Development Of A Sustainable Mechanistic Design Procedure For Flexible Pavements In The Tropics

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Abstract: - Mechanistic design approach requires proper fundamental material characterization and material failure criteria as a function of load and environmental conditions. Most of the material characterization models and failure criteria available for the analysis and design of pavement systems were developed for the temperate regions. However, there are limitations in applying these models and criteria outside the developed jurisdiction, especially in the tropical regions. Therefore, a comparative study of these models and criteria is made with a goal to developing the best for tropical regions. Based on correlation with field results collected from several randomly selected pavement sections in the tropical region, the widely used Smith and Witczak material characterization model and Claros and Ijeh failure criteria are selected for further study and incorporated into a design procedure. Modifications of the algorithms of the models selected are made in order to reflect the material properties and variability; cost-effectiveness and to give a better fit to the measured field values. The model and failure criteria are able to accurately define the states of stresses and strains in the pavement structure as well as offer good translation of the mathematical findings into performance behavior. Subsequently, the developed model gives a superior correlation with the field results obtained in the tropics. Additionally the developed design procedure is utilized to obtain important information relative to the influence of all pavement variables upon the probable performance of flexible pavement, thereby aiding in optimization of pavement variables. From this mechanistic design procedure a cost–effective pavement chart reflecting the influence of pavement variables on performance in the tropics is also developed. For a more strategic sustainable pavement system a life-cycle cost analysis framework such as RealCost can be incorporated in the design procedure. The life-cycle cost analysis takes into consideration the constriction cost, the maintenance cost, and the users cost. To ensure sustainability of the pavement throughout its life-cycle apart from incorporating in the users cost, the delay cost, vehicle operating cost, accident cost, the environmental cost (energy use, emissions, waste, noise and water pollution) are also taken into consideration.

Key-Words: Flexible pavements; Mechanistic design procedure; Cost-effective; Sustainability; Pavement performance; Strains.

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1 Introduction

The aim of a pavement is to carry traffic safely, conveniently and economically over its design life by protecting the subgrade from the effects of traffic and climate and by ensuring that no materials used in the structure suffer any unacceptable deterioration. More recently, there is a growing desire for environmental sustainability of pavement throughout its life-cycle. A sustainable pavement system is safe, healthy, affordable, and renewable, operates fairly, and limits emissions and the use of new and nonrenewable resources [1]. It is usually very difficult to quantify environmental performance and stewardship in pavement design and construction. The basis of environmental sustainability consists of the three elements shown in Figure 1: economy, society and environment. Sustainable pavement design is a subset of sustainable transportation where the impacts of the design and construction the economy, environment, and social equity are defined and evaluated.

![Fundamental Sustainability Model](image)

Fig. 1 Fundamental Sustainability Model [2]

Hitherto to the design procedure majorly used in the tropics are the empirical methods; these methods do not explicitly consider an optimization of flexible pavement components to minimize the total cost of the pavement system. Although this cost minimization must take cognizance of the serviceability and structural stability of the pavement. Therefore, with increased construction and maintenance of streets and highways in tropical developing countries, it is imperative that a mechanistic structural design procedure which can provide a wide usage of traffic wheel loadings, fundamental material properties, and variation in pavement components and failure criteria be adopted.

The design procedure will also be valuable in the probable effects of changes in loading or materials properties or pavement thickness on the performance of flexible pavement without the necessity of carrying out extensive and lengthy full-scale road tests.

Mechanistic procedures have also been useful in the investigation of structural effectiveness with pavements since they allow the identification of the part of the structure most likely contributing to the observed effect and may indicate the types of remedial action to be effective [3].

Smith and Witczak [4] developed a model that can determine the influence of pavement variables on performance. Adeyeri and Owolabi [5] further modified the model to suit tropical conditions. Ever since there have been various attempts to develop mechanics design procedure for the tropics. Furthermore, for a better sustainable decision making a life-cycle cost analysis can also be incorporated in the design procedure. The life-cycle cost analysis takes into consideration the construction cost, the maintenance cost, and the users cost to ensure sustainability of the pavement throughout its life-cycle apart from incorporating in the users cost, the delay cost, vehicle operating cost, accident cost, and the environmental cost (energy use, emissions, waste, noise and water pollution)

The purpose of this paper is to review some of the models that are used in the mechanistic design procedure for flexible pavements and compare the results obtained from these models with field data collected from a high temperature (33°C) and rainfall (2000mm) environment. The paper also aims at assessing the probable effects of changes in material properties and thickness as on the performance of flexible pavements without the necessity of carrying out expensive and lengthy full-scale road trials. This will be with a view of developing a more cost-effective pavement design in the tropics. A life-cycle cost analysis can be further used in comparing and judging the efficiency of different design alternatives.

2 Mechanistic Design Procedure

The mechanistic design procedure involves the analysis of stresses and strains in the pavement structure and the determination of the allowable stresses and strains that the pavement materials can withstand.

The essential features of the mechanistic design procedure as observed by Peattie [3] for flexible pavements are:

1) the selection of a suitable elastic or visco-elastic model to represent the pavement structure.
2) solutions to the equations for the stresses, strains and deformations in the model.

3) the characterization, in fundamental terms, of the mechanical properties of the materials in the layers under appropriate climatic and loading conditions.

4) the definition of criteria for design and performance in fundamental terms of stress, strain and deformation.

5) the presentation of the design system in a form that is convenient for use by highway engineers.

The degrees of success achieved by the systems are measured by assessing the accuracy to which these requirements are used.

2.1 Design Models
The adoption of suitable pavement models has been affected by the constant conflict between the desire to represent the pavement structure by a comprehensive and realistic model and the considerable difficulties encountered in solving the equations for stress and strain as the complexity of the model increases.

In order to provide a direct method of pavement design, the elements comprising the pavement model, the procedure for stress analysis, the properties of the pavement materials, the traffic loading and climate, and the design criteria have to be combined into a comprehensive system. To reduce the amount of work involved while still representing a pavement realistically, Peattie [3] has observed that some of the structural design systems have adopted a 3-layer model. All the bituminous layers are considered together as the top layer, all the unbound granular layers as the second layer and the subgrade as the semi-infinite mass. Traffic loading was assumed to apply a uniform stress over a circular contact area. This system incorporates a mechanistic prediction of pavement strains, material characterization and environmental factors such as temperature and rainfalls. The analytical model used for design computations is the stress-strain and Odemark Equivalency Theories. The design procedure is based on fatigue cracking and permanent deformation (rutting) criteria.

2.1.1 Stress-Strain and Odemark Equivalency Theories
In flexible pavements in which the unbound layers are the main structural layers, while the asphalt part is only to provide a smooth driving surface and to protect the unbound layers from penetration of water, stresses and strains in such pavement structures had been indicated by Molennar [6] to be more like the Boussinesq distributions than like the distributions that follow from linear elastic multi-layer theory.

It has been shown that the vertical and horizontal stresses under a circular load can be calculated using, Equation (1), i.e.

$$
\sigma_z = \sigma_0 \left( \frac{1}{1 + \left( \frac{a}{z} \right)^2} \right) \left( 1.5 \right)
$$

$$
\sigma_{x,y} = \frac{\sigma_0}{z} \left( 1 + 2\mu \right) \left( \frac{2(1+\mu)}{1+(\frac{a}{z})^2} \right)^{0.5} \left( \frac{1}{1 + \left( \frac{a}{z} \right)^2} \right)^{0.5}
$$

where

- $\sigma_0$ = vertical contact pressure \([MPa]\)
- $a$ = Radius of the loaded area (mm)
- $z$ = Depth (mm)
- $\mu$ = Poisson’s ratio
- $\sigma_z$ = Vertical stress of depth $z$
- $\sigma_{x,y}$ = Horizontal stress in $x$ or $y$ direction at depth $z$,

In case of a thin wearing course and base layers of different stiffness an equivalent depth, $h_e$ should be used to calculate the stresses. As indicated by Molennar [7] this can be done using Odemark equivalency theory [8], which is explained below, [9].

$$
h_e = 0.9h_w \sqrt{\frac{E_w}{E_s}} + 0.9h_b \sqrt{\frac{E_b}{E_s}}
$$
If the layered pavement structure has been transformed into an equivalent layer system, the stress in the equivalent single layer can be calculated which can be back transformed to stress in actual structure.

Peattie and Ullidtz [10] simplified theory for multi-layered stress-strain analysis was adopted for the computation of the tensile and compressive strains within the flexible pavement. The relevant equations are:

\[
\sigma_z = \sigma_u = \sigma_0 \left[ 1 - \frac{1}{\left[ 1 + \left( \frac{a}{h_z} \right)^2 \right]^{1.5}} \right] \\
\epsilon_z = \epsilon_u = \epsilon_0 (1 + \nu) \left[ \frac{\frac{z}{a}}{\left[ 1 + \left( \frac{1}{\epsilon_z} \right) \right]}^{(1-2\nu)} - \frac{\frac{z}{a}}{\left[ 1 + \left( \frac{1}{\epsilon_z} \right) \right]} \right] \\
\epsilon_r = \epsilon_t = \frac{1}{E} \left( \frac{1-v}{2v} \right) (\sigma - E\epsilon_z)
\]

where \( \sigma_z = \sigma_u = \) Vertical stress of depth \( z \) due to wheel load
\( \sigma_0 = \) Actual wheel load
\( \epsilon_z = \epsilon_u = \) Compressive vertical strain on top of the subgrade layer
\( \epsilon_t = \epsilon_r = \) Tensile tangential strain at the bottom of bituminous layer

2.1.2 Australian Procedure

This procedure was developed by the Australian Road Research Board [11]. The procedure incorporates nonlinear material behavior directly into linear layered models through the use of an iterative-stress-modulus approach. Sublayers within the nonlinear layer are developed and assumed moduli values initially assigned to each. Layer solutions are then performed and states of stress within each sublayer are computed. These stress results are then substituted into the modulus expression obtained from the repeated load triaxial test to obtain predicted moduli results. Iterations are pursued until a tolerable error difference between these moduli is reached.

2.1.3 Smith and Witczak Base Modular Prediction Model

This procedure is used to determine the equivalent one layer granular base modulus that would result in identical critical strain parameters as that determined from the iterative elastic layered approach that directly accounts for the non-linear granular behavior. In this case there is no need of detailed non-linear computerized solutions for normal elastic layered stress-strain-displacement procedures [4]. This model incorporates the derived material characterization for various pavement components sizes and parameters, in which unique equivalent base modulus in the vertical direction \( (E_{2v}) \) and equivalent base modulus in the horizontal direction \( (E_{2h}) \) values were determined for a particular function of \( h_v, h_b, E_a, E_s, K_1, \) and \( K_2 \) respectively.

The relationships are given below:

\[
LogE_{2v} = 0.959 - 0.430 \text{Log}h_v - 0.073 \text{Log}h_b - 0.122 \text{Log}E_a + 0.294 \text{Log}E_s + 0.843 \text{Log}K_1
\]

\[
LogE_{2h} = 1.079 - 0.511 \text{Log}h_v - 0.008 \text{Log}h_b - 0.155 \text{Log}E_a + 0.279 \text{Log}E_s + 0.888 \text{Log}K_1
\]

where

- \( a \) = Tire contact radius
- \( v \) = Poisson ratio
- \( E \) = Modulus of Elasticity
- \( Z \) = Depth of pavement
The model assumes $K_2$ to have a constant value of 0.5 because of the rather small range in $K_2$ values plus the necessity to keep the problem parameter matrix as manageable as possible [4].

### 2.1.3 Input Parameters

In order to use elastic layer for the design of pavement, it is necessary to carefully select the input parameters. The following three considerations must be kept in mind since they are important in the design analysis:

- **a)** Determination of representative modulus value for each of the pavement layers.
- **b)** Variation in the traffic loading.
- **c)** Variation in environmental conditions.

#### Determination of values of modulus

**Asphalt layer**

For this layer, various charts are available. The selected values are based on the characteristics, such as bitumen characteristics; mix composition, loading time and temperature.

**Subgrade**

If repeated load test equipment is not available, the resilient modulus of subgrade may be estimated from CBR test values by using the relationship developed by Heukelom and Klomp [12] given by:

$$M_R = K \times CBR \text{ (Megapascal)}$$

where $K$ varies between 5 and 20 with an average of 10

**Base and sub-base**

In general, the $E$ for base is stress dependent. Values obtained from the road base characterization are inputted here. Either the Smith and Witczak base modular model or the Australian Procedure is used to obtain the base modulus.

**Environmental factors**

Any mechanistic design approach requires that the effects of environmental factors be included in the analysis. The moisture and temperature variations for each sublayer within the pavement (or a representative temperature) need to be determined. In the Asphalt Institute design method [13] the following equation developed originally by Witczak [14] was used to estimate pavement temperature.

$$MMPT = MMAT \left[ 1 + \frac{1}{(z+4)} - \frac{34}{(z+4)} + 6 \right]$$

where:

- $MMPT = \text{mean monthly pavement temperature}^\circ F$
- $MMAT = \text{Mean monthly air temperature}$
- $z = \text{Depth below pavement surface (inches)}$

The temperature at a depth $z$ equal to 1/3 the asphalt layer thickness was selected as the temperature to represent the layer.

For the present Nigerian conditions, the provision of adequate drainage facilities and proper compaction of pavement material will go a long way to alleviate the effect of the environment, especially rainfall, on pavements. The environmental effect is usually measured on the moduli of the pavement materials [15].

**Traffic**

In using the mechanistic approach, only one traffic load condition is analyzed, although others can be added by superposition.

For Nigerian conditions, the traffic analysis is based on the number of axle loads in terms of and equivalent to 80kN axle load. In the analysis, since the maximum stresses are found at the center of the wheel, it is assumed to be equivalent to the effect of a single wheel load of 40kN.

The mixes of axle weights of all vehicles using a highway are converted to an equivalent factor. An equivalence factor relates the damage done by one pass of a given axle load to an equivalent number of standard axle load applications to cause the same damage. Truck axle weights data should be collected on representative samples of vehicles using the route.

Equivalence factor can be computed using the equation below:
\[ F_j = \left( \frac{L_j}{L_s} \right)^{3.8} \]  

where:

\[ F_j \] 80kN single axle load equivalent factor for axle load \( j \)

\[ L_j \] axle load \( j \), KN and

\[ L_s \] Standard axle load as follows:

- 80kN for single axle
- 160kN for tandem axles

which is based on a regression analysis of American Association of State Highways and Transportation Officials [16] equivalence factors for flexible pavement with a terminal serviceability index of 2.5 and a structural number of 3.0.

The design lane is calculated using the traffic of lane, which is expected to carry the heaviest amount of truck traffic over the design period. The design lane is calculated as the number of equivalent single axle loads (ESAL) over the design period. The design lane \( (N_D) \) is calculated by applying a directional distribution factor (DDF), a lane distribution factor (LDF), the percent of truck factor \( (P_t) \), and an average 80kN single axle load equivalent factor to the total number of vehicles over the design period. The equation to calculate the design traffic as proposed by Claros and Ijeh [17] is:

\[ N_D = (365)ADT_p \left[ \left( \frac{(1+i)^n-1}{\ln(1+i)} \right) \right] (DDF)(LDF)(P_t)(F_{avg}) \]

where

\[ N_D = \text{Past ESALS for the critical lane for estimating remaining life.} \]

\[ ADT_p = \text{Present average daily traffic} \]

\[ DDF, LDF, P_t, F_{avg} \] are as previously defined.

**Pavement Response**

The determination of pavement response to traffic load and material characteristics involves the calculation of the stresses and strains within the pavement layers. However, because of the enormity of the task involved, only certain stresses and strains are determined at certain points within the pavement. These stresses and strains are known as critical parameters and are dependent on the type of pavements.

For a flexible pavement with unbound base and sub-base materials, the critical parameters are:

1) The tensile strain at the bottom of the surface or asphalt layer. (This is considered to be the maximum strain for a thin surface and it is taken at the bottom because a thin surface is being assumed, but for a thick surface it may be anywhere within the thickness)

2) The vertical strain at the top of the subgrade.

Strains are calculated as follows:

Tensile or tangential strain

\[ \varepsilon_t = \frac{1}{E_1} \left( \sigma_t - \mu (\sigma_\theta + \sigma_u) \right) \]

and
Vertical (Rut) strain:

\[ \varepsilon_v = \frac{1}{E_1} (\sigma_v - \mu(\sigma_x + \sigma_y)) \]  
(14)

where:

\[ \sigma_v = \sigma_z, \]  
Vertical stress at depth \( z \)

\[ \sigma_x = \sigma_x, \]  
Horizontal stress in x direction at depth \( z \)

\[ \sigma_y = \sigma_y, \]  
Horizontal stress in Y direction at depth \( Z \)

\[ E = \text{Modulus} \]

**Transfer Function**

Transfer functions are the relationship between stresses and strains and the number of repetitions of standard axle loads to failure. Based on extensive research, the following models have been found to be the most appropriate under tropical conditions in Nigeria.

**Fatigue Cracking Model**

The essential development of the fatigue criteria is a fatigue curve based on actual field observation of tensile strain and number of strain applications to failure. The fatigue equation developed for Nigerian pavements by Claros and Ijeh [17] is stated below:

\[ \log N_f = 15.988 - 3.291 \frac{\varepsilon_t}{10^6} - 0.854 \log \frac{E}{10^3} \]  
(15)

where \( N_f = \text{number of allowable 80KN ESAL applications} \)

\[ \varepsilon_t = \text{Horizontal tensile strain at the bottom of the asphalt layer} \]

\[ E = \text{Dynamic modulus of the asphalt concrete in PSI} \]

**Permanent Deformation Model**

Permanent deformation (rutting) is another important mechanism leading to the deterioration of a pavement structure. In general, there are three major contributors to rutting depth:

1. shear deformation in the subgrade when excessive vertical strain is present on the subgrade,
2. permanent or viscous deformation of the asphalt bound layers and
3. partial densification of the base and sub-base layers.

The most important parameter is the vertical compressive strain on the subgrade surface. The rut model for Nigeria as developed by Claros and Ijeh [17] is stated below:

\[ \varepsilon_v = 1.36 \times 10^{-2} (N)^{-0.2126} \]  
(17)

where:

\[ \varepsilon_v = \text{Vertical compressive strain at the top of the sub-grade} \]

\[ N = \text{Number of allowable 80KN ESAL applications} \]

The numbers of allowable 80kN ESALs are compared with the number obtained from the traffic data based on the design life. If the computed value is less than the projected, the sections of the pavement or the materials properties are changed so as to obtain number or repetitions of allowable 80kN ESAL that is greater than the projected traffic.

Several of the calculation steps required for the flexible Pavement Design procedure are too time consuming to do by hand. For this reason, computer programs to perform these calculations are included as part of the procedure.

### 3 Life-cycle Cost Analysis

For better decision making in choosing from various pavement section alternatives a life-cycle cost analysis (LCCA) can be incorporated in the above design procedure. Subsequently, the LCCA can be used in comparing and judging the efficiency of different design alternatives. The life-cycle cost analysis takes into consideration the construction cost, the maintenance cost, and the users cost to ensure sustainability of the pavement throughout its life-cycle apart from incorporating in the users cost, the delay cost, vehicle operating cost, accident cost, the environmental cost (energy use, emissions, waste, noise and water pollution).

The Life-cycle cost analysis (LCCA) framework represents the governing criteria and principles by which a comparison of costs of alternative design strategies is made. Facets of the LCCA framework include the following:

1. Economic analysis technique
2. Real versus nominal dollars
3) Discount rate

4) Analysis period

5) Cost factors

6) Approach to risk and uncertainty in LCCA

Detailed discussions on these facets and guidance in addressing them can be found in [18]

3.1 Estimating Costs

The most important components of the LCCA are the Life-Cycle Cost which include two types of costs: Agency cost and user cost. Agency costs include initial, maintenance, rehabilitation, support and remaining service life value costs. User costs include the additional travel time and related vehicle operating costs incurred by the traveling public due to potential congestion associated with planned construction throughout the analysis period. Due to the growing need to add environmental sustainability factors to the pavement design process, environment costs such as emission and water pollution costs are taken into consideration in the user cost. Environmental cost estimates are applied to estimate pollution damage costs over the entire life-cycle of the pavement system. These cost are related to both direct and indirect impacts to human health from air pollution, either due to the inhalation of air pollutants detrimental to human health or greenhouse gas emissions that results in global warming [19].

3.2 LCCA Software

A typical LCCA framework that is recommended for incorporation into the design procedure in order to compare pavement design alternatives is the Federal Highway Administration (FHWA) RealCost. This software provides a tool to perform LCCA for pavement selection in accordance with FHWA best practice methods. These best practices are outlined in the FHWA's "Life-Cycle Cost Analysis Primer," [20] and the software methodology is fully documented in the FHWA's "Life-Cycle Cost Analysis Technical Bulletin," [21] publication number FHWA-SA-98-079. Both of these documents are available in electronic format on the FHWA's LCCA website (http://www.fhwa.dot.gov/infrastructure/asstmgmt/lcca.cfm).

4 Validation and Comparison of Models

To assess the validity of the models, field data were collected from the test sections in Nigeria, which falls in a tropical environment. Comprehensive and reliable field and laboratory tests were performed on the entire road network of the country by the Pavement Evaluation Unit Kadanu, Nigeria, in conjunction with Texas Research and Development Foundation [22]. Field measurement on a factorial of thirty-six master test sections covering the observed ranges of thickness, surface, traffic and climatic variables were carried out. A careful selection from all the field and laboratory test results performed on the thirty six master stations of the country was done to make sure that only stations where resilient modulus tests were performed on the base were selected. The field strain values were calculated from the deflection basin matching determined from the deflectometer; and from the field and laboratory data.

4.1 Results and Observations

The results of strains obtained by the Smith - Witczak and Australian models were compared with the field values. Figures 2 and 3 reveal the level of closeness of the horizontal strain (fatigue strain at the bottom of the asphalt layer) and the vertical strain (rut strain at the top of the subgrade) of the predicted values and the field values. From Figure 3 it can be easily seen that the Smith-Witczak predicted values are closer to the field results than the Australian model. However, the level of closeness cannot easily be distinguished in Figure 2 for horizontal strains. Consequently, to accurately determine the closer of the models, significant test by using the Student-t analysis at 95 percent confidence interval was performed. From the test Smith-Witczak method gives a better prediction of the field value. The analysis shows that there is no significant difference between the field values and the predicted values, whereas for the Australian method there is a significant difference in the results obtained.

4.2 Comparison of Strains with the Field Values

As it can be seen in Figure 2 both models give a better prediction of the horizontal strains, i.e., the fatigue strain at the bottom of the asphalt layer, this is because they both give higher moduli at the top of the granular sublayer (similar to actual field values).
The reason being that for classical flexible pavement structure, the modulus of the granular sublayer at the top of the base is much higher than at the bottom of the base course as a result of rapid attenuation of the bulk stress with depth. For this reason most of the predicted values for vertical strains are higher, resulting from the lower moduli obtained at the bottom of the granular layer by the two models. Additionally, as shown in Figure 3 the vertical strains estimated by the Australian method are extremely high because of the very low estimated moduli for the bottom of the granular sublayer. This resulted from the fact that the constituent equations of the method (Australian) do not depend solely on the base quality \( K_1 \) and the limitation of the modular ratio to four in which other properties of the layers above the subgrade were not taken into consideration. The Smith-Witczak method, on the other hand, gives a better estimation of the base moduli because it incorporates most of the variables that affect pavement behavior. The Smith-Witczak method was therefore adopted for fine-tuning for tropical conditions because of the above reasons. Besides, the method requires less computing time than the Australian, since the iterative elastic layered approach is not involved.

4.1.1 Modification of the Selected Model

After a careful observation of the material properties of the representative cross-sections studied, it was noted that there is a wide range of \( K_2 \) value for tropical granular materials, in the studied area the values range from –0.33968 to 0.40200. This is in contrast to the small range proposed by Smith and Witckaz, [4] resulting from observation of the soils in the temperate environment. Consequently, the assumed constant value of 0.5 for \( K_2 \) by their model may not be applicable to tropical conditions. Therefore, the model was modified to accommodate the wide range of \( K_2 \) for tropical soils as revealed in this study.

To accomplish the above, a back calculation technique by deflection basin matching was adopted to give the resilient base moduli value that match the fatigue and rut strains obtained from the field results. The shortfall between the predicted and actual moduli was then accounted for by the inclusion of \( K_2 \) as a variable in the equations. To incorporate the \( K_2 \) values in the Smith-Witczak model, a statistical regression analysis was performed. The shortfall between the field values and the predicted values was regressed against the \( K_2 \) values. The analysis yielded the following predictive equations for the Equivalent Base Modulus in the horizontal direction \( (E_{s2h}) \) and Equivalent Base modulus in the vertical direction \( (E_{s2v}) \), respectively.

\[
\log E_{s2h} = 1.079 - 0.511 \log h_s - 0.008 \log h_b - 0.155 \log E_s + 0.279 \log E_i + 0.888 \log K_1 + 1.257 K_2
\]
\[
\log E_{s2v} = 1.105 - 0.430 \log h_s - 0.073 \log h_b - 0.122 \log E_s + 0.294 \log E_i + 0.848 \log K_1 + 1.338 K_2
\]

where:

\[
E_{s2h} = \text{Equivalent Base Modulus in the horizontal direction (psi)}
\]
\[
E_{s2v} = \text{Equivalent Base modulus in the vertical direction (psi)}
\]
\[
h_s = \text{Asphalt/Bituminous surfacing thickness (ins)}
\]
\[
h_b = \text{Base Course thickness (ins)}
\]
\[
E_s = \text{Modulus of asphalt/bituminous surfacing psi}
\]
\[
E_i = \text{Modulus of the subsurface layer (psi)}
\]
\[
K_1 = \text{Intercept of regression constant from the modulus test plot (psi)}
\]
\[
K_2 = \text{Gradient regression constant from the modulus test plot (psi)}
\]
From the regression analysis, the high $r^2$ value of 0.70 obtained for the two determinations shows a good level of reliability on the modified algorithms. This means that about 70 percent of the total variation of the discrepancy or shortfall between the field results and the predicted value was accounted for by the inclusion of the $K_2$ variable. Additionally, significant tests performed on the coefficient of $K_2$ also revealed a high level of reliability.

5 Application of The Modified Model in The Mechanistic Design Procedure

The modified model was then implemented in a mechanistic pavement design procedure. The results obtained were very reasonable for tropical conditions and this was also used to obtain the influence of all pavement variables (layer thickness, environmental effects of temperature and moisture upon moduli, granular material quality as measured by the laboratory resilient moduli test) upon the probable performance of flexible systems throughout various periods of the year [23].

5.1 Influence of Pavement Variable on Performance.

As mentioned earlier one of the advantages the Smith-Witczak model has over other models is that the influence of pavement variables on performance can be easily determined. This can be achieved by partial derivatives.

It is obvious from Equations (18) and (19) that only the $K_1$ and $K_2$ parameters (quality of base course material), and the subgrade modulus $E_s$ give a positive correlation, i.e., the values of the equivalent base moduli in both the vertical and horizontal directions ($E_{2e}$ and $E_{1e}$) increase as the subgrade modulus $E_s$ and $K_1$ and $K_2$ parameters increase for constant values of the other road parameters. While the other variables $h_a$ (Asphalt surface thickness), $E_a$ (Asphalt surface modulus) and $h_b$ (Base thickness) have a negative correlation; which means there is a decrease of the Base moduli for increase in these variables.

From Table 1, the most significant variable affecting the $E_{2a}$ and $E_{1a}$ values is the $K_1$ parameter (quality of base course material). There is about 88.8% percent increase in $E_{2a}$ for an increase in $K_1$ while other variables are kept constant. This is quite logical as this parameter plays a significant role in determining the non-linear sublayer modulus for any given bulk stress value ($M_r = K_1 \theta^{52}$). The high negative variation on the $E_{2a}$ and $E_{1a}$ by the $h_1, E_a, \theta, K_1$ values suggests that there will be no significant increase in the pavement performance as these variables increase. As a result of these the major focus of this study was on the influence of the most significant variables -$K_1$ and $E_s$ (Subgrade modulus) on pavement performance.

Table 1. Influence of pavement variable on Performance

<table>
<thead>
<tr>
<th>Variable</th>
<th>Pavement Performance Indicator</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent horizontal Modulus ($E_{2e}$)</td>
<td>Equivalent Vertical Modulus ($E_{2v}$)</td>
</tr>
<tr>
<td>$K_1$-value (quality of Base)</td>
<td>88.8 %</td>
</tr>
<tr>
<td>Subgrade Modulus ($E_s$)</td>
<td>27.9 %</td>
</tr>
</tbody>
</table>

From the results obtained from the study it was found out that the most significant variable affecting the pavement performance is the $K_1$ parameter (quality of base course material), followed by the subgrade modulus $E_s$. A more cost effective design chart was then developed for tropical regions in which the least significant variables of base thickness, asphalt layer modulus and thickness are kept at optimum constants of 150mm, 2000Mpa and 50mm, respectively. The choice of 2000Mpa as asphalt modulus was chosen because of the ambient temperature experienced in the tropics. Figure 4 shows the chart that was developed and also from the chart another model was developed for tropical environment, which relates number of repetitions of Standard 80KN axle load to $K_1$ and $E_s$ (Equation 20)

$$
\log N_f = 4.0161 \log K_1 + 1.4048 \log E_s - 3.4721
$$

(20)

where:
$N_f = \text{No of Repetitions of Standard 80kN axle load}$

$K_1 = \text{Intercept regression constant in MPa}$

$E_s = \text{Subgrade modulus in MPa}$

From a partial derivatives Analysis conducted on Equation 20 an increase in Base quality ($K_1$ parameter) will lead to about four times increase in pavement performance ($N_f$ value). This is in conformity with field tests carried out by AASHTO in 1977 and as also observed by Burns and Ahlvin [24]. Their observations showed that high quality materials found in two instances were more than double the number of wheel load repetitions required to reduce the serviceability of the pavement to a given level. It is therefore imperative that in pavement design and construction the monitoring of the quality of the base must be given the highest priority. Studies on the various pavement variables that effect quality of granular bases are very essential in pavement Test Sections. Of such is the one carried out by Phang, [25] and Potts et. al [26]; their observations revealed that the field performance of granular bases tend to vary considerably more than either bituminous or cement treated bases since an unstabilized base is considerably more affected than stabilized materials by such variables as material type, grain size, gradation compaction density and frost susceptibility. Similarly several closely controlled field studies [24] have shown other similar factors, e.g., a crushed stone base usually performs significantly better than a natural gravel base that has not been crushed.

Equally some pavement test sections revealed these phenomenon; of such is the AASHTO Road Test which consisted of two similar pavement sections consisting of 80mm asphalt concrete surfaced and 100mm unstabilized subbase. One section had a 360mm well-graded crushed limestone base and the other a well graded uncrushed gravel base of similar thickness. At a Present Serviceability Index (PSI) of 2.5 upon a scale of 5.0, the section having the gravel base had withstood 400,000 repetitions of an 80- kN single axle load, whereas the crushed stone base withstood 1,000,000 repetitions. Also at waterways Experiment Station [27] a heavily loaded section constructed with a 80mm asphalt concrete surfaced and 530mm crushed limestone base failed at 5,000 coverages while a section having a 530mm gravely sand subbase overlaid by a 150mm crushed limestone base failed at only 1,500 coverages even though this section was 150mm thicker.

Similarly, the partial derivatives analysis shows that the pavement performance will increase by almost one and half times for a corresponding increase in sub-grade modulus $E_s$ and vice versa. This explains why poor sub-grades have a very significant detrimental effect on pavement performance [27, 28 and 29].

For a more strategic sustainable pavement system a life-cycle cost analysis framework such as $RealCost$ can be incorporated in the design procedure. The life-cycle cost analysis takes into consideration the construction cost, the maintenance cost and the users cost. To ensure sustainability of the pavement throughout its life-cycle apart from incorporating in the users cost, the delay cost, vehicle operating cost, accident cost, the environmental cost (energy use, emissions, waste, noise and water pollution) are also taking into consideration.

6 Conclusion

The Smith and Witzczak base moduli predictive model has been adopted as part of the mechanistic design model in the tropics because of its good correlation with field results. The algorithms of the model was then modified by the inclusion of the $K_2$ in the mathematical equation and then included in the mechanistic design model for the tropics. Subsequently the developed model gives a superior correlation with field results obtained in
the tropics. The developed model was used to determine the influence of all pavement variables upon the probable performance of flexible systems throughout the year. A cost effective design chart was also developed for tropical roads.

From the model it was confirmed that the use of high quality base materials is very important in achieving optimum pavement performance, since an increase in base quality can result in a quadruple increase in structural strength of the section. Therefore stabilizing base materials will be more resourceful than increasing the thickness. It was also observed that there is a limiting subgrade modulus of 300 Mpa for optimum pavement performance. Any increase above this may be uneconomical most especially for the values below 1000 Mpa.

The design procedure developed allows extrapolation to any set of design conditions, since many variables are accommodated and taken into consideration in the model.

A life-cycle cost analysis framework, such as RealCost can be further used in comparing and judging the efficiency of different design alternatives.

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