the Enhanced Integrated Climatic Model (EICM) is incorporated [2]. The EICM itself is a model of models comprising;

- The Climatic-Materials-Structures(CMS) model, developed at the University of Illinois
- The Infiltration and Drainage(ID) model, developed at Texas A and M University and

- The Cold Regions Research Engineering Laboratory (CRREL) model developed by the US Army.

The EICM models are discussed in 6.3.2., but the Guide [2] describes the EICM thus;

> "The EICM is a one dimensional coupled heat and moisture flow model based on both finite difference and finite element techniques with its thermal boundary conditions as the climatic and solar inputs at the surface together with the constant deep ground heat source."

The study of heat and moisture flow in soils has elicited a lot of interest among engineers and scientists over the years, with several models proposed. Some of these include the one dimensional heat flow equation due to Wilson [41] describing heat flux due to conduction and the latent heat transfer due to a change of phase, and the air pressure gradient (in one dimension) model developed by Dakshanamurthy and Fredlund [34]. The models take into account such soil properties as the latent heat, thermal conductivity, specific heat capacity and diffusion coefficient of the water flowing through the soil. Combined with ambient parameters like the atmospheric pressure, rates of evaporation and the air temperature, such models have been used to describe heat flow underground.

Moisture flow on the other hand has led to the formulation of equations that attempt to account for the moisture content at different depths in a soil profile. Darcy's law and its affiliated equations have found use in harnessing phreatic resources. For pavement design, the soil water characteristics curve (SWCC) equations have been more applicable with those developed by Fredlund and Xing [2] being included in the EICM. A more elaborate treatment of the SWCC's and other related models, and their central role in the design process is considered in Section 6.2.

Wilson attempted combining the effects of moisture and temperature in a single model giving rise to the composite Wilson equation expressed as;

$$\frac{\partial u_w}{\partial t} = -C_w \frac{\partial u_a}{\partial t} + C_v^w \frac{\partial^2 u_w}{\partial z^2} + C_v^{wv} \frac{\partial^2 \bar{u}_v}{\partial z^2} \tag{2.0.8}$$

where;

u_a= air pressure

u_w= water pressure

$\bar{u}_v$= the vapor pressure

C_w= interaction coefficient associated with liquid water phase

C_v^w= consolidation coefficient with respect to water phase

C_v^{wv}= consolidation coefficient with respect to vapor phase

None of the above models is as comprehensive as the EICM [2] in accounting for the climatic effects in pavement design since in addition to the moisture and temperature, it also incorporates wind speed, sunshine, humidity and ground water depth in the process. This study thus centred on the EICM as a component of the overall MEPD and how best to position Kenya to transition to the new design paradigm.

Chapter 3

Fundamental Concepts

3.1 Introduction

A pavement refers to the layered structure forming the surface of a road [2]. This chapter introduces pavement-related concepts that are helpful in the understanding of the ideas we have come up with in the later chapters.

In Section 3.2, we briefly describe the conventional types of pavements with a specific focus on those in Kenya since this will help us in understanding the implications of different design approaches.

In Section 3.3, the philosophy of pavement design is highlighted, with emphasis put on its uniqueness and the twin processes of response and performance, and their evaluation.

In Section 3.4, we contrast the structural and functional conditions of a pavement to clarify the ambiguity common in them, while Section 3.5 gives an overview of some pavement design approaches.

The chapter concludes by giving a numerical example of an empirical pavement structural design (of the type the Kenyan manuals are based on) in Section 3.6.

3.2 Pavement types

Broadly, pavements are classified into two categories, with the determining factor being their reaction to the loads they are subjected to. All hard surfaced pavement types can be termed as flexible or rigid. Flexible pavements, generically referred to as asphalt concrete (AC) pavements, are those which are surfaced with bituminous (or asphalt) materials [11]. These can be either in the form of pavement surface treatments (such as a bituminous surface treatment (BST) found on roads with lower traffic volumes) or, hot mix asphalt (HMA) surface courses (generally used on higher volume roads such as the Northern Corridor linking the port of Mombasa to Uganda). These types of pavements are called "flexible" since the total pavement structure "bends" or "deflects" due to traffic loads. An AC pavement structure is generally composed of several layers of materials which can accommodate this "flexing". Some properties of this bending that will be useful to our study are the resilient modulus, M_r, and dynamic or complex modulus, E, as described later in section 3.6. This study focussed on AC pavements since nearly all roads in Kenya are of this type.

On the other hand, rigid pavements are composed of a portland cement concrete (PCC) surface course. Such pavements are substantially "stiffer" than flexible pavements due to the high modulus of elasticity of the PCC material. Further, these pavements may be reinforced with steel, which is used to reduce or eliminate joints. The more famous of the PCC road sections in Kenya is Mbagathi Road in Nairobi.

AC and PCC pavements distribute load over the subgrade in different ways. Rigid pavement, because of PCC's high elastic modulus (stiffness), tends to distribute the load over a relatively wide area of subgrade. The concrete slab itself supplies most of a rigid pavement's structural capacity. Flexible pavement uses more flexible surface course and distributes loads over a smaller area. It relies on a combination of layers for transmitting load to the subgrade. These load distribution mechanisms are illustrated in the Figures 3.2.1 and 3.2.2 in the subsequent pages. It is generally more expensive to construct a rigid pavement than a flexible one, but the rigid ones usually have a design life nearly twice that of the flexible and lower maintenance costs [9]. The construction costs for a pavement decreases along the depth profile and is a function of the materials used. Figure 3.2.3 illustrates the property of the layers of a generic pavement structure.

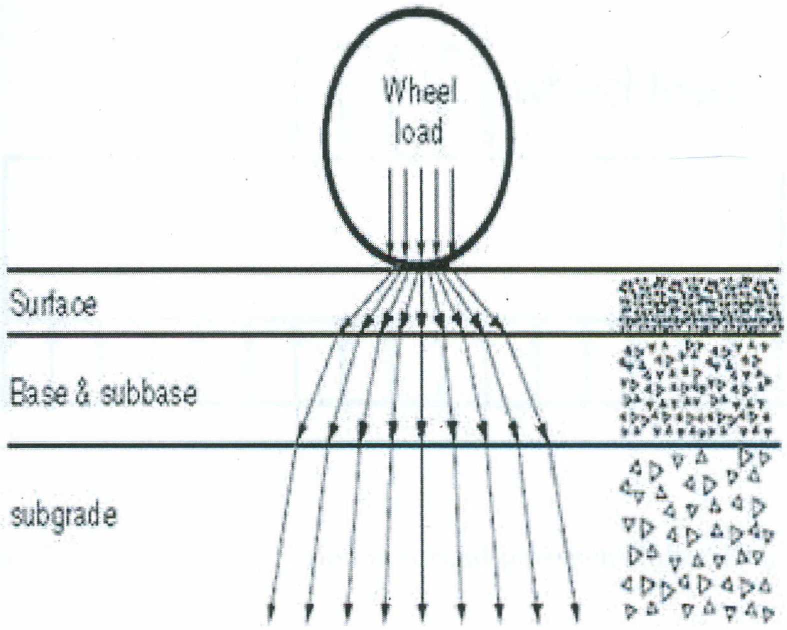


Figure 3.2.1: Load distribution in a flexible pavement (after Fwa[11])

3.3 Philosophy of pavement design

The design of pavements is an act in optimization that seeks to develop the most economical combination of pavement layers (in relation to both thickness and material gradation) to suit the soil foundation and the traffic to be carried during the design life. Whatever the design approach employed, it is necessary to consider that, as structures, pavements are subjected to moving and repetitive traffic loads with each load causing an amount of damage which finally accumulates to cause pavement failure.

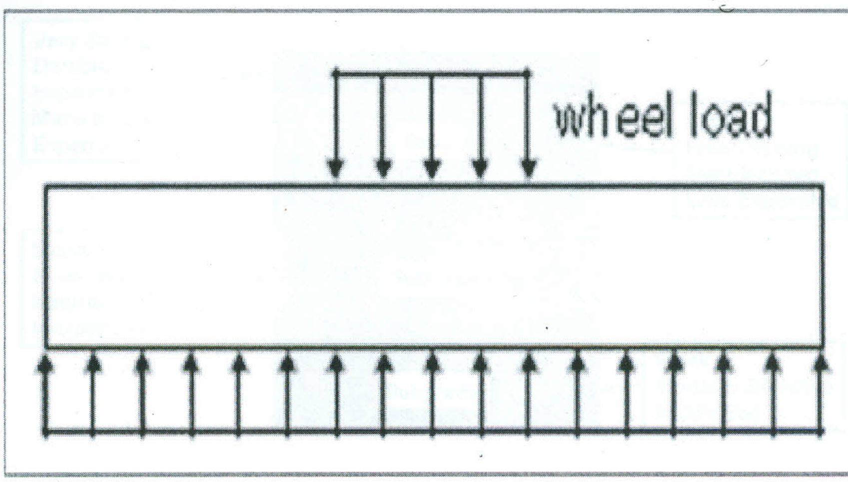


Figure 3.2.2: Load distribution in a rigid pavement (after Fwa[11])

3.3.1 Uniqueness

From the foregoing, pavements can thus be said to be 'designed to fail', since it is a fact that the cumulative loading eventually leads to a deterioration to an unacceptable level at some point [38]. This is unlike other engineering designs which are intended to last forever or fail suddenly, the latter usually being accidental. In this regard, the concept of 'design life' has to be included. Defined as the period that a newly constructed pavement takes before deteriorating to an unacceptable functional and structural condition where there would be need for a reconstruction or major repair work, the average design life for flexible pavements is about 20 years [42].

It should be noted here that the concept of design life does not preclude minor routine maintenance, which should be done on the road from right

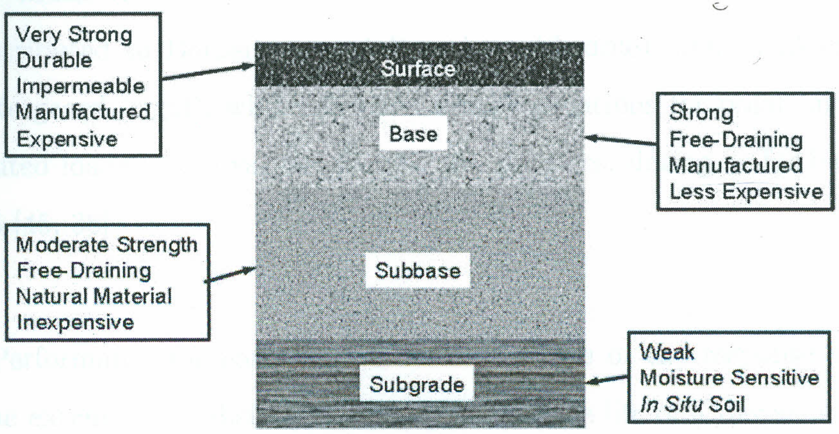


Figure 3.2.3: Typical pavement profile (after von Quintus[35])

after the construction. Indeed the Kenyan road design manual [28] states thus;

> "It is assumed that, during the design period, only ordinary maintenance work will be carried out, this will comprise shoulder and drainage system maintenance, erosion and vegetation control, localized patching and periodic resealing."

3.3.2 Response and performance

The response of a pavement refers to the mechanics-based analysis of its reaction to the traffic loading and environmental influences. This involves the calculation of the stresses, strains and deflections, more so at critical locations throughout the pavement profile. Clearly a daunting procedure, response determination thrives on a number of assumptions e.g. that the pavement is homogenous, linear elastic, isotropic and semi-infinite

[38]. Much of the early work on response determination for pavements is attributed to Boussinesq and Burmister (flexible), and Winkler and Westergaard (rigid), who developed sets of equations for point and distributed loads in the various dimensions and axes, dating as far back as 1885 [45, 38].

Performance for pavements is a consequence of the response and it is the extent of the distresses developed over the life of a pavement [15]. Such distresses include rutting, cracking, roughness and skid resistance among others. Unlike response, it is much more difficult to analytically quantify performance owing to the time scale considered, the variabilities and uncertainties in parameters involved and the repetitive and cumulative nature of the inputs required. For this reason, we often talk of performance prediction and in most design methods include a defined level of reliability (in percentages) explicitly in the process.

Reliably correlating the performance to the responses has been a challenge for most design systems, but the ME method has gained wide acceptance in recent years as a trailblazer in this effort [2].

3.4 Functional and structural conditions of roads

Two viewpoints exist in judging the state of a roadway, and these depend on either the users or the engineers and other specialists involved in the design process. To a user, the utility aspect also referred to as the functional condition of the road overrides other concerns. In this, the 'ride quality' comes into focus as judged by the roughness, safety considerations (skid resistance, visibility) and noise emission. Overall, a road user desires to minimize the costs associated with using the road.

In addition to the functional condition, the pavement engineering community concerns itself with the structural condition, more often referred to as the 'bearing capacity' [38]. This is a factor of the materials used in the construction and the thicknesses of the various layers, and is directly related to the expected traffic levels the pavement will carry.

It is worthy to note that any condition assessment for pavements must build distinct indices for both functional and structural analysis. The complexity in this task is more evident when one considers that the two conditions can act in reverse at times e.g. a road currently thought of as functionally sound (as judged by surface smoothness) may hide structural deficiencies in the underlying subgrade that would lead to degradation after a short while. Similarly a structurally robust rigid pavement usually has discontinuities at the joints leading to subtle bumpiness and increased noise during use. Further, some of the factors involved are difficult to quantify mathematically.

3.5 Some pavement design paradigms

As stated in Chapter 2, design methods have evolved over the years in response to the changing traffic and as a function of advancement in technology. Some of the conventional pavement design procedures used in the past and at present include the Catalogues, the California Bearing Ratio (CBR) method, the Shell Pavement design method, Road Notes 29 and 31 (British in origin), the Highway Design Manual III (HDM3) of the World Bank, the Aircraft Classification Number (ACN) method for airfield pavement and the American Association of State Highway and Transportation Office (AASHTO) 1972, 1986, and 1993 guides [38]. Most of the procedures are based on data obtained from test sites, from which empirical mathematical equations are formulated to guide future design processes. The most famous of these tests is the American Association of State Highway Officials (AASHO) Road Test [26].

Past and present designs in Kenya have been hybrids, leaning in parts towards the Road Notes, the CBR and, largely, the AASHTO guidelines [28]. In most countries in the world, purely empirical methods have been dominant, and to more clearly grasp the process involved, the next section is devoted to showing how to structurally design a pavement using the empirical approach.

3.6 Example of empirical design

The following example is abridged from the Geotechnical Reference Manual [15], and is for the structural design of a high volume AC pavement using the 1993 AASHTO Guide. The general empirical equation used is given as;

$$\log(W_{18}) = Z_R S_o + 9.36\log(SN+1) - 0.20 + \frac{\log[\frac{\Delta PSI}{4.2-1.5}]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\log(M_R) - 8.07 \qquad (3.6.1)$$

where;

W_{18}= number of 18kip equivalent single axle loads (ESALs)

Z_R= standard normal deviate (a function of design reliability level)

S_o= average standard deviation (a function of overall design uncertainty)

ΔPSI= allowable serviceability loss at the end of the design life

M_R= subgrade resilient modulus

SN= structural number (a measure of the required structural capacity)

While it is implicit in the equation, the structural number SN, is the unknown required output in empirical design with all the other four being known input parameters. Equation 2.6.1 is thus solved implicitly for SN, whose significance is in determining the required layer thicknesses using the equation;

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad (3.6.2)$$

where:

D_1, D_2, D_3= thickness in in. of the surface, base and subbase courses respectively.

a_1, a_2, a_3= structural layer coefficients of the surface, base and subbase courses respectively.

m_2, m_3= drainage coefficients of base and subbase courses respectively

As stated before, this design method puts emphasis on the various layer thicknesses, and listed below is a step-by-step procedure for determining the thicknesses. Numerical values are assigned where necessary.

1. First, the analysis period corresponding to the design life is set. In our case this is 30 years.

2. The design traffic in terms of the ESAL is evaluated. An ESAL is a measure of the damage caused in one pass by the axle of a truck of 18,000 pounds (18kip). For this example, the projected traffic cause damage equal to 11.6 million ESALs in 30 years. The equation used to estimate ESALs is given as;

$$ESAL = (ADT_0)(T)(T_F)(G)(D)(L)(365)(Y) \tag{3.6.3}$$

where:

ADT_0= average daily traffic at start of design period

T= percentage of trucks in ADT

T_f= truck factor (number of 18kip ESALs per truck)

G= truck growth factor

D= directional distribution factor

L= lane distribution factor

Y= design period in years

3. Since this is a heavily trafficked road, a high reliability level of 90% is next assigned, giving rise to standard normal deviate, Z_R, and overall standard deviation ,S_0, of -1.282 and 0.45 respectively.

4. We then determine the allowable loss of serviceability as a result of the traffic, Δ PSI=1.7

5. Next, the seasonal average of the subgrade resilient modulus M_r, is determined. A more comprehensive treatment of M_r follows in section 5.2.2. This example picked on a value of 7,500psi.

6. Depending on the materials used, the layer structural coefficients, a_i, are then determined for each layer. We take a_1=0.44 for AC and a_2=0.17 for aggregate base layers. Similarly the unbound layers' drainage coefficients are determined (m_2=1.0)

7. Equation 2.6.1 is then solved to obtain the overall structural number for the pavement system. A calculation of the same gives this value at 5.07.

8. Finally, the layer thicknesses are obtained. This involves using equation 2.6.1 with the M_r set equal to the granular base resilient modulus, E_{BS}=40,000psi, then solving for the required AC layer structural number giving, SN_1=2.62. SN_1 is then converted to thickness

using the relation $D_1 = \frac{SN_1}{a_1}$=5.95 in. For the granular base thickness, the remaining required SN is assigned, and a similar procedure carried out to obtain its thickness.

From above, it is clear that the empirical design method is a fairly simple algorithm based upon generalized equations largely drawn from the AASHO tests. This generalization provides one of its weaknesses as it fails to consider unique environmental conditions since the tests were done in one geographical area. An example on ME design in section 4.7 will contrast this.

Chapter 4

Mechanistic-Empirical Pavement Design

4.1 Introduction

In this chapter, we briefly consider the variants of the mechanistic-empirical pavement design approach in different countries in Section 4.2. Section 4.3 critically examines the American (AASHTO) version of ME, on which the study is based, while Section 4.4 highlights the significant role the environmental factors play in ME design. The response and performance mathematical models contained in the ME design suite are presented in Sections 4.5 and 4.6 respectively. Finally, the chapter ends by giving a typical example of ME design in Section 4.7.

4.2 Historical perspectives

The mechanistic empirical (ME) design has long been recognized as the most promising approach to road design and management in a number of countries. Ullidtz [38] notes that it was introduced in Denmark in early sixties as Danish Standard, adopted by Shell, and introduced in the USA by the Asphalt Institute in 1982. Closer home, Theyse et al [37] report that the method has been available in South Africa since the 1970's. However, its use as a design tool has been hampered by difficulty in determining the elastic moduli of the pavement materials, over-sensitivity to changes in input variables leading to counterintuitive results and thus unrealistic structural capacity estimates. In nearly all instances, the failure of the early ME versions to dynamically relate environmental influences to pavement material characteristics and hence performance over many years of service have curtailed their reliability and use.

In recognition of this and the shortcomings of the other design methods, the pavement engineering practice drawn from academia, industry and government road agencies in the USA in 1996, under the aegis of AASHTO Joint Task Force on Pavements (JTFP) laid the foundation for a new ME pavement design method, which adds a more scientific approach to pavement design and improves on the earlier versions [30]. This gave rise to the National Cooperative Highway Research Program (NCHRP) Project 1-37A which developed the design guide and its accompanying software. Our study focussed on the AASHTO version of ME design, and any reference to the method is construed to be of this approach.

4.3 Overview of methodology and its adoption

The ASHTOO ME design brings together two hitherto separate approaches in the design of roads and uses them in a stepwise manner. It is heavily reliant on mathematical models for its implementation. Based on laboratory pavement material properties, the mathematical models are used to predict deflections, stresses and strains (collectively referred to as responses) within the pavement when subjected to wheel loadings [30] and changes in environmental conditions. This makes up the first step of the analysis (mechanistic) which relies on physical laws of mechanics. In the mechanistic stage, the mathematical tools employed are either finite element analysis (FEA) or the multi-layer elastic theory (MLET) . FEA and MLET are used to analyze the pavement responses to both loadings and environmental influences.

Another set of mathematical models (transfer functions) use the values of responses computed to empirically predict pavement performance. This is usually through the determination of the distresses for both flexible and rigid pavements including roughness, rutting, cracking, faulting and punchouts. The step involving the evaluation of the performance is called the empirical stage, and depends on the statistical prediction of the pavement condition at a future time based on the critical responses obtained earlier.

In using the AASHTO ME design method, the following steps are followed [2]:

1. All the inputs are defined. These include traffic, environment, materials and other general design inputs.
2. A trial pavement part is picked for analysis.
3. The properties of the materials in the pavement layers are defined.
4. The pavement responses, both environmental and structural are analyzed based on traffic loadings and environmental influences on a season by season basis. This is done to include the variations in traffic loadings, environmental conditions and material behavior over the period.
5. The critical pavement responses are empirically related to damage and distress. The damage is computed cumulatively over the design life.
6. A reliability level is assigned and the predicted distresses adjusted accordingly.
7. At the end of the design life, the predicted distresses are compared with the design limits. If the set limits are not met, the trial pavement section is adjusted and steps 3) through 7) repeated until they are met.

The schema on figure 4.3.1 gives an outline of the ME procedure.

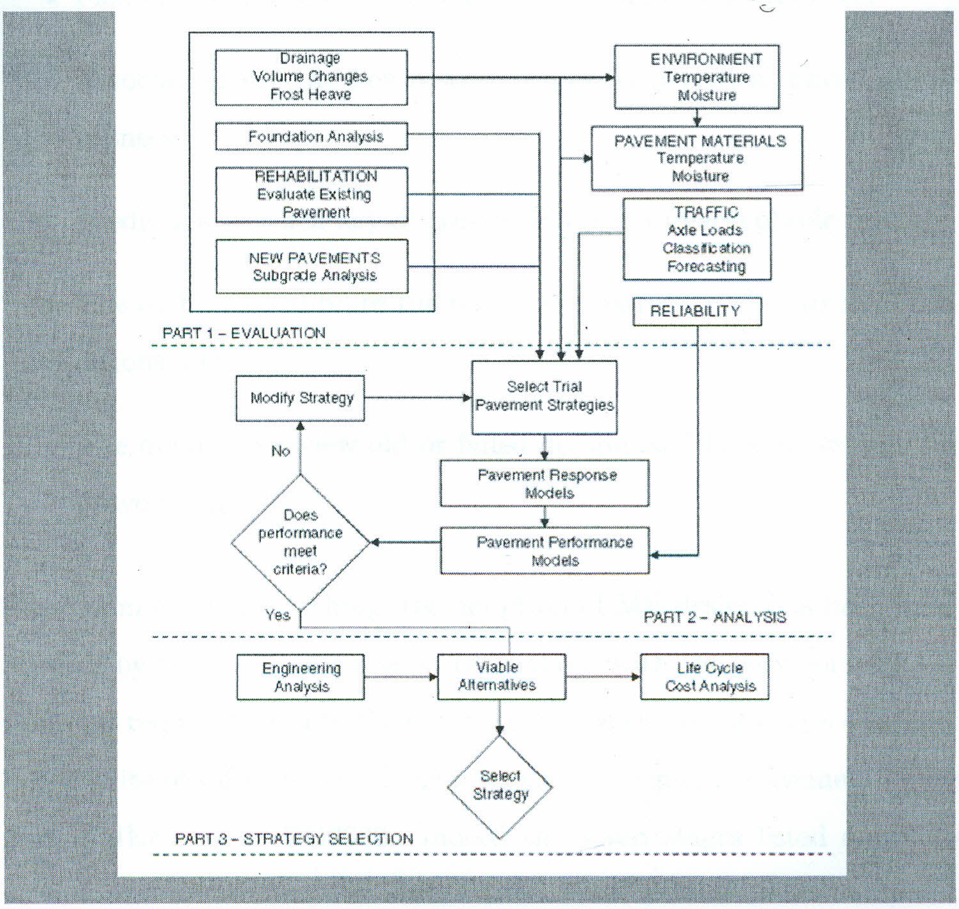


Figure 4.3.1: MEPDG Scheme (after Fwa[11])

The strengths of the ME design method lie in seven key areas as noted by Witczak et al [43]:

- Use of site-specific data on materials, pavement thickness, and the environment for custom pavement design.
- A comprehensive set of mathematical models that serve both rigid and flexible pavements in one suite.

- Flexible use for new designs, rehabilitation, or maintenance.
- Associating specific key distress types with critical pavement responses.
- Prediction of when key distresses will reach unacceptable levels.
- The ability to calibrate the pavement distress models to local conditions and
- The ability to review old or failed designs and look at ways to improve them.

Perhaps more than anything, the adoption of ME design has been made possible by the rapid advances in technology, with the very complex calculations required for both the mechanistic and empirical stages combined in one suite of software called the mechanistic-empirical pavement design guide (MEPDG) or DG2002. Indeed the seven stages listed above involve a number of computer simulations. From an economic standpoint, the ME methodology offers the most cost effective approach yet in the design, construction and overall management of pavements, costly undertakings that have to compete with other key sectors (health, education etc.) for scarce public finances.

While its use is desirable, one of the drawbacks of the ME approach is the sheer large requirements in inputs needed by the models, including the climatic ones, and this militates against its adoption. An innovative approach the ME procedure proposes to overcome part of this is the use of inputs at different levels [2]. This stratification works on the basis that the level of engineering effort put in a process should match the importance of the project. 3 input levels are used. Level 1 inputs are the most accurate ones and are found by conducting actual laboratory or field testing. This means such inputs are expensive to get, and their use should be for those projects where early failures would be very costly e.g. heavily trafficked roads with dire safety and economic failure concerns. Level 3 inputs on the other hand are estimates or typical averages, providing the lowest level of accuracy with minimal consequences for early failures. Level 2 inputs are intermediate between levels 1 and 3 [26].

4.4 Environmental factors in ME design

Compared to other approaches, the AASHTO ME design method mainstreams environmental effects into the design process more prominently than any other. The previous AASHTO guide (1993 in particular) devoted only 4% (25 pages out of 575) to climatic effects. The South African version of ME design, while generally older than other ME approaches, does not fully factor the environmental effects as noted by Theyse et al [37]. The Kenyan manual [28] is even more muted, directly dedicating only one page (page 3.1 under the shortest chapter titled 'The Natural Environment'). The ME pavement design guide (MEPDG) documentation reserved 659 pages out of 1735 to it [29], representing 38%, a major improvement from the others and a tacit recognition of the vital role the environment holds in the design process.

Several challenges hampered complete integration of the environmental factors in design as noted in 6.2, with Carvalho [6] reporting that the only ones included in the empirical 1993 AASHTO guide and its predecessors being the layer drainage coefficients and the unbound layers' resilient moduli. In most instances, even these two parameters are estimates obtained from correlations. This is highlighted by the numerical example in 3.6.

ME design on the other hand has a complete component within the MEPDG devoted to environmental effects called the enhanced integrated climatic model (EICM). The EICM is a one dimensional coupled heat and moisture flow model that is in itself a model comprising of 3 others as noted in 2. The place of EICM in ME design is core and its function is described by Witczak et al [43]thus:

> "The Enhanced Integrated Climatic Model (EICM) is a fundamental component of the new Mechanistic-Empirical Pavement Design Guide (MEPDG) developed under the NCHRP Project 1-37A. The model is intended to predict or simulate the changes in behavior and characteristics of pavement and unbound materials in conjunction with environmental conditions over several years of operation."

Given Kenya's geographic location, one of the constitutive models in the EICM (the CRREL) was not considered in this study, as it applies to regions that experience winter conditions only.

4.5 Pavement response models

The structural responses due to traffic are evaluated using the FEA and MLET modules, while EICM evaluates the moisture and temperature profiles in the pavement in the MEPDG [6]. Their outputs then act as the inputs to the transfer functions. The choice of FEA or MLET in analysis is guided by the pavement layer, with the AC layer (treated as exhibiting linear-elastic behaviour) analyzed using the latter. The unbound layers and subgrade exhibit more stress-dependent tendencies and are analyzed through FEA. The MLET method is grounded in both the Boussinesq equations and Burmister solutions as earlier mentioned in 3.3.2.

MLET offers the advantage of simplicity, requiring few material properties (usually elasticity) and layer thicknesses for every layer, together with the tire pressures and contact areas [6]. However, it does not consider the anisotropy in materials, and FEA overcomes this even though it requires more rigour and computation time. The model FEA uses to account for the stress-dependency of materials is the resilient modulus one obtained from laboratory tests and given as;

$$M_r = k_1 P_a (\frac{\theta}{P_a})^{k_2} (\frac{\tau_{oct}}{P_a} + 1)^{k_3} \tag{4.5.1}$$

where;

M_r= resilient modulus of pavement layer (considered in section 6.2.3)

k_1, k_2, k_3= regression parameters

P_a= atmospheric pressure

θ= bulk stress

τ_{oct}= octahedral shear stress

The EICM predicts the temperature distribution throughout the pavement profile. This is then used to determine the elastic moduli, E, (a measure of stiffness) of the asphalt mixture at different depths. Such E values are then sent forward to the distress prediction models, and this depends on the types of distresses considered. For moisture, the EICM computes a correction factor F_{env}, at different points in the unbound layers and subgrade. F_{env} is used to modify the M_r depending on the state of the soil and on the season. A lengthy algorithm for the determination is provided in [2], while further treatment of the environmental effects is made in chapter 6.

A third component of the EICM (the CRREL), not considered in this study, computes the freezing index and predicts the formation of ice lenses at different depths for winter conditions in Temperate Zones.

4.6 Empirical performance models

As noted above, the performance models have as their inputs the outputs of the response ones. The different distresses that are quantifiable mathematically and have models include cracking (longitudinal, thermal and fatigue), permanent deformation and roughness. It is worth mentioning that the said models vary depending on the pavement layer (specifically are a function of the materials). The models were developed as an effort of Project 1-37A under North American conditions, but even then are not adequate for the individual states' requirements, thus need exists in calibrating them to local conditions to be reliably used.

In the following subsections, we examine the models with specific significance to Kenyan conditions, and are climate-related as contained in the MEPDG documentation.

4.6.1 Longitudinal cracking model

Also called top-down cracking, it initiates at the surface and propagates downward. For AC pavements, tensile strains atop the surface AC layer caused by traffic loads causes the appearance of cracks. Overall damage is computed using Miners Law (discussed below), while cracking, in units of ft/mi, is given by;

$$FC = \left(\frac{C_4}{1 + \exp(C_1 - C_2 \log(100D))}\right) X 10.56 \tag{4.6.1}$$

where;

FC= longitudinal cracking

C_1, C_2 and C_4= regression parameters

D= damage

The calibration coefficients are a function of the HMA thickness, while the damage D, is computed using Miner's law. Also known as the principle of linear summation of damages [38], its thrust is that the total damage on a pavement after a time interval is the cumulative sum of the individual smaller damages caused during the interval, and is mathematically expressed as;

$$D = \sum_{i=1}^{T} \frac{n_i}{n_{fi}} \tag{4.6.2}$$

where;

T= total number of seasonal periods

n_i= actual traffic for period i

n_{fi}= traffic repetitions of a given load to cause cause failure at period i

The damage, being incremental, has an index that is influenced by the median temperature for the five temperature intervals used to subdivide each analysis period [35]. This involves modeling the temperature distribution using a normal curve to account for the extreme values not captured by the mean. A detailed procedure to undertake this is provided in [2].

4.6.2 Thermal cracking model

This distress is caused by the heating and cooling cycles experienced by the pavement. Differences in rates of cooling between the surface and the inner HMA layers leads to cracking in a direction perpendicular to the wheel paths. It is also called transverse cracking. EICM generates the temperature-depths profiles at hourly intervals during the analysis period, and the viscoelastic transformation theory is used to determine the compliance $D(t)$, which is related to the relaxation modulus, E_r, of the HMA mix [2]. Knowledge of E_r together with the temperature data from the EICM allow for the prediction of thermal stress as a function of time and depth, given by the equation below;

$$\sigma(\xi) = \int_0^{\xi} E(\xi - \xi')\frac{d\varepsilon}{d\xi}d\xi' \tag{4.6.3}$$

where;

$\sigma(\xi)$= stress at reduced time ξ

ξ'= variable of integration

$E(\xi - \xi')$= relaxation modulus at reduced time $\xi - \xi'$

$d\varepsilon$= strain at reduced time $\xi = \alpha(T(\xi') - T_0)$

$T(\xi')$= pavement temperature at reduced time ξ'

T_0= pavement temperature when σ=0

α= linear coefficient of thermal expansion

The viscoelastic properties of the HMA mixture is then represented as a generalized Maxwell model via a Prony series expansion [35], with the relaxation modulus function obtained through transforming a creep compliance function. The growth in crack depth is modeled using the equation;

$$\Delta C = A(\Delta K)^n \tag{4.6.4}$$

where;

ΔC= change in crack depth due to cooling cycle

A, n= fracture parameters of the HMA mixture

ΔK= change in stress intensity factor due to cooling cycle

A two stage regression is performed, first to obtain the temperature shift factors and for the Prony series parameters and then to fit a second functional form for the master creep compliance information [2]. The final form of the cracking model implemented in the MEPDG is given as;

$$TC = \beta_{t1} N_z[\frac{1}{\sigma}\log(\frac{C}{h_{HMA}})] \tag{4.6.5}$$

where;

TC= observed amount of thermal cracking, ft/mi

β_{t1}= regression coefficients obtained through field validation

N_z= standard normal distribution evaluated at z

σ= standard deviation of the *log* of the depth of cracks in the pavement

C= crack depth, in

h_{HMA}= thickness of the surface layer, in

Using the model has a limitation. Theoretically it can predict thermal cracking up to only four-fifths of the total pavement length [35]. Practically however, it cannot go more than 50% of this value as the failure criterion assigned is breached.

4.6.3 Fatigue cracking model

This load-related distress manifests itself as a series of short interconnected cracks that look like the back of an alligator, hence sometimes referred to as alligator cracking. The cracks initiate from the tensile strains at the bottom of the HMA layers induced by wheel loads, and propagate upwards to the surface. The MEPDG has the model implemented mathematically as;

$$N_f = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E)^{k_{f2}\beta_{f2}} \tag{4.6.6}$$

where;

N_f= allowable number of load repetitions

ε_t= tensile strain at the critical location

E= dynamic modulus measured in compression

k_{f1}, k_{f2}, k_{f3}= global field parameters

$\beta_{f1}, \beta_{f2}, \beta_{f3}$= global field parameters

C= correction factor

C_H= thickness correction factor

Like in longitudinal cracking, the damage accumulation is computed as a function of temperature.

4.6.4 Permanent deformation models

The models considered earlier were for the HMA layers only. The rutting models however includes both the unbound layers and subgrade in addition to the surface. This is because rutting, also called permanent deformation occur in all the layers of a pavement system [45]. Quantifying it means summing the rutting in different layers via the equation;

$$RD_{total} = RD_{HMA} + RD_{base} + RD_{subgrade} \tag{4.6.7}$$

The models for the different layers are given as in [2];

- **For the HMA layers**: Computation of rutting is done for each sub-season at mid-depth of each sub-layer using the cumulative damage approach. The rutting in a sub-layer is expressed as a ratio between the plastic and elastic strains thus;

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_{r1} k_z 10^{k_{r1}} T^{k_{r3} * \beta_{r3}} N^{k_{r2} * \beta_{r2}} \tag{4.6.8}$$

where;

ε_p= accumulated permanent strain

ε_r= resilient strain

k_z= depth confinement factor

k_{r1}, k_{r2}, k_{r3}= global calibration factors

$\beta_{r1}, \beta_{r2}, \beta_{r3}$= local calibration factors

T= temperature, ^oF

N= number of load repetitions

Most rutting occur at elevated temperatures when the stiffness of asphalt is lowest, and the total rutting for the HMA layer is given by Quintus et al as [35];

$$PD = \Sigma_{i=1}^{n} \varepsilon_p^i h_i \tag{4.6.9}$$

where;

PD= permanent deformation, in

n= number of sub-layers

ε_p^i= total plastic strain in sub-layer i

h_i= thickness of sub-layer i

- **For the unbound layers**: All layers with unbound granular materials are divided into sub-layers, and a similar approach to HMA layer used to compute the total rutting. The rutting for any sub-layer is given as in [6];

$$\sigma_i = \beta_i k_i \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \exp -\left(\frac{\rho}{N}\right)^{\beta} \varepsilon_v h_i \tag{4.6.10}$$

where;

σ_i= permanent deformation for sub-layer i

β_i= field calibration coefficient

k_i= regression coefficient determined form laboratory permanent deformation test data

$\frac{\varepsilon_0}{\varepsilon_r}, \beta, \rho$= material properties

N= number of repetitions of a given load

ε_v= computed vertical resilient strain at mid-thickness of sub-layer i for a given load

h_i= thickness of sub-layer i

Unlike the HMA surface, the base is directly affected by the soil moisture and this is accounted for in the equation above by the material parameter, β, which is expressed as;

$$\log \beta = -0.61119 - 0.017638 W_c \tag{4.6.11}$$

where; W_c= percentage water content in the soil.

The other material properties $\frac{\varepsilon_0}{\varepsilon_r}$ and ρ are functions of the water content, the bulk stress and resilient moduli of the sub-layer considered. Variants of the equation above thus exist depending on whether the soil is fine or coarse grained, and much detailed analysis of this is found in [2].

- **For the subgrade**: The subgrade is the native earth on which other layers are placed and may be considered as of infinite depth. Subdividing into sub-layers for computation of strains is thus impractical. The approach developed and adopted for use in the guide [35] gives rise to the transition model;

$$\varepsilon_p(z) = (\varepsilon_{p,z=0} \exp(-kz)) \tag{4.6.12}$$

where;

$\varepsilon_p(z)$= plastic vertical strain at depth z (measured from the top of the subgrade)

$\varepsilon_{p,z=0}$= plastic vertical strain at the top of the subgrade (z = 0)

z= depth measured from the top of the subgrade, in

k= constant obtained from regression analysis

k is limited by a value of 0.000001 to prevent inconsistencies, with the total permanent deformation in the subgrade obtained by solving the integral;

$$\sigma_{soil} = \varepsilon_{p,z=0} \int_0^{h_{bedrock}} \exp(-kz)dz = \frac{1 - \exp(-kh_{bedrock})}{k} \varepsilon_{p,z=0} \quad (4.6.13)$$

where;

σ_{soil}= total plastic deformation of the subgrade, in

$h_{bedrock}$= depth to bedrock, in

4.6.5 Roughness model

Roughness is a distress that leads to loss in the quality of ride and can be felt by the road users. Levels of road roughness are the main indicator informing maintenance and rehabilitation practices by many pavement management agencies. As described in 2, quantifying roughness mathematically is very complex. The MEPDG methodology implements roughness measurement using the international roughness index (IRI) , a regression model with 4 main contributing factors expressed as [2];

$$IRI = IRI_0 + \Delta IRI_D + \Delta IRI_F + \Delta IRI_S \quad (4.6.14)$$

where;

IRI= international roughness index

IRI_0= initial IRI

ΔIRI_D= increase in IRI due to distress

ΔIRI_F= increase in IRI due to subgrade frost heave potential

ΔIRI_S= increase in IRI due to subgrade swell potential

The equation above is generic, with the parameters exhibiting variations depending on the site, construction type (whether new or overlays) and the types of distresses considered. The more specific and final forms of the smoothness prediction models are given by Quintus et al as [35];

1. **For New Flexible and HMA Overlays on AC Pavements:**

$$IRI = IRI_0 + 0.015(SF) + 0.4(FC) + 0.008(TC) + 40(RD) \quad (4.6.15)$$

where;

RD= average rut depth, in

TC= length of transverse cracking, ft/in

FC= area of fatigue cracking (total of alligator and longitudinal cracking in the wheel path), % of total lane area

SF= site factor=$Age(0.02003(PI + 1) + 0.007947(Rain + 1) + 0.000636(FI + 1))$

Age= pavement age in years

PI= plasticity index of soil, %

FI= average annual freezing index, ^{0}F

$Rain$= average annual rainfall, in

2. **For HMA Overlays on PCC Pavements:**

$$IRI = IRI_0 + 0.00825(SF) + 0.575(FC) + 0.0014(TC) + 40.8(RD) \tag{4.6.16}$$

4.7 Example of Mechanistic-Empirical design

To illustrate the ME design procedure and the centrality of the climatic inputs and related parameters, the following example obtained from the guide [2] is used. The graphs and screen shots, due to Quintero [34], are illustrative of the possible outcomes even though they may not be specific to the example considered.

Design Problem:- Need to design an AC pavement with a 10 year design life. Base and subgrade are to be done in August, surface laid in September and pavement opened to traffic in October of the same year (2003).

Requirements:- The pavement should have an initial IRI=63 in/mile. It is expected that by end of design life: IRI$\leq$172 in/mile, longitudinal cracking, LC$\leq$1000 ft/mile, fatigue cracking, FC$\leq$25 per cent, thermal cracking, TC$\leq$1000 ft/mile and rutting (AC layer)$\leq$0.25 in, rutting (total)$\leq$0.755 in. All these criteria are to be satisfied at a reliability level of 90%.

Location: The 5 mile road is in Indiana, with an annual ground water table depth of 15 ft. (*Remark:* By specifying the location, the software retrieves all the required climatic parameters from its database. It similarly has an option for actual entry of the data. Used for regions whose data is not included in the database (like Kenya), the user has to manually feed in the details in the EICM window shown in figure 4.7.2

Traffic:- The 2-way average annual daily truck traffic (AADTT) is 1500 during the first year. Two lanes exist in the design direction with 90%

of trucks in the design lane, and the traffic is equally distributed in both lanes. The operational speed is 60 mi/hr. The road is heavily trafficked with the % of the AADTT in each vehicle class same as that in the default classification. (*Remark:* This default value is retrieved from a table specifying the distribution based on past trends. Unique to every country, Kenya should build its own default values based on traffic counts). The vehicle class daily and monthly patterns are assumed to be the same all year round, while the hourly distribution is obtained from default values for each hour based on past observations (for instance, more vehicles during peak hours). The traffic is assumed to grow at the rate of 4% compounded annually after the base year. The axle load distribution and spectra are then set (yet again from default values, obtained from long periods of observation, LTPP program). The tire pressure is set at 120 psi.

Drainage Properties:- The cross-slope=12%, drainage path=12 ft from the centreline and the surface shortwave absorptivity=0.85.

Asphalt Properties:- The AC uses default value in software, the sieve analysis for the different sizes set. The volumetric mix design has 12% binder content, 6% air void content, with a density of 143 lb/ft^3. The thermal conductivity=0.67 $Btu/hr - ft -^0 F$, the specific heat capacity=0.23 $Btu/lb-^0 F$, the Poisson's ratio=0.35, and the mixing temperature=$70^0 F$. **Subgrade and Other Layers:-** These have their properties like the optimum resilient moduli, M_{ropt}, plasticity indices, PI, and the sieve analysis test values entered into the system.

After all these values have been entered, the analysis is done by the computer at the press of a button. The outputs are displayed in graphical form, with each predicted distress plotted usually against time. If the predicted distress is within the design limits (as the rutting curve shows), then it is adopted. On the other hand if it is beyond the limits (see the thermal cracking curve), a new trial design is picked iteratively until the required conditions are met. This is achieved by adjusting the various parameters that are input into the program and considered here.
The figures and graphs that follow illustrate the discussions above.

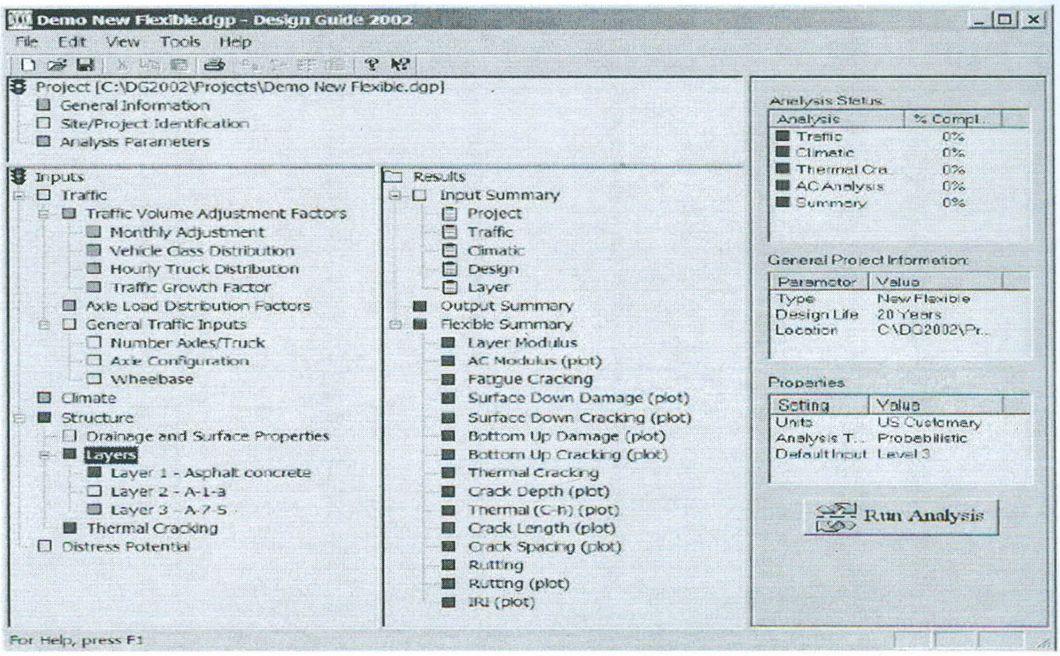
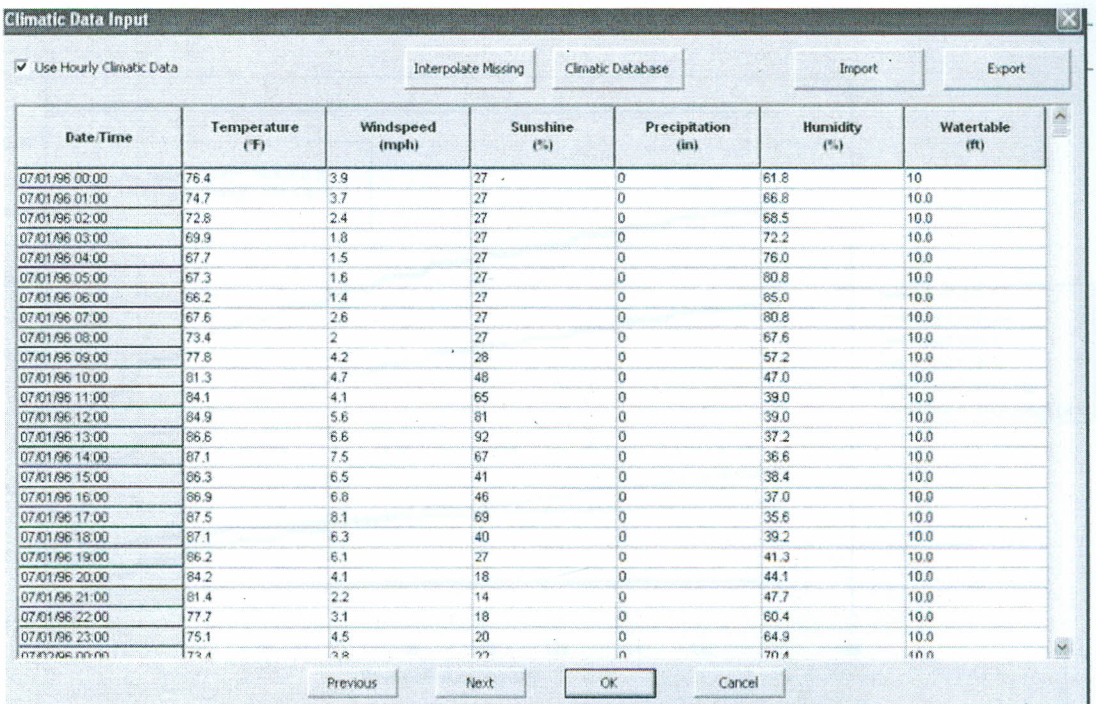


Figure 4.7.1: The MEPDG Input Windows (after Quintero[34]).



Date/Time	Temperature (°F)	Windspeed (mph)	Sunshine (%)	Precipitation (in)	Humidity (%)	Watertable (ft)
07/01/96 00:00	76.4	3.9	27	0	61.8	10
07/01/96 01:00	74.7	3.7	27	0	66.8	10.0
07/01/96 02:00	72.8	2.4	27	0	68.5	10.0
07/01/96 03:00	69.9	1.8	27	0	72.2	10.0
07/01/96 04:00	67.7	1.5	27	0	76.0	10.0
07/01/96 05:00	67.3	1.6	27	0	80.8	10.0
07/01/96 06:00	66.2	1.4	27	0	85.0	10.0
07/01/96 07:00	67.6	2.6	27	0	80.8	10.0
07/01/96 08:00	73.4	2	27	0	67.6	10.0
07/01/96 09:00	77.8	4.2	28	0	57.2	10.0
07/01/96 10:00	81.3	4.7	48	0	47.0	10.0
07/01/96 11:00	84.1	4.1	65	0	39.0	10.0
07/01/96 12:00	84.9	5.6	81	0	39.0	10.0
07/01/96 13:00	86.6	6.6	92	0	37.2	10.0
07/01/96 14:00	87.1	7.5	67	0	36.6	10.0
07/01/96 15:00	86.3	6.5	41	0	38.4	10.0
07/01/96 16:00	86.9	6.8	46	0	37.0	10.0
07/01/96 17:00	87.5	8.1	69	0	35.6	10.0
07/01/96 18:00	87.1	6.3	40	0	39.2	10.0
07/01/96 19:00	86.2	6.1	27	0	41.3	10.0
07/01/96 20:00	84.2	4.1	18	0	44.1	10.0
07/01/96 21:00	81.4	2.2	14	0	47.7	10.0
07/01/96 22:00	77.7	3.1	18	0	60.4	10.0
07/01/96 23:00	75.1	4.5	20	0	64.9	10.0
07/02/96 00:00	73.4	3.8	22	0	70.4	10.0

Figure 4.7.2: The EICM Input Window: Notice the hourly-interval requirement for the 6 climate parameters, and the constant value for Groundwater Depth (after Quintero[34]).

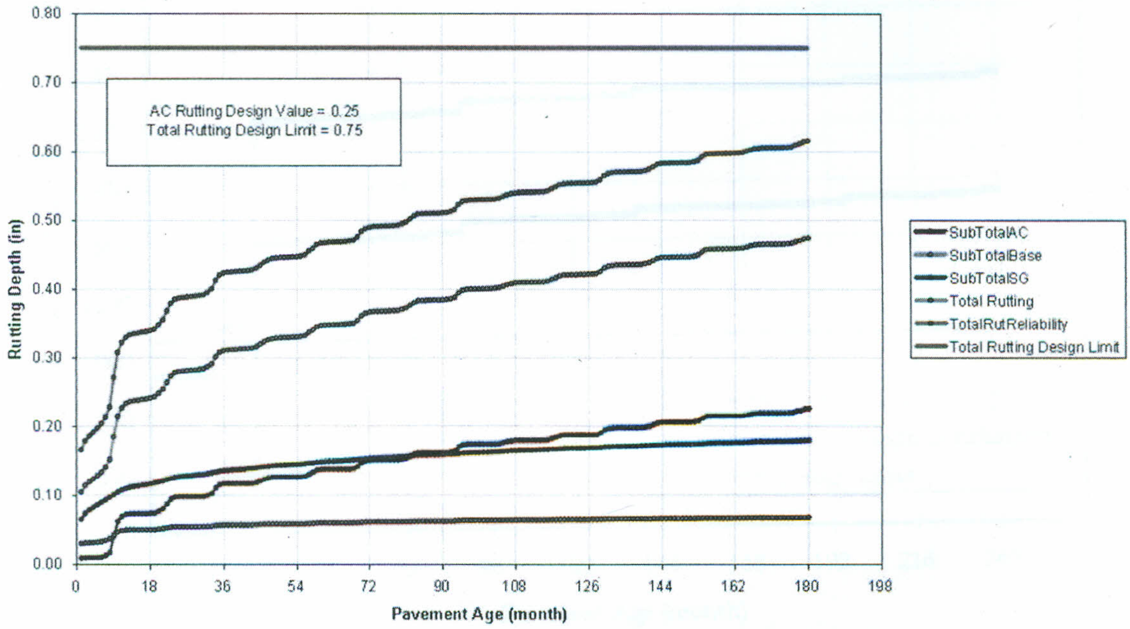


Figure 4.7.3: MEPDG Output predicting rutting: A case of distress within design limits, thus adoptable (after Quintero[34]).

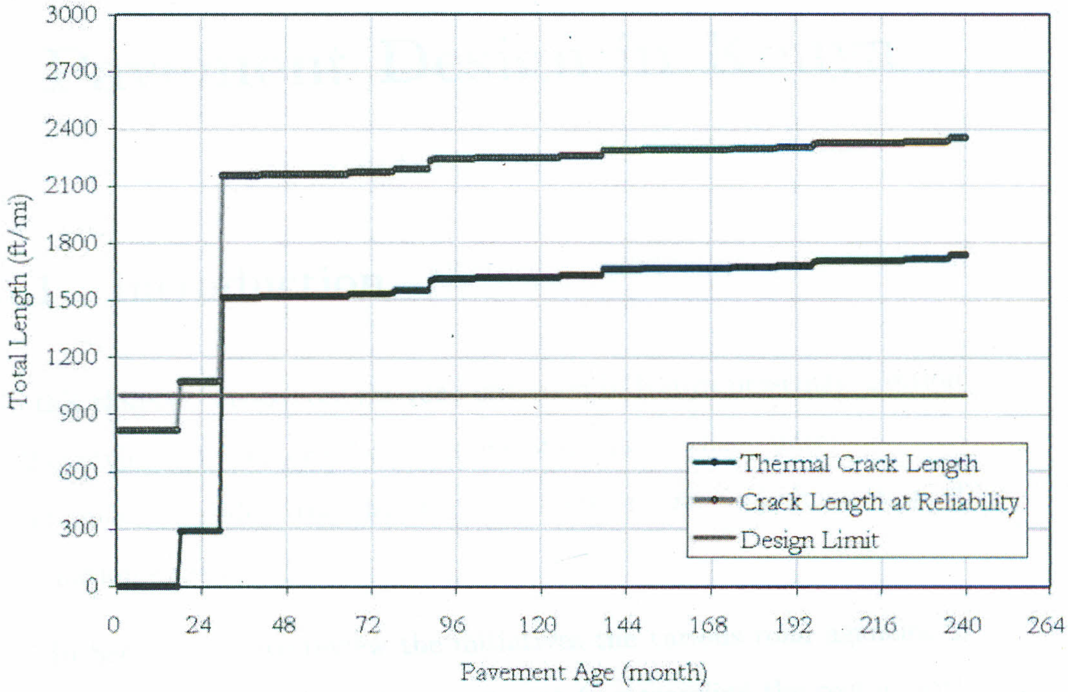


Figure 4.7.4: MEPDG Output predicting thermal cracking: A case of distress beyond design limits, thus new trial design taken (after Quintero[34]).

Chapter 5

Pavement Design in Kenya

5.1 Introduction

In this chapter, we explore the road situation in Kenya presently. Section 5.2 overviews the historical dimensions of design in the country. Section 5.3 considers the classification, financing and administration of the nation's roads.

In Section 5.4, we review the initiatives the various road agencies in Kenya have undertaken in the recent past to streamline the sector, with a specific focus on the design, administration and overall management of roads in the country. The chapter ends in Section 5.5 by outlining some of the challenges that still exist in the sector in spite of the various interventions taken so far.

5.2 Historical perspectives

Kenya initially designed her roads based on the Road Notes 29 and 31 until about 1977 during which an extensive research programme was carried out into the performance and characteristics of Kenyan materials and pavements. This was necessary because the Road Notes produced by the British Transport and Road Research Laboratory (TRRL) were not specifically designed to cater for Kenyan conditions [28]. The first country-specific pavement design manual for Kenya were published in 1981, with the manual divided into the following 5 parts [28];

- Part 1: Geometric Design of Rural Roads
- Part 2: Geometric Design of Urban Roads
- Part 3: Materials and Pavement Design for New Roads
- Part 4: Bridge Design
- Part 5: Pavement Rehabilitation and Overlay Design

The initial manuals were revised in 1988 to accommodate for changes in traffic, materials and improved design methods, but both versions were leaning towards the AASHTO ones (whose basis was the 1950's Road Test), and the pavement designs in them were presented in the form of catalogues [28].

Road testing programmes in Kenya have in the past mainly centered on one-off pavement condition surveys, evaluation of vehicle weight statistics, rut depth and crack index measurements, assessment of embankment conditions, earthwork and subgrade survey. In more recent years, in situ performance evaluation has been carried out. Mwea and Gichaga [32] in 2004, detail a performance study done for flexible pavements around Nairobi through deflection testing of both low and high volume roads largely using the Benkelman beam. Used in design of roads, such tests help in the understanding of the responses and performance independently.

5.3 Roads in Kenya

The significance of a good road network to the development of a country cannot be overstated, but for Kenya, it is even more crucial considering that other transport sectors are not fully developed. Overall, the transport system in Kenya is estimated to contribute about 11% of the GDP, with an average annual growth of 14.7% [1]. It is noted that road transport makes up about 85% of all domestic travel [1]. As an engine for commerce and development, it is necessary to keep the road network in good shape, and the design process offers the first crucial step.

5.3.1 Classification

Kenya has a network of 177,800km of roads. 63,575km of these is classified, with only 9,273km paved, the rest consisting of loose earth [21]. Estimates have it that about 70% of the classified network is in good condition and maintainable while the remaining requires rehabilitation or reconstruction. Of the classified roads, a stratification system is adopted based on the significance of the road. Alphabetically listed, road classes A, B and C are the most significant and heavily trafficked, with class A roads comprising international trunk roads linking centers of international importance and crossing international boundaries or terminating at international ports. Only 2,886 km of class A roads are paved [21]. The Mombasa-Malaba highway for instance fall in category A.

Class B roads are national trunk roads that link provincial headquarters and other important centres to the capital, to each other or to the international road network. 1,339 km of the national trunk roads are paved. The class C ones are primary roads that link district headquarters to each other and higher level roads or centers. The rest of the other roads, numbered from D to W, comprise minor roads linking rural centers, utility roads serving farmlands, private roads and those that traverse national parks and reserves.

From a design viewpoint, nearly all of the paved roads are of AC type and empirically designed, while the unpaved ones have either compacted gravel for surfacing or native earth.

5.3.2 Management, financing and training

The management of the road network in Kenya is shared by several bodies. These include the Roads Ministry (in charge of road classes A, B and C), District Roads Committees (classes D, E and all special purpose roads) and the Kenya Wildlife Services (roads through the national parks) [1]. The Local Government Ministry through Civic Authorities are responsible for unclassified roads in urban areas, and the Forestry Department oversees unclassified roads within forests [21]. The Road ministry in turn has 3 agencies to manage the roads under it. These are; Kenya National Highways Authority (KeNHA) for national highways countrywide, Kenya Urban Roads Authority (KURA) for roads within urban centers and Kenya Rural Roads Authority (KeRRA) for roads in rural areas [1].

Financing and development of the overall network is the mandate of The Kenya Roads Board (KRB) . It achieves this through funds obtained from the Road Maintenance Fuel Levy (RMFL), a surcharge on the oil products used in the country for transportation purposes. A large part of road funding is also sourced from both bilateral and multilateral development partners like the European Union, the World Bank and the African Development Bank [1]. Over the past years, funding for road sub-sector has steadily risen with KSh. 182 billion allocated to infrastructure development this financial year (2010/2011) [12]. This represents an increase of 20.1 % over the amount allocated in the previous fiscal year. Of this, the roads sub-sector had the highest allocation at KSh. 78.6 billion [12].

Manpower development is achieved through training of personnel at universities and at the Kenya Institute of Highways and Building Technology (KIHBT) , the latter offering special diploma courses in civil, structural and highway engineering [21]. An evaluation of the curriculum of KIHBT shows it is steeped towards the empirical procedures that have defined road building in the country in the past.

5.4 Recent developments and initiatives

The road condition survey statistics in 5.3.1 above is a recent event. Hitherto, poor planning and management practises saw emphasis put on construction of new roads at the expense of maintenance, a situation that led to pavement failures in most of the constructed roads. Freezing of donor funding in the 1990's exacerbated the situation further. However maintenance has recently been mainstreamed with about a third of this year's roads budget going to it [12].

A wide range of reforms in the transport sector in general and road sub-sector in particular have seen improvements in the operation and maintenance of the roads in Kenya. Administratively, these include the formation of the various road agencies like KRB, KeNHA and KURA [21]. The KRB has developed a strategic plan (2008-2013) that seeks to strengthen public-private partnership in road financing, conduct a comprehensive road inventory and condition survey and develop road management system (RMS) [3]. Some of these propositions have been actualized.

On a more technical level, enhancing the design of roads has been recognized. In March 2008, the Kenyan government embarked on its latest road design manual revision with funding from the European Commission under the Stabex 1991 Framework of Mutual Obligations Programme [14]. The scope of the review work was an upgrade of the empirically based 1988 manuals. Contact made with the reviewers pointed to a need to re-orientate the existing design manuals towards the ME procedures to benefit from their superior performance prediction capabilities [16] as described earlier. The outcome of the review is an improved manual but still based on the empirical procedures that have some inherent limitations.

5.5 Challenges in the roads sub-sector

A number of challenges face the road sub-sector in Kenya. Some highlighted by [3] and [21] include;

1. Financing- Construction and rehabilitation works require a lot of funding which is often difficult to obtain, since other sectors are competing for the same scarce sources. The road maintenance backlog, for instance, is estimated at about KSh. 100 billion.

2. Non compliance with axle load limits- Heavy trucks that overload lead to early pavement failures and estimated to cause up to 60% of damage on roads.

3. The sharp growth in traffic in recent years- This puts a strain on existing pavements projected to carry lower volumes during their design lives.

4. Safety- Increased growth in vehicular traffic has led to high incidences of road accidents. The situation has been exacerbated in recent days by steep rise in the number of motorcycles whose operators have low levels of knowledge of the highway code.

From the perspective of this study, challenges exist in gathering relevant reliable data that can be used to aid the design process. The traffic count on the roads, a major determinant and input in the design process, is done intermittently and not continuously. Moreover, the traffic inputs required by the ME approach are more wholesome, needing both the loading spectra and the numbers, while that in use in the country uses the ESALs [28]. The shift to using ME models equally pose some challenges, one of which is the rigidly defined inputs required by the mathematical models in the ME design. In the next chapter and in Chapter 7, the study explores options of addressing these concerns and how to position Kenya to embrace the new cost effective road design method.

Chapter 6

Formulation of Mechanistic-Empirical Climatic Models for Kenya

6.1 Introduction

This chapter (Section 6.2 through 6.7) considers the 6 environmental factors which influence the response and performance of pavements, and how they individually do so as contained in both the EICM and the MEPDG. Surveyed too are the ways of measurement of the elements currently and proposals made on new ways of measurement that should be adopted specifically for road design.

Each section contains an overview of the types of records of each weather factor that the Kenya Meteorological Department (KMD) keeps and finally concludes by proposing the best possible ways of accommodating the available data in the MEPDG design process sensibly without loss in the reliability of the estimates of performance.

6.2 Precipitation

That precipitation is one of the key environmental factor in pavement design has been recognized for several decades [2]. However, along with other weather factors, its complete integration into the design process is a recent development as noted in 4.4. The difficulty of incorporating precipitation, and by extension other climatic effects, into the design process may be attributed to the complex interplay between them and other design factors (materials, structural design etc). Significantly, the collection of and/or the availability of climatic records also pose a challenge since roads are built to last several years and reliably backcasting and projecting the often fickle weather patterns is a difficult task. Precipitation takes several forms including frost, hail and rainfall. This study considers all forms of precipitation as rainfall since this is what is prevalent in Kenya.

6.2.1 Effects of precipitation and its measurement

For both AC and PCC pavements, their bound layers (surfacing) are not as susceptible to changes in moisture as the unbound. Indeed, the effects of excess moisture is often limited to stripping in asphalt mixtures and compromising the structural integrity of cement bound materials [2]. In its most extreme forms, like during flash flooding, rainfall can mechanically abrade whole pavement structures. However such situations are not explored in this study.

Unlike the bound surfacing, other lower pavement layers consist of natural materials like gravel for bases and raw earth for subgrade, and these bear the brunt of variations in moisture content. The effects of moisture are noted as [2];

- Moisture affects the soil through destruction of cementation of the soil particles thus making them susceptible to yielding on loading.
- Moisture affects the state of stress in the soil through suction or pore water pressure, and this is more pronounced for unsaturated soils. Later subsections explore this relation fully.
- For temperate locations, freezing of water within the soil leads to expansion and the formation of ice lenses, which in turn heaves the pavement resulting into transverse thermal cracking
- A resultant modifying effect of rainfall events is the reduction in the ambient and hence pavement surface temperatures. This is discussed further in 6.3.1

Consideration of moisture effects on unbound pavement layers hinges on a measure of the ability of the layers to deflect then regain their original shape when subjected to loading-unloading cycles, as quantified by the resilient modulus, M_r. Due to the significance of M_r, our analysis will centre on it as the next sections will show.

The determination of the amount of rainfall has been through the use of the rain gauge. This consists of a funnel atop a graduated cylinder set on a secluded and unobstructed ground. Readings are taken once a day and the gauge is reset by emptying the cylinder for further recording. This is the predominant rainfall-measuring device in Kenya as was established by a visit to the meteorological agency. The need for shorter interval rainfall recording, driven largely by the aviation industry, has led to the automation of the process. While the principle behind their operation remains similar to their predecessors', automatic rain gauges channel the precipitation into buckets that tip over at timed intervals, usually one hour. These are wired together with a datalogger that records the readings, eliminating the possible inaccuracy and delay that would have been incurred if done manually [34].

Moisture flow in soils, and thus in unbound pavement layers is a complicated process and the equations governing it depend on whether the soil is saturated or not. Even then, such equations are mere estimates at best [18]. For reliable determination of pavement moisture content, instrumentation of actual pavement sections is done [2]. This involves drilling of cores vertically or obliquely into the pavement structure and inserting a time domain reflectiometer (TDR). The TDR is a device that uses electromagnetic waves to detect the variation in conductivity in a medium [34]. It is a form of a ground penetrating radar (GPR), sending incident waveforms and receiving back the reflected waves. Water has a markedly different electrical conductivity (as measured by its dielectric constant, defined as the capacity of a material to store electrical charge relative to a vacuum's) from that of air or the soil materials and its presence and quantity between the pores will be reflected in the resulting waveforms registered. The volumetric moisture content of different soil types are then obtained using the relation below proposed by the Federal Highways Authority (FHWA) [34];

$$\theta = a_1 + a_2 K_a + a_3 K_a^2 + a_4 K_a^3 \tag{6.2.1}$$

where;

a_i= correlation coefficients dependent on soil type; either coarse or fine

K_a= dielectric constant of the soil

θ= the volumetric moisture content

6.2.2 Soil moisture and soil suction

Moisture content refers to the amount of water in a soil. It may be expressed in two ways; gravimetric or volumetric. Gravimetric moisture content, w is the the ratio between the mass of water and that of the solids, while volumetric moisture content, θ is the ratio between the volume of water to the volume of solids. We note here that w and θ are related by the equation below given by Quintero [34];

$$\theta = w \frac{\rho_d}{\rho_w} \tag{6.2.2}$$

where;

θ= volumetric moisture content

w= gravimetric moisture content

ρ_d= dry soil density

ρ_w= density of water

Related to the terms above is saturation, S, defined as the ratio between the volume of water to the volume of air voids in the soil. It is often expressed as a percentage. A fully saturated soil sample therefore has all the voids filled with water. The equation below is used to determine the degree of saturation:

$$S = \frac{w}{\frac{\rho_w}{\rho_d} - \frac{1}{G_s}} \tag{6.2.3}$$

where;

G_s= specific gravity of the solids

Movement of water in soils is governed by forces of attraction between the water and soil particles, and the resultant pressures that depend on the saturation of the soil. Wilson [41] defines suction as the difference between the water and air pressures if osmotic pressure due to dissolved salts is not included. These relationships are expressed by the equation below;

$$\psi = h_m + h_s \tag{6.2.4}$$

where;

ψ= total suction

h_m= matric suction $= u_a - u_w$

h_s= solute suction or osmotic pressure due to dissolved salts

u_a= pore-air pressure

u_w= pore-water pressure

The air and water pressures are affected by temperature, thus moisture flow in partially saturated soils is a function of temperature, hydraulic conductivity, air and water pressures [34]. Several equations may be used to describe the flow of moisture in soil. The classical Darcy's law is one such expression and is given as;

$$v = -k(\theta)\frac{\partial \phi}{\partial x} \tag{6.2.5}$$

where;

v= the velocity of water

$k(\theta)$= the hydraulic conductivity as a function of θ

θ= the volumetric moisture content

$\frac{\partial \phi}{\partial x}$= the gradient of potential in the x direction

Similarly showing this relation is the more detailed air pressure gradient model in 1-dimension formulated by Dakshanamurthy and Fredlund [34];

$$\frac{\partial u_a}{\partial t} = -C_a\frac{\partial u_w}{\partial t} + c_a^v\frac{\partial^2 u_a}{\partial z^2} + C_{at}\frac{\partial T}{\partial t} \tag{6.2.6}$$

where;

u_a= the air pressure

u_w= the water pressure

T= the temperature

C_{at}= the temperature interaction coefficient associated with the air phase

c_a^v= the consolidation coefficient with respect to air phase

C_a= the interaction coefficient associated with the air phase

Of interest to pavement engineers over the years has been the moisture content of the various pavement layers which does not necessarily have an immediate direct correlation with the incident precipitation. Han et al [18] observed that;

> "Past research has not established a clear and practical relationship between precipitation and the change in ground water content. This is because of the 'pause' or 'postponement' between rainfall and the change of moisture content that happens and the complexity of the analysis. It might take four to six weeks for water to infiltrate the subgrade from the surface and cause any variations in moisture content, although increases may not be measurable"

Soil water characteristic curve, (SWCC)

The soil water characteristic curve (SWCC) is a graphical representation describing the relationship between the soil suction and the moisture content, and is defined by Witczak et al. [43] as the variation of water storage capacity within the micro and macro pores of a soil with respect to the suction. This relationship is often plotted graphically featuring either w, θ or S against the suction. A typical SWCC has the profile shown below; From the figure, SWCC is dependent on the soil type. Know-

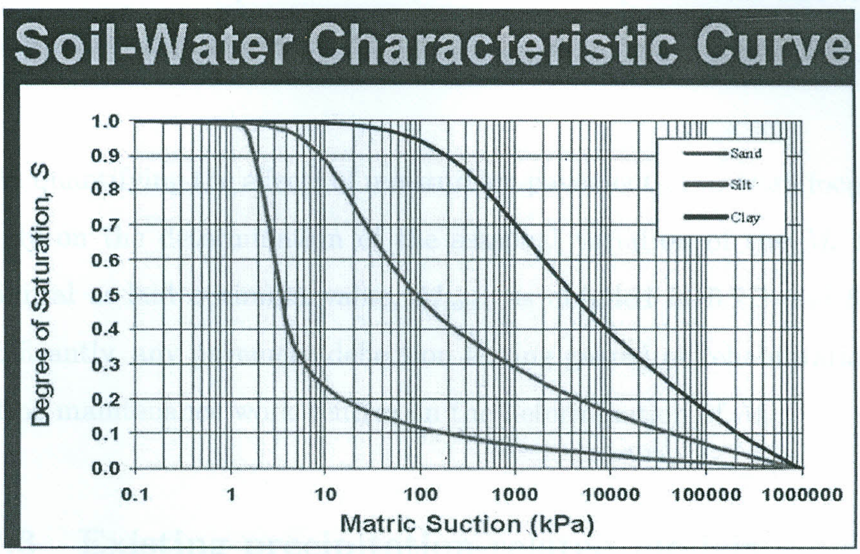


Figure 6.2.1: Soil water characteristic curve (after Zapata[43])

ing SWCC is vital in modelling the flow of moisture in unsaturated soils and thus predicting the moisture content in pavement design [43]. For this study, the SWCC bear a major significance since the SWCC model is the main moisture-prediction module implemented in EICM. A more elaborate treatment of the SWCC model follows in 6.2.3.

Resilient modulus, M_r

This is the maximum energy recoverable per unit volume that can be elastically stored [2], and it is represented by the area under the curve in the elastic region of a stress-strain curve. It is a measure of pavement response and its value is highly influenced by the variation in moisture content of the unbound layers. Applied both in empirical and in ME design, it is used to define fundamental material properties, formulate constitutive models, predict stress, strain and displacements and to develop performance models.

In quantifying the effects of moisture on pavements, our work focussed mainly on the determination of the seasonal variation of the M_r from an initial as-laid optimum value, M_{ropt}, as detailed in 6.2.3 and 6.2.4. Significantly, any pavement deflection testing geared at rehabilitation or routine maintenance work centres on the determination of M_r.

6.2.3 Existing precipitation-related models

As detailed in Chapter 2, the Collaborative Historical African Rainfall Model (CHARM) is a rainfall-specific weather prediction tool relying on reanalysis fields, interpolated gauge data and outputs of orographic models[10]. It is used in weather backcasting, more so in developing strategies in famine early warning systems. This study recognized the possibility of extending its use to road design.

In response and performance evaluation, the determination of the unbound layers' moisture content, and thus their resilient moduli, is central to the prediction of the permanent deformation (rutting) and other related distresses. Our analysis will thus focus on examining the transfer functions as they relate to moisture in the pavement.

A lot of engineering effort has been devoted to tracking the changes in the resilient moduli of pavement materials with fluctuations in their moisture content. Good AC and PCC surfacings are regarded as moisture-proof, hence such evaluations are confined to the unbound layers. The complexity in these evaluations arise due to the fact that different soil types react differently to moisture, leading to separate models for the fine and coarse-grained aggregates. Among the preeminent M_r models are the Li and Selig [24] ones which are of the quadratic form expressed as;

$$\log(\frac{M_r}{M_{ropt}}) = 0.98 - 0.28(w - w_{opt}) + 0.029(w - w_{opt})^2 \qquad (6.2.7)$$

where;

M_r= resilient modulus at moisture content w, and same dry density as M_{ropt}

M_{opt}= resilient modulus at maximum dry density and optimum moisture content, w_{opt}

The coefficients in the equation above vary depending on whether the same compactive effort is considered for M_r and M_{opt} or not [2], and on the material gradation. Irrationality exist in the models for the case where $w - w_{opt}$=0, since they give the value of the ratio $\frac{M_r}{M_{ropt}}$=0.98 (or lower for fine-grained soils) instead of unity as is expected.

In the EICM [2], the seasonal variation of the resilient modulus is obtained using the equation below;

$$\log(\frac{M_r}{M_{ropt}}) = a + \frac{b-a}{1+\exp[\ln\frac{-b}{a} + k_m(S - S_{opt})]} \tag{6.2.8}$$

where;

$\frac{M_r}{M_{ropt}}$= resilient modulus ratio

M_r= resilient modulus at any given time

M_{ropt}= resilient modulus at a reference condition

a= minimum of $\log(\frac{M_r}{M_{ropt}})$

b= maximum of $\log(\frac{M_r}{M_{ropt}})$

k_m= regression parameter

S= degree of saturation at any time

S_{opt}= optimum degree of saturation

M_{ropt} above is supplied by the user into the EICM module, and is the value obtained at optimum moisture content and at maximum dry density of the solids in the soil. Equation 6.2.8 above assumes no changes occur in the stress-state of the soil, thus the M_{ropt} is constant. However, the effects of stress are accounted for in computing the M_{ropt}, and this is given by the universal constitutive model;

$$M_{ropt} = k_1 P_a (\frac{\theta}{P_a})^{k_2} (\frac{\tau_{oct}}{P_a} + 1)^{k_3} \tag{6.2.9}$$

where;

k_1, k_2, k_3= regression parameters

P_a= atmospheric pressure

θ= bulk stress

τ_{oct}= octahedral shear stress

A lot of research effort has been expended on developing better M_r models with the most recent being that by Liang et al. [25] in 2008. An all-inclusive moisture and stress-state equation, it is expressed as;

$$M_r = k_1 Pa(\frac{\theta + \chi_w \psi_m}{P_a})^{k_2}(\frac{\tau_{oct}}{P_a} + 1)^{k_3} \tag{6.2.10}$$

where;

ψ_m matric suction

χ_w the Bishop's effective stress parameter

The other terms in the Liang et al. model have the same sense as in equation 6.2.9. Used for cohesive soils which predominate in Kenya, the predictive equation applies the soil suction concept and offers the advantage that fewer tests are required to determine its regression coefficients. Further, it combines the moisture content and stress state thus avoiding the need to determine the M_{ropt}, and it is hence more accurate. The term χ_w represents the contribution of matric suction to effective stress and has the numerical limit as the saturation (in decimal).

The EICM was developed with the regions experiencing winter conditions in mind (evident from the inclusion of the CCREL model). Freezing and thawing have major impacts on the resilient moduli of soils, with the former increasing the modulus by between 20 to 120 times its value from the unfrozen condition [2]. Associated with this is the formation of ice lenses which may lead to heaving of pavements in winter. When thawing occurs, cavities are left in the pavement leading to lowered structural integrity and susceptibility to damage.

More importantly, and complicating the analysis further, it is possible to have a single pavement structure being partially frozen with the different layers responding uniquely to the changes. To account for these variations, the EICM engine subdivides the pavement into smaller cells with nodes and assigns an environmental adjustment factor, F_{env} to each node depending on whether the node is unfrozen (F_u), freezing (F_f) or recovering/thawing (F_r). The F_{env} is a composite index for all the three soil states. It is one of the major outputs of the EICM and is used to determine the instantaneous resilient modulus of the overall pavement as a function of time t, and position or depth x, using the relation;

$$M_r(x,t) = F_{env}.M_{ropt} \tag{6.2.11}$$

To account for the effects of stress and the environmental factors together on the resilient modulus, equation 6.2.9 is included in the above equation to give;

$$M_r = F_{env}.k_1 P_a(\frac{\theta}{P_a})^{k_2}(\frac{\tau_{oct}}{P_a}+1)^{k_3} \tag{6.2.12}$$

The guide [2] details an elaborate algorithm for the determination of the F_{env} using a serial-spring analogy, and relying on sub-layering of the pavement.

The soil water characteristic curve (SWCC) is used to obtain the instantaneous volumetric moisture content as a function of the suction. SWCC is obtained for a given soil by measuring the suction of a representative sample in the laboratory using filter paper or pressure plate tests [43], a tedious process that takes between days and weeks. Laboratory determination of SWCC is not widely performed due to the special equipment required and the difficulty associated with the process. When carried out, a family of curves depending on the soil types is obtained. Different SWCC models have been used, but the Fredlund and Xing one [2] was picked for implementation into the EICM owing to its robustness to sensitivity and reliability. It is represented as;

$$\theta = \theta_s(\frac{1}{\ln(e + (\frac{\psi}{a})^n)})^m \tag{6.2.13}$$

where;

θ is the volumetric moisture content (vmc)

θ_s is the residual volumetric moisture content

ψ is the suction

a, n, m are SWCC parameters.

The average of the vmc values are fed into the structural analysis program of the ME engine for the prediction of permanent deformation of the unbound layers. These and other distresses are covered in chapter 4. The schema on figure 6.2.2 illustrates the interrelation between the M_r and the SWCC, and their use in the determination of the F_{env}. The acronym TMI refers to the Thornthwaite Moisture Index, a measure of the aridity/humidity of a soil-climate system. More detailed treatment of the TMI is made by Quintero [34].

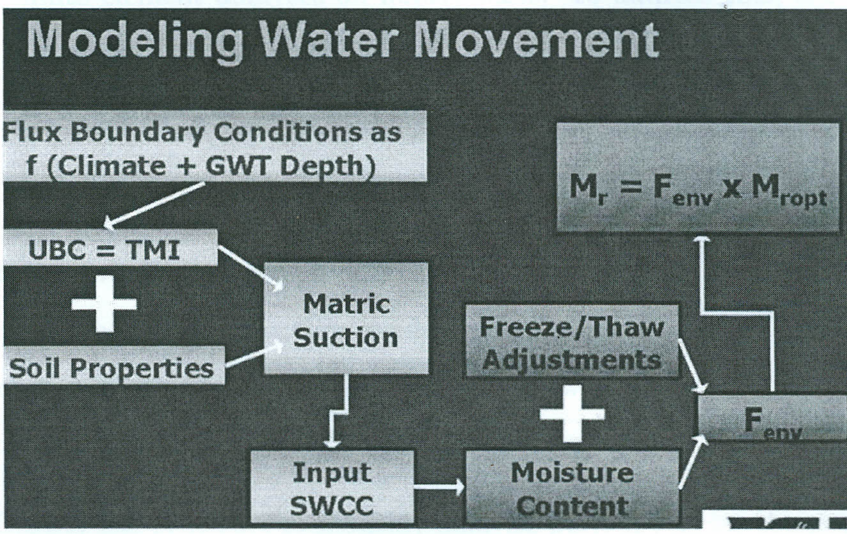


Figure 6.2.2: Modelling water movement in soil (after Witczak et al[43])

6.2.4 Adaptation to Kenyan case

From the perspective of precipitation/moisture inputs into the EICM and analyses of both the EICM and the rainfall patterns and records in Kenya, the study found that the existing records can be used in the models without much loss in their prediction capacities.

While inputs into the EICM are hourly, the outputs and their use in the analysis of response and performance predictions are of far much longer intervals. The guide [2] states thus;

> "Although the EICM provides environmental data on an hourly basis, it is obviously impractical to perform linear elastic or

finite element analyses on hourly basis. To address this, the analysis period (design life) is subdivided into 1-month or 2-week periods, at the end of which stress-strain analyses are performed."

Carrying out the analysis on hourly basis would be ideal, but the cost in terms of the computation time would be enormous (Consider for a design life of 30 years: 30X365X24=262800 analyses would be required).

Clearly the bi-weekly or monthly analysis periods rely on the averages to give more representative estimations. Indeed the shorter period (2 weeks) is confined to those seasons associated with freezing and thawing [43], since rapid changes occur in the material properties then. Kenya on the other hand never experiences such extreme weather, with any freezing confined to mountain peaks (specifically Mount Kenya), where paved roads are seldom built. Along with this, it would be easy to compute the environmental adjustment factor for the determination of the resilient modulus, as only the unfrozen component would be required i.e. $F_{env}=F_u$.

Of all the weather parameters collected in Kenya, rainfall is the most common. The Kenya Meteorological Department (KMD) [20] classifies the weather stations into 3:

Rainfall Stations Those that exclusively record rainfall. It is noted that as far back as 1977, 2000 of these station were actively collecting data, but currently only 700 are operating.

Temperature Stations These collect both temperature and rainfall data, with 62 of them presently active.

Synoptic Stations The most comprehensive and professionally manned by KMD staff, they collect rainfall, temperature, evaporation, wind speed and direction, sunshine hours, visibility, solar radiation, atmospheric pressure and humidity. 27 stations of this type are spread around Kenya and are in working order.

An analysis of their distribution within the country shows they can adequately cater for the site-specific nature required for pavement design. For those regions not covered, the generation of virtual weather stations by weighted averaging using the five nearest stations as described in the [2] is recommended. This algorithm uses the inverse square method.

Ninety six (96) years' rainfall patterns for major centres in Kenya as obtained from records off the Weatherbase website [39] show a distribution with highest values recorded in the month of May while the least values correspond to February.

From the foregoing, it is clear that the available rainfall records as contained in the KMD database can adequately be applied to the models in ME design with little loss in the reliability of their predictions. We thus believe that daily rainfall data can be as good as hourly for use in road design.

6.3 Temperature

6.3.1 Significance and Measurement

As stated in 4.4, temperature is one of the main climatic factors affecting the response behaviour, and thus performance, of a pavement structure. The influence of temperature variations is mostly felt by the bound materials than the lower layers [2]. This is largely because asphalt exhibits a viscoelastic property, usually 'stretching' easily with the increase in temperature and hardening when cooled. Indeed it displays a liquid-like property at elevated temperatures, a phenomenon exploited during the paving process to have it bind the aggregates, and thus the term hot mix asphalt (HMA).

The fluctuations in temperature for a pavement thus have a major influence on the extent to which the surface will react to the wheel loads traversing it. The determinant in this case is the stiffness property of the AC, more specifically referred to as the dynamic modulus, E, and defined as the ratio between the stress and strain under continuous sinusoidal loading [45]. Temperature also influences the plastic strains and the fatigue life of the pavement [38].

The traditional weather station features the liquid-in-glass thermometers for temperature determination. However, recent trends have seen the development of more robust heat sensors like the thermistor [34]. The latter is particularly useful where the pavement structure needs to be instrumented, and relies on the electrical conductivity of the material of which it is made varying with fluctuations in temperature. This relationship is as shown [8];

$$R = R_0 \exp[\beta(\frac{1}{T} - \frac{1}{T_0})] \tag{6.3.1}$$

where:

R= resistance at temperature T

R_0= resistance at temperature T_0

β= a constant depending on electrical conductivity of the material.

Some of the pavement distresses associated with temperature include thermal cracking (usually in temperate climates), rutting and fatigue cracking.

6.3.2 Existing temperature models

The relationship between the air temperature and the pavement temperature is not necessarily a linear one. This is because predicting the temperature profile of the asphaltic material from the air temperature is complicated, often requiring detailed information about the thermal properties of the asphalt and the ambient conditions [7]. Further, many other factors like the solar radiation intensity, ambient wind conditions, pavement geometry and orientation need to be considered [44].

The problem is further compounded when other pavement layers (base, subbase and subgrade) with varying material properties are considered, observing that the centre of the earth itself is continuously dissipating heat towards the surface. One of the models describing the movement of heat in the ground is the one-dimensional heat equation developed by Wilson [41] and given as;

$$\zeta\frac{\partial T}{\partial t} = \frac{\partial}{\partial t}(k_t\frac{\partial T}{\partial z}) - L_v(\frac{\bar{u}_a + \bar{u}_v}{\bar{u}_a})\frac{\partial}{\partial z}(D_v\frac{\partial \bar{u}_v}{\partial z}) \qquad (6.3.2)$$

where;

T= the temperature

$\bar{u}_a$= the atmospheric pressure

$\bar{u}_v$= the vapor pressure

ζ= the volumetric specific heat capacity as a function of the moisture content

k= the soil's thermal conductivity as a function of the moisture content

L_v= the latent heat

D_v= the diffusion coefficient of water flowing through the soil.

The equation above describes the heat flow due to conduction and the latent heat transfer due to a change of phase. Yavuzturk and Ksaibati [44] developed a model that could reliably predict the asphalt pavement temperature at the surface, at 20mm depths and horizontal locations based on ambient air temperatures using a two-dimensional numerical procedure giving both maximum and minimum temperatures that design engineers could use for in situ back-calculation of pavement modulus values. Their model is expressed below;

For minimum pavement temperature:

$$T_{surface} = -1.56 + 0.72T_{air} - 0.004Lat^2 + 6.26\log_{10}(H+25) - z(4.4 + 0.52\sigma^2)^{\frac{1}{2}} \quad (6.3.3)$$

where;

Lat= latitude of the location

T_{air}= the low air temperature in ^{0}C

H= depth to surface in mm

z= 2.055 for 98% reliability

$T_{surface}$= temperature at the pavement surface

σ= standard deviation of the mean low air temperature

For maximum pavement temperature:

$$T_{20mm} = (T_{air} - 0.00618Lat^2 + 0.2289Lat + 44.2)(0.9545) - 17.78 \quad (6.3.4)$$

where:

Lat= latitude of the location

T_{air}= the seven-day average high air temperature in ^{0}C

T_{20mm}= pavement temperature at a depth of 20mm.

The necessity of a model for maximum temperature is driven by the fact that for pavement engineers and designers in arid and other hot climates, like those experienced in Northern Kenya, Coast and around the Lake Victoria basin, this is the primary consideration [44]. The key limitation of the model above for the consideration of maximum temperatures is that it was confined to depths of 20mm.

The temperature profiles within the different layers of a pavement are passed on to the transfer functions (damage prediction models). In analysing the distresses, one approach is to assign a single representative temperature value called the effective temperature T_{eff}, which can be defined as [13];

> "a single test temperature at which an amount of a given type of distress within a given pavement system would be equivalent to that which occurs from the seasonal temperature fluctuation throughout the annual temperature cycle."

This value would then be input into the transfer functions for an analysis period. The initial T_{eff} model, a multiple regression predictive equation, was formulated by Witczak [42] and was of the form;

$$T_{eff} = 58.0 - 5.5(z) + 0.92(MAAT) \tag{6.3.5}$$

where:

T_{eff}= effective temperature in ^{0}F

z=any desired critical depth

$MAAT$=mean annual air temperature

Validation of the model showed reasonable agreement between field observed and predicted rut-depth propagation for some cases. However for hotter climates, large and unexpected disparities existed between the two values exposing their insufficiency in performance prediction.

El-Basyouny and Jeong [13] later refined the T_{eff} model incorporating the influences of traffic loading frequency, precipitation, sunshine and wind. More significantly,they also reduced the interval of the temperature measurements, including the average monthly value in addition to the annual one. Their final model is given as;

$$\begin{aligned} T_{eff} &= -13.995 - 2332(Freq)^{0.5} + 1.006(MAAT) + 0.876(\sigma MMAT) - \\ &\quad 1.186(wind) + 0.549(sunshine) + 0.071(rain) \end{aligned} \tag{6.3.6}$$

where:

$Freq$= traffic loading frequency, Hz

$rain$= annual cumulative rainfall depth, in

$wind$= mean annual wind speed, mph

$sunshine$= mean annual percentage sunshine

$\sigma MMAT$= standard deviation of the mean monthly air temperatures

As noted in 4.4, the EICM is a one dimensional coupled heat and moisture flow model. The heat component of it is implemented as a finite difference equation expressed as [35];

$$T_{(i,t+\Delta t)} = T_{(i,t)} + (\frac{k\Delta t}{\gamma_d} C\Delta z^2)[T_{(i+1,t)} + T_{(i-1,t)} + T_{(i,t)}] \tag{6.3.7}$$

where:

T= temperature in F

t= time

Δt= time increment

Δz= depth increment

k= thermal conductivity in $BTU/(hr - ft^2 - F)$

γ_d= dry density

C= mass specific heat in $BTU/(hr - F)$

z= vertical co-ordinate

Recognizing the significance of hourly weather data in design, this study developed a model to simulate hourly data from daily (maximum-minimum). This is discussed below.

By its equatorial geographical positioning, the country experiences near-equal lengths of days and nights all year round. From the perspective of temperature distribution, this study conducted a comparative analysis of the hourly air temperature distribution during the equinoxes(21 March and 23 September) over 4 years for a location in the US (Hawaii). The choice of the location was based on its geographical similarity to a typical Kenyan coastal town using the Köppen-Geiger climate classification [22], while that for the duration of recorded temperatures was motivated by the fact that for the two dates, the day and night lengths equal. In particular, Hawaii falls under the tropical wet and dry or savannah climate type, As, with the dry season occurring during the time of higher sun and longer days. This is the same case with Mombasa in Kenya [22]. The values were then plotted as shown in Figure 6.3.1 By inspection, it is clear that if continuity is enforced in the plots (assuming the curves for the next days are placed to start where the previous days' end), a roughly sinusoidal pattern is obtained. Taking into consideration the unique climate of Kenya, more so the near-equal day and night lengths, the study developed the following cosine function model to simulate hourly temperature records from daily minimum-maximum ones;

$$T_h = (\frac{T_{max} + T_{min}}{2}) + (\frac{T_{min} - T_{max}}{2})\{-cos[\frac{(H - H_o)\pi}{12}]\} \qquad (6.3.8)$$

where:

T_h= instantaneous temperature at hour h

T_{max}= maximum temperature recorded in a given day

T_{min}= minimum temperature recorded in a given day

H= hour number from midnight

H_o= hour number corresponding to maximum temperature

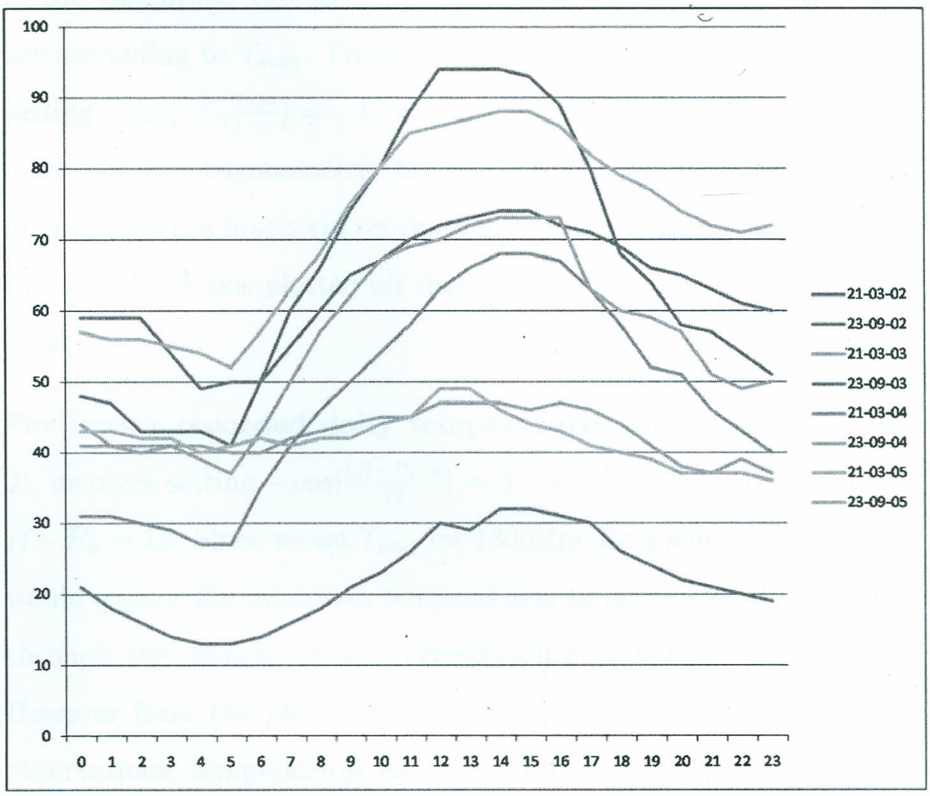


Figure 6.3.1: Temperature (^0F) variations over 24 hours in Hawaii during the Equinoxes (2002-2005)

A consideration of the extreme values of the model follows as under:

1. **For highest recorded daily temperature**: An analysis of the 8 temperature series plots in the figure above reveals that the maximum temperatures are recorded between 1200Hrs and 1400Hrs. This is consistent with the case in Kenya where gauge readings hit their peaks during the same interval barring any rainfall events or extreme circumstances like prolonged cloud cover. For the purpose

of the validation, the mean of the two i.e 1300Hrs is set as time corresponding to T_{max}. From the model then, T_h is maximized by setting $-cos[\frac{(H-H_o)\pi}{12}] = -1. \Rightarrow \frac{(H-H_o)\pi}{12} = 0$, or $H = H_o$. Furthermore, for our trigonometric function curve, the mean position is provided by the first term on the righthand side of the model, with the horizontal axis plotted off this value on vertical (temperature) axis.

2. **For lowest recorded daily temperature**: The minimization of T_h involves setting $-cos[\frac{(H-H_o)\pi}{12}] = 1. \Rightarrow \frac{(H-H_o)\pi}{12} = \Pi$, or when $H - H_o = 12$. Since we set T_{max} for 1300Hrs, for a smooth curve, we would expect the minimum temperatures to occur exactly halfway through the 24-hour cycle, corresponding to a time of 0100Hrs. However from the plots, and from both empirical and anecdotal observations, temperatures hit their nadirs two to five hours after this time. Consequently, the lowest hourly temperature in any typical day was set to occur at 0400Hrs. To account for this anomaly, a temperature shift factor equivalent to $\frac{\pi}{4}$ is included in the model for the periods between 0000Hrs and 0600Hrs. The final form of the model for the six hour interval then becomes;

$$T_h = (\frac{T_{max} + T_{min}}{2}) + (\frac{T_{min} - T_{max}}{2})\{-cos[(\frac{(H - H_o)\pi}{12}) + \frac{\pi}{4}]\} \tag{6.3.9}$$

The addition of $\frac{\pi}{4}$ to the last term in the equation above ensures that it corrects for the 3-hour lag time represented by the value.

The model proposed however has constraints as it assumes that no rainfall events occur. Rainfall events in Kenya generally lower the ambient temperature, and the model does not account for this.

6.3.3 Adaptation to Kenyan case

The quest to bridge the climatic data collection gaps has seen many attempts devised using interpolation and empirical correlations. Wilson [40] for instance modeled daily pavement temperature using a non-linear interpolating function. The ultimate objective of any such endeavour is to obtain shorter time intervals for use in the discretization of the finite difference grids, since the finer the grids, the more accurate the approximations.

Although it lies between the tropics and straddles the Equator, Kenya has a very diverse climatic distribution. The Kenyan road design manual [28] describes it thus;

> "Kenya has a very wide variety of climates comprising; afro-alpine, equatorial, wet tropical, semi-arid, arid and very arid climates. Moreover, the pattern of the climatic zones is rather complex since the Kenyan climates are largely governed by altitude."

In spite of the climatic disparity noted above, temperature variations in a typical day in Kenya generally follow a similar trend to the one shown on the curve. This study thus recommends the use of the model developed to simulate hourly temperature data from daily, due to the criticality of the estimates and distresses involved.

6.4 Percentage sunshine

6.4.1 Measurement

The recording of the duration of sunshine in Kenyan stations is achieved using the Campbell-Stokes recorder [20] consisting of a glass orb suspended on an axle above a bowl-shaped container with a graduated card at the bottom. This is exposed to the sun and the orb refracts the rays into a sharp focus of high temperature which burn holes through the card. Counting through the burnt out holes, the percentage sunshine for the day is then computed.

6.4.2 Sunshine in ME design

Sunshine duration is used to compute the cloud cover. Unlike temperature and moisture, it is not a stand-alone weather parameter in pavement design, since it has a modifying effect on temperature (thus heat energy) for which it provides the upper boundary conditions, in the EICM models [2]. The largest source of thermal energy in pavement layers is the sun, and the primary modes of its transfer are incident solar radiation, convection (between pavement surface and fluid in contact with it), the conduction within the pavement, long-wave and thermal radiation between pavement surface and sky [44].

With such diverse transfer mechanisms, the need to conserve the energy has given rise to many energy balance models. At the pavement surface, the basic concept in all the models is that the sum of all the heat energy gained must equal the heat conducted in the pavement. The generic energy balance equation has the form [2];

$$Q_i - Q_r + Q_a - Q_e \pm Q_c \pm Q_h \pm Q_g = 0 \tag{6.4.1}$$

where:

Q_i= incoming short-wave radiation

Q_r= reflected short-wave radiation

Q_a= incoming long-wave radiation

Q_e= reflected long-wave radiation

Q_c= convective heat transfer

Q_h= effects of transpiration, evaporation, sublimation, and condensation

Q_g= energy absorbed by the ground

The expression above shows that the pavement both absorbs and emits/reflect heat energy depending on the ambient temperature, with the equation serving as the upper boundary condition for the CMS model, the lower boundary conditions being the constant deep ground heat source. The net all-wave radiation at the surface is then given by;

$$Q_n = Q_s - Q_l \tag{6.4.2}$$

where:

Q_s= net short-wave radiation

Q_l= net long-wave radiation,

with $Q_s = Q_i - Q_r$ and $Q_l = Q_a - Q_e$.

The presence of clouds enhance the reflection of the solar energy radiated by the sun so that it does not reach the pavement, yet it similarly contributes to the greenhouse effect, trapping the reflected and emitted radiations off the pavement above the ground. The percentage sunshine, S_c, contributing to the cloud cover is implemented in the MEPDG as a function of material property like short and long-wave absorptivity, emissivity, and reflectivity. The equation used is that developed by Baker and Haines [2] as;

$$Q_s = a_s R^*[A + B\frac{S_c}{100}] \tag{6.4.3}$$

where:

a_s= short-wave absorptivity of pavement surface

R^*= extra-terrestrial radiation incident on the horizontal surface at the outer atmosphere

A, B= constants accounting for diffuse scattering and adsorption of the atmosphere

S_c= % of sunshine accounting for the influence of the cloud cover

R^* in the equation above depends on the position of the sun N or S of the Equator, and is a function of the time of the year only.

Accounting for both outgoing and incoming long-wave radiation involves a cloud cover correction factor equal to $1 - \frac{NW}{100}$, so that the values in equation 5.4.1 become;

$Q_a = Q_z(1 - \frac{NW}{100})$ and $Q_e = Q_x(1 - \frac{NW}{100})$, thus $Q_l = (Q_z - Q_x)(1 - \frac{NW}{100})$, with;

$$Q_z = \sigma_{sb} T_{air}(G - \frac{J}{10^{\rho p}}) \tag{6.4.4}$$

where:

N= cloud base cover(approximately 0.9-0.8 for clouds between 1000-6000ft)

$W = 100 - S_c$ = average cover during day and night

T_{air}= air temperature in Rankine

σ_{sb}= Stefan-Boltzman constant$=1.72X10^{-8}Btu.hr^{-1}.ft^{-2}.^0R^{-1}$

G, J,andρ are constants=0.77, 0.28 and 0.074 respectively

p= air vapour pressure, ranges between 1-10mmHg

Q_x= outgoing long wave radiation without cloud cover correction

The guide [2] shows that the uncorrected long-wave radiation Q_x, is given by the formular;

$$Q_x = \sigma_{sb} \varepsilon T_s^4 \tag{6.4.5}$$

where:

ε= emissivity of the pavement surface (typical value=0.93)

T_s= pavement surface temperature in Rankine ε in the equation above is a function of the pavement colour, texture and temperature.

6.4.3 Adaptation to Kenyan case

It is clear from sections 6.4.1 and 6.4.2 that the percentage sunshine and the cloud cover exhibit a linear inverse relationship. Indeed the term W in the correction factor shows that the percentage cloud cover is maximum when the no sun shines. For Kenya that enjoys near-equal day and night lengths, this means that we can confidently assign a 100%-value for cloud cover to each of the 12 hours from sunset to sunrise, solving half of the problem in the process.

An overview of the climatic data from the American weather agency, the National Climatic Data Center (NCDC), used in the LTPP pavement analysis shows that most recording of variability of the percentage sunshine is achieved by adopting a quarter-based data collection system. In this, the values are given as; 0%, 25%, 50%, 75% or 100%. This study strongly favours this approach for Kenya whose few synoptic stations (27 in number) keep sunshine data [20].

For those stations totally lacking this weather parameter, two alternatives are proposed:

- An interpolation procedure using nearby weather station records to generate virtual weather data, similar to that suggested in [2] for weather record-poor locations. This involves weighted averaging.

- Simulation of hourly percentage sunshine. As stated earlier, only the daily hourly records need to be determined for Kenya. Assuming no adverse weather events and usual mild air movements prevail, a positive correlation exist between the ambient daytime temperature and the percentage sunshine, with the latter rising from a minimum at daybreak, peaking when the sun is directly overhead and falling back to zero at nightfall. The sunshine recorders are noted to barely scorch the graduated papers in the morning and evening [20]. Under these conditions, this study recommends the use of a normal distribution curve to assign the hourly sunshine values.

6.5 Wind

6.5.1 Measurement

Only the synoptic stations in Kenya keep wind-related records, and this is achieved using the cup-counter anemometers [20]. The records obtained include the direction and the speed. In more automated stations, the windrun is measured using an alternating current (AC) sine wave-generating propeller, with the wave frequency proportional to the wind speed. The direction is then measured using a potentiometer whose response to an excitation voltage is directly proportional to the azimuth [34].

6.5.2 Wind in ME design

In the design process, wind speed is a necessity in computing the convective heat transfer coefficient, H [2]. Generally, an increase in the speed of wind in contact with a surface will enhance the transfer of heat away from the surface with the attendant cooling effect on the same surface. In this regard, the temperature of a pavement surface will be reduced when wind blows over it. It follows that warmer inner layers will conduct heat outwards, leading to the overall gradual cooling of the whole pavement.

Patel [33] noted that wind is never steady at any site, being influenced by the weather system, the local land terrain, and the height above the ground surface. The variation in wind speed occurs after very short term intervals of the order of seconds. This poses a major challenge in its measurement leading most authors to favour a probabilistic approach to its computation. He suggested the best way to describe the variation in wind speed is to use the Weibull probability distribution function h, with two parameters, the shape parameter k, and the scale parameter c. Using this method, the probability of wind speed being v during any time interval is given by;

$$h(v) = (\frac{k}{c})(\frac{v}{c})^{(k-1)}e^{-(\frac{v}{c})^k} \tag{6.5.1}$$

Basically, this works from the premise that the probability that the wind speed will be between zero and infinity during a given period is unity, mathematically expressed as;

$$\int_0^\infty h.dv = 1 \qquad (6.5.2)$$

where:

$v=$ average daily wind speed

$h=$ probability distribution function

Mathematically, accounting for the wind effects implies the quantification of the term, Q_c in equation 5.4.1 above. This is expressed as [2];

$$Q_c = H(T_{air} - T_s) \qquad (6.5.3)$$

where:

$Q_c=$ rate of heat transfer by convection

$H=$ convective heat transfer coefficient

$T_{air}=$ air temperature, in ^0F

$T_s=$ pavement surface temperature, in ^0F

The guide [2] noted that determining H is difficult owing to the many variables that influence it, but suggest an alternative formulation of it that directly involves the speed of wind, expressed as;

$$H = 122.93[0.00144T_m^{0.3}U^{0.7} + 0.00097(T_s - T_{air})^{0.3}] \qquad (6.5.4)$$

where:

U= average daily wind speed, in m/s

T_{air}= air temperature, in ^0C

T_s= pavement surface temperature, in ^0C

T_m= mean value of T_{air} and T_s

The stability criteria imposed in the EICM finite difference computations partly controls the maximum value of H even though the guide [2] recommended a value of 3.0 $Btu.hr^{-1}.ft^{-2}.^0F^{-1}$.

6.5.3 Adaptation to Kenyan case

Generally, the wind speeds in Kenya are mild, with most parts of the country recording values ranging between 9km/h and 16km/h [39]. The exception to this is the Lake Turkana basin that has high winds whose speeds average 39km/h [23], a phenomenon that has the largest ever wind-powered electricity generation plant in Africa being constructed in the area.

The KMD keeps the average daily wind speeds for the 27 synoptic stations. For those stations that totally lack wind speed data, this study recommends the use of a special case of the Weibull probability distribution called the Raleigh distribution. This distribution generates the hourly wind speeds from the daily averages, but since the stations have no wind records, regional averages should be assigned as individual stations'. The complete procedure for this simulation is described by Patel

[33]. However, for situations where actual daily records are available, we recommend their use for the predictions. Indeed the guide [2] suggests that in instances where the hourly data is not available, daily mean values suffice for the analyses. Using either of these approaches, Kenyan pavement designers could reliably infuse the wind speed parameter into the ME design procedure.

6.6 Humidity

6.6.1 Measurement

This is the quantity of the water vapour present in the atmosphere and it is measured using a hygrometer [20], although recent times have seen the development of thermohygrometers that measure it in addition to temperature. It is often expressed as a percentage of some assigned value, thus we talk of the relative humidity.

6.6.2 Humidity in ME design

Flexible pavements are not susceptible to humidity [2], thus any consideration of this factor on them is moot. PCC pavements on the other hand exhibit warping and curling, processes affected by the amount of moisture in the air, greatly influencing the pavement fatigue behaviour. The contribution of relative humidity to PCC distresses, chiefly warping and curling, is quantified by converting them to the equivalent contribution wrought by temperature fluctuations [2]. Mathematically, this is calculated as;

$$ETG_{SHi} = \frac{3(\varphi\varepsilon_{su}).(S_{hi} - S_{havg}).h_s(\frac{h}{2} - \frac{h_s}{3})}{\alpha.h_s.100} \tag{6.6.1}$$

where:

ETG_{SHi}= temperature difference equivalent of deviation of moisture warping in month i from annual average %

φ = reversible shrinkage factor

ε_{su}= ultimate shrinkage

S_{hi}= relative humidity factor for month i, with;

1. S_{hi}=1.1XRH_a for RH_a < 30%
2. S_{hi}=1.4X0.01XRH_a for 30% ≤ RH_a < 80%
3. S_{hi}=3.0X0.03XRH_a for RH_a > 80%

RH_a= ambient average relative humidity

S_{havg}= annual average humidity factor

h_s= depth of shrinkage zone

h= PCC slab thickness

α= PCC coefficient of thermal expansion

6.6.3 Adaptation to Kenyan case

As the equation above shows, computing the extent of the PCC slab shrinkage due to humidity is done once during the analysis period (one month). Even though EICM takes in hourly values, the effective one used in the analysis is an arithmetic mean monthly value [2]. In the Kenyan context, and as noted earlier, the available length of rigid pavement is negligible compared to the AC ones, hence using the daily averages from which monthly ones are obtained is cost effective.

6.7 Ground water table

6.7.1 Measurement and significance in ME design

The measurement of the ground water table depth is vital since its fluctuation has a direct influence on the resilient modulus of the unbound pavement materials similar to those considered under precipitation. In this, it is regarded as the lower boundary condition for the moisture flow models [2]. Its measure is achieved using the TDR discussed earlier with several models like the Darcy's law used to describe its flow.

6.7.2 Consideration for Kenya

Hydrogeological maps for the country show good potential for ground water availability in Western Kenya, mid-North Rift, the coastal basin and the Kilimanjaro and the Merti aquifer regions shared by Tanzania and Somalia respectively [31]. The rest of the country can be regarded having low potential for reasons ranging from having deep free-draining sandy soils to lacking in aquifers to trap the infiltrated water. Inputs of ground water depths into the EICM are hourly, even though for analysis of the response and performance, the seasonal averages are adequate [2]. This study proposes that the available geohydrological averages be used since they are adequate to meet the ME design requirements outside of the EICM.

Chapter 7

Conclusions and Recommendations

7.1 Conclusions

With reference to the objectives, the main objective for this research was to devise mechanisms of incorporating the mechanistic-empirical pavement design in Kenya, infusing weather-related mathematical models in the design process. Overall, we have established that with suitable adaptation, the existing climatic data can be used as inputs into the models with little loss in the reliability of their predictions. Specifically, we see no mathematical reason why the lack of hourly climatic data should hinder the country from embracing the ME pavement design approach.

It was clearly apparent that the most important climatic variables were rainfall and temperature with sunshine, wind, humidity and ground water depth playing supplementary roles to the two. Determining the influence of rainfall was made easy by adopting the new resilient modulus model proposed by Liang et al. on the one hand, while the analysis involving it in the transfer functions was shown to work equally well with daily averages on the other. Temperature requires hourly intervals for both thermal cracking and rutting predictions, but further investigation showed that the sine wave model should be good enough for producing the estimates needed to fill in the missing records. We further believe that the other four weather variables such as precipitation, sunshine, humidity and groundwater table depth can be used in their current forms without grossly undermining the performance predictions.

7.2 Recommendations

Based on this study, we recommend that an interdisciplinary study programme be initiated to validate the findings for Kenya. Further, such study should involve the integration of the climatic records from the Kenya Meteorological Department in their present form for use in the models.

For road design purposes, the mathematical simulation of hourly temperature data from daily was is in this study and is found to be adequate. However, for other uses this might not suffice. In this regard, we propose that more research work be carried out on improving the model for other instances. Such study might involve both mathematical and statistical simulations including the effects of random rainfall events and prolonged cloud cover on the temperature predictions.

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