DEVELOPMENT OF TRANSITIONAL PROBABILITY MATRIX MODEL FOR PAVEMENT DESIGN

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DEDICATION

This work is dedicated to Hajiya Maryam Zubairu and in loving memory of Late Alhaji Aminu Salihu, Late HajiyaSafiyaZakariYau and Late ArchSiniatuIsa Salihu. May their souls continue to rest in peace.

DECLARATION

I hereby declare that this dissertation has been composed by me and it is a record of my own research work. It has not been presented for the award of degree anywhere. However as required by research laws and ethics all sources of information are duly acknowledged by means of references.

Adamu Aminu Salihu ……. …….

StudentSignature Date

CERTIFICATION

This dissertationentitled “A Development of Transitional Probability Matrix Model for Pavement Design” by Adamu Aminu Salihu, meet the requirement governing the award of the degree of Master of Science (MSc) in Civil Engineering of the Department of Civil Engineering, Ahmadu Bello University, Zaria, and is approved for its contribution to knowledge and literary presentation.

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SYMBOLS/ABBREVIATION

AASHTO	American Association of State Highway and Transportation Officials
ADT	Average Daily Traffic
ASTM	American Society for Testing and Materials
CBR	California Bearing Ratio
COV	Coefficient of Variation
DDF	Directional Distribution Factor
ESAL	Equivalent Single Axle Load
FORM	First Order Reliability Method
FTG	Future Traffic Growth
HMA	Hot-Mix Asphalt
LDF	Lane Distribution Factor
Q_f	Total Number of Equivalent Vehicles Over the Design Lane
MATLAB	Matrix Laboratory
M-E	Mechanistic-Empirical
MEPDG	Mechanistic Empirical Pavement Design Guide
M_R	Resilient Modulus

M_{RB} Resilient Modulus of Base Layer

M_{RSB} Resilient Modulus of Sub base Layer

NEMPADS Nigerian Empirical-Mechanistic Pavement Analysis and Design System

N_FFatigue Allowable Number of Equivalent Single Axle

N_R Rutting Allowable Number of Equivalent Single Axle

PEU Pavement Evaluation Unit

E_t Horizontal Tensile Strain at Bottom of the Bituminous Layer

E_V Vertical Compressive Strain on the Subgrade

MVFOSM Mean Value First Order Second Moment

MCS Monte Carlo Simulation

FOSM First Order Second Moment

ABSTRACT

A model of transitional probability matrix for Mechanistic-Empirical Pavement Design was developed with the aim of tarnishing the traditional method obtained from the judgment of an expert panel. The model was developed using fatigue and rutting as the critical factors in pavement failure. Model performance was assessed using first order reliability method (FORM) with the concept of genetic search algorithm developed with Matrix Laboratory(MATLAB). The proposed model allowed the distressed model to be model individually so that immediate measures for correcting deficiencies can readily be identified. The model formulation was followed by applying it into a real data. The data consist of three layered Hot-Mix asphalt (HMA) consisting of 100mm surfacing underlay by 200mm base course, 250mm sub base which is purported to have a life span of 15 years. Result indicate a decrease in system transitional probability of about 3% and 6% for light and heavy traffic considered respectively with age as against the constant homogeneous Markov-chain Matrix. The components probabilities also trends the same. Therefore, the assumption of time-invariance matrix of transitional probability was disregarded. The performance of the original pavement structure (100mm) was compared to 125mm and 150mm surfacing respectively. The result shows that the heavy traffic which are characterized by larger axle load and higher truck percentage has lower transitional probability than the lighter one because of its higher damages on the pavement structures both for the component and system models.

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CHAPTER ONE

INTRODUCTION

1.1 General Overview

The Increasing demand for road transport in Nigeria and the huge government investments on roads have generated the need to encourage road maintenance and sustainability. Effective road maintenance program reduces vehicle operating costs, extends the life of pavements and results in significant savings on rehabilitation and re-construction(Abdulkarim, 2003).

With the corresponding increase in traffic volumes on our roads, characterized by varying composition and axle loads, environmental changes and material characterization, road deterioration and failure are experienced (Adeoti, 2000).

This ugly development started affecting service delivery to our pavements. Our road transportation system began to suffer losses due to high operating costs leading to some being grounded. The need to place emphasis on road maintenance started to manifest (Abdulkarim, 2003).

Unfortunately the financial and technical requirements for effective maintenance, rehabilitation and reconstruction became so staggering that the rate of maintenance could not be matched with the deterioration. This development has necessitated many highway agencies to develop a pavement management system (Adeoti, 2000).

A pavement management system is considered as a programming tool that collects and monitors information on current pavement, forecasts future conditions, and evaluates and prioritizes

alternative reconstruction, rehabilitation and maintenance strategies to achieve steady state of system preservation at a predetermined level of performance (Prozzi and Madanat, 2000).

Effective implementation and utilization of pavement management systems in generating and evaluating various alternative strategies based on engineering and economic principles is largely dependent on the ability to predict the future condition of the pavement. This pavement prediction is complex in nature because of high level of uncertainties and variability. This makes the performance or deterioration of pavement to vary greatly showing uncertain or random characteristics. Uncertainty can arise from inability to quantify the factors that affect the deterioration process, and to model the true deterioration process of the material. Thus pavement deterioration process shows stochastic characteristics (Sachez-Silva *et al*, 2005).

One of the challenges facing existing probabilistic models is the difficulties in establishing the Transitional Probability Matrix (TPMs). A TPM is a square matrix with a possible number of states in the system. The matrix contains the probabilities of transitioning from state i to state j, that is, the probability of something being in one state and then changing to another state over a fixed time interval. The TPM can be established using historical data or subjective opinions of experience engineers through individual interviews and questionnaires, which are bound to have unforeseeable and unpredictable errors and usually takes considerable time and expenses(Dilip*et al*, 2013)

A more rigorous and accurate method is to examine the effects of the factors affecting the pavement structure from the basic principles of mechanics through stress-strain responses of the pavement. In this study, both the effect of traffic impact and the growth of these traffic over time

will be studied. Environmental changes and material characterization will also be incorporated into the model.

1.2 Statement of the Research Problem

Pavement management is one of the major problems facing many highway agencies, especially in developing countries like Nigeria. The traditional methods of forecasting the performance of pavement is through establishing transitional probability matrix by using historical data or subjective opinions of experienced engineers through individual interviews and questionnaires, which takes considerable time and expenses. The more rigorous and less time consuming method is to consider the impact of axle load from the basic principle of mechanics by calculating stress-strain responses of the pavement structure through Mechanistic-Empirical approach.

1.3 Justification of the Research

The accurate prediction of pavement performance is very important for efficient management of the road infrastructure. By reducing the prediction error, pavement deterioration agencies can obtain significant budget savings through timely intervention and accurate planning (Prozzi and Madanat, 2000). Pavement performance prediction has been the key component of pavement management systems which is considered as a programming tool that collects and monitors information on current pavement, forecasts future conditions, and evaluates and prioritizes alternative reconstruction, rehabilitation and maintenance strategies to achieve steady state of system preservation at a predetermined level of performance. The performance prediction can be a difficult task due to the complex interactions of the distress and are bound to have significant differences under a certain condition. Therefore, the need to develop a model to scrutinize the

actual behavior of pavement under different conditions would be useful in the planning of maintenance activities.

1.4 Aim and Objectives

1.4.1 Aim

The aim of the research is to develop a model to predict the probability of pavement deterioration using Mechanistic-Empirical Approach.

1.4.2 Objectives

The specific objectives include to;

i. generate stresses and strains using a Multilayer elastic computer program, ELYSM-5

ii. compute the allowable stresses in terms of fatigue and rutting failures using distress models.

iii. calculate the cumulated damage for fatigue and rutting due to generated stresses.

iv. calculate the probability of failure using First order Reliability methods (FORMs) for both component and system distress with the aid of MATLAB.

v. evaluate the variations of probability of transition on both the component and system distress

1.5 Scope of the Study

This research considered only probability of transition for fatigue and rutting distress sub models. The statistical data of the variables will be generated from the literature. The reliability analysis will be undertaken using First Order Reliability Method (FORM) through the concept of genetic algorithmMATLAB toolkit would be developed to implement the FORM.

CHAPTER TWO

LITERATURE REVIEW

2.1 Mechanistic-Empirical Pavement Design

A mechanistic–empirical design method for a flexible pavement means application of the principles of engineering mechanics to evaluate the response of pavement structures to traffic loading and much improved design methods to carry out distress prediction or how performance changes with time. Using a method based on the principles of engineering mechanics would ensure a fundamental understanding of how the pavement structure responds to certain action or loading conditions. This more realistic approach would also secure the needed flexibility; in other words, the method should be able to deal with new situations such as new pavement materials and loading situations. A very important factor when using a mechanistic–empirical design method is the need to use testing equipment and set-ups in the laboratory which adequately simulate the most important aspects of the real behavior of a pavement. Otherwise we can't expect that our predictions will reflect real-world factors and results or predict actual pavement performance.

This more meaningful approach would rule out some of the classical tests which have long been used in the structural design of pavements, such as the CBR test, and would replace these with tests that would yield more reliable results and therefore lead to more efficient decisions regarding road construction. The most important factors influencing the performance and distress development of pavement structures are i) the cross section of the pavement structure, ii) the traffic (axle) loading of the structure, iii) the climatic conditions the pavement will be exposed to during its entire service life, and iv) the material properties of the different layers in the pavement structure. To be able to predict the functional and structural conditions of the road

structure over time a good understanding of these factors is needed and including, importantly, how they are linked mechanistically to performance and distress development.

Some of the benefits of the mechanistic–empirical design method over the empirical methods can be summarized below:

i. inherently better suited to treat the real-life variety of environmental and wheel loading conditions.

ii. estimation of new loading conditions, e.g. the damaging effects of increased loads, high tyre pressure and multiple axles.

iii. better utilization of available materials. Seasonal effects can be included in the design.

iv.benefits of providing improved drainage systems.

Furthermore, an additional important benefit of the mechanistic–empirical based approach includes ease of implementing future enhanced or improved knowledge.

2.2 Factors Affecting Mechanistic-Empirical Pavement Design

2.2.1 Traffic loading

An important factor affecting pavement performance is the number of load repetitions and the total weight a pavement experiences during its lifetime. In the empirical design procedure all axle loads were converted into a single number of Equivalent Single Axle Loads or ESAL's. As this does not correspond to the real loading, more detailed information is needed or the full axle load spectra for single, tandem, tridem and quad axles. Based on Weigh in Motion (WIM) data the full axle load spectra can be established(Sigurdur, 2004)

2.2.2 Material characteristics

The ability to calculate the response of pavement structures to vehicle load depends on the understanding of the mechanical properties of the constituent materials. This is essential because most pavements in Nigeria are flexible pavements. They are usually built up of bitumen bounded or unbounded granular layers. In general they show complex non-linear viscoelastic-plastic behavior under external loading. Furthermore, bituminous bounded materials are temperature dependent and the response of unbound material depends on the moisture content. This must be taken into account in a mechanistic–empirical pavement design method in order to achieve a good prediction of pavement performances. However this is both complicated and requires sophisticated laboratory test results and the design analyses become time consuming(Sigurdur, 2004).

2.2.3 Climatic conditions and seasonal variation of material properties

The mechanical parameters of both bounded and unbound layers in pavement structures are seasonally affected. It is therefore important to understand their seasonal variation in order to be able to predict their effect on pavement performance. For instance, asphalt concrete shows visco-elastic-plastic behavior which is temperature dependent, and stiffness and permanent deformation characteristics of unbound materials show some moisture content dependency. Environmental monitoring programs where temperature, moisture content and frost penetration in pavements are collected and related to bearing capacity are therefore of great importance(Sigurdur, 2004).

2.3 Existing Models for Pavement Performance

Most of the pavement performance models developed in the early stages of pavement research are deterministic (Haas and Hudson, 1982). Currently, deterministic pavement performance models, such as the American Association of State Highway and Transportation Official (AASHTO, 1962) regression performance model and various *S*-shaped curves, are still widely used. Based on the AASHO Road Test data, the initial pavement performance equation was developed to predict the loss of the serviceability by capturing the comprehensive effects of applied traffic loadings, material characteristics, and environmental conditions (AASHO, 1962). In order to accommodate the impact of the routine maintenance actions, the *S*-shaped curve which provides more accurate long-term prediction was proposed to reduce the deterioration rates at the end of pavement design period (Garcia-Aiaz and Riggians, 1984). However, such models are unable to effectively accommodate measurement errors and unobserved factors. As a consequence, the prediction error could go as high as 1 unit of the PSI value by using the AASHTO performance equation (Prozzi, 2001).

As part of the effort to improve such models, other regression models (Paterson, 1987; Prozzi,2001) were proposed to consider more explanatory variables, such as pavement strength over different subgrades, environmental conditions, and maintenance actions, and different model structures based on the field data. Paterson (1987) developed a number of incremental empirical model at different levels of complexity to explain the real physical phenomena of pavement deterioration. Prozzi (2001) developed a mechanistic-empirical pavement performance model by using a two-step approach. An initial incremental nonlinear pavement performance model was developed based on the AASHO Road Test data by using the random-effects estimation methods. Then, with the integration of the joint estimation method, the bias of the

parameter estimation in the prediction model was corrected by incorporating the in-service pavement data sets.

Although these models can provide good prediction results by considering the effects of the heterogeneity in the data sets or the maintenance activities, their deterministic prediction results are still used and hence they are not used to capture the inherent uncertainty in the process of pavement deterioration. In other words, despite the various efforts in improving the accuracy of deterministic models, these models are still constrained by the fact that they cannot effectively take the stochastic nature associated with pavement performance into consideration.

Many probabilistic or stochastic models have been developed in order to characterize the uncertain characteristics of pavement deterioration processes. These previously developed probabilistic models can be summarized into three categories: econometric models, Markov Chain models, and reliability analysis. Each category covers a range of more specific applications. For example, Markov Chain models include homogeneous and non-homogeneous Markov Chain models. The details of the classification is illustrated in Figure 2.1

In the last decade, econometric models were widely used to correlate the pavement distresses with their explanatory variables. Madanat *et al*, (1995)proposed a joint discrete-continuous model to characterize the appearance of cracking and the propagation process of those cracks, while the binary logit model was used to determine whether the cracking appeared, and then a continuous model was developed to model the propagation process. The explanatory variables in the model include the structural number (SN) of the pavements, the thickness of the surface layer, and the number of wheel passes per unit strength of pavement (Madanat*et al*, 1995)

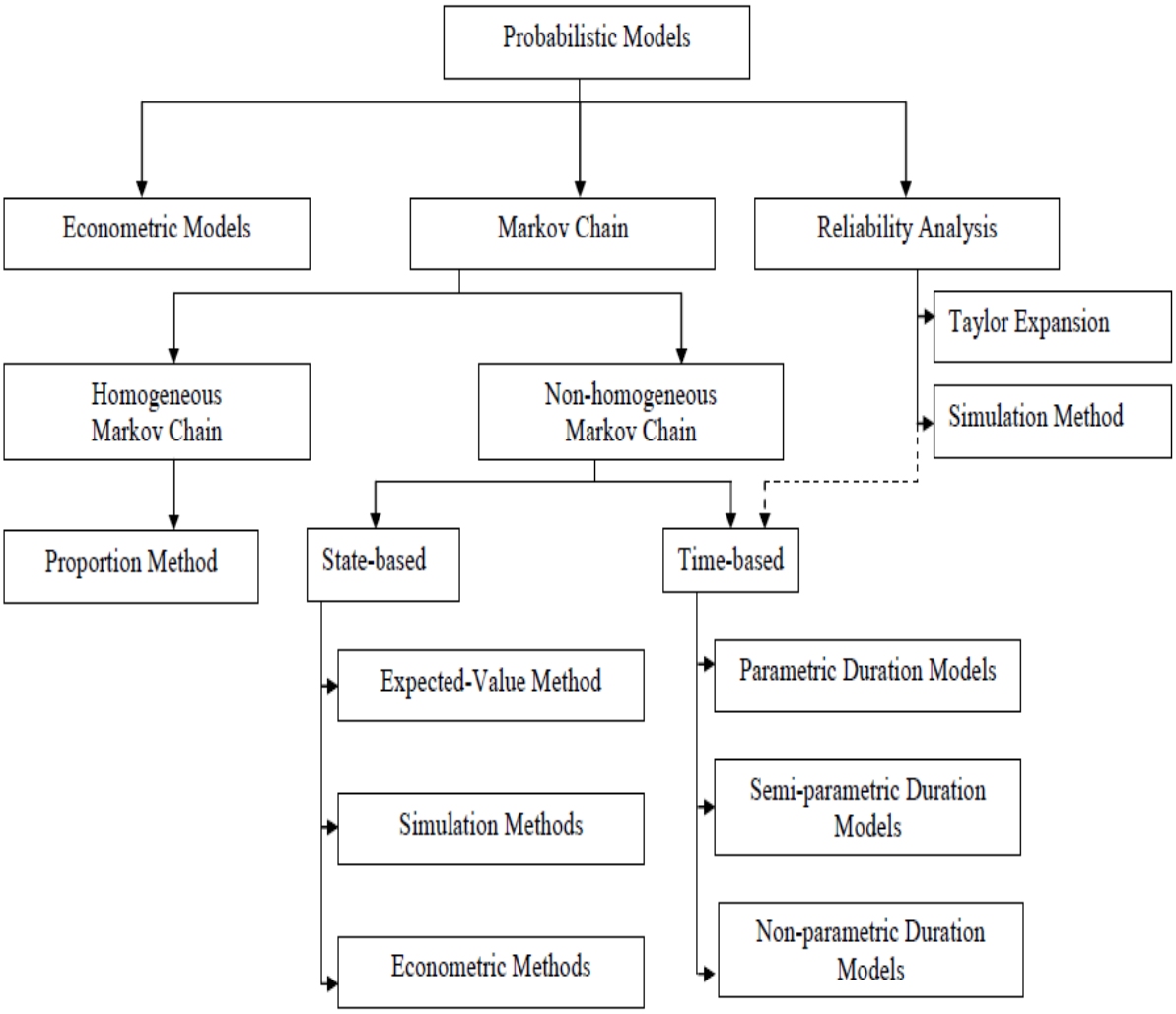


Figure 2.1: Probabilistic Model for Pavement Deterioration

(source: Li, 2005)

Other econometric models were proposed to develop the Markov Chain models. Another popular category of performance models is the Markov Chain.Golabi *et al*, (1982)proved the effectiveness of using the Markov Chain method by developing Markov Chain performance

models in the state of Arizona (Golabi*et al*, 1982).In those Markov Chain models, the discretization of the continuous variable was undertaken based on different schemes because not much detailed information is needed at the network level of management (Madanat*et al*, 1995). Two types of Markov processes have been proposed according to different assumptions. The first is homogeneous Markov Chain process which assumes that the present condition state is only related to the previous state or the impact variables are constant during the analysis period (Golabi*et al*, 1982). In other words, the Markov Chain model has no memory of the entire past. On the other hand, the non-homogeneous Markov Chain models characterize the changes of the pavement deterioration rates over time. The Markov Chain models can be developed using the state-based or time-based models. The state-based models quantify the transition probabilities from one condition state to another in a predefined period of time, while the time-based models estimate the probability distributions of time it takes to change from one condition state to another (Mishalani and Madanat, 2002).

The state-based models are widely developed in practice, because they require less frequency of data collection. The core of the state-based Markov Chain models is the development of Transitional Probability Matrices (TPMs). Research methods, varying from the simplest proportion method (Wang *et al*,1994)and the expected-value method (Jiang *et al*, 1989; Butt *et al*, 1987) to the complicated econometric techniques (Madanat, 1995), were used to develop the TPMs.

The simplest approach used for developing a homogeneous Markov Chain model is a proportion method used by Wang *et al*, (1994)which directly calculated the transition probabilities from one condition state to another. However, the prediction results of the homogeneous Markov Chain process are questionable, since the deterioration rate is not

constant(Butt *et al*, 1987).Therefore, non-homogeneous Markov Chain model is more appropriate in the deterioration process.

. The widely used non-homogeneous Markov Chain models were developed using the expected-value method in the 1980s. The expected-value method segments the pavements into different groups and then minimizes the differences between the expected values calculated using the TPMs and those obtained from the regression model with time as its explanatory variable (Jiang *et al*, 1989; Butt *et al*, 1987).

Another way of developing the state-based non-homogeneous Markov Chain model is the simulation approach which assumes design variables to follow different statistical distributions. The Monte Carlo simulation technique was used to produce the probability vectors representingthe transition from one condition state to another, consisting of the TPMs.

The calculated TPMs of pavement deterioration process determine the time-related non-homogeneous Markov Chain processes (Li *et al*, 1996). This simulation method can save a significant amount of money and effort compared with the previously discussed proportion and expected-value methods, because the collection of multi-year performance data is not required.

However, the above discussed methods cannot directly consider the impact of pavement types, environmental factors, traffic loading, and other relevant factors. The improved econometricmethods such as ordered probit model, Poisson model, and random-effects probit models are proposed to connect the relevant explanatory variables to the transition probabilities (Madanat*et al*, 1995*;* Carnahan, 1987; Jiang *et al*, 1989;Madanat *et al*, 1997).

These models employed statistical techniques to develop the relationship between the explanatory variables and dependent variables, providing more accurate prediction results than

the previously discussed methods (Madanat*et al*, 1995).However, variables such as traffic and facility age could cause the TPMs to vary with time, resulting in the non-homogeneous facility deterioration process. Therefore, it is difficult to use these TPMs as the input to a stochastic Markov decision-making process, since most of the decision-making models are developed with the assumption that the deterioration process is stationary (Durango and Madanat, 2002).

The time-based models are considered as alternatives to develop the Markov Chain models. The time-based models focus on estimating the probability distributions of the time taken to transit from one condition state to another using the duration models (DeStefano and Grivas, 1998; Mauch and Madanat, 2001; and Mishalani and Madanat, 2002). Therefore, they also belong to the category of reliability models. These duration models can account for the censoring problems associated with data collection in the parameter estimation process. The hazard rate defined as a transition rate out of a certain state can be assumed to be a function of explanatory variables. Based on the assumption of the hazard rates, the duration models are further classified as: parametric duration models, semi-parametric duration models (Cox proportional hazard models), or nonparametric duration models. Most of parametric models assume that the hazard rates follow the Weibull distribution (Prozzi and Madanat, 2000; Vandem*et al*., 1997). They can characterize the nonlinear accumulated hazard rates. The estimated parameters of the Weibull distribution can be used to test whether the homogeneous Markov assumption is valid or not. But the Weilbull distribution assumption for the hazard rates in these parametric duration models is questionable because of the lack of explanations of the underlying assumption. Both Cox proportional hazard models and nonparametric models were proposed to theoretically solve the problems stemming from the predefined distributions for the baseline hazard (Mauch and Madanat, 2001; DeStefano and Grivas, 1998). Although the Cox

proportion hazard model relaxes the parametric assumption of hazard specification and also considers the impact of the covariates, the baseline hazard cannot be estimated using a partial likelihood estimator. The nonparametric duration model is attractive because of its simplicity and accuracy in estimating hazard rates, but it cannot relate the dependent variable to the relevant explanatory variables. For the efficiency of the models, Meyer reported that the nonparametric estimation does not suffer from substantial loss of efficiency even for situations where parametric models are appropriate (Meyer, 1987). Therefore, it is recommended that the test of the nonparametric hazard baseline be performed before conducting any parametric analysis with duration data.

The time-based and state-based modeling methods are complementary in the sense that the state-duration probability density function used to calculate the transition probabilities can be estimated using a time-based model. The selection of the modeling approach primarily depends on the nature of the available data. The time-based model requires accurate observations of performance data spanning the whole deterioration period. If the measurements are not made frequently in short time windows, the measurement errors would result in the inaccurate time-based models (Mauch and Madanat, 2001). In reality, the data set satisfying these strict requirements is not easy to obtain. Therefore, these time-based models are not commonly used in practice.

To overcome the limitations associated with the previous models, one possible solution is to directly predict the pavement condition states by using a probabilistic approach. This approach can accommodate the stochastic characteristics of pavement performance and can also link the causal variables to pavement performance regardless whether the deterioration process is homogenous or not.

2.4Concept of Structural Reliability

The study of structural reliability is concerned with the calculations and prediction of the probability of limit state violation for engineered structures (Melchers, 1999). Ditlevsen and Madsen (2005) considered structural reliability as a method that attempt to treat rationally, the various sources of uncertainties. According to Tsompanakis and Papadrakakis (2000) structural reliability analysis is a tool that assists the design engineer to take into account all possible uncertainties during the design and construction phases and the lifetime of a structure in order to calculate its probability of failure. To Stanley *et al*, (1978)reliability-base design considers the probability that the structure will last a given length of time against the agents that can cause it to fail.

Reliability-based design is a probabilistic design process where the loads and the strengths of materials and sections represented by their known or postulated distributions, defined in terms of distribution type, mean and standard deviation. The probability of failure (P_f) for a specific design case is calculated as the probability that the maximum total load effect exceeds the resistance to failure. This is opposed to limit state design (LSD) method, which is a semi-statistical design process in which the probabilistic aspects are treated at the code development stage (Kadry and Smaili, 2007) in order to define characteristic values and partial safety factors for load and resistance that are used to ensure, on average an acceptably low probability of failure across a full spectrum of design cases.

In reliability-based concept, the performance function of a structural system according to a specified mission is given by:

M = performance criterion – given criterion limit

$$M = g(X_1, X_2, \ldots\ldots, X_n) \quad (2.1)$$

in which the X_i (i = 1, ...,n) are the n basic random variables (input parameters), with g( ) being the functional relationship between the random variables and the failure of the system. The performance function can be defined such that the limit state of failure surface, is given by M = 0. The failure event is defined as the space where M > 0. Thus a probability of failure can be evaluated by the following integral.

$$P_f = \iint \ldots \int f_x(x_1, \ldots, x_n) dx_i \ldots dx_n \quad (2.2)$$

Where f_x is the joint density function of x_1, x_2, . . .,x_n and the integration is performed over the region where M < 0. Because each of the basic random variables has a unique distribution and they interact, the integral cannot be easily evaluated. Use is made of approximate method.

Traditionally, the concern of researchers was on the evaluation of structural reliability of steel and concrete structures and/or components. (Rosowsky and Ellingwood, 2002; Jinquan and Baidurya, 2007; Afolayan and Abubakar, 2003; Afolayan and Opeyemi, 2008; Holicky and Retief, 2005; Chinwedu, 2002). Due to the performance of highway structures during recent extreme events, much attention is being tailored on the reliability of highway structures under different traffic and environmental influence (Mustapha, 2014).

2.5 Different Methods of Structural Reliability

2.5.1 Monte Carlo simulation method

One of the approximate methods of evaluating Equation (2.9) is the method of Monte Carlo. The method is used to build probability density function (pdf) of the structural system response, as well as to assess the reliability of components or structures or to evaluate the sensitivity of parameter. Monte Carlo simulation consists of drawing samples of the basic variables according to their probabilistic characteristics and then feeding them into performance function (Rubistein,

1981; Kadry and Smaili, 2007). The major advantage of the Monte Carlo method is that this method is valid for static, but also for dynamic and probabilistic models, with continuous or discrete variables. The main drawback of this method is that it requires often large number of calculations and can be prohibitive when each calculation involved a long and onerous computer time (Kadry and Smaili, 2007).

To avoid the problem of long computer time in the method of Monte Carlo, it can be interesting to build an approximate mathematical model in the form of a polynomial function, called response surface method (Rajeshekhar, 1993). When a response surface has been determined, the system reliability can be easily assessed with Monte Carlo simulation. The practical problems encountered by the use of the response surface method are in the analysis of strongly non-linear phenomena where it is not obvious to find a family of adequate functions and in the analysis of discontinuous phenomena. Likewise, this approach may also be time consuming when there are larger numbers of random variables (Cordoso*et al*, 2008).

2.5.2 First and second order reliability method

First and Second Order Reliability Method (FORM/SORM) are other approximation methods. The Safety level of a structure is measured by a reliability index. There are different definitions of reliability index, β. The first was introduces by Cornell in the late 1960s (Cornell, 1969). Given the performance function as"

$$\mathbf{M = R - S = g(X_1, X_2, \ldots\ldots, X_n)} \quad (2.3)$$

The mean and the standard deviation of the margin can be defined as

$$\mu_M = \mu_R - \mu_S \quad (2.4)$$

$$\sigma_M = \sqrt{(\sigma^2 + \sigma^2)} \quad (2.5)$$

and the safety index is given by:

$$\beta_C = \frac{\mu_M}{\mu_S} \quad (2.6)$$

This reliability index is defined as the shortest distance between the failure surface defined by G(x) = 0 and the standardized origin (Figure 2.2).

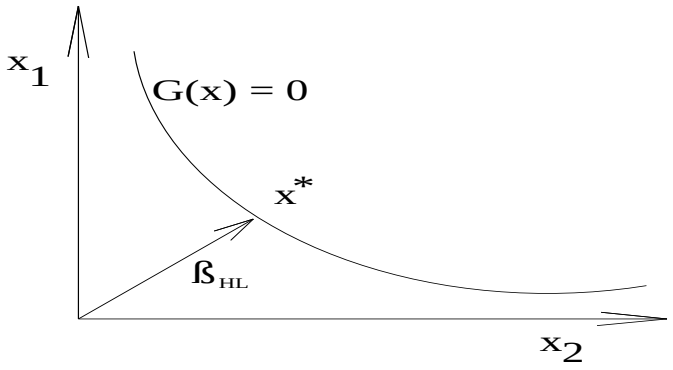


Figure 2.2 Reliability index β and the limit state function of the standardized space

The standardized variables are expressed as:

$$X = \frac{X - \mu_x}{\sigma_x} \quad (2.7)$$

The point x^* on the failure surface is of considerable importance. It is the design point. If a design is carried out using values from the design point this will result in the highest probability of failure, thereof the name. If the limit state function is linear and the variables are non-correlated, the shortest distance, that is β_{HL}, can be found directly using basic algebra. However, this is seldom the case. When the limit state function is non-linear, approximate methods must be considered. For example, geometrical optimization of the problem can be solved numerically or analytically. The analytical methods are usually based on a first order Taylor approximation of

the limit state function. That is why the method is called First Order Reliability Method (FORM) (Thoft-christensen and Baker, 1982).

An iterative method that can be used to find β_{HL}, x^*_i and an estimate of the probability of failure is the Rackwitz algorithm (Ang and Tang, 1984). The design point can be expressed in scalar form.

$$x_i^* = -\alpha_i^* \beta_{HL} \qquad (2.8)$$

where α_i^* are the direction cosines (the unit vector) in the x_i^* direction.

$$\alpha_i^* = \frac{\mathbf{A}}{\mathbf{B}} \quad (2.9)$$

where $\mathbf{A} = \frac{\partial \mathbf{g}}{\partial x_i^*}$ (2.10)

and $\mathbf{B} = \sqrt{[\sum(\frac{\partial \mathbf{g}}{\partial x_i^*})^2]}$ (2.11)

First Order Reliability Method (FORM) and the Second Order Reliability Method (SORM) are based on first order approximation of the limit state at the design point (Cheng, 2007; Melchers, 1999; Ditlevsen and Madsen, 2005; Rackwitz, 2000).

2.5.3 Probabilistic transformation method (PTM)

The probabilistic Transformation Method (PTM) developed by Kadry, Chateauneuf & El-Tawil, 2006 and Kadry and Chateauneuf, 2006 is a combination between the finite-element method and the probabilistic transformation method. It evaluates the probability density function (pdf) of a function by multiplying the input pdf by the Jacobian of the inverse function. The idea of PTM is based on the following formula (Hogg and Craig, 1978).

$$F_u(u) = f_p(p) \cdot \left| J_{p,u} \right| = f_b(p) \cdot \left| \frac{\partial \varphi^{-1}(u)}{\partial u} \right| \quad (2.12)$$

Where p is the input parameter, u is the response (solution), and $\varphi^{-1}(u)$ is the inverse transformation, which is determined either analytically or numerically.

One advantage of the PTM-FEM technique in the context of reliability analysis is the evaluation of the pdf, of the response in a closed form as opposed to other numerical methods which give only the first and second moment of response under some condition (Seifedine and Khaled, 2007).

2.6 Structural Reliability using Genetic Algorithms

Many human inventions were inspired by nature (Dyer, 2003), notably, the use of artificial intelligence in civil engineering domain. Advancement in artificial intelligence and its applications have led to the development of comprehensive search algorithms that lend themselves to immediate use in the reliability evaluation of structural systems and the identification of dominant failure modes. Areas that attracted considerable research outputs are the genetic algorithms and artificial neural networks (Lagaros and Papadopoulus, 2006).

According to Rackwitz (2001), a genetic algorithm (GA) is known to be capable of detecting global minima during the search for the reliability index. The Genetic algorithm is an adaptive heuristic search method based on population genetics. Genetic algorithm were introduced by John Holland in the early 1970s (Manoj*et al*, 2010; Black, 1996) It is an iterative procedure that consists of a constant size population of individuals, each one represented by a finite string of symbols, known as 'genomes', encoding a possible solution in a given problem space. This space referred to as the search space, comprises all possible solution to the problem at hand.

GA-base structural reliability method was first developed by Shao and Murotsu (1999). It consists of generating search directions in the space formed by the random variables affecting the reliability of a structural system. By following a search path until failure is reached, the

reliability index, β, corresponding to the search direction is obtained. Genetic search algorithm simulates the biological process of evolution in an attempt to identify the parameters that is genes that define an optimum system.

Genetic algorithm is an iterative procedure that consists of a constant-size population of individuals, each one represented by a finite string of symbols, known as the "genome", encoding a possible solution in a given problem space, comprises all possible solutions to the problem at hand.

The standard genetic algorithm proceeds as follows (Black, 1996): an initial population of individuals is generated at random or heuristically. Every evolutionary step, known as generation, the individuals in the current population are decoded and evaluated according to some predefined quality criterion, referred to as fitness. The fundamental principle for building an artificial system that reproduces and mimics the working of evolution date back to the biologist Barricelli who formulated an article about artificial methods to realize evolutionary processes (Baricelli, 1957).

The issue of efficiency of GA, which is of concern in all types of applications, is the subject of continuous research (Goldberg and Samtani, 1986; Goldberg, 1989; Grigoriu and Turkstra, 1979). Deng *et al,* (2005) proposed a method for improving the efficiency of genetic algorithm during the search for the minimum reliability index of a structural system. This is based on a splitting technique that shreds a chromosome into many small fragments, which can be easily identified.

A dominant structural failure mode normally occurs when a set of structural members fail. Also, the failure of subsets of those members would lead to the occurrence of 'partial failure' can be identified, and then some of these are very likely to contribute to the dominant failure mode.

Conversely, a possible solution that does not contain any important ‘partial failure’ is unlikely to be the solution that would lead to a dominant failure mode. Linkage shredding genetic algorithm for reliability of structural system was proposed by Wang and Ghosn (2005).

The genetic algorithm (GA) that differs from other classical optimization in four ways (Goldberg, 1989) is a part of evolutionary computational technique and probabilistic and global search method. Due to these advantages, the GA has been preferred in wide ranges of optimization problems. In order to apply the genetic algorithm, a population of solutions within a search space is initialized on the contrary of the traditional optimization methods that starts from a single point solution.

The population can be viewed as points in the search space of all solutions to the optimization problem. Each individual in population has a fitness value defined by a fitness function. Then the artificial evolution processes called the genetic loop which mimic natural evolution are applied to produce new candidate solutions. At the end of the process, the newly created generation replaces previous generation and revolution is repeated until a satisfying solution to the problem is obtained ensuring certain design criteria are satisfied or a maximum number of generations are reached.

CHAPTER THREE

MATERIALS AND METHOD

3.1 Introduction

This chapter provides an overall methodology to compute the transitional probability of flexible pavement subjected to traffic, material and environmental degradation. There are various components that make up the overall model, which is analogous to "input-process-output" for characterizing a system. The chapter begins by explaining about the inputs of the model and the sources from which these inputs could be obtained. Traffic information can be obtained from actual traffic count to obtain Average Daily Traffic, Annual Average Daily Traffic and subsequently the total number of vehicle over the design period of the pavement. Environmental condition due to seasonal changes in temperature and moisture can be observed by influencing the subgrade stiffness. The material characterization can be used to ascertain the Poisson ratio and the modulus of elasticity. This input will result in Pavement Performance Prediction model. In order to obtain the transitional probability, the pavement must be classified based on the severity of the damages on the pavement with upper and lower limits well specified. The

probability of remaining and exceeding these limits will easily be computed. The output from the model is the probability of transition from one condition state to another.

3.2 Model Inputs

3.2.1 Traffic

Traffic data are required in the M-E pavement design procedure. Design traffic is expressed in terms of 8, 200 kg (80 kN) equivalent single axle loads (ESAL). Estimate of ESAL projected for the design life of the pavement (future traffic) is made using information from historical data to predict future growth parameters. Future traffic can be estimated from traffic growth rate factors as follows:

$$Q = ADT(FTG) \quad (3.1)$$

Where

Q_f = total number of vehicles over the design period

ADT = average daily traffic

FTG = future total growth factor

$$FTG = \frac{365[(1+i)^n - 1]}{\ln(1+i)} \quad (3.2)$$

Where i = growth rate

The future traffic is distributed by direction and lanes for the purpose of design. It has been found in Nigeria that the direction distribution factors (DDF) are between forty and sixty percent (Claros, Carmicheal & Harvey, 1986).For multilane facilities, the controlling lane is generally the outside lane. A lane distribution factor (LDF) is applied depending on the number of lanes

e.g. for 2 lanes both direction LDF = 1.0 is used. The design traffic is therefore calculated as number of equivalent single axle load (ESAL) expected to be carried on the design lane over the design period. The design traffic (n) is calculated by applying a directional distribution factor (DDF), a lane distribution factor (LDF), the percent truck factor (Pt), and an average 8, 200 kg single axle load equivalent factor to the total number of vehicles over design period. The equation to calculate the design traffic is:

$$n = Q_f(DDF)(LDF)(P_t)\left(F_{avg}\right) \quad (3.3)$$

Where

n = number of cumulative ESALs to be carried by critical lane over design period

Q_f= total number of estimated future vehicles during the design period, in both directions

DDF = directional distribution factor (between 0.4 and 0.6)

LDF = lane distribution factor

P_t= percent trucks, and

F_{avg}= *average 8, 200 kg single axle load equivalence factor from the TRUKWT program.*

3.2.2 Material characterization

3.2.2.1 Asphalt concrete material

The required asphalt concrete material properties are the Poisson's ratio and dynamic modulus of elasticity. While modulus of elasticity values must be estimated, Poisson's ratio values are assumed constant. The dynamic modulus of the asphalt concrete can be measured in the lab using indirect tensile test (ASTM d-4123). In order to characterize asphalt mix for new pavement the asphalt mix design for new pavement is computed and six samples material can be molded to

design density and tested in the dynamic indirect tensile device to obtain the resilient modulus of elasticity (ER) at room temperature. Then ER is corrected to the temperature condition in the field.

3.2.2.2 Bases and sub-bases

For unbound bases and sub-bases, the resilient modulus MR can be estimated from in-situ CBR test using the following relationships developed for Nigerian soils (Claros *et al*, 1986), which now expressed in S.I. units are presented below:

For base layers

M_{RB} = Resilient modulus of Base layer *kPa*

FLDCBR = Field *CBR* of the Base material

For Sub base Layers

M_{RSB} = Resilient modulus of Sub Base layer *kPa*

$FLDCBR_{SB}$ = Field *CBR* in the Sub Base material

Sub grade Material: Usually samples are tested in the lab for resilient modulus (M_R).

3.3 Mechanistic Pavement-Performance Model

The transition probabilities are determined within the framework of a mechanistic pavement performance model of Chua (1992), wherein the pavement is modeled as a structure consisting of various components acted upon by wheel loads. A structural analysis of the pavement is first performed to determine the controlling structural response that governs the extent of damage for each distress mode. This may be adequately achieved using the multilayered model (Monismith and Finn 1978). In particular, the ELSYM5 program for multilayered elastic analysis is employed for the present purpose. The remaining life of the pavement before the distress exceeds prescribed levels can then be determined from a distress sub model based on the controlling

structural response. In the present study, only the distress modes corresponding to fatigue cracking and rutting due to repetitive axle loads are considered. The implementation may be similarly extended to other modes of distress, differing only in the exact details of the mechanistic model representation. In the case of fatigue cracking, the maximum tensile strain in the asphalt bound layer is the governing structural response.

3.3.1 Fatigue cracking

Miner's (1945) cumulative damage concept has been widely used to predict fatigue cracking. It is generally agreed that the allowable number of load repetitions is related to the tensile strain at the bottomof the asphalt layer. The amount of damage is expressed as a damage ratio, which is the ratio between predicted and allowable number of load repetitions. Damage occurs when the sum of damage ratio reaches one. The major difference in the various design methods is the transfer functions that relate the hot mix asphalt (HMA) tensile strains to the allowable number of load repetitions. The allowable number of load repetitions (N_f) can be computed using Equation below

$$N_f = 1.66\ x\ 10^{-10}(E_t)^{-4.32} \quad (3.4)$$

(source: Murana, 2010)

Where E_t is tensile strain at the bottom of HMA

3.3.2 Rutting

Rutting models are used to limit the vertical compressive strain on the top of the subgrade and are widely used. The allowable number of load repetitions (N_d) to limit rutting is related to the vertical compressive strain (E_V) on top of the subgrade by Equation given below;

$$N_R = 2.56\ x\ 10^{-8}(E_V)^{-4.5337} (3.5)$$

(source: Murana, 2010)

WhereE_V is the vertical compressive strain at the top of the subgrade.

3.4 Stochastic Framework

3.4.1 Individual and system distress

Generally, a distress state for a given mode may be characterized by two damage indexes, L_{wk} and L_{wj}, representing the lower and upper bounds, respectively. In the case of a failed section (severely cracked or rutted) only a lower bound applies. The damage index for fatigue cracking denoted by superscript f is defined so that a value of unity denotes 100% cracking, and $L^f_w$is determined by the ratio N/N_d for any arbitrary w% extent of cracking. A similar damage index L^R_wdenoted by superscript R taking values ranging from zero to unity for any arbitrary w% of critical rut depth may be defined so that a value of unity would denote severe rutting. Next, consider limit-state functions $g^f_{wT}(x)$ and $g^R_{wT}(x)$ for fatigue and rutting, respectively, where

$$g^f_{wjT}(X) = L^f_{wj} - \sum_{t=1}^{T}\sum_{i}^{n}\frac{\left[Q_f(DDF)(LDF)(P_t)\left(F_{avg}\right)\right]}{N_f} \quad (3.6)$$

$$g^f_{wkT}(X) = L^f_{wk} - \sum_{t=1}^{T}\sum_{i}^{n}\frac{\left[Q_f(DDF)(LDF)(P_t)\left(F_{avg}\right)\right]}{N_f} \quad (3.7)$$

$$g^R_{wjT}(X) = L^R_{wj} - \sum_{t=1}^{T}\sum_{i}^{n}\frac{\left[Q_f(DDF)(LDF)(P_t)\left(F_{avg}\right)\right]}{N_R} \quad (3.8)$$

$$g^R_{wkT}(X) = L^R_{wk} - \sum_{t=1}^{T}\sum_{i}^{n}\frac{\left[Q_f(DDF)(LDF)(P_t)\left(F_{avg}\right)\right]}{N_R} \quad (3.9)$$

x = vector of input variables in the mechanistic performance model; n_{it} = actual number of load applications at strain level ε_t in the *t*th year; N_f and N_R = allowable number of load applications at strain level ε_t for fatigue cracking and rutting modes, respectively. Using the generic form g_{wT} for limit-state functions, if $g_{wT}(x) < 0$, the linear sum of cycle ratios after T years of load applications would have exceeded the damage index, and the extent of distress would have exceeded *w*%.

Thus, for an individual distress mode the probability of transition P_F into a distress state bounded by L_{wj} and L_{wk} would be given by

$$P_F = P\left[g_{wjT}(X) < 0\right] - P[g_{wkT}(X) < 0] \qquad (3.10)$$

The limit state functions are derived from the individual distresses. The probability of "failure" of the pavement section is defined as follows:

$$P_F = P\{g(X) \leq 0\} = \int_{g(X)\leq 0} f_x(X)dx \quad (3.11)$$

Where $f_x(x)$ is the joint probability density of variables $x_1, x_2, \ldots, x_n$. The reliability is then the probability that the design criterion is not exceeded, or $1 - P_F$. An analytical evaluation of the integral in Equation 3.9 is possible in only a few special cases, and hence numerical integration is necessary. However, the limits of integration become intractable whenever the number of random variables exceeds two or three.

3.5 Evaluation of the Limit State Functions

During the last two decades much efforts has been directed towards reliable and efficient methods for reliability analysis of structures. Among these methods, first order and second order reliability (FORM and SORM) methods are the earliest and most widely used. FORM and SORM are based on first-order or second-order approximations of the limit-state at the design point. Extensive reviews of these methods are found in (Rackwitz, 2000). These methods require

the evaluation of the derivatives of limit state functions with respect to the random variables. When these functions are explicit functions of the random variables, it is easy to compute the derivatives of these functions.

In many cases of practical importance, particularly for complicated structures, the limit-state functions are usually implicit in terms of the random variables. Therefore, derivatives of the limit-state functions are not easy to derive. Moreover, a significant disadvantage of these methods is that they may lead to erroneous results when the limit-state function has multiple local minimal distance points (Cheng, 2007).

In this study, a different route for structural reliability employing genetic algorithm (GA) is adopted. The genetic algorithms calculate the probability of failure by using the genetic search technique based on a natural selection process and following a search path until failure is reached (Cheng, 2007; Rackwitz, 2000; Wang and Ghosn, 2005). Compared with FORM/SORM, the genetic algorithm has advantage that it does not involve the difficulties of computing the derivatives of limit state functions with respect to the random variables and has the capability of identifying global optimum values of the limit state functions. A genetic algorithm based reliability problem can be formulated in the following form:

Minimise $\beta = ||\mu||^2 = \mu^T.\mu$ (3.12)

Subject to: $g(\mu) = 0$

Where μ is the vector of standard normal variates; $g(\mu)$ is the limit state function; β is the reliability index. The problem in Equation (3.12) is a constrained nonlinear optimization problem. The genetic operation is continous until the following stopping criteria is achieved.

1. The average reliability index of the current generation does not show significant improvement over the former generation.

   $\gamma\beta^{(k+1)generation} > \gamma\beta^{kgeneration}$

γ can be set to 0.95 (Wang and Ghosn, 2005)

2. The first three different minimum reliability indices of the current generation remain the same as those of the previous generation.

3.6 GA Based First Order Reliability Analysis Formulation

Considering the independent standard normal U-space where failure is defined by a number of failure modes, g_1,.g_2, , , , ,g_k. These modes are reached by following the failure paths in direction $\boldsymbol{I}_i$. In the genetic algorithm (GA) according to Shao and Murotsu (1999), $3^n - 1$ discrete search directions are used in n-dimensional space. The search direction vectors are expressed by a three-digit system where each of the digits belong to the binary set (-1,0,1) as illustrated in Fig 3.1.

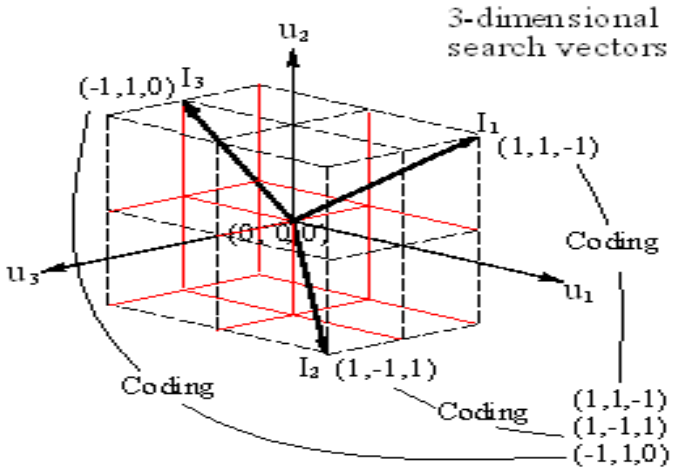


Figure 3.1: Coding of search direction vectors

The rectangular coordinates of a direction vector constitute a string of digits known as chromosomes in GA parlance. GA simulates the natural selection process based on the survival of the fittest (in this case, which direction should survive and which direction should be killed off). In the GA-based reliability analysis, a fitness is defined as the inverse of the reliability

index, β, where β is the distance from the origin of the U-space to the failure surface as shown in Fig. 3.2.

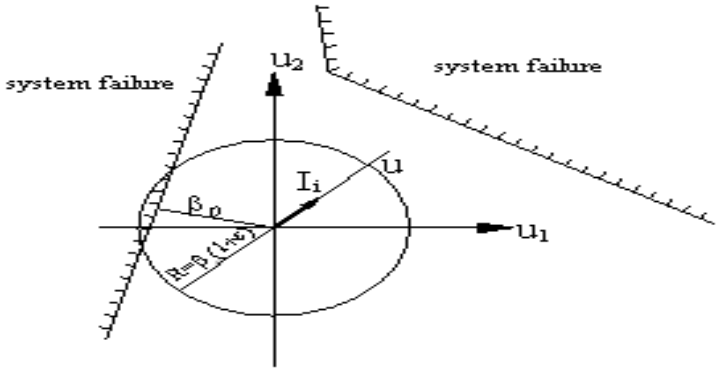


Figure 3.2Illustration of GA search strategy

To initiate the search process, GA randomly generates a set of chromosomes to form a first generation of directions. The search is refined by creating a "new generation' of chromosomes using three operators: Selection, Crossover and Mutation. The objective is to improve the fitness of the population of chromosomes with each generation.

Selection simulates the evolution process that allows the chromosomes with best fitness to survive and mate. The chromosomes in a new generation are the offspring (or children) of selected chromosomes of the previous generation. The selection is done randomly such that chromosomes with high fitness have the higher probabilities to contribute offspring in a new generation.

Crossover simulates the natural mating phenomena whereby offspring in a new generation are obtained by crossing the chromosomes of two "parents" directions. Each offspring will keep some of the genes from each of its two parents as shown in Figure 3.3

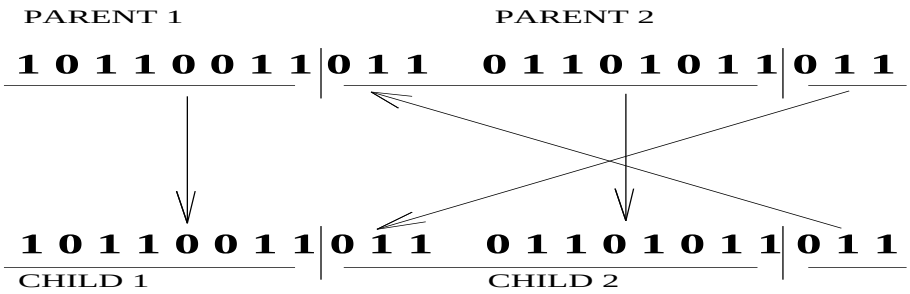


Figure3.3 Illustration of crossover operation

Mutation is secondary operator that simulates an inheritance that goes astray. It is an explorative operator that allows the search to wander into regions of the solution space which may never have been reached from the crossover operation. The purpose of the mutation operator is to reduce "in-breeding" that may misdirect the search process. This, new "genetic material" is occasionally introduced into the new generation by randomly changing digits in each chromosome. For example, a 1 is randomly changed to 0, or vice versa. This is done only with a very low probability to generate new genetic material without losing track of the important directions that the search has produced so far. Consider a typical chromosome that is formed by a total of n genes (or variables). For example (1 0 1 1 0 0 1 1 1 0 …….) is one representation of such a chromosome. The m genes associated with 1 in the sequence are called active variables. In this example, the active variables are located in positions 1,3,4,7,8,9,……. The other genes are not active.

The reliability index associated with the failure modes described by the chromosome is β. An approximation to the probability of failure P_f can be obtained by assuming that the safety margin M (or failure function) is linear in the space defined by the n variables (X_1, X_2, . . .,X_n). This linear equation can be expressed as:

$$M = \frac{1}{\sqrt{m}}(X_1 + X_3 + X_4 + X_7 + X_8 + X_9 + \ldots) + \beta \qquad (3.13)$$

Only the active variables appear in Equation (3.13), because the non-active variables are known constant (deterministic) parameters that do not affect the safety index.

For a given search direction, on the U-space, the failure point (design point) is designated by **A** and its coordinates are expressed as:

$$\mathbf{OA} = \beta(k_1\mathbf{x}_1, k_2\mathbf{x}_2, \ldots\ldots\ldots, k_n\mathbf{x}_n) \qquad (3.14)$$

Where $k_1, k_2, \ldots\ldots, k_n$ are **OA**'s unit vector coordinates in direction $x_1, x_2 \ldots x_n$ such that:

$$(k_1^2 + k_2^2 + \ldots\ldots + k_n^2)^{1/2} = 1 \qquad (3.15)$$

For the coding adopted in this work, as outlined in Fig. 4.8, the k's can be represented as:

$$(k_1, k_2, \ldots\ldots k_n) = (\boldsymbol{I}_1\boldsymbol{I}_2 \ldots\ldots \boldsymbol{I}_n)\frac{1}{\sqrt{m}} \qquad (3.16)$$

Where $\boldsymbol{I}_i = 1$ or 0. A value of 1 indicates an increase in the direction of variable x_i. 0 indicate that x_i remains at its mean value (origin). The index i = 1,2, ...n, identifies the variable. m gives the total number of non-zero terms in the set $(k_1, k_2, \ldots, k_n)$. This would lead to the alternate representation of OA as:

$$OA = \frac{\beta}{\sqrt{m}} = (\boldsymbol{I}_1 x_1 \boldsymbol{I}_2 x_2 \ldots\ldots \boldsymbol{I}_n x_n) \qquad (3.17)$$

Each set of $\boldsymbol{I}_i$ values defines a chromosome that represents the direction along the n-dimensional space $x_1, x_2, \ldots\ldots x_n$. The n variables X_n in turn represents the sequence of random variables $X_1, X_2, \ldots . X_n$ in tha order in the normalized U-space. These random variables are denoted $l_1, l_2, .$

. . . . .,l_n, An approximate number of chromosomes define the population of each generation. If too few chromosomes are generated, GA has only few possibilities to execute the crossover operation. On the other hand, if there are too many chromosomes, GA slows down. Research has shown that increasing the population size beyond a certain limit will not help improve convergence (Deng 2005). Following the approach of Cornell (1996), it is suggested that an optimum population size would be on the order of:

$$N_{popsize} = order(r\frac{L}{2k}3^k) \qquad (3.18)$$

Where $n_{popsize}$ is the optimum population size, L is the length of the chromosome, and k is the average number of bits per parameter. The value that each active variable takes when failures occur is given by:

$$L_{active} = \beta\frac{i}{\sqrt{m}} = \frac{(x_{active} - \mu_{active})}{\sigma_{active}} \qquad (3.19)$$

Where m is the number of non-zero (active) genes, μ_{active} is the mean value of the active variable, σ_{active} is the standard deviation of the active variable, β is the safety index.

3.7 System Reliability Analysis Formulation

Reliability models applied to highway pavement were always related to individual components but not the system. As any real structure is a complex system, system behavior must be of particular interest. All components in structural systems, must satisfy the basic code requirement, that is,the load effect cannot exceed the resistance. The component-based design is highly conservative because of redundancy. When the load in a critical component approaches the ultimate value, other components can take additional loads and prevent a failure. However, the

quantification of this load sharing requires a special approach using the system reliability models. A considerable effort was directed on the development of the reliability analysis procedures for structural components. The formulation of the limit state function for a structural system is much more difficult than for a component

In the study, two failure modes were considered (i.e Fatigue and Rutting modes). Consequently, the two value of Hasofer-Lind reliability index corresponding to two modes of failure were calculated. The system probability of failure of structural systems is calculated from (Melchers, 1999; Afolayan, 2005; Detlevsen and Madsen, 2005):

$$P_{sys} = P(E_i U ... U E_n)$$

$$= \sum_{i=1}^{n} E_i - \sum_{j>i=1}^{n} (E_i \cap E_j) + \sum_{k>j}^{n} (E_i \cap E_j \cap E_k) + (-1)^{n-1} P(E_i \cap \ldots \cap E_n) \qquad (3.20)$$

Where E_i, I = 1,2,…,n represent the violation of the established failure modes for the component under consideration.

3.8 Implementation of the Reliability Analysis

The reliability analysis in this study was implemented through a developed program using MATLAB Simulink.

3.8.1 Program flowcharts

The first step for writing a computer program is the formulation of the appropriate flowchart, to show the necessary steps for realizing the algorithms for each of the given problem at hand. In this study, Genetic Algorithms (GA) based First Order Reliability Methods (FORM) was used.

In GA based FORM, each of design random variables is considered as a gene that define the inherent uncertainty. The set of random variables is known as chromosome. Unlike the conventional FORM, in which single solution is obtained at a time, the GA-based FORM

generate population of solution through the process of natural selection based upon survival of the fittest, as a simulation of the heredity and genetics first proposed by Charles Darwin. In this way, the search of the global probability of failure is enhanced by the genetic operators of reproduction, cross over, mutation and elitisms. The compact flowchart for the program development is presented in Figure 3.4

3.8.2 Program documentation

The developed MATLAB programs consist of main program and twenty subroutines as follows;

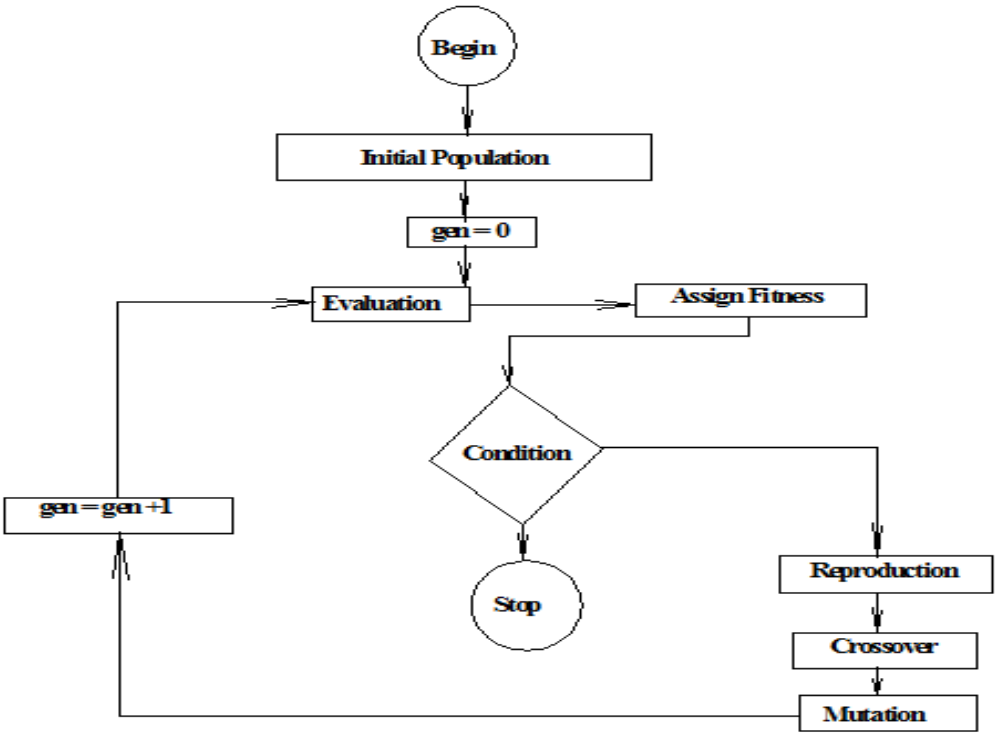


Figure 3.4 Flowchart for genetic algorithms

i. Main program **'adamu.m'**: This is the program that coordinates the entire process of the reliability analysis. It accept the input parameter and call all the required subroutines in order to evaluate the limit state functions and determine both the component as well as the system probability of failure. There are two types of parameters; those that are

embedded in the program and those that are entered via keyboard. The detailed definition of the random and deterministic parameters used in the program is presented in the table for the stochastic models of the basic variables.

ii. Subroutine 1**'condition_state_1_fatigue.m'**: This is the subroutine that evaluates the limit function for the fatigue condition state 1.

iii. Subroutine 2 **'condition_state_2_fatigue.m'**: This is the subroutine that evaluates the limit function for the fatigue condition state 2.

iv. Subroutine 3 **'condition_state_1_rutting.m'**: This is the subroutine that evaluates the limit function for the rutting condition state 1.

v. Subroutine 4 **'condition_state_2_rutting.m'**: This is the subroutine that evaluates the limit function for the rutting condition state 2.

vi. Subroutine 5 **'invnorm.m'**: This is the subroutine that determines the cumulative distribution of a random variable.

vii. Subroutine 6 **'gamm.m'**: This is the subroutine containing a function required for rackwitz fiessler transformation of non-normally distributed random variables to equivalent normally distributed random variables.

viii. Subroutine 7 **'gumbel.m'**: This is the subroutine that transforms gumbel to normal variables.

ix. Subroutine 8 **'frechet.m'**: This is the subroutine that transforms frechet to normal variables.

x. Subroutine 9 **'lognormal.m'**: This is the subroutine that transforms lognormal to normal variables.

xi. Subroutine 10 **'weibull.m'**: This is the subroutine that transforms weibul to normal variables.

xii. Subroutine 11 **'system_reliability.m'**: This is the subroutine that performs the required system reliability analysis.

xiii. Subroutine 12 and 14 **'distribution_model_1.m and distribution_model_2'**: These are the subroutines that assign appropriate distribution models to each basic design variable.

xiv. Subroutine 13 and 16 **'covariance_model_1.m and covariance_model_2'**: These are the subroutines that assign appropriate coefficients of variations to each basic design variable

xv. Subroutine 17 **'cross.m'**: This is the subroutine that performs cross over operation required for the genetic algorithm implementation of the reliability analysis.

xvi. Subroutine 18 **'mutate.m'**: This is the subroutine that performs mutation operation required for the genetic algorithm implementation of the reliability analysis.

xvii. Subroutine 19 **'initialise_population.m'**: This is the subroutine that generate the initial population of chromosomes with genes using binary coding system with monte carlo simulation (MCS).

xviii. Subroutine 20 **'tournament_select.m'**: This is the subroutine that selects the best for individual and kills the less fit individuals through the process of natural selection in GA parlance.

3.9 Illustration

In order to demonstrate and evaluate the applicability of the proposed model, data obtained from Pavement Evaluation unit (PEU) for the 10KM (CH 25+000 to CH 35+000) length of the road, from Rigachikun, PEU, Federal Ministry of Works Kaduna , Kaduna- Kano road project, 1987.

In this project the recommended NEMPAD Fatigue and rutting distress model equations evaluated in Murana (2010) were used, in order to compute probability of transition from one bound state to another.

Table 3.1 Variability of Design Input Parameters

Variable Description	Distr. Type	Light Truck Traffic		Heavy Truck Traffic	
		Mean	COV	Mean	COV
Annual Daily Traffic	Lognormal	1818	0.05	1818	0.05
Asphalt stiffness	Lognormal	4827	0.05	4827	0.05
Horizontal tensile strain	Normal	3.34E+09	0.05	3.34E+09	0.05
Ver. Compressive strain	Normal	2.71E+12	0.05	2.71E+12	0.05
Truck percentage	Lognormal	27.4	0.05	34.5	0.05
Load equivalent factor	Normal	2.5	0.05	2.5	0.05
Lane Distribution factor	Deterministic	1	-	1	-
Dir. Distributional factor	Deterministic	0.5	-	0.5	-
Traffic growth rate	Lognormal	5	0.05	5	0.05
Asphalt conc. Thickness	Lognormal	Varies	0.05	Varies	0.05
Damage index	Deterministic	Varies	-	Varies	-
Estimated future vehicle	Lognormal	Varies	0.25	Varies	0.25
Actual load application	Lognormal	Varies	0.25	Varies	0.25

CHAPTER FOUR

RESULTS AND DISCUSSION OF RESULTS

4.1 Introduction

The result of the overall component and system reliability are presented in this chapter

4.2 Discussion of Result

4.2.1 Cumulative damage analysis

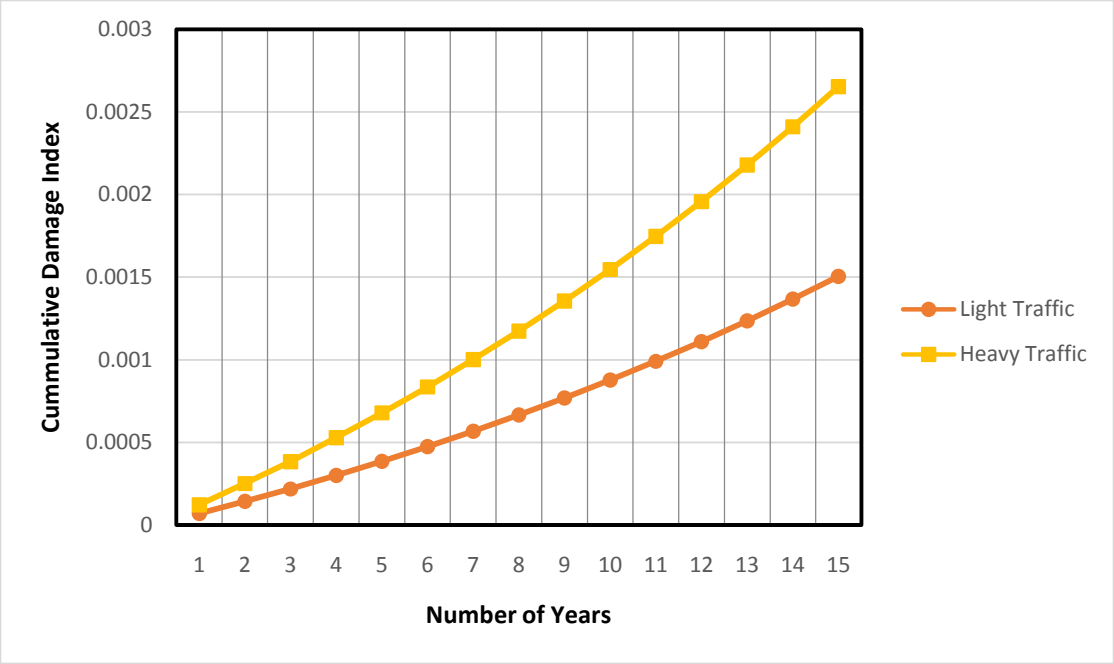


Figure 4.1Cumulative Damage for Fatigue Distress

The pavement is a four layered section comprising of 100mm asphalt concrete layer over a 200mmbase course and 250mm sub base underlain by 7620mm subgrade. The inputs variable mechanistic pavement performance model include pavement configuration, the material properties for each layer, the environment and traffic factors. The model is capable of handling variety of distribution types, all value assume their distribution from literature.

The effect of truck traffic volume is also considered. Data for light and heavy traffic were shown in Figure 4.2. The heavy traffic is characterizes by higher truck percentage, and higher axle factor, which is representative of higher percentage of truck with five or more axle. The result assume the damage indices were set to 0.2 – 0.4 for fatigue and 0.01 – 0.02 for rutting

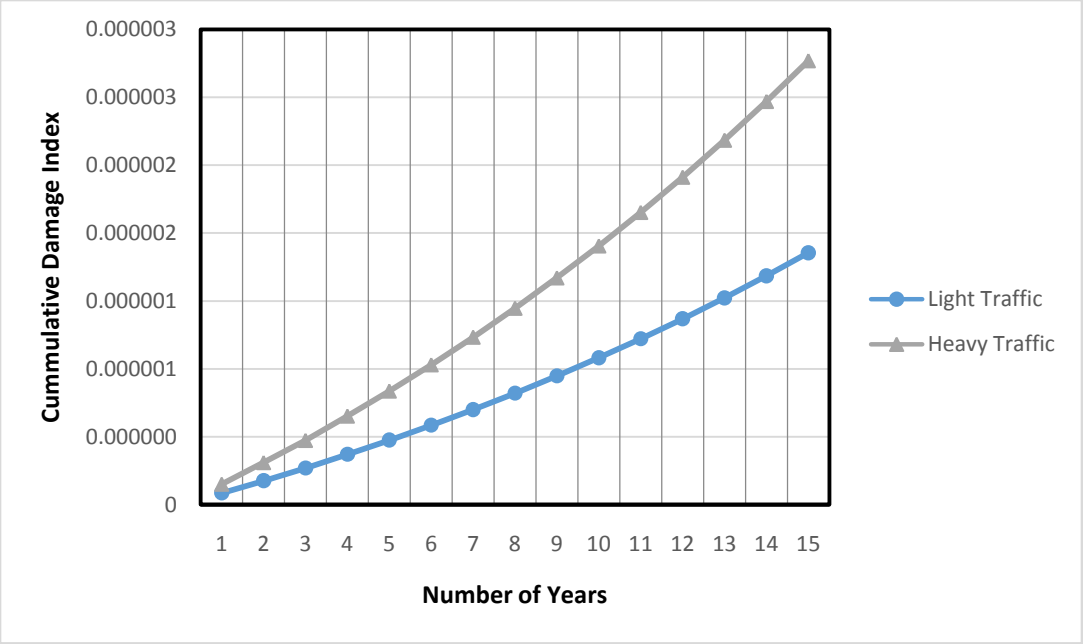


Figure 4.2 Cumulative Damage forRutting Distress

The cumulative damage result in figure 4.1 shows a recorded damage of 6.97 x 10^{-5} at first year to 1.50 x 10^{-3} at 15th year for light traffic and 1.23 x 10^{-4} at first year to 2.65 x 10^{3} at the 15th year for heavy traffic. This is evident as heavy traffic causes more damage as compared to light traffic because of higher truck percentage and axle factor. Similar behavior was also observed with the rutting damage shown in Figure 4.2

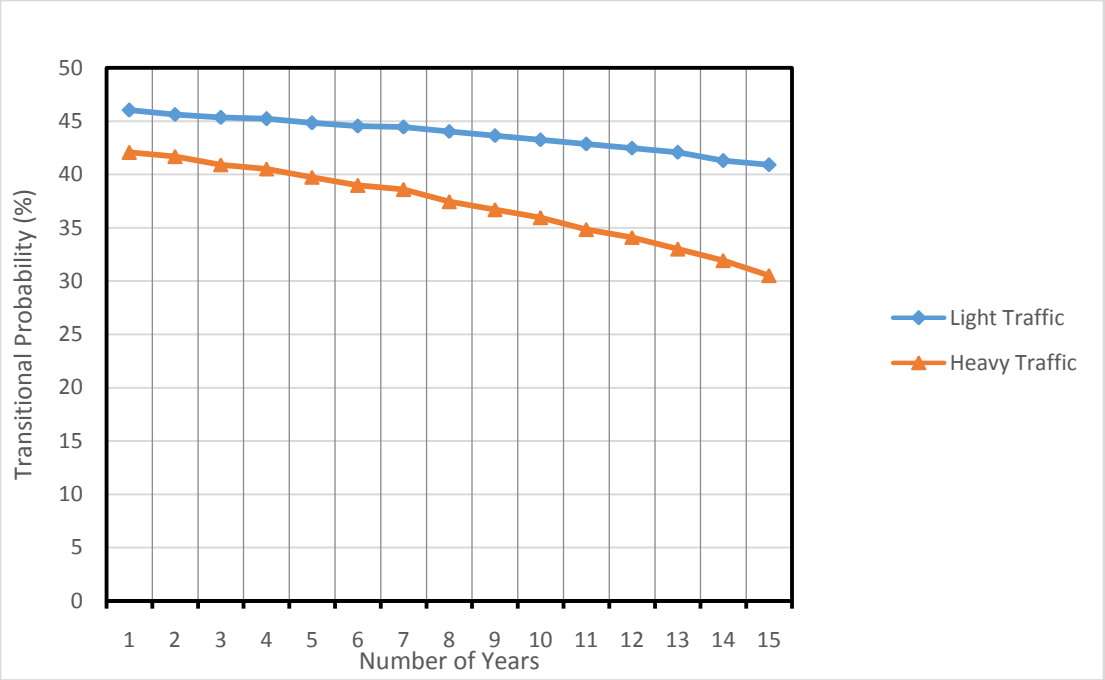


Figure 4.3 Transitional Probabilities for Fatigue Mode

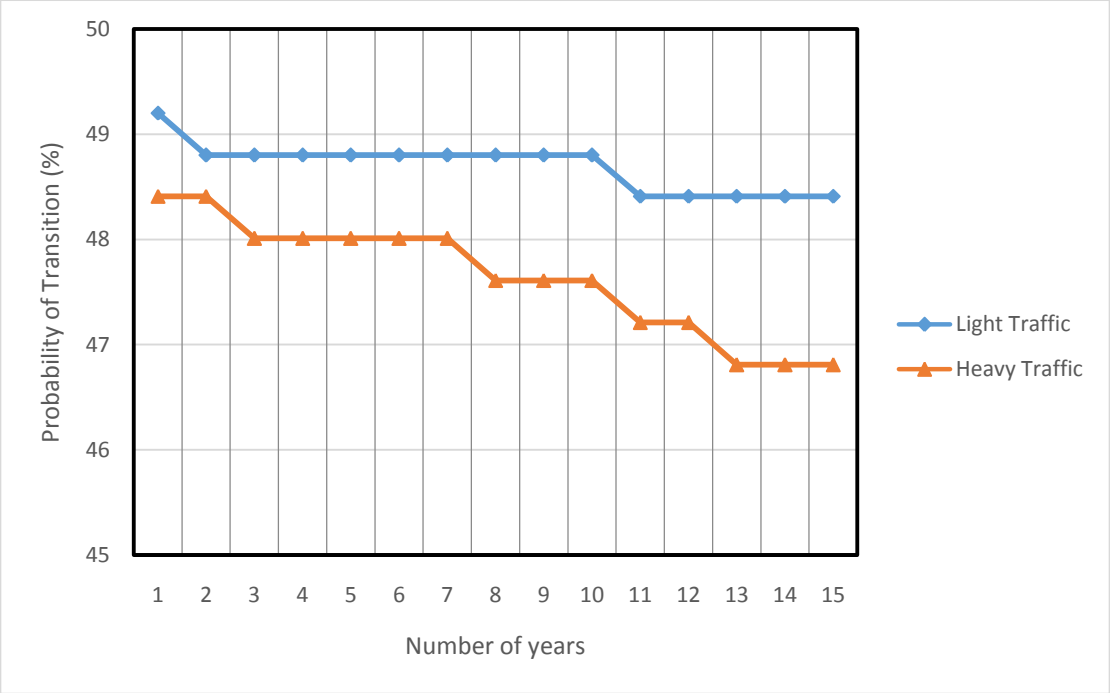


Figure 4.4 Transition Probabilities for Rutting Mode

4.2.2 Individual (Component) transitional probability analysis

The probability of remaining in the assumed condition state for fatigue is shown in Figure 4.3 beginning from year one to year fifteen. The result recorded a transition of 46.02% at year one to 40.91% at year 15th for light traffic and 42.07% at year one to 30.50% at year 15th for heavy traffic. This shows that heavy traffic has lower probability of transition as compared to light traffic because of the associated higher damage. Similar behavior was also observe in Figure 4.4.

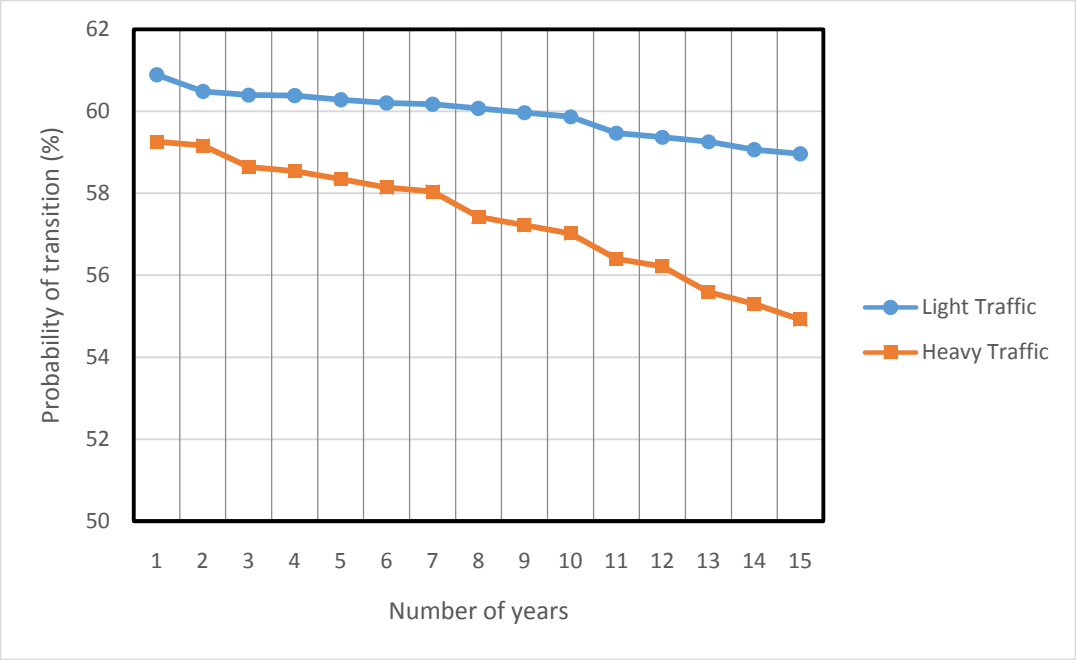


Figure 4.5 Combine Transitional Probability (Fatigue & Rutting)

4.2.3 Combine (System) transitional probability analysis

Figure 4.5 represent the combine probability of transition, the decrease in graph shows that the probability of transition is not time-invariant. It decreases by as much as 5% over the 15thyear. This arises as a result of the annual growth in the traffic volume. The difference will be larger with higher traffic growth. Thus, the assumption of a constant transition matrix in the Markovian

decision process in the existing pavement management system may not be valid when traffic growth is considered.

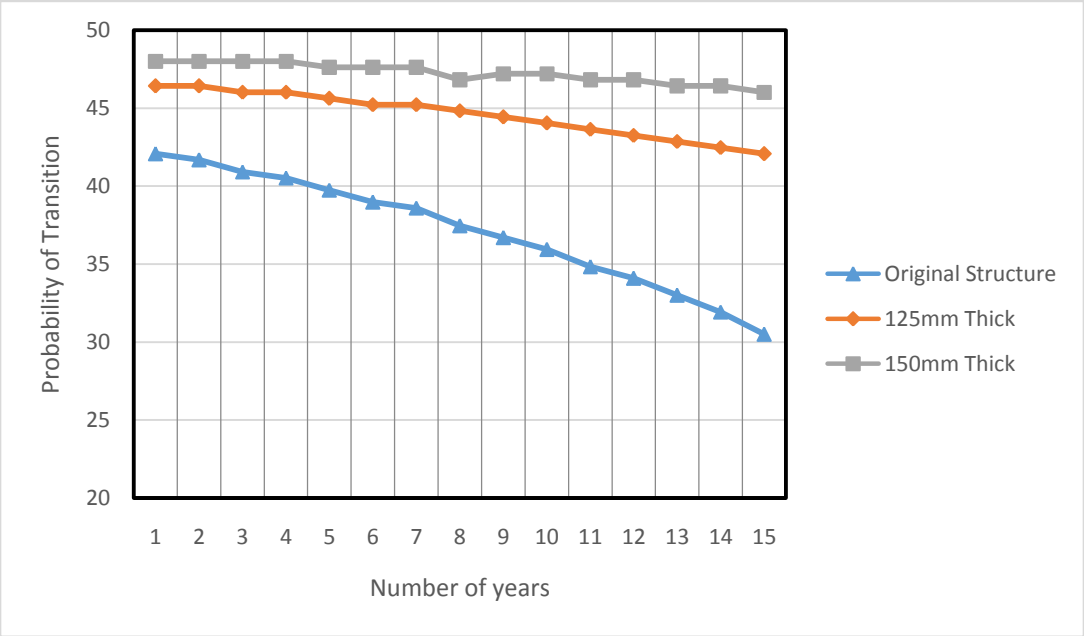


Figure 4.6 Effect of Pavement Structure on Probability of Transition (Heavy Traffic)

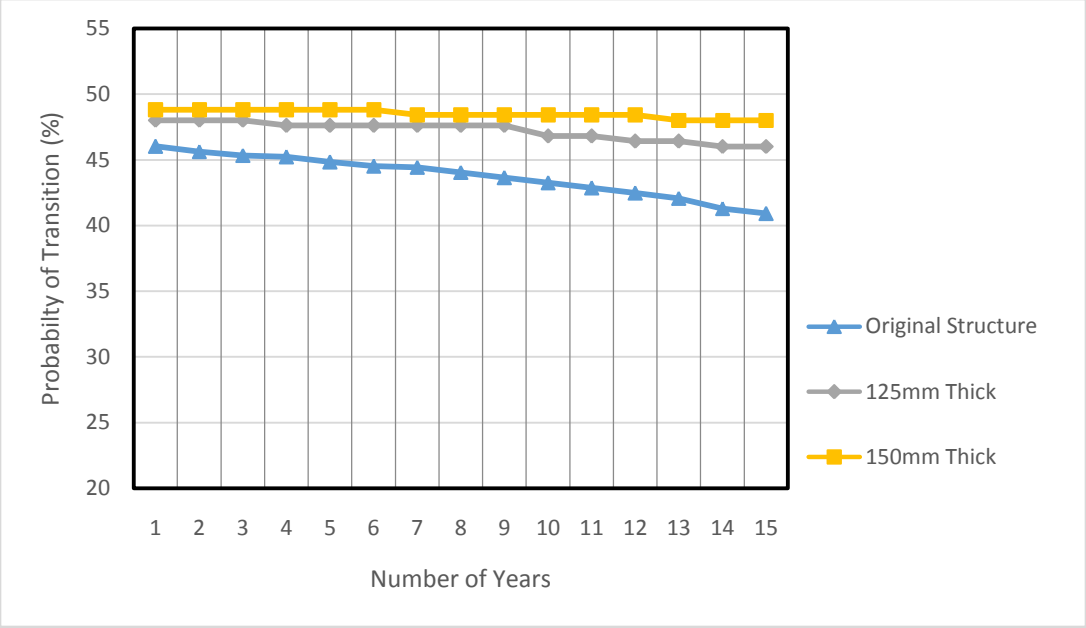


Figure 4.7 Effect of Pavement Structure on Probability of Transition (Light Traffic)

4.2.4 Effect of increasing pavement thickness

The effects of asphalt thicknesses on transition probability for both heavy and light traffic were also examined. The performance of the original pavement structure is compared to one with 125mm thick and one with 150mm. The 125mm thick increase the probability of remaining in fatigue condition state by about 4% at the beginning and 12% after the 15th year. However, with 150mm thick, the probability of remaining in fatigue condition stateonly increases by 2% at the beginning to 4% after 15th. The effect of increased thickness in the asphalt bound layer does not have a linear effect on the strains in the pavement structure. Obviously, increasing thicknesses in the asphalt bound layer have a nonlinear effect on strengthening the pavement when considering the probability of remaining on fatigue condition state at least for thickness above 125mm for the present pavement structure.

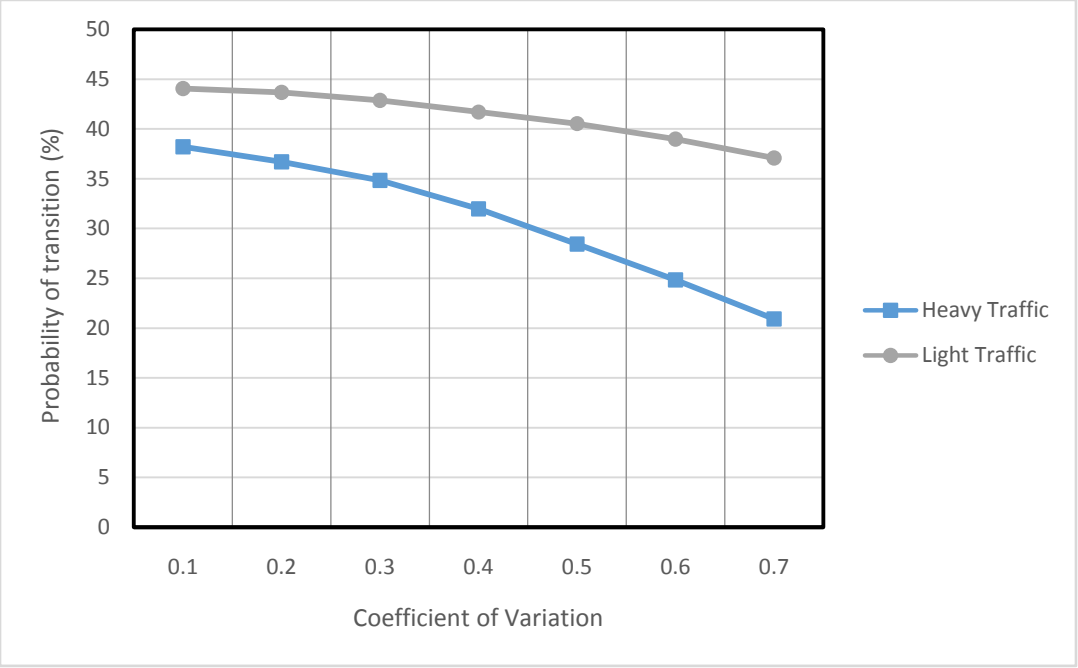


Figure 4.8 Effect of Coefficient of Variation on probability of transition.

4.2.5 Effect of variation of the coefficient of variation

The coefficients of variation (COV) for total number of estimated future vehicle during the design life were checked against probability of transition. It shows that as the coefficient of variation (COV) is increasing, the probability of transition is decreasing as shown in Figure 4.8.

CHAPTER FIVE

SUMMARY, CONCLUSION AND RECOMMENDATION

This chapter briefly summarizes the research process and findings. Then, major conclusions are presented. Recommendations for future research are also included in this chapter.

5.1 Summary

The purpose of this research is to develop a comprehensive framework for modeling the deterioration process of pavements, in order to assist engineers and administrators in effectively managing pavements through better performance prediction. The transitional probability model established serves as inputs in pavement management system. The formula is based on the state of the art structural reliability method, wherein first order reliability method were used using the concept of genetic algorithm. Furthermore, the response of the damage as a result of repeated load is based on mechanistic pavement performance model that predicts pavement distress in terms of underlying pavement behavior

The numerical illustration showed the capability of model to handle variables with various distribution types. The effect of truck traffic, coefficient of variation of input parameter and pavement structure were demonstrated. Since it is assumed that the traffic growth increases yearly, the assumption of a time-invariant matrix of transition probability of traditional method is no longer true with the proposed model.

5.2 Conclusion

From the study the following conclusion are observed;

1) Multilayer elastic computer program is an effective way of computing pavement responses due to changes in traffic and material characterization over time to predict pavement performance
2) The calculated number of load application to cause fatigue failure is less as compare to rutting failure under the same traffic and environmental condition
3) The cumulative damage increase with time and with higher axle load or higher truck percentage of heavy trucks
4) The probability of transition changes over time when traffic growth is considered which is in contrary to the constant Markovian transitional probability matrix used in many pavement management system (PMS).
5) The accurate prediction of transitional probability matrix should be based on system distress not based on the component distress

5.3 Recommendation

The proposed methods capture the stochastic and dynamic nature of pavement deterioration and have the ability to better predict the pavement performance compared with other methods, but some limitations still exist and should be further researched.

Key recommendations for future research are discussed as follows;

1) The model can be extended to other modes of distress, differing only in the exact details of the mechanistic model representation.
2) Poor construction method and poor drainage system can also be incorporated into the model and investigated.

3) Other artificial intelligent method like Neural Network can be a useful aid for comprehensive prediction of pavement performance.

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