JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Evaluation of Flexible Pavement Performance under Traffic Load Using Mechanistic-Empirical Design Approach.

A Research Submitted to Jimma University, Jimma Institute of Technology, School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Highway Engineering).

By:

Wubetu Yenegeta

March, 2020

Jimma, Ethiopia

JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Evaluation of Flexible Pavement Performance under Traffic Load Using Mechanistic-Empirical Design Approach.

A Research Submitted to Jimma University, Jimma Institute of Technology, School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Highway Engineering).

By:

Wubetu Yenegeta

Advisor: - Eng. Elmer C. Agon (Asso. Prof.)

Co-Advisor: Eng. Salem H/Kristos(MSc)

March, 2020

Jimma, Ethiopia

Acknowledgment

First, Praise and Glory to Almighty **GOD** for giving me strength, health and power. Next, I would like to express my sincere thanks and appreciation to my main advisor **Eng. Elmer C. Agon (Asso. Prof.)** and co-advisor **Eng. Salem H/Kristos (MSc.),** for their effort and commitment in sharing their knowledge towards me.

Secondly, my deepest gratitude goes to Jimma Institute of Technology, especially Department of Civil and Environmental Engineering and Highway Engineering Chair for enabling me to do this work and creating this beautiful environment for education.

Declaration

I, the undersigned, declare that this research entitled "**Evaluation of Flexible Pavement Performance Under Traffic Load Using Mechanistic-Empirical Design Approach**." is my original work, and has not been presented for an award of a degree in this or any other University, and all sources of material, if used, have been duly acknowledged.

Wubetu Yenegeta ______________ ______________

Candidate Signature Date

As an Advisor, we have read and evaluated this MSc thesis prepared, under our guidance, by Wubetu Yenegeta entitled: "**Evaluation of Flexible Pavement Performance under Traffic Load Using Mechanistic-Empirical Design Approach**." Therefore, I hereby recommend that it can be submitted as MSc thesis, fulfilling the requirements.

Eng. Elmer C. Agon (Asso. Prof.) ______________ _____________

Main advisor Signature Date

Eng. Salem H/Kristos (MSc.) ______________ ______________

Co- advisor Signature Date

JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Evaluation of Flexible Pavement Performance under Traffic Load Using Mechanistic-Empirical Design Approach.

By: Wubetu Yenegeta

APPROVED BY BOARD OF EXAMINERS

Prof.Dr.-Ing. Alemayehu Gebissa	______________	___________
External Examiner	Signature	Date
Engr. Bushirelkerim Omer (Msc)	______________	___________
Internal Examiner	Signature	Date
Engr. Abel Tesfaye (Msc.)	______________	___________
Chair Person	Signature	Date
Engr. Elmer C. Agon, (Asso. Proff.)	______________	___________
Advisor	Signature	Date
Eng. Salem H/Kristos (MSc.)	______________	___________
Co - Advisor	Signature	Date

Abstract

The development of countries all over the world is dependent to a large extent on the quality of its road transportation system. Particularly, in Ethiopia road is extensively used infrastructure for access to rural and urban areas. Because of this, a huge amount of money is being invested in road sector development. However, conditions of some roads are largely deteriorated and in poor condition. Empirical design method, which is used in Ethiopia now days, could not provide immediate evaluation of the ever increasing traffic in volume, load magnitude and axle configuration. It has been proposed that mechanistic design method offers this advantage over empirical one. However, it is a realization that Ethiopia is left behind to use Mechanistic-Empirical design approach. Therefore, it is obvious from the above that the need of this approach, despite its problems, is unquestionable.

The aim of this research was to evaluate flexible pavement performance under traffic load and development of a new framework for flexible pavement design with Mechanistic-Empirical Method. To access the effects of physical factors such as vehicular load, material properties and environment on pavement responses, phenomena such as stress, strain and deflection, and performance was one of the aims of the paper. At the meantime ERA design method was reviewed. A pavement structure designed using ERA manual is analyzed and evaluated using Mechanistic-Empirical approach. In this study KENLAYER, a Mechanistic-Empirical flexible pavement ML program has been used and ABAQUS, a 3D FEM program, has also used for load configurations that could not be modeled in KENLAYER.

In this research, an effort was made to review various research results and use them in the structural design method development to flexible pavement design which is mechanistic Empirical. Austroads criteria and shell 95% distress models have been used in this research for fatigue cracking and rutting respectively. According to the analysis, Allowable repetition of standard axle loads for rutting increases by 113% over the range of 1500-3500mpa AC modulus. However, allowable repetition of standard axle loads for fatigue cracking increases only by 64% over the same range of AC modulus. Only 12.69 % increase in allowable repetition of ESA for poisson's ratio in a range of 0.15-0.4 recorded.

According to this research, among all catalogue structures analyzed here, 54% performs below the design ESA of which 80% are due to fatigue cracking. This implies that Allowable Load Repetitions and in turn performance period predicted by KENLAYER is less from the number of standard design Load Repetitions expected by the ERA method. Therefore, it is necessary for Ethiopia to adapt mechanistic empirical design approach. Accordingly, an effort has been made to develop a new framework for design of flexible pavements. Finally, A new flexible pavement design tool was developed which is mechanistic Empirical based. The basis of the method is the KENLAYER computer program for the linear elastic stress analysis of the structure. The developed method treats the pavement structure as system of pavement layers and designing it accordingly.

Keywords: KENLAYER, ABAQUS, Mechanistic-Empirical, Finite Element Method, Rutting, Fatigue Cracking, Pavement performance.

Contents

List of Tables

List of Figures

ACRONYMS

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway Officials
AC	Asphalt concrete
BISAR	Bitumen structures analysis in roads
BRRC	Belgian Road Research Center
CBR	California bearing ratio
ELF	Equivalent load factor
ERA	Ethiopian Roads authority
ESAL	Equivalent Standard Axle Loads
FEM	Finite Element Method
HDM	Highway Design Manual
HMA	Hot mix Asphalt
M-E	Mechanistic-Empirical
ML	Multi-Layer
MLE	Multi-Layer Elastic
MR	Resilience Modulus
ORN	Overseas Road Notes
SAMDM	South Africa Mechanistic Design Method
TRRL	Transportation and Road Research Laboratory
VBA	Visual Basic for Application

CHAPTER ONE

INTRODUCTION

1.1 Background of the Study

The development of countries all over the world is dependent to a large extent on the quality of its road transportation system. Particularly in Ethiopia, road is the extensively used infrastructure for access to rural and urban areas and plays crucial role for economic growth. Due consideration of this, a huge amount of money has been invested in road sector development and the road network is improved in quality and size. But in the face of this huge investment in the sector, conditions of some roads are largely deteriorated and in poor condition yet and many of them have become sources of economic. Many researches have been undertaken on causes of pavement failure and causes are attributed to traffic load [**1**] and improper design of pavements. To alleviate from this problem several professionals have conducted a research on pavement design and construction and associated issues in their particular area and developed design methods which involve immediate evaluation of the ever increasing traffic in volume, load magnitude and axle configuration.

As technology advances, researches in the area are ever increasing and many agencies have been prompted to move toward development and use of new design methods. Empirical design method at one extreme, mechanistic-empirical design method at the middle and Mechanistic-Empirical design at the other extreme are the developed deign methods evolved in past era of pavement technology. Now a day, because of technology advancement and for reasons of the aforementioned advantages and many others, the trend worldwide is to use the Mechanistic-Empirical design approach [**1**]. The newly developed design methods i.e. mechanistic-empirical design have also many other advantages than the previously used empirical methods [**2**].

In mechanistic design, there are a number of different types of models available today of which the multi layered elastic model and the finite elements model (FEM) are models typically used. Both of these models can easily be run on personal computers and only require data that can be realistically obtained [**3**]. There are various types of computer programs available worldwide for mechanistic analysis of pavements. ABAQUS, a 3-D FE program [**4**] , has been used successfully in structural analysis of pavements. KENLAYER, a package for flexible pavement in KENPAVE, is another ML program developed by Haung Yang H. [5]

Elastic/Resilience modulus and Poisson's ratio are frequently used design parameters for Mechanistic-Empirical design. For preparation of design inputs in the Mechanistic-Empirical design and analysis, material properties are characterized by laboratory testing. However, laboratory tests for characterization of pavement materials are expensive and time consuming. For this reason, researches are developing predicting models in different environmental conditions [6]. Therefore, for this study input data were collected from predicting models and standard technical material specifications. In pavement design, Traffic is characterized as fixed traffic, fixed vehicle or variable traffic and vehicle [5], where each characterization is used differently. The first is rarely used. Many of the design methods [5] and manuals in use today including ERA design manual uses the second approach [7,8]. The latter, i.e. currently referred to as axle load spectrum, is best suited for Mechanistic-Empirical methods of design.

Another important thing frequently used in pavement performance analysis is failure criteria sometimes called transfer functions, an equation that is used to predict pavement life in terms of a number of repetitions (or loading cycles) to failure, designated as N_f[9]. The most common transfer functions relate pavement responses to either structural rutting, as opposed to surface rutting, and fatigue cracking. The asphalt fatigue criterion developed in Australia is recommended for use in Ethiopia [8]. The subgrade strain criteria, transfer function for the number of load cycles to failure in structural rutting (permanent deformation), is related to the vertical strain at the top of the subgrade layer (ε_v), the constant M_R. For the specific definition of failure, Different agencies have developed their own models some of which do contain M_R, and the rest contain εv only. However, the precision with which subgrade criteria are known is poor and the range of the published criteria is very wide indeed. Selecting the most appropriate is essentially a matter of engineering judgment [8].

The aim of this research was to evaluate flexible pavement performance under traffic load and development of a new framework for flexible pavement design with Mechanistic-Empirical method. At the meantime ERA design method is reviewed and a pavement structure which was designed using this manual is analyzed using mechanistic-empirical approach.

Effects of physical factors (such as vehicular load, material properties and environment) on pavement responses (phenomena such as stress, strain and deflection) and performance has been studied. Finally, economical pavement layer composition has been identified for predetermined repetition of axle loads (no of cycles or target design life) and a new framework for design of flexible pavements has been devised.

For this study secondary data from standard technical specifications, predicting models and engineering judgments, were used as an input to software analysis and in the development of the new design framework. Traffic and temperature data were collected for Jimma area. For achievement of the objectives of this study KENLAYER, a Mechanistic-Empirical flexible pavement ML program, has been used. ABAQUS, a 3D FEM program, was also used for comparison with KENLAYER and for load configurations that could not be modeled in KENLAYER.

1.2 Statement of the Problem

The development of countries all over the world is dependent to a large extent on the quality of its road transportation system. Particularly in Ethiopia, Road is the extensively used infrastructure for access to rural and urban areas and plays crucial role for economic growth. Due consideration of this, a huge amount of money has been invested in road sector development and the road network is improved in quality and size. But in the face of this huge investment in the sector, conditions of some roads are largely deteriorated and in poor condition yet and many of them have become hazardous and sources of economic drain in terms of high road users cost, loss of lives and property. This is because of that roads have been heavily trafficked by freights and public busses and many other factors. This traffic is ever changing in volume, load magnitude and axle configuration. Immediate evaluation of changes in this vehicle loading and inclusion in pavement design guidelines is difficult in empirical design method. However, it has been proposed that mechanistic design method offers these advantages [**2**].

Empirical design uses design catalogue and local experience and works only for standard materials and pavement structures. Mechanistic (analytical) design avoids this shortcoming and supports engineering judgment in the design process, allows for comparative analysis of technical similar variants and for parameter studies to better understand material/structural performance[**10**]. Now a day, for reasons of the aforementioned advantages and many others, the trend worldwide is to use the Mechanistic-Empirical design approach [**1**]. Similarly advances in technology have prompted many agencies to move toward development and use of mechanistic-empirical design methods. However, Ethiopia is sticking to purely empirical method of design with catalogue of structures despite the inclusion of the concept in latest edition of the design manual. In order not to be left out, Ethiopia should adopt its own mechanistic-empirical design procedures so that pavement design procedures are more in line with other advanced countries.

The motivation behind of this research is based on the facts that ERA has included mechanistic design approach in the design manual at least as method of providing confidence in the extended (relatively high traffic levels) design of flexible pavements which were not included in 2002 edition of the manual. Empirical evidence for pavement designs that involve relatively high traffic levels is much less than for the structures of low and intermediate levels of traffic [**7**,**8**]. For such designs, theoretical (or mechanistic) analysis techniques have to be referred [**8**]. Despite the many problems that mechanistic design method has, it has been discussed in appendix H of ERA 2013 pavement design manual and recommended for high traffic levels. As can be concluded from the above statements, the previous manual was purely empirical and mechanistic basis of design has been included in 2013 edition at least for comparison purpose. This is an indication a need for transition in design methods. Secondly, and just as importantly, an article by Research and technology for national asphalt pavement association on mechanistic pavement design which states: mechanistic design methods offer many advantages over the empirical method such as the following [**2**]:

i. It allows an evaluation of changes in vehicle loading on pavement performance;
ii. New materials can be evaluated through their design properties;
iii. The impact of variability in construction can be assessed;
iv. Actual engineering properties are assigned to the materials used in the pavement;
v. Pavement responses related to actual modes of pavement failure are evaluated;
vi. Databases of materials used as input in pavement design can be developed and updated as information becomes available.

However, it is a realization that Ethiopia is left behind to apply Mechanistic-Empirical design approach. Therefore, it is obvious from the above that the need of this approach, despite its problems, is unquestionable.

1.3 Research questions

For the purpose of this study, the following questions were addressed:

1. What is the performance of empirically designed Pavement structures in ERA design manual when evaluated using Mechanistic-Empirical design method?
2. What is the effect of physical factors on response and performance of flexible pavement?
3. What is the sensitivity of rutting and fatigue to changes in thickness and material property of pavement layers?
4. What is the economical pavement layer composition of a flexible pavement under traffic load in Mechanistic-Empirical design approach?

1.4 Objectives of the study

1.4.1 General objective

The general objective of this study was to evaluate flexible pavement performance and develop a new approach for flexible pavement design using Mechanistic-Empirical method.

1.4.2 Specific objectives

1. To evaluate the performance of pavement structures designed as per ERA's Empirical guideline using Mechanistic-Empirical Design Approach
2. To determine the effect of physical factors on flexible pavement response and performance
3. To study sensitivity of rutting and fatigue to changes in thickness and material property of pavement layers
4. To determine economical pavement layer compositions for a target design life (repetition of standard axle loads) using Mechanistic-Empirical method

1.5 Significance of the Study

This study tried to introduce new framework for flexible pavement designs with mechanistic-empirical method. This enables designers to accurately represent effects of physical factors such as vehicular load, material properties and environment on pavement responses and performance. Therefore, designers will be benefited from the research and in turn transport departments. This is because, in Mechanistic-Empirical design physical factors are represented accurately during design and this enables the pavement to serve long and hence reduction in maintenance coast.

The research shows advantage of mechanistic-empirical design over empirical and therefore will initiate the adaption of this design methodology in design manuals and curriculums of academic institutions. At the meantime adaption of design tools (software's) such as KENLAYER, a MLE program, and other finite element programs is required. While adapting the methodology, it is must to have material properties, traffic and environmental data. Therefore, the study clearly emphasizes the necessity of data base of material properties, traffic and environmental data for Ethiopian condition. This research could also pledge further research and in adaption of this design methodology.

1.6 Scope and limitation of the study

Basically, this research focuses on evaluation of flexible pavement performance under traffic load and development of a new framework for flexible pavement designs with Mechanistic-Empirical method. At the meantime ERA design method is reviewed and a pavement structure designed using this manual is analyzed using Mechanistic-Empirical approach. Effects of physical factors (such as vehicular load, material properties and environment (i.e. Temperature)) on pavement responses (phenomena such as stress, strain and deflection) and performance has been studied. Finally, economical pavement layer compositions were determined for predetermined repetition of axle loads (number of cycles or target design life) and a new design method is devised. In this research, linear elastic theory has been used.

Main limitation of this study is, other distress models except rutting and fatigue are not considered for performance evaluation of flexible pavements and development of the design. However, this will not affect this research significantly because rutting and fatigue cracking, which are the major distresses in the structural design of asphalt pavements, are considered. Another limitation was in availability of local axle load configuration data.

CHAPTER TWO

RELATED LITERATURE REVIEW

2.1 Introduction

Several professionals have conducted a research on pavement design and construction and associated issues in their particular area and developed design methods. As technology Advances researches in the area are ever increasing and many agencies have been prompted to move toward development and use of new design methods. Empirical design method at one extreme, mechanistic-empirical design method at the middle and mechanistic design at the other extreme are the developed deign methods evolved in past era of pavement technology. These are now adapted by different countries and agencies in their design manuals and standard technical specifications. Reviewing such literatures is important to compare theories and adapt to our particular case in the way which suits the condition. Among many others, the following are some of the issues studied by researchers:

- Material characterization and predicting models
- Physical phenomena (loading and environment)
- Transfer functions/Distress models
- Pavement response and predicting models
- Pavement damage analysis etc…

This all are reviewed in subheadings as follows.

2.2 Review of Flexible Pavement Design Methods

2.2.1 General

According to literatures, there are two well know pavement design approaches. These are empirical, which is the mostly used approach and mechanistic approach. However, no pure mechanistic approach is developed yet [**11**]. Now a day the mechanistic-empirical approach is used by many countries and state agencies.

Empirical method of design procedures is based on experience or observation alone. They disregard system behavior or pavement theory or mechanics of materials and there is no way of relation of design inputs to outputs/responses such as strain and stress. However, for a given geographic location and climatic condition, there is an empirically derived relationship between performance, load and pavement thickness. This is the basis of the empirical design approach [**1**].

On the other hand, the analytical or mechanistic design approach is based on mechanics of materials that relates physical factors/input parameters to outputs/responses such as strain and stress [1]. this approach involves characterization of materials for input for theoretical models, calculation of parameters that have primary influence on selected aspects of pavement performance and then to use this values in performance or distress models for evaluation of structural adequacy of the pavement under consideration [**12**].

2.2.2 Empirical design Approaches

a. ERA Design manual

The first standardized Road design manual of ERA 2002 has been updated according to the prevailing conditions in Ethiopia and experiences gained during the last 10 years till the publication of the current 2013 manual. This manual gives recommendation for the structural design of flexible pavements in Ethiopia. The preparation of this pavement design manual is based on a review of design standards of several countries such as TRRL's ORNs (ORN31, ORN19, ORN3 and others), AASHTO design of pavement structures as revised in 1993, asphalt institute and South African Pavement Design manuals. In this manual pavement design charts have been reorganized into four groups (A, B, C and D) according to surface types and pavement structures [**7**,**8**]. Appendix H is a new concept introduce in the later design manual

i. Theoretical and Empirical Basis of design

The empirical basis of the manual is results of full-scale experiments and studies of the performance of as-built existing road networks. However, empirical evidence for pavement designs that involve relatively high traffic levels is much less than for the structures of low and intermediate levels of traffic. For such designs, theoretical (or mechanistic) analysis techniques have to be referred [**8**].

ii. Design principles

Road Pavements are designed mainly to limit stress and deformations at subgrade level due to traffic travelling on the pavement surface. The principle of design here is to limit distresses which affects the riding quality of the road to acceptable levels during the design period considered. The design method assumes that routine and periodic maintenance are performed during the design period and at the end strengthening overlay is required.

The following are structural requirements considered as principal by the manual [**7**,**8**]

1) The subgrade should not deform excessively
2) Bituminous materials and cement-bound materials used in roadbase design should not crack under the influence of traffic
3) The roadbase must be able to sustain the stress and strain generated within itself without excessive or rapid deterioration of any kind.
4) Internal deformation of bituminous materials must be limited.
5) The load spreading ability of granular sub-base and capping layers must be adequate to provide a satisfactory construction platform.

Requirement 3&5 are controlled by simple trial an error tests such the CBR and index tests and standard technical material specifications formulated from experience while requirement 4 is controlled mainly by marshal stability test and others such as bituminous content tests, air voids and aggregate quality tests etc...

The 1st requirement is controlled by limiting the vertical compressive stress or strain at this level and the 2nd is controlled by the horizontal tensile stress or strain at the bottom of the bound layer. This is illustrated in Figure 2.1 below

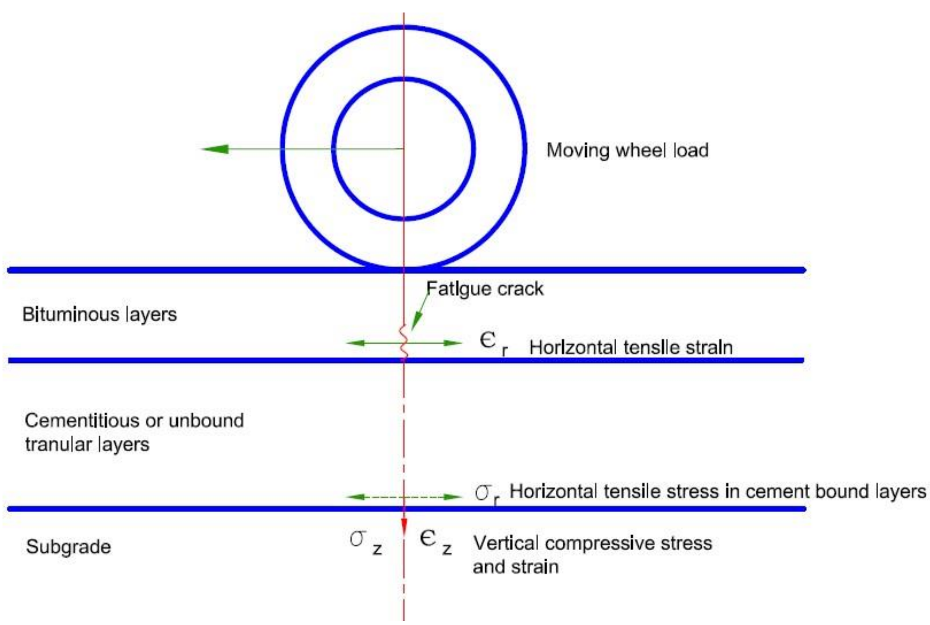


Figure 2. 1 Location of Critical stresses and strains [**8**]

However, the Design manual has not provided a means of controlling distress resulted from compressive stress or strain at top of subgrade and horizontal tensile stress or strain at the bottom of the bound layer in the design procedures.

iii. Traffic

For paved roads, traffic is defined in terms of equivalent standard axles (ESAs). To determine ESA both traffic volume count and axle load survey are required. Design period is the important factor to be considered while forecasting ESA. For structural design of pavements, it is the one directional cumulative traffic of each vehicle class which is considered. For given vehicle class 'm'

$$T(m) = 0.5 \times 365 \times AADT(m1) * \frac{[(1+i)^N - 1]}{i} \qquad (2\text{-}1)$$

T (m) = the cumulative traffic of traffic class m

AADT (m1) = The AADT of traffic class m in the first year

N = the design period in years

i = the annual growth rate of traffic in percent

The cumulative traffic volume of each vehicle class is multiplied by the corresponding average number of equivalent standard axles (or sometimes called equivalent standard axle load factor (ESALF)) and then added together to calculate the total cumulative ESA. Number of equivalent standard axles (ESAs) is the damaging power of a particular vehicle (EF) when compared with a standard axle load which is 80KN according to this manual.

The number of equivalent standard axles (EF) of an axle is related to the axle load as follows:

$$EF = \left(\frac{L}{8160}\right)^n \quad \text{(for loads in kg) or} \qquad (2\text{-}2)$$

$$EF = \left(\frac{L}{80}\right)^n \quad \text{(for loads in KN)} \qquad (2\text{-}3)$$

Where:

EF = number of equivalent standard axles (ESAs)

L = axle load (in kg or kN)

n = damage exponent (n = 4.5and n=4 for low volume roads).

The sum of individual EF of axle loads of a vehicle gives the equivalency factor for a vehicle as a whole. A likely average of equivalent load factor for vehicles is provided in ERA design manual derived from historical data in Table 2.6 of this manual. Finally, the design live of pavements is expressed in terms of ESA they are designed for to carry.

In this manual, for vehicles with multiple axles such as tandem and tridem, each axle in the multiple set is treated separately however the exact ESA is different from this. Similarly, average ESA of a vehicle class is used rather than ESA for average axle load (incorrect and leads to large errors according to the manual).

In any of the ERA design manuals there is no way to consider for spectrum of axle loads

iv. Subgrade

In this manual, the strength of subgrade for flexible pavement is assed in terms of CBR which in turn depends on type of soil, its density, and its moisture content.

v. Limitations

Traffic load characterization

For vehicles with multiple axles such as tandem and tridem, each axle in the multiple set is treated separately, which may be very conservative. In any of the ERA design manuals there is no way to consider for spectrum of axle loads. In contrast to AASHTO, average ESA of a vehicle class is used rather than ESA for average axle load.

Distress control mechanism

There is no way of protecting and controlling the individual layers of pavement system against excessive deformation and shear. However, material specifications are specified in ERA standard technical specification and in the design manual. For example, grading and CBR requirements for unbound layers and subgrade to achieve a specific strength. For bituminous bound layers, asphalt content and air void are specified.

The Design manual has not provided a means of controlling distress resulted from compressive stress or strain at top of subgrade and horizontal tensile stress or strain at the bottom of the bound layer.

b. AASHTO (1993)

i. Background

Based on the performance data from the AASHO Road Test, AASHTO, 1993 design method was developed. This version was preceded by several other versions. This is the last empirical version followed by the proposed AASHTO mechanistic-empirical design method. Although this is not the latest, it is used by several states in US and countries abroad out of US [**5**,**11**,**13**].

The main requirement in AASHTO design is to determine thickness of pavement layers to satisfy the following design objectives [**11**].

a. Maximum economy, safety, and serviceability over the design period
b. Maximum or adequate load-carrying capacity in terms of load magnitude and repetitions
c. Minimum or limited deteriorations over the design period

ii. Design procedures

First of all, equivalent axle load factor is calculated using the following equation (2-4).

$$\text{EALF}=\left(\frac{Wtx}{Wt18}\right) \tag{2-4}$$

Where $\left(\frac{Wtx}{Wt18}\right)$ is expressed by the following equation:

$$\text{Log}_{10}\left(\frac{Wtx}{Wt18}\right) = 4.79*\log_{10}(18+1) - 4.79*\log_{10}(L_x+L_2) + 4.33*\log_{10}L_2 + \left(\frac{Gt}{bx}\right) - \left(\frac{Gt}{b18}\right) \tag{2-5}$$

$$\text{Gt}=\log10\left[\frac{4.2-Pt}{4.2-1.5}\right] \text{ and} \tag{2-6}$$

$$\beta_x=0.40+\frac{0.081(Lx+L2)^{3.23}}{(SN+1)^{5.19}*(L2)^{3.23}} \tag{2-7}$$

Lx = load on one single tire or one tandem-axle set (kips)

L2 = axle code (1 for single axle, and 2 for tandem axle)

SN = structural number

p_t = the terminal serviceability

β_{18} = value of βx when Lx is equal to 18 and L2 is equal to 1

w_{tx} = total applied load from a given traffic

w_{t18} = total applied load from an 18-kip single axle

After having EALF and repetition of each axle load group Ni, initial or opening ESALo is given by:

$$\text{ESALo} = \sum_{i=1}^{m} (\text{Ni} * \text{EALFi}) \tag{2-8}$$

Where m is the no of axle load groups

Then, the cumulative ESAL during the designed life of the pavement, W_{18}, is calculated as:

$$W_{18} = \sum_{i=1}^{n} \text{ESAL} = \frac{\text{ESALo} * 365}{\text{loge}(1+\text{i})} [(1 + \text{i})^{\text{n}} - 1] \tag{2-9}$$

Alternatively, W_{18} is given by

$$\text{log}W_{18} = \text{ZR} * \text{So} + 9.36 * \text{LOG}(\text{SN} + 1) - 0.2 + \frac{\text{LOG}\left(\frac{\Delta \text{PSI}}{2.7}\right)}{0.4 + \left(\frac{1094}{(\text{SN}+1)^{5.19}}\right)} + 2.32 * \text{LOG}(\text{MR}) - 8.07 \tag{2-10}$$

W_{18} Cumulative expected 18-kip ESAL during the designed life in the design lane

Z_R Normal deviate for a given reliability R

So Standard deviation

M_R Roadbed soil resilient modulus

SN Pavement structural number

SN1, SN2, SN3.etc are calculated using equation (2-10). First SN3 is calculated using MR value of subgrade. To obtain SN2 and SN3, MR Value of subbase and base is used respectively

Where:

SN1 structural number required above base

SN2 structural number required above subbase

SN3 structural number required above subgrade

To calculate the required thickness of each pavement layer the following equations are used

$$\text{SN} = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \tag{2-11}$$

Where:

a_1, a_2 and a_3 are Structural layer coefficients

D_1, D_2 and D_3 are Thicknesses of surface, base and subbase, respectively.

m_2, m_3 are Drainage coefficients

Equivalently

$$SN1 \leq a_1D_1 \quad (2\text{-}12)$$

$$SN2 \leq a_1D_1 + a_2D_2m_2 \quad (2\text{-}13)$$

$$SN3 \leq a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \quad (2\text{-}14)$$

2.2.3 Mechanistic-Empirical design approaches

a. Asphalt institute

Before the 8th edition (1970) of the manual it was empirical in nature. It is after the 9th edition in 1981 has become mechanistic. After then, Asphalt institute starts to apply a mechanistic-empirical methodology and a mechanistic multilayer theory [**5**]. This analytical method of design uses a multilayer elastic program DAMA. This program contains parameters that should be included in pavement design and deemed appropriate such traffic, environment and material properties. It also uses a laboratory distress models/transfer functions such as fatigue cracking model which are calibrated to actual field conditions after AASHTO road tests. The asphalt institute design manual contains five design charts developed using DAMA results for climatic regions of United States [**5**,**14**].

This manual uses two criteria for design of pavements. one is a critical horizontal strain at the bottom of asphalt layer which causes fatigue cracking and vertical compressive strain at the top of subgrade which is responsible for formation of rutting of the pavement [**5**].

b. South Africa

Background:-It has been reported that South Africa has a long history of using mechanistic approach design since 1970's [**1**,**14**,**15**]. Therefore, it seems that South Africa is among the 1st countries to use ME approach.

Material characterization: - this including layer thickness and elastic material properties. The manual uses typical available database of moduli and other properties where there are no laboratory or field measurements. Poisson's ratio is assumed to be 0.44 and 0.35 for asphalt granular layers respectively [**14**].

Structural analysis:-In this manual linear elastic, static analysis of multilayer system is used and this results in pavement responses such stress and strain at critical locations in the pavement system. For this purpose, linear elastic multilayer analysis program is used. The standard design load for SAMDM is 40kN dual wheel with 350cm spacing and 520 kpa uniform contact pressure [**15**].

Pavement life prediction: -The design life to strengthening is selected on economic grounds. The cumulative traffic in terms of 80 kN equivalent standard axle load is estimated using the fourth power law. Pavement responses results from the structural analysis are further used as an input to the transfer functions evolved in 1977,1978 and 1995 to predict the service life [**14**,**15**].

c. United Kingdom

The design method used in UK is analytical based. It uses a linear multilayer elastic model. Unlike the AI method critical horizontal strain is located at the bottom of bituminous base course while critical compressive strain is located on top of subgrade like any other ME design methods. Critical strains are used to predicted service life of the pavement in terms of standard cumulative number of 80kN standard axle load. The design method uses a single wheel load of 40 kN for determination of critical strains [**14**,**16**]. This method can be used to design roads that can carry up to 200 MSA.

d. Australia

This manual has 1st published by national association of Australian state road authorities (NAASRA) and in 1987. Later on by Austroads in 1992, 1997, 2004 and the current version is 2017. It evolves through time by revising and adding new procedures of design traffic calculations, materials characterization, and thickness design procedures [**17**,**18**,**19**].

It follows an analytical approach by using the linear elastic program, CIRCLY. The software uses a dual wheel load with 330cm spacing and 550-700 kpa (typically 550 kpa) uniform contact pressure which was later revised to be in a range of 500-1000 kpa (typically 750 kpa) in 1997 [**17**]. Materials are characterized by laboratory and from existing literatures. For example, The Shell nomogram can be used to estimate asphalt stiffness modulus. Granular materials are considered to no linear or stress dependent [**17**,**18**,**19**].

The Shell asphalt fatigue criteria after a little modification (reliability and shift factor) has been used to establish design charts [**17**,**18**,**19**,**20**].

2.3 Factors Affecting Flexible Pavement Design

2.3.1 Traffic Loading

a. Types of traffic loading

Many types of traffic use the highway. However, the destructive effect comes from trucks because other vehicles such as passenger cars significantly lighter. Again trucks also consist of verity of loads and axles. Loads and dimensions of trucks are limited by road agencies. For example, In Ethiopia, it is dimension and loads of any vehicle are limited by council of ministers by its proclamation as shown in Table 2.1 and 2.2 below [**21**].

Table 2. 1Vehicle size limits

No	Description	Limit(Tons)
1	Total **width** of any vehicle including load thereon	2.5
2	Total **Height** of any vehicle including load thereon	4.2
3	Total **Length** of any **single** vehicle	12
4	Total **Length** of any **combination [Truck Trailer | semitrailer]** vehicle	17
5	Total **Length** of any **combination [Truck with drawbar Trailer]** vehicle	18
6	Combination of Motor Vehicles	2 units

Adapted from Ethiopian council of minister's regulation No 11/1990 under publication of Negarit Gazeta [**21**]

Table 2. 2 Axle load limits

No	Description	Limit(tons)
1	Steering Axle | Single Axle Single Wheel	8
2	Single Axle dual Wheel	10
3	Tandem Axle dual Wheel , C/C spacing between axles <1.3m	17
4	Tridem or more axles, load per axle If C/C spacing between axles >1.3m Load per single Axle	10 each

Adapted from Ethiopian council of minister's regulation No 11/1990 under publication of Negarit Gazeta [**21**]

b. Measurement of traffic loads

In Ethiopia traffic is counted in 4 rounds per year and the data is stored as a database by road asset management directorate and in ERA website. The load on each and every axle of each and every vehicle can be determined using the help of weigh bridges. The axle configuration, the number of axle applications for each axle type and axle load group is not determined. A typical truck axle load configuration used by ERA can be found from ORN40.

c. ESA and Load equivalency

In empirical design process a mixed stream of vehicles is used by converting the different axles with different loads to an equivalent number of standard axles, for example 8160 kg (80kN). LEF, which is defined as ratio of damage caused by one pass of the load/axle to a single pass of a standard 18-kip single axle is used for converting the different axles to an equivalent number of standard axles [**22**].

In AASHTO design process, damage is considered as loose in present serviceability index (PSI). terminal serviceability index Pt and structural number SN are utilized to determine LEF using the regression equations 2.5, 2.6 and 2.7 developed based on AASHTO road test for flexible pavements [**13**].

In Ethiopia Load equivalency factor (LEF) or number of equivalent standard axles (EF) is determined using 4^{th} power rule. The number of equivalent standard axles (EF) of an axle is related to the axle load using equation (2-2) or (2-3) [**7**,**8**]:

Alternatively, LEF can be determined using the damage concept, note that the ratio of damage by a single pass of the axle in question to a standard axle is the LEF. If Na passes of an axle causes a pavement to fail as Ns passes of standard axle load do [**22**]. Then

Damage due to axle in question=1/Na

Damage due to standard axle=1/Ns

Then According to definition of LEF, LEF= (1/Na)/ (1/Ns) = Ns/Na

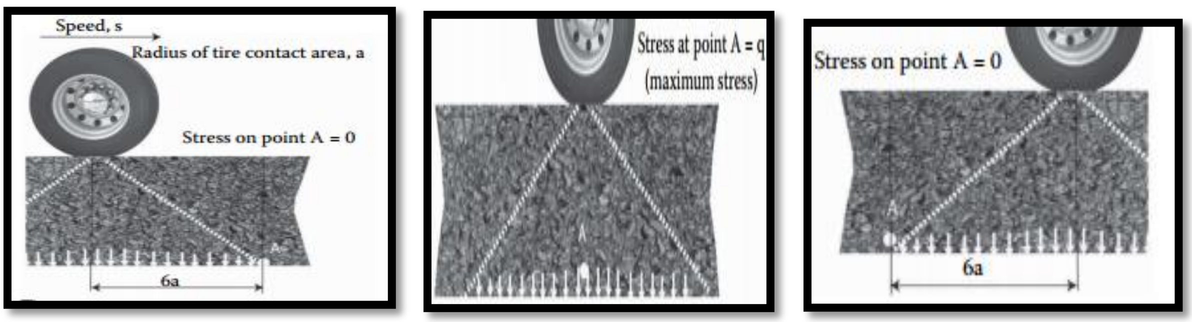
Tire pressure Can range from 90 psi (621 kpa) to 120 psi (828 kpa), with loads ranging from 3600 lb. (16kN) to 6600 lb. (30 KN)

d. Speed

In asphalt mixes the effect of loading time varies with speed and depth. Points in a pavement layer have different loading time at the same speed due to a depth difference. Points at depth have longer loading time than points near to surface [**22**].

For an asphalt mix layer loading time is determined by one of the following ways [**22**].

1. assume that at the middle of the asphalt layer, the load is uniformly distributed over a circular area with a radius, r, as shown in Figure 2.2 below



Time t_1 Time t_2 Time t_3

Figure 2. 2 Stress at appoint due to moving load [**22**].

$$r = a + \frac{h}{2} \tag{2-15}$$

a = the radius of the tire contact area

h = thickness of the asphalt layer and the loading time t is expressed as

$$t = \frac{2a+h}{v} \tag{2-16}$$

2. Assume that the moving load varies as a haversine function with time. In this case the pressure is considered to be zero when the load is at a distance greater than 6a from that point, where a is radius of contact area [**5**,**22**]

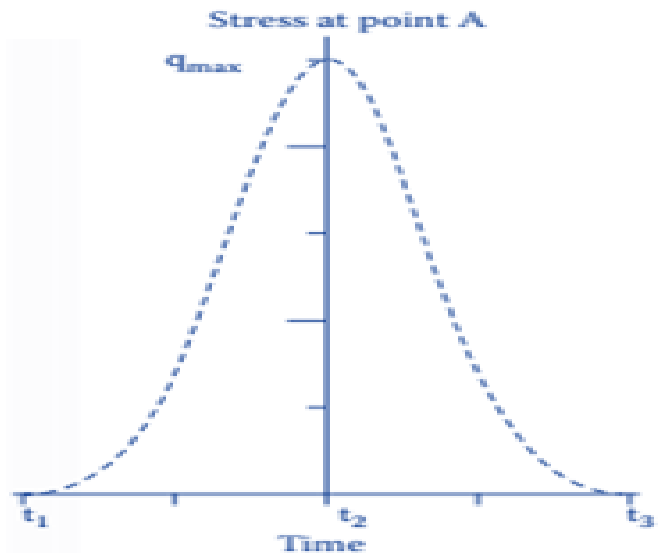


Figure 2. 3 Stress at appoint A [**22**]

Then the loading time is approximated by

$$t=\frac{12*a}{v} \tag{2-17}$$

Where v is speed

2.3.2 Response/Structural Models

a. General

Pavement response at a number of locations is used during pavement design and performance evaluation procedures [15]. If this is the case, response models are important factors in the design process.

b. Multilayer models

In1885 BOUSSINESQ marked the first step in the development of mechanistic design procedures. He has developed a mathematical equation used to determine stress and strains for one-layer system (semi-infinite half space) subjected to point load on the surface. Following this several other professionals has continued to the development of multilayer theory. A two-layer system by WESTERGAARD in 1938 and three-layer system by BURMISTER in 1945 has evolved [**5**,**14**]. The development of computers 1960's has contributed to the mark of multilayer systems greater than 5 layers [**14**].

c. Finite element models

Finite element method can be used for pavement responses to both static and dynamic loads. In contrary to multilayer models where by the pavement is modeled by layer of continuum layers, the construction of the model is divided into virtual parts i.e. finite elements connected by nodal points. Nearly all geometric conditions (such as layer thickness, local discontinuities and special distribution of material properties) and any material law (elastic, plastic linear and nonlinear and viscous) are studied. Discretization of the continuum pavement into finite element is time consuming and analytically costly in terms of memory consumption [**14**].

d. Comparison of the two models

According to reference--- this models are known as mechanistic models. Among the various types of mechanistic models available today, the multi layered elastic model and the finite elements model (FEM) are models typically used [**3**,**23**]. Both of these models can easily be run on personal computers and only require data that can be realistically obtained. In the mechanistic design analyses, flexible road pavements are represented most often by physical multi-layered half-space [**24**]. Even though, input parameters and process of modeling is a little bit different, outputs are similar in both models which are explained as follows [**3**].

- **Stress**. The intensity of internally distributed forces experienced within the pavement structure at various points. Stress has units of force per unit area (N/m^2, Pa or psi).
- **Strain**. The unit displacement due to stress, usually expressed as a ratio of the change in dimension to the original dimension (mm/mm or in/in). Since the strains in pavements are very small, they are normally expressed in terms of micro strain (10^{-6}).
- **Deflection**. The linear change in a dimension. Deflection is expressed in units of length (mm or µm or inches or mils).

There are many finite element programs to model flexible pavement structures starting from specialized to general purpose software's such as ANSYS and ABAQUS.

ABAQUS, a 3-D FE program [**4**], has been used successfully in structural analysis of pavement and used an elastic multilayer analysis program called BISAR for validation of static analysis results and highly correlated [**25**].

ABAQUS provides more realistic representation of flexible pavement system and has the capability to simulate actual pavement loading conditions [**25**]. However, because of its simplicity and speed, multilayer elastic analysis software's has been accepted in the industry and this can be used as justification for the relative difference between predicted and measured/actual response/ obtained during analysis. It has to be noted here that the remaining discussions has not given emphasis to ANSYS rather to ABAQUS and KENLAYER.

e. KENLAYER as a ML pavement Analysis Software

KENLAYER is a multilayer pavement analysis and Design software which is part of KENPAVE package. This software is developed by Huang, Y. H, the author of Pavement Analysis and Design book. Details of The software for ML pavement Analysis can be accessed from this book [**5**].

f. ABAQUS as a FE pavement analysis Software

ABAQUS is a general purpose commercial finite element program. It has also used widely for modeling of pavement analysis [**4**]. Chen et al [**26**] has compared ABAQUS with other 4 pavement analysis programs and has found comparable results. Zaghloul and White [**25**] has used ABAQUS 3D for simulation of the pavement responses under both static and dynamic loading. Validation of the static case has been made by an elastic multilayer program BISAR and for the latter case a test data under moving trucks made in Canada has been used.

ABAQUS has been used successfully for pavement analysis in axisymmetric (2D), planar (2D) and 3D. Although 2D models are easier to generate mesh and use less computational time and computer memory, they have inherent limitations. 3D models can be applied to model nearly the actual pavement structure and predict responses accurately in relative terms. However, they are computationally costly and complex to model. The accuracy of finite element analysis has been reported highly dependent to geometric properties and mesh refinement [**27**].

Kim M.[**27**], after modeling a 3-layer system using linear and nonlinear material properties, he has found insignificant difference for horizontal strain at bottom of AC when both cases i.e. case 1(linear base and subgrade) and case 2 (nonlinear base and subgrade) are compared. AC is taken as linear elastic and isotropic for both cases. In the same literature it has been reported that linearity of subgrade has significant effect on deflection response of the pavement and strain on top of subgrade but has in significant effect on other locations.

The same study [**27**], shows that the results of 2D and 3D analysis has not been differing significantly, which is a good new that confirms both models can be used for analysis purpose. However, it needs nonlinear material characterization.

2.3.3 Material Characterization

a. General

Mechanical properties of pavement materials are characterized by using laboratory testing. However, instrumentation and testing procedures of some mechanical properties requires huge investment and skilled manpower. Due to this limitation prediction models has been developed. Therefore, these prediction models are used where there are no reliable data and instrumentation for testing. There are various terms used to express these mechanical properties of pavement materials as defined below [**28**].

Stiffness: Stiffness is resistance of a material to stress, the rigidity of a material, or how it resists deformation if load is applied. It is defined as ratio of load to deformation, for example ratio of wheel load to pavement deformation.

Elastic modulus: It is the most common measure of stiffness and defined as the ratio of stress to induced strain .it assumes that properties of a material are linear, i.e. if stress is doubled then strain is doubled. However, this is not the usual case for pavement materials.

Stiffness modulus and resilient modulus: Pavement materials couldn't be characterized by a single elastic property i.e. elastic modulus (young's modulus) because of different factors. For example, asphalt is a viscos material and its modulus depends on temperature and time of loading and therefore best expressed by stiffness modulus which is defined as exactly the same as elastic modulus but, at a given temperature and loading rate[**28**]. This term is used by shell similarly as the term Dynamic modulus (which is defined as the ratio of stress and strain under dynamic condition) is used by AI and other agencies.

The term resilient modulus is devoted to soils and unbound granular materials. It is used to separate the component of behavior which is approximately elastic from the plastic (unrecoverable or not linear and stress dependent). It is defined as exactly the same as elastic modulus i.e. Ratio of stress to induced strain but at a given set of stress conditions. In other words, it can also have defined as the ratio of stress to recoverable strain due to cyclic loading condition [**28**].

Poisson's ratio, shear modulus and bulk modulus

Stress and strains are assumed in a single direction. However, the fact that the world is three dimensional cannot be ignored. The same is true for pavement layers. A quantity known as Poisson's ratio is used to deal with stress and strain effects in different directions. It controls the degree to which a material compresses under load. It has a value of 0.5 for incompressible materials. For saturated clays it values is in a range of 0.45-0.5. Due to a higher void content for granular materials it is in a range of 0.3-0.35 and values for asphalt tend to similar and in a range of 0.15-0.2 for hydraulically bound materials. The other quantities, shear modulus and bulk modulus, are related to modulus and Poisson's ratio [**28**]. It should be noted here that we will not deal with for this properties in more detail.

b. Characterization for Bituminous bound materials

i. Modulus

One of the basic and mostly used parameters used for characterization of pavement materials is the modulus. However, there are many moduli such as Young's, shear, bulk, complex, dynamic, double-punch, resilient, and Shell monograph moduli that are used in a specific condition. Mamlouk M.S. and Sarofim R.T. [**29**] have discussed these moduli. They also evaluated the suitability of each for a particular condition. At the meantime he has mentioned that the term modulus has been used loosely in literatures on pavements and "elastic stiffness" should be recommended. Similarly, In Austroads manual, resilience modulus is defined as "The ratio of stress to recoverable strain under repeated loading conditions; also referred to as elastic stiffness" [**18**].

On the same literature [**29**], it has been stated that Due to different test conditions the same material may result in different value of modulus. Therefore, to incorporate these values in mechanistic-empirical design procedures extreme care should be exercised. Finally, on their discussion, "it has been concluded that the resilient modulus is more appropriate for use in multilayer elastic programs than are other moduli because it represents the elastic stiffness of the material after many load repetitions" [**29**].

But later on it has been reported that both resilience modulus and dynamic modulus could be used as an input for multilayer elastic programs. For example, in Australia [**18**,**30**], resilient modulus is used and dynamic modulus is used in AASHTO's MEPDG.

There are various test procedures at various conditions for the same modulus and the same material and may result in different value. However, the same equipment can be used for resilient modulus and the dynamic modulus tests [**5**]. The difference between a resilient modulus test and a dynamic modulus test for bituminous mixtures is that the former uses loadings of any waveform with a given rest period, while the latter applies a sinusoidal or haversine loading with no rest period. The dynamic modulus varies with the loading frequency [**5**].

ii. Effect of loading time on modulus

Resilient modulus is determined in time domain while dynamic modulus is in frequency domain. Similarly, master curve for dynamic modulus is drawn in frequency domain. Therefore, when comparing dynamic modulus and resilience modulus, conversion of time to frequency and vice versa is required. The conversion can be made as follows [**30**].

Angular frequency $$f = \frac{1}{2\pi t} \tag{2-18}$$

Time frequency $$f = \frac{1}{t} \tag{2-19}$$

Several literatures [**5**,**22**,**30**,**31**] have revealed that there is a relationship between dynamic modulus and resilient modulus of AC pavements. But comparison is made where test conditions, in terms of temperature and frequency/loading time, are the same. Based on these works, the resilient modulus is equivalent to the dynamic modulus at the same temperature when equation (2-18) was used for equivalent comparison of the two moduli. For example, at a certain temperature and frequency of 5 Hz, resilient modulus and dynamic modulus are equivalent as shown in Figure 2.4. Such a relationship means that dynamic modulus of asphalt mixtures from predicting models could be used as resilient modulus or elastic modulus in multilayer elastic programs such as in KENLAYER.

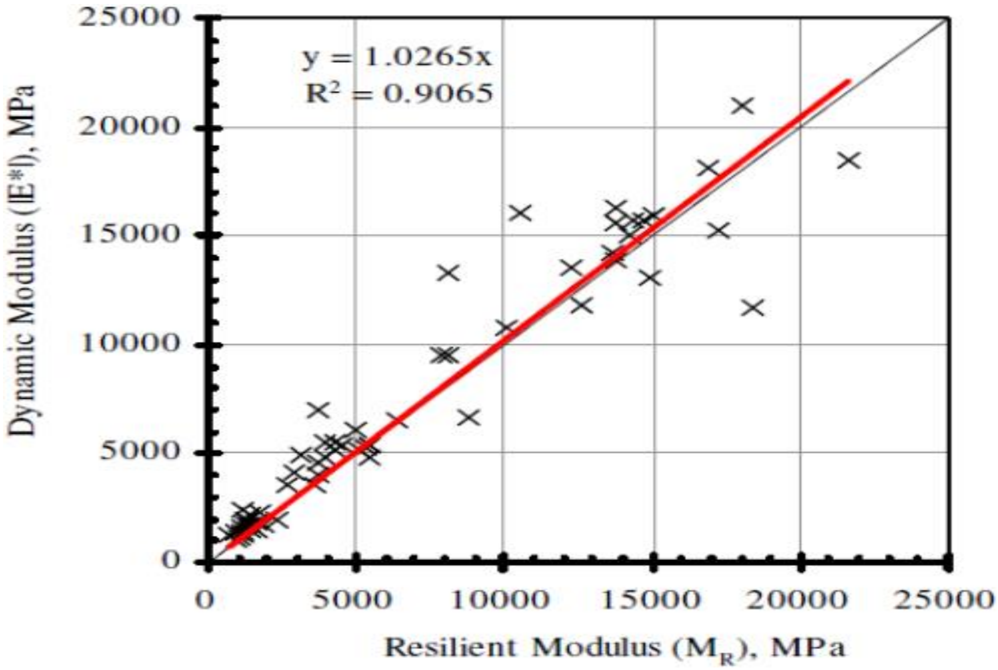


Figure 2. 4 Relationship between resilient and dynamic modulus [**31**]

For design purpose a frequency that represents the actual traffic load should be selected for the test so that the dynamic modulus will be equivalent to resilient modulus [**5**]. In asphalt mixes the effect of loading time varies with speed and depth. Points in a pavement layer have different loading time at the same speed due to a depth difference. Points at depth have longer loading time than points near to surface [**22**]. For an asphalt mix layer loading time is determined as explained in section (2.3.1.d). NCHRP 9-19 report also uses the effective frequency concept as Explained in [**30**] which is given by the following equation.

$$t=\frac{2(a+h)}{v} \tag{2-20}$$

Where parameters are as defined in Section (2.3.1.4)

iii. Creep compliance

AC is characterized by creep compliance curve or mechanical models and these are further related. If one is known, for example creep compliance, it can be converted to the other form i.e. mechanical model. AC is a viscoelastic material and therefore its property depends on temperature and time of loading. Therefore, Time and temperature dependence is reflected in determining modulus. In KENLAYER, creep compliance is determined using collocation method, which is an approximate method to collocate the computed and actual responses at a predetermined number of time durations [**5**].

iv. Effect of Temperature on creep compliance

Creep compliances of visco-elastic materials are determined by 100 sec creep test with compliances measured at 11 (0.001,0.003,0.01,0.03,0.1,0.3,1,3,10,30 and 100sec) different time durations at a certain reference temperature, Then fitted with a Dirichlet series as given in the following equation . Fitting means determination of constant coefficients of the series (i.e. G_i) [**5**].

$$D(t) = \sum_{i}^{n} Gi\ exp\left(-\frac{t}{Ti}\right) \tag{2-21}$$

Where, Ti is retardation time. Using time temperature superposition principle, Creep compliance at any other temperature is determined. In doing so a time temperature shift factor which is defined in Equation (2-22) is used

$$\alpha_T = \frac{tT}{tTo} \tag{2-22}$$

t_T= time to obtain a creep compliance at temperature T

t_{To} = time to obtain a creep compliance at reference temperature To

According to various laboratory tests on asphalt mixes, log a_T versus temperature results in a straight line. The slope of the straight line β is given by Equation (2-23) and varies from 0.11(0.061) to 0.306(0.170) with an average value of 0.203(0.113) (FHWA, 1978) [**5**]

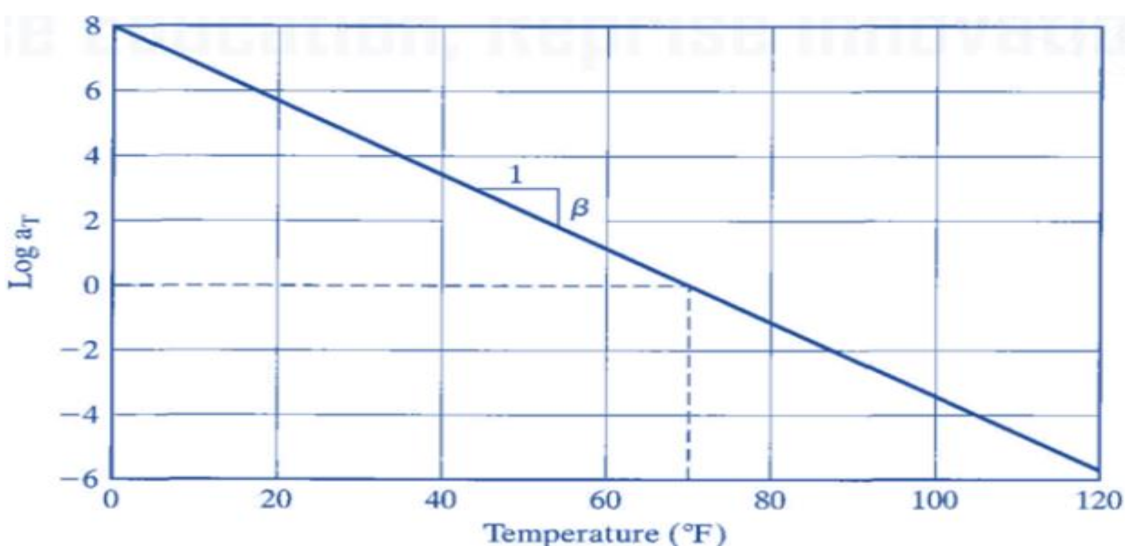


Figure 2. 5 Log a_T (time Temperature shift factor) versus temperature [**5**]

$$\beta = \frac{\log(\frac{tT}{tTo})}{T-To} \text{ and} \tag{2-23}$$

$$T = t_{To} * \exp [(2.304\ \beta(T-To)] \tag{2-24}$$

If Creep compliance at a reference temperature is given by

$$D\ (t) = \sum_i^n Gi\ exp(-\frac{tTo}{Ti}) \tag{2-25}$$

Then at any other temperature it is given by

$$D\ (t) = \sum_i^n Gi\ exp(-\frac{tT}{Ti}) \tag{2-26}$$

v. Effect of loading Time on creep compliance

There are varies methods for analysis of moving loads. For example, the elastic-viscoelastic correspondence principle as reported by hang in 1973 in multilayer system, however, the complexity for analysis and high amount of computer time required make this method unsuited for practical use. Therefore, a simplified method has been used in KENLAYER [**5**].

The viscoelastic solution for moving loads is not so simple because the creep compliances must be expressed in a Dirichlet series by the collocation method so that the Boltzmann's superposition principle can be applied. The use of Dirichlet series for creep compliances also makes possible the prediction of creep compliances at any temperature from those at a given temperature [**32**].

In KENLAYER, it is assumed that the intensity of load varies with time according to a haversine function as give in Equation (2-27) and Figure 2.6.

$$L\ (t) = q\ sin^2\left(\frac{\pi}{2} + \frac{\pi t}{d}\right) \tag{2-27}$$

Where d is the duration of load. Pressure is considered to be zero when the load is at a distance greater than 6a from a point. Where, a is radius of contact area [**5**,**22**].

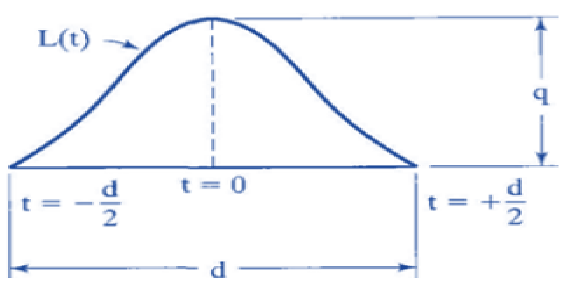


Figure 2. 6 Pressure distribution and loading time [**5**]

When a load is at a considerable distance from a point (at t=-d/2) the load above the point is zero i.e. L (t) =0 and will be max (i.e. = q) when t=0 [**5**]

Then the loading time is approximated by equation (2-16). The response under static load can be expressed in Dirichlet series

$$R\ (t) = \sum_{i}^{7} Ci\ exp(-\frac{t}{Ti}) \qquad (2\text{-}28)$$

By applying Boltzmann's superposition principle, the response under moving load can be obtained as given in the following Equation.

$$R = \frac{q\pi^2}{2} \sum_{i=-\frac{d}{2}}^{0} \left[Ci\, \frac{1+\exp\left(-\frac{d}{2Ti}\right)}{\pi^2+\left(\frac{d}{2Ti}\right)^2}\right] \qquad (2\text{-}29)$$

For the viscoelastic case, the elastic modulus of some layers is time-dependent and these layers must be identified. The reciprocal of the elastic modulus is called the creep compliance and their values at a number of times must be specified.

For stationary loads, the viscoelastic solution at these times can be easily obtained by inverting the creep compliances to the elastic moduli and the elastic solution thus obtained can be considered as the viscoelastic solution.

c. Characterization for Unbound Materials and Subgrade

Stiffness is the most important mechanical characteristic of unbound materials in pavements. It is characterized in terms of resilient modulus M_R which is defined as the unloading modulus in cyclic loading. Elastic modulus (the most important) and poisonous ratio are mostly used unbound material property as an inputs in current pavement design procedures. However, the influence of Poisson's ratio v on computed pavement response is normally quite small. Consequently, use of assumed values for v often gives satisfactory results, and direct measurement in the laboratory is usually unnecessary [**6**].

The elastic modulus for unbound pavement materials is most commonly characterized in terms of the resilient modulus, MR which is defined as the ratio of the applied cyclic stress to the recoverable (elastic) strain after many cycles of repeated loading (Figure 2.7) and thus is a direct measure of stiffness for unbound materials in pavement systems[**33**].

The resilient modulus as measured in a standard resilient modulus cyclic tri-axial test is shown in Figure 2.8, in which σa and εa are the stress and strain in the axial (i.e., cyclic loading) direction and The resilient modulus (MR) is defined simply as the ratio of the cyclic axial stress to resilient axial strain [**33**]:

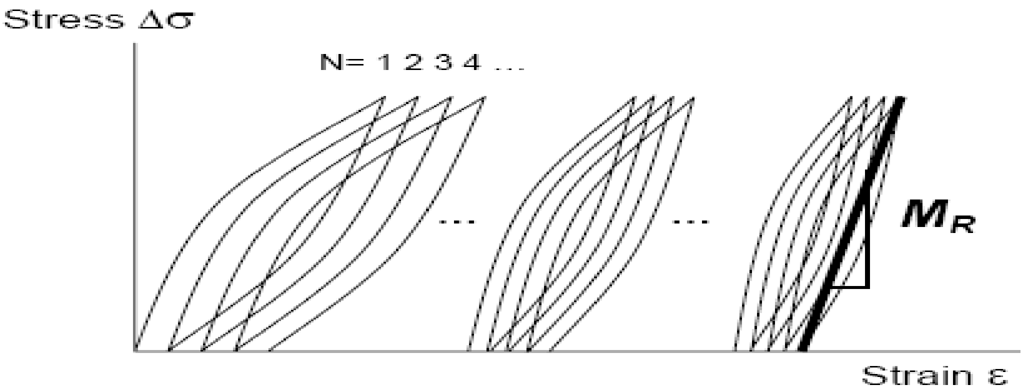


Figure 2. 7 Resilient modulus under cyclic loading [**33**]

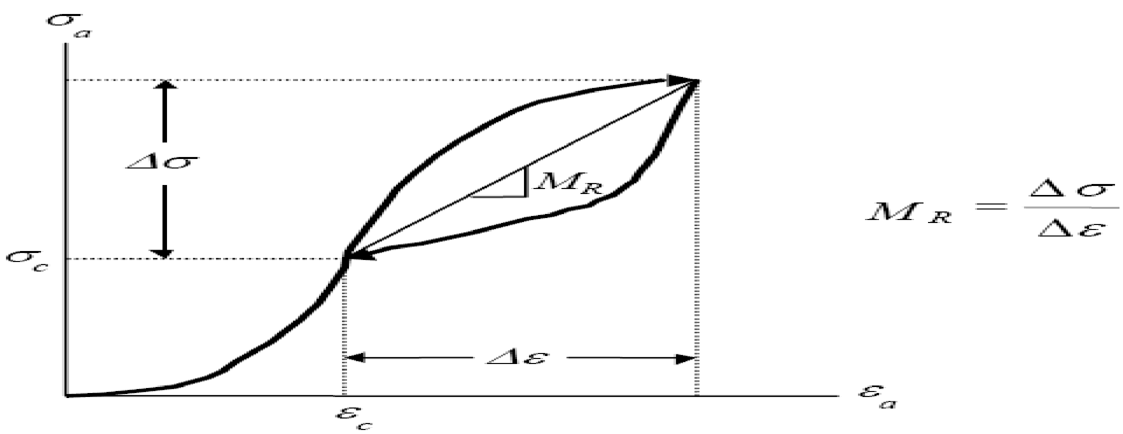


Figure 2. 8 Definition of resilient modulus MR for cyclic triaxle loading [2].

One of the most important part of Mechanistic-Empirical flexible pavement design, is the determination of the resilient modulus (MR) to characterize the mechanical behavior of road structures. Because of the complexity and cost of the test, correlations have been established to predict resilient modulus. California bearing ratio (CBR) is the used parameter to estimate resilient modulus since this parameter is not expensive and is easy to obtain [**33**].

In 1962, Heukelom collaborated with Klomp developed the following relationship between MR and CBR for prediction of Elastic (resilience) modulus of subgrade [**33**].

$$M_R\,(psi) = 1500*CBR \qquad (2\text{-}30)$$

M_R: Resilient modulus (psi) CBR: California Bearing Ratio (%) <10%

This relationship can be expressed (in MPa): as M_R (MPa) =10.34CBR

TRRL establishes the following relations as follows:

$$M_R \text{ (psi)} = 2555 \times CBR^{0.64} \tag{2-31}$$

$$M_R \text{ (MPa)} = 17.6 \times CBR^{0.64} \tag{2-32}$$

In reality, however in some instances it is assumed linear, elastic modulus of granular materials and subgrade soils varies with state of stress. In layered systems, it is the resilient modulus obtained from tri-axial compression or repeated unconfined test which is used as elastic modulus. For granular materials, elastic modulus increases with increase in state of stress but the reverse is true for fine-grained soils [**5**].

For Granular Materials [**5**]**,** the relationship between the first stress invariant and resilient modulus E can be expressed by equation (2-33) as follows

$$E = K_1 \theta^{K2} \tag{2-33}$$

Where K_l and K_2= experimentally derived constants and θ is the stress invariant, which can be expressed as follows

$$\theta = \mathbf{s}_1+\mathbf{s}_2+s_3 = \mathbf{s}_x+\mathbf{s}_y+\mathbf{s}_z \tag{2-34}$$

Including the weight of a layered system gives

$$\theta = \mathbf{s}x+\mathbf{s}y+\mathbf{s}z+\gamma z\,(1+2K_0) \tag{2-35}$$

γ = the average unit weight

z = distance below surface at which the modulus is to be determined, and

K_O = coefficient of earth pressure at rest.

Typical values of Nonlinear Constants K_l and K_2 for several granular materials has tabulated in table…. and table……of [**5**]. In equation (2-35), which includes the weight of layered systems, only normal stresses are used. This is because of the reason that principal stresses may not be in the same direction as the geostatic stresses. The resilient moduli of base and subbase aggregates can be represented by equation (2.33) with K_2 equal to 0.6 and K_l ranging from 3200 to 8000 psi, depending on moisture contents [**5**,**13**].

For Fine-Grained Soils [**5**]**,** in laboratory tri-axial tests, $\mathbf{S}_2 = \mathbf{S}_3$, so the deviator stress is defined as follows in equation (2-36).

$$S_d = S_1 - S_3 \tag{2-36}$$

And in layered systems, S_2 may not be equal to S_3, so the average of S_2 and S_3 is considered as S_3. Including the weight of layered system yields

$$S_d = S_1 - 0.5(S_2 + S_3) + \gamma Z\,(1 - K_o) \tag{2-37}$$

However, geostatic stresses and principal stress may not be in the same direction. Therefore, equation (2-37) is not theoretically correct. **KENLAYER** [**5**] uses the three normal stresses, $\mathbf{S}_X$, $\mathbf{S}_Y$, and $\mathbf{S}_Z$, to replace the three principal stresses, $\mathbf{S}_1$, $\mathbf{S}_2$, and $\mathbf{S}_3$ in equation (2-37) in the case of subgrade. The relationship between resilient modulus and deviator stress of fine-grained soils has a bilinear behavior which can be expressed as follows [**5**].

$$E = K_l + K_3\,(K_2 - \mathbf{S}_d) \text{ when } \mathbf{S}_d < K_2 \tag{2-38}$$

$$E = K_1 - K_4\,(\mathbf{S}_d - K_2) \text{ when } \mathbf{S}_d > K_2 \tag{2-39}$$

Where K_1, K_2, $\mathbf{K}_3$, and K_4 are material constants.

Environment

Temperature

Moisture

2.4 Pavement performance

2.4.1 Performance Concept

The capability of a pavement to serve traffic reasonably over time is known as Pavement performance [**34**]. Pavement performance evaluation or prediction is important in road asset management activities such as in selection of new pavement options and appropriate maintenance and rehabilitation techniques. In doing so, pavement performance models are extremely important and they are the most important components of PMS [**35**,**36**]. Accurate/good prediction of pavement deterioration can result in significant budget savings through timely intervention and accurate planning. It has been acknowledged that pavement roughness is the best measure of pavement performance however it may not be important in identification of type of maintenance and rehabilitation requirements. Distress such as rutting and cracking are more specific for this purpose [**35**]. The general types of approaches used to predict pavement performance are discussed in section ------. Pavement performance can be measured using different techniques.

2.4.2 Measures of Pavement Performance

Pavement performance can be measured in terms of pavement distress such as rutting, cracking etc., PCI, which in turn involves measurement of functional and structural condition, and PSI, which involves condition evaluation by users of the road. In project level, it is important use the 1st approach i.e. to measure individual distress however in network level some measure of performance (performance indicator) is required that may be PCI (measured on a scale 0 to 100) or PSI (measured on a scale 0 to 5). Pavement Condition Rating (PCR) and Pavement Quality Index (PQI) which are measured on a scale of 0 to 100 and 0 to 10 were also introduced as pavement performance measures [**34**,**37**].

Measurement or prediction/estimation of pavement service life for rutting and fatigue cracking was first introduced by Shell's pavement design method. This also involves damage concept which uses miner's principle [**37**]. **Damage** is defined as deterioration of pavements due to traffic under different environmental conditions. This can be estimated by miner's hypothesis which can be stated as [**38**]:

> "*If the fatigue life at stress σ_1 is N_1 and that at stress σ_2 is N_2, then one application of stress σ_1 uses up σ_1/N_1 of life and n_1 applications of load σ_1 uses up a fraction n_1/N_1. Similarly, one application of stress σ_2 uses up σ_2/N_2 of life and n_2 applications of σ_2 uses up a fraction n_2/N_2. Failure occurs when all of the fatigue life is used up, Thus, if the distribution of axle loads (the ns and σs) in the traffic is known, and the fatigue relationship between σ and the number of cycles to failure (N) is determined from laboratory measurements, the life of the pavement layer can be calculated*".

The damage at any increment can be expressed as follows [**22**]:

$$D = \frac{n}{N} \tag{2-40}$$

Where

n is the calculated load application

N is the allowable number of load applications

The total damage at any point is computed by summing up all of the damages over time, up to that time, as follows

$$\text{Total damage} = \sum_{i=1}^{n}\sum_{j=1}^{m}\sum_{k=1}^{o}\frac{\text{nijk}}{\text{Nijk}} \qquad (2-41)$$

Where i, j, and k are the different categories over which the summation of the damage is made. Such categories may include different time increments and traffic (may be different truck class). This principle i.e. miners law has been applied in KENLAYER for pavement life prediction [**5**].

Pavement performance depends on different factors which are reviewed in the following subsections

2.4.3 Factors Affecting Pavement Performance

Despite the efforts in pavement engineering, accurate prediction of pavement life is not possible yet. This has been attributed to the fact that factors that influence pavement performance are difficult to predict. Climatic conditions are not constant through the year, uncertainties in traffic prediction and characterization and variability of pavement material properties [**37**].

There are several parameters that have large impacts on long term performance of pavements. Moisture content, the Ground Water table and temperature are the main environmental factors reported as having significant effect on pavement performance [**34**]. Moisture content affects pavement performance significantly. A dramatic increase in water content of subgrade and unbound pavement layer materials will result in decrease of their modules. This in turn affects pavement performance. Ground water table is directly results in increase in moisture content of subgrade and unbound layers and in turn decrease in modules. In flexible pavements, layer moduli and deflection are affected by temperature. Temperature effect is directly reflected in the moduli of asphalt concrete layers. As temperature increases moduli of AC layer decrease which makes it less resistant to wheel loads and a higher stress will be transferred to lower pavement layers and subgrade. It should be noted that study of the impact of ground water table and moisture content is not the intention of this study.

Traffic load is another factor which affects pavement performance and is responsible for tensile strain and stress at bottom of AC layer and compressive stress and strain on top of subgrade. Theses stresses are further results in fatigue cracking and rutting. This distress also allows moisture to ingress from surface to down layers and subgrade. Repeated application of traffic load results in further deterioration. Traffic can be described in terms of axle load, Traffic volume, the number of ESAL's, tire pressure, configuration, loading time, and mechanism [**5**,**22**,**34**]. Properties of pavement layer materials also play have critical role in long term performance of flexible pavements. Asphalt mix in particular should have a good property to resist cracking. Stiffness of Other unbound layers such as base and subbase should be good enough to resist deformation. Good subgrade resilience modulus will result in good pavement performance because subgrade is the foundation for all other pavement layers [**5**,**34**].

Other Parameters such as geometric features, design and construction and level of maintenance have their own effect on pavement performance [**34**]. Here the concern of this study is method of design used.

2.4.4 Pavement Performance Evaluation/Prediction Models

According to Martin, T.C [**35**], two general approaches has been used by different countries and agencies. The following are some of the approaches used to predict pavement performance [**35**].

Probabilistic: - considers the stochastic nature of pavement performance base on the distribution of dependent variable

Deterministic: - predicts a single value of dependent variable from pavement performance prediction models based on the statistical relationship between dependent and independent variable

Table 2. 3 Classification of Performance Prediction models [**35**]

	Deterministic				**Probabilistic**		
Levels of Pavement Management	Primary responses * Stress * Strain * Deflections etc.	Structural and distress * Rutting * Cracking * Pavement condition (PCR&PSI)	Functional * Serviceability index (PSI) * Skid Loss * Roughness	Damage *Damage Factors	Survivor Curves	Transition process Models	
						Markov *Pavement Condition PSI	Semi-Markov *Pavement Condition PSI
National				✓	✓	✓	✓
State		✓	✓	✓	✓	✓	✓
District		✓	✓	✓	✓	✓	✓
Project	✓	✓	✓	✓			

As can be shown in Table 2.3 above deterministic models are either empirical or mechanistic-empirical. Mechanistic models are depending primary responses. Until recent years there were no pure mechanistic pavement performance prediction models. However mechanistic-empirical models are based on theoretical postulation of pavement performance and calibrated using regression analysis. These models can involve interactive forms of distress at the end of pavement life such as cracking and rutting. If this are calibrated correctly and are theoretically sound, they can be used beyond the data range they were developed. Empirical models are developed from regression analysis of observed or experimental data [**35**].

Saba R.G [**36**]Also discuss in three general categories as empirical, mechanistic-empirical and probabilistic models which is similar to the above classification as mechanistic-empirical and empirical models are deterministic.

a. Empirical Models

Empirical models are developed from regression analysis of observed or experimental data [**35**]. One of the well-known empirical models is The World Bank's Highway Design and Maintenance Standards Model (HDM) which is developed and improved through time the current version is HDM-4 Distress Models. The other example of empirical model is the AASHTO serviceability equation developed from AASHTO road test [**36**].

b. Mechanistic Empirical (M-E) Models

Mechanistic-empirical models are based on theoretical postulation of pavement performance and calibrated using regression analysis. These models can involve interactive forms of distress at the end of pavement life such as cracking and rutting. If this are calibrated correctly and are theoretically sound, they can be used beyond the data range they were developed [**35**].

i. Permanent deformation/rutting models

According to [**5**] and [**22**] two procedures can be used to limit rutting/deformation. One is to limit vertical compressive strain on top of subgrade. This method assumes that rutting failure is due to permanent deformation within the subgrade layer and the deformation from other layers (ac and base/subbase layers) is controlled by mix design and construction specifications and, the other to limit TAPD on pavement surface using the permanent deformation property of each pavement layer. The 1st approach is adopted by many design procedures [**5**].

The subgrade strain criteria, sometimes called transfer function, relates the number of load cycles to failure in structural rutting (permanent deformation) to the vertical strain at the top of the subgrade layer (ε_v), the constants and M_R [**22**][20]:

$$NF = b_1\beta\ (\varepsilon_v)^{-b2}\ (MR)^{-b3} \tag{2-42}$$

Where

N_f is the number of load repetitions causing failure

MR is the design resilient modulus of the subgrade soil

ε_v is the vertical compressive strain at the top of the subgrade

b1, b2, b3 are the coefficients derived from tri-axial tests

β is the field calibration factor

For the specific definition of failure, Different agencies have developed their own models based on this model, some of which do contain MR, and the rest contain εv only. U.S. Army Corps of engineers, for example, has developed the following model later modified by Rauhut et al., in 1984 and Von Quintas et al., 1991 for failure definition of 13 mm (0.5 in. rutting) [**22**].

$$Nf = 1.259 * 10^{-11}\ (\varepsilon v)^{-4.082}\ (M_R)^{0.955} \tag{2-43}$$

The following are some of the rutting models without M_R term [**8**]

The Australian criterion:

$$\left(\frac{9300}{\mu\varepsilon}\right)^{7} \tag{2-44}$$

An average criterion for strong subgrades by recent study in the USA

$$\left(\frac{11000}{\mu\varepsilon}\right)^{5.7} \tag{2-45}$$

The Shell 50% criterion:

$$\left(\frac{28000}{\mu\varepsilon}\right)^{4} \tag{2-46}$$

Overseas Road Note 31:

$$\left(\frac{7300}{\mu\varepsilon}\right)^{6.2} \tag{2-7}$$

Where: $\mu\varepsilon$ is vertical compressive micro strain

However, majority of models are expressed in the following form [**5**] which contains ε_v term only and summarized in Table 2.4.

$$Nf = f4\ (\varepsilon_v)^{-f5} \tag{2-48}$$

Table 2. 4 Summary of rutting failure criteria

	Model	Rutting Failure definition (mm)	f4	f_5
1	Asphalt Institute (1984)	13	1.365×10^{-9}	4.477
2	Australian (Austroads,1997)		$6.017*10^{-15}$	7
3	USA (Janooet al, 2003)		$6.854*10^{-12}$	5.7
4	Shell 50% (1985)	18	$6.150*10^{-7}$	4
5	Shell 85% (1985)	15	$1.94*10^{-7}$	
5	Shell 95% (1985)	13	$1.050*10^{-7}$	4
6	TRRL,UK;85%reliability ; Powell, 1984)	10	$6.180*10^{-8}$	3.95
7	Belgian Road Research Center (Verstraeten et al., 1977)	10	$3.05*10^{-9}$	4.35
8	RN31		5.6572E-14	6.2

The precision with which subgrade criteria are known is poor and the range of the published criteria is very wide indeed. Selecting the most appropriate is essentially a matter of engineering judgment [**8**].

ii. Fatigue cracking Models

The shell fatigue cracking model

The shell laboratory fatigue relationship was developed from laboratory tests results from a broad range of mixes containing conventional binders. There are two models developed in different loading modes, one is under controlled strain (displacement) and the other under controlled stress (load). The following has been developed under controlled strain [**20**,**12**].

$$N = \left[\frac{6918(0.856*Vb+1.08)}{\mu\varepsilon * E^{0.36}}\right]^5 \tag{2-49}$$

Where: V_b = proportion of bitumen by volume in the mixture in %

E = elastic modulus of mixture in MPa

$\mu\varepsilon$ = horizontal micro strain in the asphalt

N = number of strain repetitions to failure.

Using structural analysis, this model was adapted to field conditions to account for temperature gradients in AC layer and wheel wander (transverse distribution of loads). Laboratory tests were adjusted for intermittent loading. To account for these effects, the fatigue life predicted by the above equation is multiplied by 10. The computer version of the shell design procedure uses this factor for prediction of fatigue life [**20**,**12**].

Austroads fatigue criteria

The asphalt fatigue criteria used by Austroads is the laboratory fatigue relationship developed by shell in 1978. Austroads has adjusted this further by applying shift factors and project reliability factor. On moderate to heavily trafficked pavements, For AC containing conventional bituminous binders, the relationship between maximum strain produced under a certain traffic load and repetition of this particular load is given by [**19**].

$$N = \frac{SF}{RF} * \left[\frac{6918(0.856*Vb+1.08)}{\mu\varepsilon * E^{0.36}}\right]^5 \quad (2\text{-}50)$$

Where: V_b = proportion of bitumen by volume in the mixture in %

E = elastic modulus of mixture in MPa

$\mu\varepsilon$ = horizontal micro strain in the asphalt

N = number of strain repetitions to failure.

SF = shift factor between laboratory and in-service fatigue lives (likely value = 6)

RF = reliability factor for asphalt fatigue

Table 2. 5 Suggested reliability factors (RF) for asphalt fatigue [**19**]

Desired project reliability					
50%	**80%**	**85%**	**90%**	**95%**	**97.50%**
1	**2.4**	**3**	**3.9**	**6**	**9**

The above relations can be expressed in basic form in which many major models are expressed

$$N_f = f1\,(\varepsilon_t)^{-f2}\,(E_1)^{-f3} \quad (2\text{-}51)$$

Where: f1, f2, f3 = regression constants εt = horizontal strain in the asphalt

Nf= number of strain repetitions to failure.

E_1= elastic modulus of asphalt

Assuming a standard mix with V_b=11%, The Australian fatigue law for asphalt is

$$N_f = 0.507\, (\varepsilon_t)^{-5}\, (E_1)^{-1.8} \qquad ((2\text{-}52)$$

E_1= elastic modulus of asphalt in kpa

ε_t = horizontal strain in the asphalt

Other models are summarized in Table 2.6

Table 2. 6 Summary of fatigue failure criteria models

Model	Literature	f1	f2	f3
asphalt institute (AI,1991)	pavement analysis and design (yang.H,2004)	0.416	3.291	0.854
Australia	Ethiopian Roads Authority (ERA,2013)	0.507	5	1.8
Shell	Pavement engineering, principles and practice (Rajib B. etal,2013)	6.564	5.671	2.363
TRRL	Pavement engineering, principles and practice (Rajib B. etal,2013)	1.66E-10	4.32	0

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Study Area

Because this study uses a hypothetical pavement layer composition, it has no specific study area. However, for temperature and traffic, data were collected from Jimma area. Specifically, traffic data is collected along Jimma-Addis road (a case along Jimma-Sekoru road). The Distance between cities Jimma (located in Oromia, Ethiopia) and Sekoru (located in SNNP Region, Ethiopia) on public roads is 106.13 km. The distance between the points in the coordinates is 71 km.

3.2 Study Design

At the every initial this research framework involves selection of study area and problem definition. This research type is exploratory in nature for being little is known about mechanistic design methods, although it might be confirmatory for others. Figure 3.1 shows the research design diagrammatically.

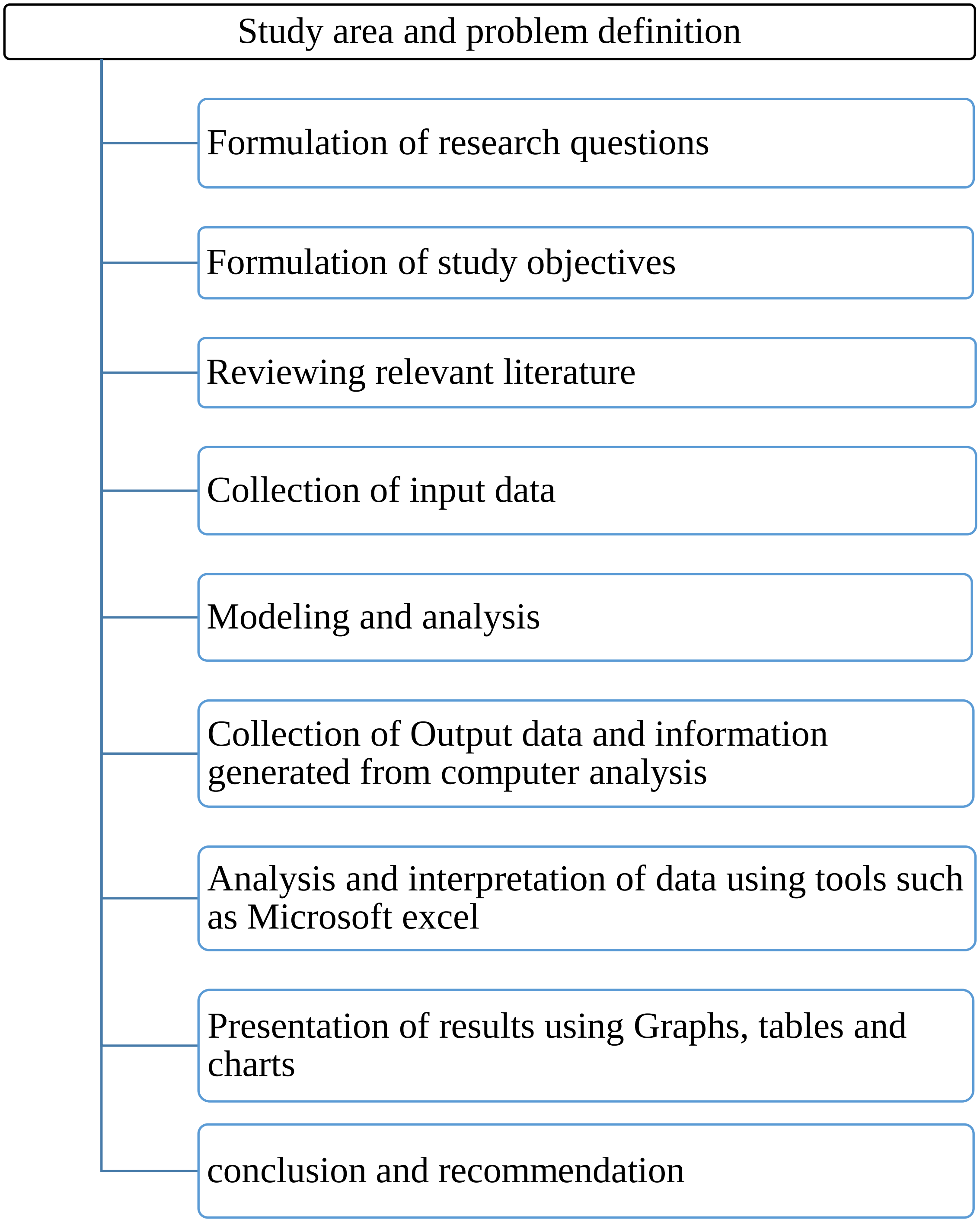


Figure 3. 1 Research Design

This research frame involves collection of secondary data and software simulation and the approach is presented as follows.

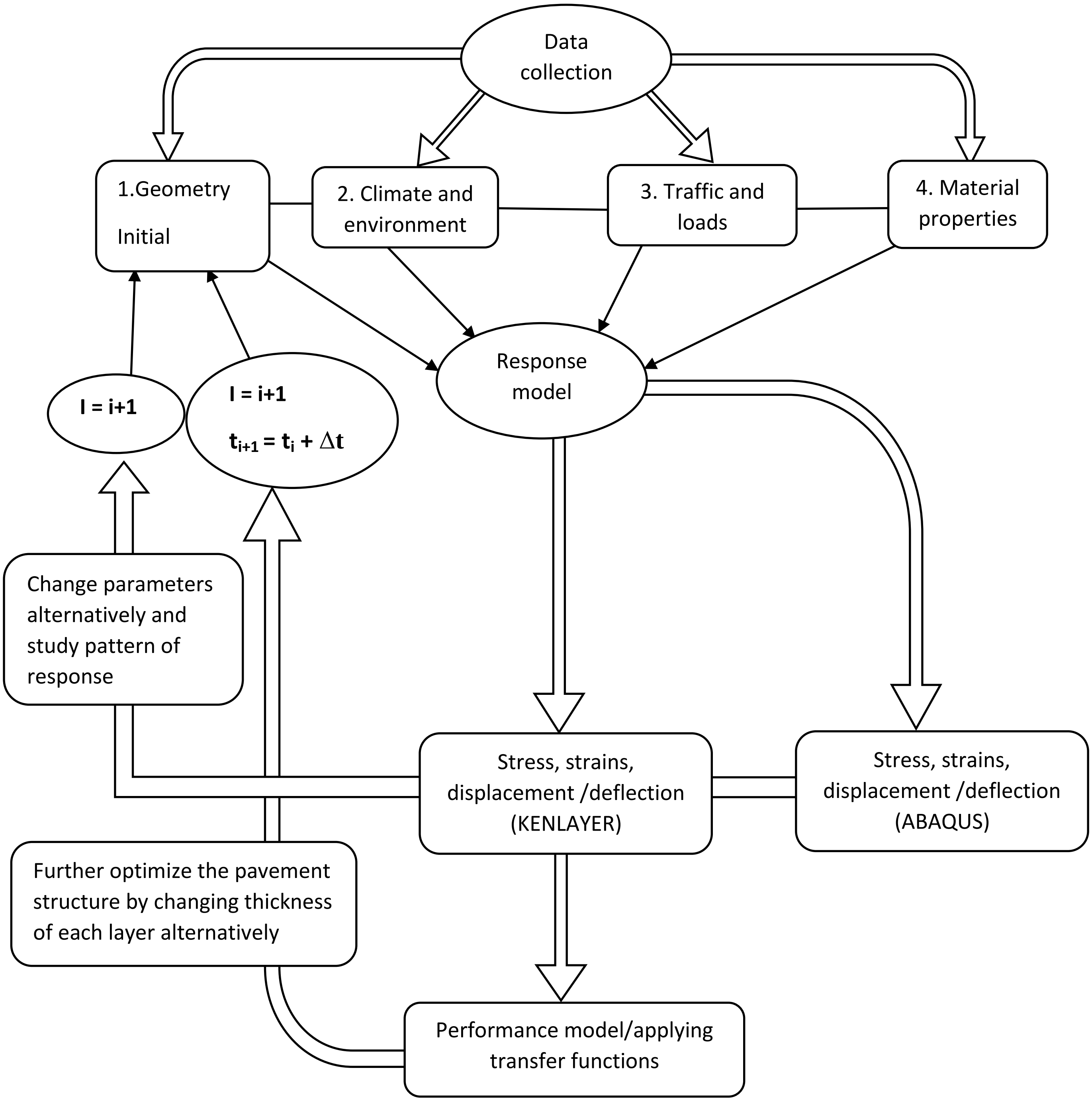


Figure 3. 2 Research Methods

3.3 Study population and sampling technique

The study population for this research was pavement structures in catalogues of ERA pavement design manual. The sampling technique that was used for this research is purposive sampling which is a non-probability sampling technique. This method is appropriate when the study places special emphasis upon the control of certain specific variables. The Idea is to pick out the sample in relation to some criteria, which are considered important for the particular study. The criteria here are traffic level, subgrade class and different layer composition.

3.4 Study variables

3.4.1 Dependent variable

Pavement responses

Pavement performance

3.4.2 Independent variables

Temperature

Traffic load in terms of ESAL

Tire pressure

Material properties

Pavement thickness

3.5 Methods of data collection

3.5.1 General

This study is curried out to evaluate performance of pavement structures designed as per ERA Design Manual against the mechanistically designed structure using KENLAYER, mechanistic-empirical design software and a new design framework was developed using this approach. This can be done either by directly taking the pavement structure from ERA catalogue or after designing using ERA guide lines and then analysis these pavement structures using KENLAYER. ABAQUS, a 3D FEM program, was also used for comparison to check the accuracy of KENLAYER and for load configurations which could not modeled in Kenlayer.

By taking a reasonable hypothetical pavement structure from ERA design catalogue, the effect of material properties, traffic and environmental factors on pavement performance were studied. Finally, an economical pavement structure (pavement layer composition) was established for pre-determined designed period which could be expressed in terms of cumulative ESA.

Data for the following parameters listed from 1-8 and others were collected from literatures, material specifications, design manuals and predicting models with appropriate engineering judgment. Most of the data used were Secondary in source. However, data related to traffic such as axle load and AADT were collected from ERA Database by official request attached on appendix. Tire pressure was collected on field.

1. CBR Data
2. Elastic modulus of bituminous surface layer
3. Elastic (resilience) modulus of subgrade
4. Elastic (resilience) modulus of granular base and subbase
5. Poisson's ratio
6. Traffic
7. Temperature
8. Transfer function (distress model)

For performance prediction of catalogue of structures, Data types listed above from 1-5 were taken from ERA Flexible Pavement Design Manual 2013 as shown in Table 3.1 below

Table 3. 1 Material characteristics for mechanistic analysis [**8**]

Material	Parameter	Value	Comment
Asphaltic concrete wearing course and binder course	Elastic modulus (MPa)	3000	A balance between a value appropriate for high ambient temperatures and the effect of ageing and embrittlement
	Volume of bitumen	10.5%	
Asphaltic concrete roadbase	Elastic modulus (MPa)	3000	
	Volume of bitumen	9.5%	
Granular roadbase	Elastic modulus (MPa) Poisson's ratio	300 0.30	For all qualities with CBR > 80%

Granular sub-base	Elastic modulus (MPa) Poisson's ratio	175 0.30	For CBR ≥30%
Capping layer	Elastic modulus (MPa) Poisson's ratio	100 0.30	For CBR ≥15%
Subgrades S1 S2 S3 S4 S5 S6	Elastic modulus in MPa	 28 37 53 73 112 175	Poisson's ratio for all Subgrades was assumed to be 0.4
Hydraulically stabilized material	Elastic modulus (MPa)	CB1 = 3500 CB2 = 2500 CS =1500	Poisson's ratio assumed to be 0.25 The modulus of CS is assumed to decrease with time hence a conservative low value of 1000MPa has been used

Source ERA [**8**]

The other parameter used in this research is transfer functions which are labeled as No. 8 in the above list. To use transfer functions for performance prediction, it needs to choose appropriate distress models for the major distresses which are considered in the structural design of asphalt pavements which are fatigue cracking and permanent deformation (rutting). Because these are most devastating asphalt pavement distresses, researchers have focused on developing prediction models for these distresses. Distress models are also called transfer functions which relate pavement structural response to different types of distress such as fatigue cracking and rutting. Therefore, choosing the appropriate model is a matter of engineering judgment.

3.5.2 Fatigue cracking model

There have been many fatigue cracking models used and it is the major difference of many design methods. Fatigue cracking is related to the tensile strain at the bottom of AC layer and miner's damage concept is used to predict fatigue cracking. Here damage is defined as the ratio between predicted and allowable load repetitions [**5**]. As shown in Table 2.6, many major models are expressed in a basic form as in equation (2-51) [**5**].

After drawing the four alternative models on log-log scale as shown below comparison has been made. Accordingly, at similar calculated strain the allowable repetition of loads is very low for

AI model and intermediate for TRRL and Austroads model. This implies that The AI model is very conservative and the allowable repetition of loads for Shell is very high which in turn may leads to wrong design i.e. thin AC thickness. The TRRL is developed by Powell ital. for UK dense bitumen macadam base course not for surface course. The weather condition in UK is temperate therefore leave this as an alternative.

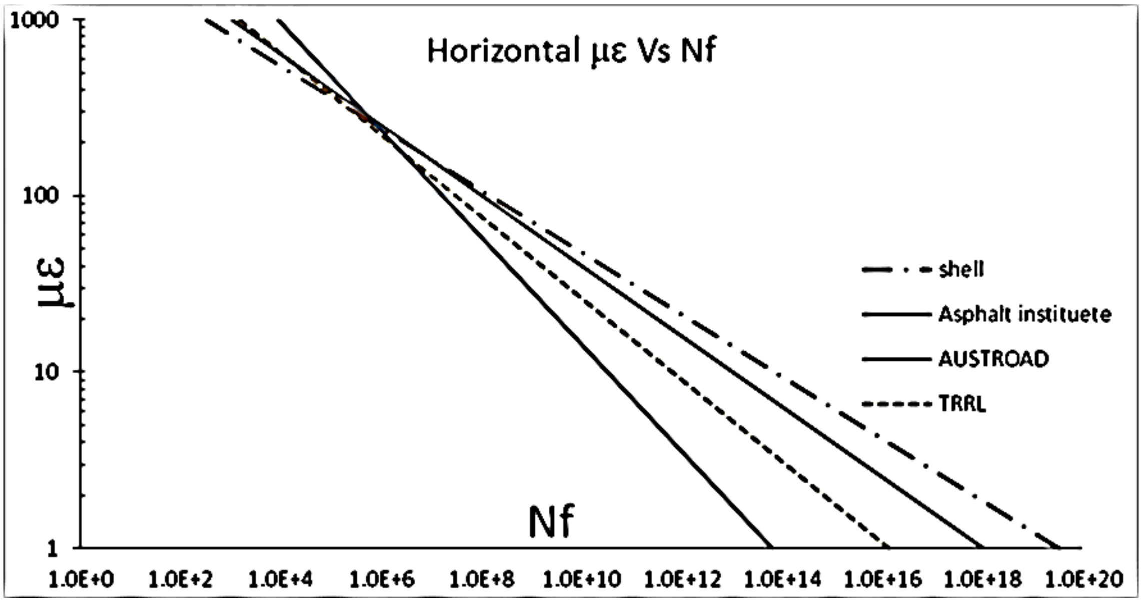


Figure 3. 3 Comparison of fatigue cracking models on log-log scale graph

Of the many models explained and listed in the literature review and here, the Shell (1978) model or later adapted by the Austroads is selected for our particular condition because this is based on an evolving research program and best available data. Furthermore, the climatic conditions in Australia are suitably tropical [**8**]. This is further recommended by ERA design manual. The Australia fatigue law for asphalt is as shown in equation (2-49) or (2-50)

For this study, Assuming a standard mix with V_b=11%, this fatigue law for asphalt is given in the form of equation (2-51)

$$N_f = 0.507\, (\varepsilon_t)^{-5}\, (E)^{-1.8}$$

E= modulus of asphalt mix in kpa

εt = horizontal strain in the asphalt

Other models are summarized in the literature review Table 2.6.

3.5.3 Rutting Model

Two procedures [**5**,**22**] have been used for mechanistic modeling of asphalt pavements rutting as discussed in the literature review The 1st approach is adopted by many design procedures [**5**]. However, the 2nd approach has not been used widely because of the difficulty of materials characterization (elasto-plastic or visco-plastic characterization) [**22**].

The subgrade strain criteria, also called transfer function, relates the vertical strain at the top of the subgrade layer (ε_v), the constants and resilience modulus to the number of load cycles to failure in structural rutting (permanent deformation) [**22**]. Majority of models are expressed in the basic form as shown in equation (2-48)[**5**], which contains ε_v term only and summarized in Table 2.4.

For this particular study the 1st approach is used. As can be inferred from Table 2.4, the range of the published criteria is very wide indeed and selecting the most appropriate is therefore a matter of engineering judgment. The Road note 31 model is derived from the empirically design charts that are already adapted by ERA. This model is recommended by ERA but a rough Evaluation of the rutting performance of catalogue structures in ERA design manual shows that all structures are within the design range i.e. they satisfy the intended repetitions of standard axle loads and by far performs beyond that requirement. Therefore, this model seems unnecessarily non conservative. After drawing alternative models on log-log scale as shown in Figure 3.4 comparison has been made. Accordingly, Analysis of Figure 3.4 reveals that this model is similar to the shell 50% model and a model developed in USA for strong subgrade materials but more conservative by about 3 times in traffic terms at a subgrade strain of 645 $\mu\varepsilon$.

At the same level of subgrade strain it is more conservative than the Australian model by a factor of 40. Other models (such as BRRC, TRRL, Asphalt institute, Shell (95%) and Shell (85%)) are also more conservative than RN31 by a factor of 15, 14,13,6 and 3 respectively in traffic terms. From this analysis the Shell (95%) model would be a suitable compromise for this study.

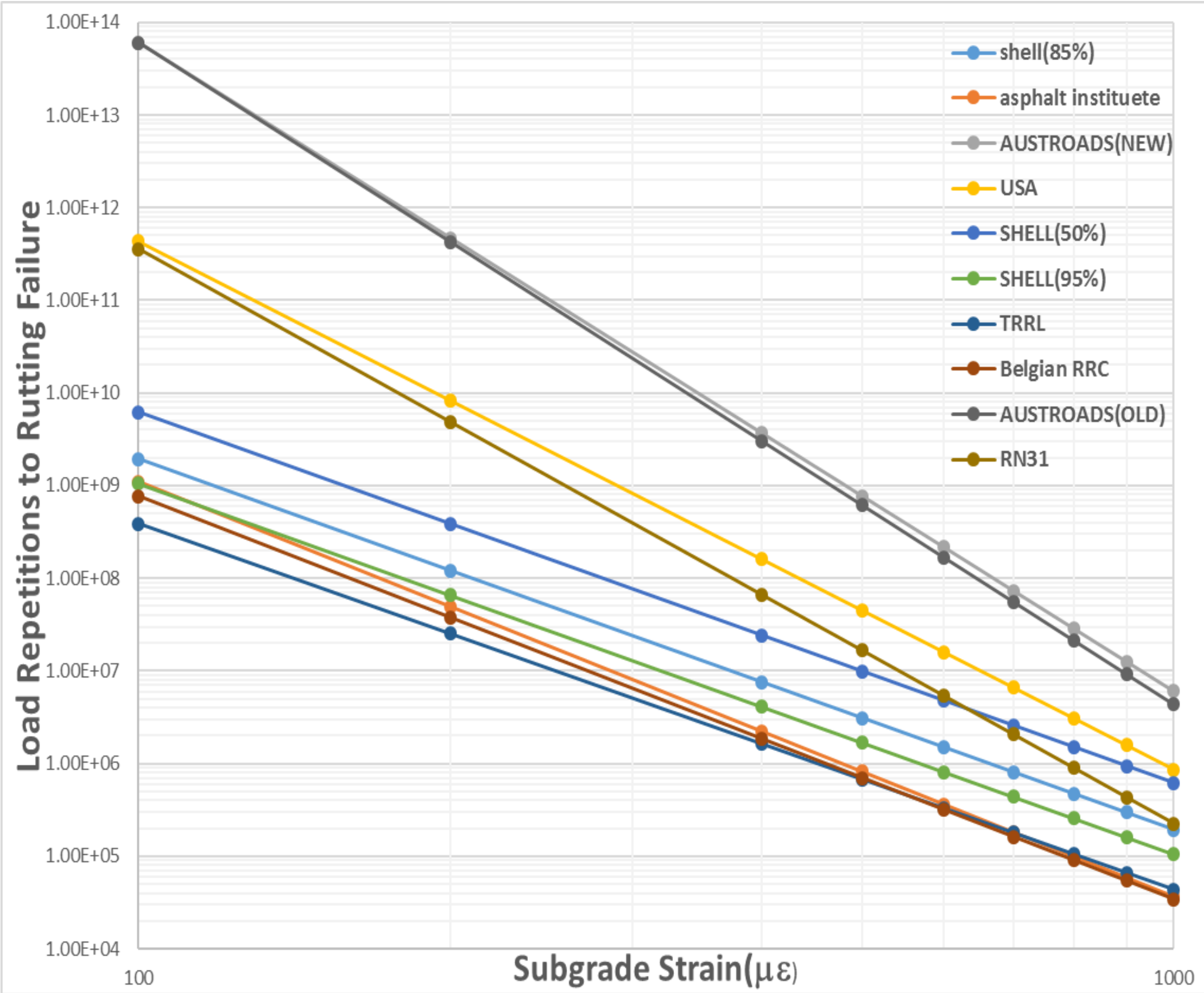


Figure 3. 4 Comparison of rutting models on log-log scale graph

3.6 Data processing and analysis

Data to be processed here were software outputs. However, software outputs are only data until analyzed and used intelligently. It was after then they become evidences. Software outputs are numbers (quantitative). If available data to be analyzed is a number, Microsoft Excel is an ideal tool for analysis. Therefore, for this study Microsoft excels was used for analysis of data obtained from software's and predicting models. Then the data is presented by graph, tables and charts.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Effect of physical factors on response and performance of flexible pavements

4.1.1 Modulus of AC

Modulus May be expressed in terms of resilient modulus elastic modulus, stiffness modulus or dynamic modulus as expressed in literatures. Whichever term is used, it varies considerably over place to place and due to any other factors. For the purpose of this analysis we have taken a range of modulus of 1500-3500. Other properties of AC such as μ and any property of other pavement layers are kept constant as given in Table 4.5.

Table 4. 1 Property of pavement layers

AC layer	Modulus (MPa)	Poisson's Ratio (μ or ν)
AC layer	Variable	0.25
Base course	300	0.35
Subbase	175	0.4
Subgrade	37	0.45

Tire and contact pressure is assumed to be equal. However, contact pressure varies considerably from 500-1000 kPa according to literatures and the survey conducted for purpose of this research. But for this analysis a value of 700 kpa is used. A hypothetical pavement structure as shown in Figure 4.1 is used

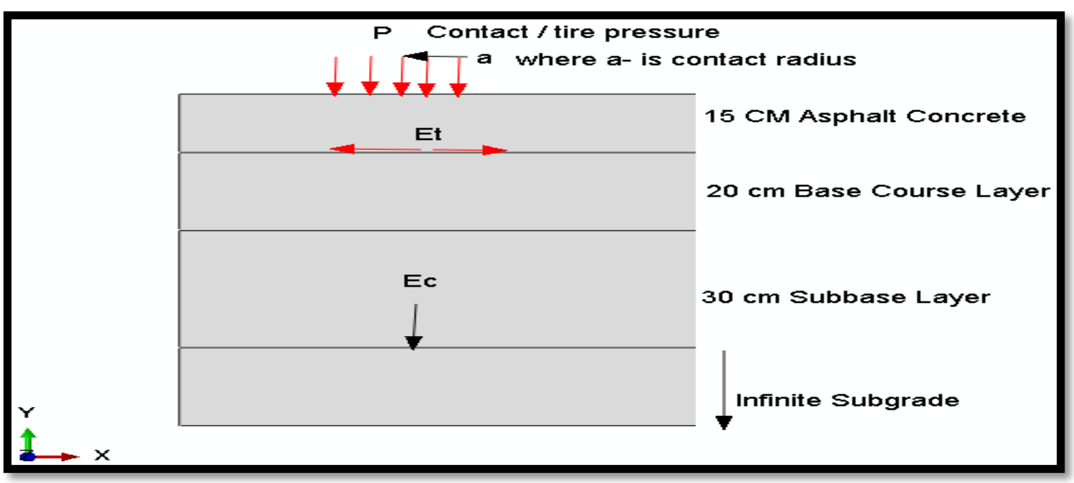


Figure 4. 1 A hypothetical pavement structure used for analysis of effect of modulus

The results of the analysis are presented in Table 4.2. In this table only critical strains and pavement deflection at three locations and two depths are presented. When modulus of AC is

1500mpa, Horizontal strain and vertical strains are maximum. Further increase in modulus results in decrease of horizontal strain and vertical strain on top of subgrade but with different rate of decrease (sensitivity) which was discussed in section 4.2 . At bottom of AC layer, pavement deflection is maximum at corner of contact area until the modulus of ac layer increases to 2700mpa. Beyond this modulus value of deflection becomes maximum between dual wheels. As the AC becomes stiff, location of max deflection changes from corner of contact area to between dual wheels. However, at a lower depth i.e. at depth of 65.001cm (or at top of subgrade) deflection is maximum between the dual wheels through all modulus values. This is an indication of that the effect of dual wheels at location 3 is more pronounced when depth increases. As can be shown in Table 4.2, increase in allowable repetition of standard axles per unit increase of modulus (MPa) increases as AC layer becomes stiff. The effect of AC layer stiffness is more pronounced when the stress location is near to surface. As a summary all strains and deflection has decreased when AC modulus increases and in turn increase allowable repetitions of standard axles.

Table 4. 2 Effect of AC modulus change on pavement response

Modulus	Pavement Response at bottom of AC				Pavement responses at Top of Subgrade				allowable repetitions of standard axles	Increase in allowable repetitions of standard axles per unit increase of modulus (in mpa)
	H. strain	Deflection (cm)			V. Strain	Deflection (cm)				
		1	2	3		1	2	3		
1500	-2.22E-04	0.05567	0.05635	0.05618	3.56E-04	0.04214	0.04286	0.04311	6.56E+06	-
1800	-2.09E-04	0.05442	0.05515	0.05505	3.45E-04	0.04146	0.04215	0.0424	7.04E+06	-
2100	-1.97E-04	0.05337	0.05414	0.05409	3.36E-04	0.04091	0.04157	0.04181	7.13E+06	300.00
2400	-1.87E-04	0.05246	0.05326	0.05325	3.29E-04	0.04043	0.04107	0.04131	7.35E+06	713.33
2700	-1.77E-04	0.05167	0.05248	0.05251	3.22E-04	0.04002	0.04064	0.04088	7.64E+06	990.00
3000	-1.69E-04	0.05097	0.05178	0.05184	3.15E-04	0.03965	0.04025	0.04049	8.00E+06	1193.33
3300	-1.62E-04	0.05033	0.05115	0.05122	3.09E-04	0.03932	0.03991	0.04015	8.41E+06	1350.00
3500	-1.58E-04	0.04994	0.05076	0.05084	3.06E-04	0.03912	0.0397	0.03993	8.70E+06	1460.00

Note: 1. Response location at center of one of the wheels

2. Response location at corner of one of the wheels contact area

3. Response location between dual wheels

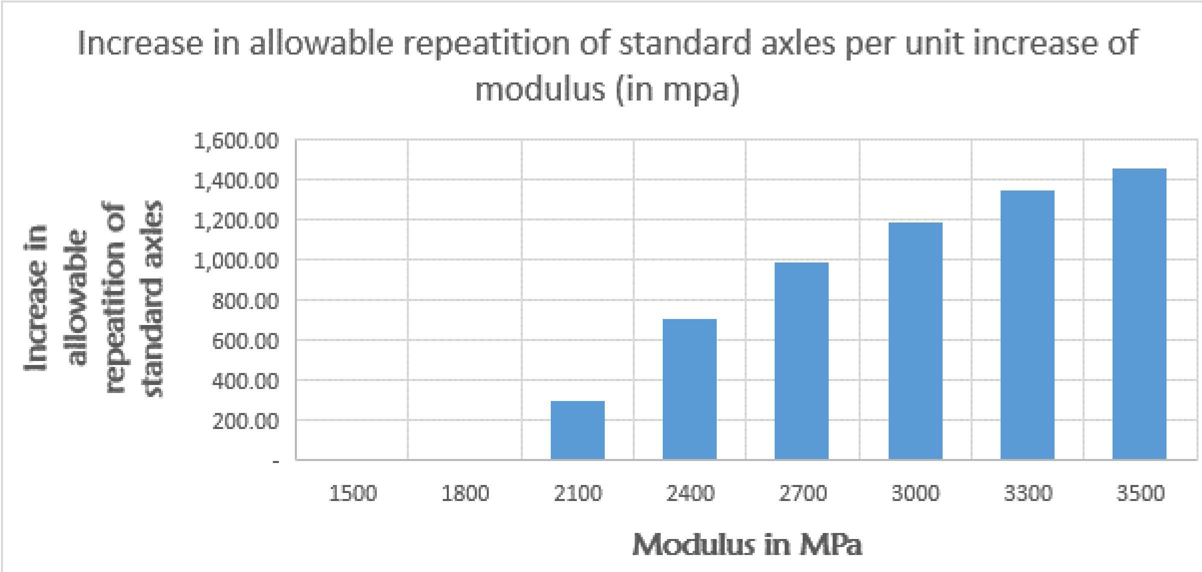


Figure 4. 2 Increase in allowable repetition of standard axles per unit increase of modulus

4.1.2 Poisonous ratio of asphalt concrete

To assess the effect of Poisson's ratio of AC on pavement response and in turn performance, Other properties of AC layer such as E and any property of this layer and other pavement layers are kept constant as given in Table 4.3.

Table 4. 3 Parameters used for analysis of effect of AC Poisson's ratio on pavement response

AC layer	Modulus (MPa)	Poisson's Ratio (μ or ν)
AC layer	3000	variable
Base course	300	0.35
Subbase	175	0.4
Subgrade	37	0.45

Pavement contact pressure and the pavement structure used here is the same as in Section 4.1.1 . As can be illustrated in Table 4.4 increase in poisson's ration results in decrease of pavement responses and in turn increase in allowable repetitions of standard axle loads but the rate of increase (sensitivity) is less compared with the sensitivity to modulus. To confirm this, it is enough to see an increase in allowable standard axle loads of 32.69% over the range of 1500-3500 MPa but only 12.69% increase for poisson's ratio in a range of 0.15-0.4.

Table 4. 4 Effect of poisson's ratio on pavement response and performance

Poisson's Ratio(μ	Pavement Response at bottom of AC				Pavement responses at Top of Subgrade				allowable repetitions of standard axles
	H. strain	Deflection (cm)			V. Strain	Deflection (cm)			
		1	2	3		1	2	3	

or ν)	V. strain	Deflection (cm)							
		1	2	3		1	2	3	
0.15	-1.71E-04	0.0515	0.0523	0.0524	3.20E-04	0.0400	0.0406	0.0409	7.72E+06
0.2	-1.70E-04	0.0514	0.0522	0.0522	3.19E-04	0.0399	0.0405	0.0408	7.73E+06
0.25	-1.70E-04	0.0512	0.0520	0.0521	3.17E-04	0.0398	0.0404	0.0406	7.83E+06
0.3	-1.69E-04	0.0510	0.0518	0.0518	3.15E-04	0.0397	0.0403	0.0405	8.00E+06
0.35	-1.68E-04	0.0507	0.0515	0.0516	3.13E-04	0.0395	0.0401	0.0403	8.28E+06
0.4	-1.67E-04	0.0504	0.0512	0.0513	3.10E-04	0.0393	0.0399	0.0401	8.70E+06

Note: 1. Response location at center of one of the wheels

2. Response location at corner of one of the wheels contact area

3. Response location between dual wheels

4.1.3 Modulus of base course

Resilient modulus of base course layers could be as high as 350 MPa as given in Cost 333 [**14**] and in the user interface of WESLEA software for windows (version 3.0). Depending on its nature Resilient modulus of base can be as high as shown in Table 4.5. In ERA design manual 300 MPa is used for all types of granular base course [**8**]. For this purpose 100-350 MPa is used and other parameters are kept constant. The hypothetical pavement structure is the same as used in Section 4.1.1.

Table 4. 5 Typical Range of Resilient modulus of base depending on its nature

Material	Elastic Modulus (E or $M_{r.}$), MPa	Poisson's Ratio (μ or ν)
Crushed stone base (clean, well-drained)	150 – 600	0.35
Crushed gravel base (clean, well-drained)	150 – 600	0.35
Uncrushed gravel base (Clean, well-drained)	70 – 400	0.35
Uncrushed gravel base (Clean, poorly-drained)	20 – 100	0.40

Analysis results for change in Modulus of base course are presented in Table 4.6. As can be seen in the table, all pavement responses has decreased as modulus of base course increases but when compared to AC modulus, increase in standard axle load repetition per unit increase of base course modulus is higher. Over the range of 100-350MPa horizontal strain at bottom of Asphalt Layer decreases from $-2.40\text{x}10\text{-}^{04}$ to $1.59\text{x}10\text{-}^{04}$ which is a 34% change. Over the same range vertical strain at top of subgrade changes only in 18% and change in pavement deflection at a depth of 15cm and 65.0001cm is 15% and 12 % respectively.

Table 4. 6 Effect of base course on pavement response & performance

Modulus of Base	Pavement Response at bottom of AC				Pavement responses at Top of Subgrade				allowable repetitions of standard axles	
	H. strain	Deflection (cm)			V. Strain	Deflection (cm)				
		1	2	3		1	2	3		
100	-2.40E-04	0.060	0.061	0.062	3.60E-04	0.044	0.045	0.045	1.40E+06	-
150	-2.15E-04	0.056	0.058	0.058	3.47E-04	0.042	0.043	0.043	2.41E+06	20,220
200	-1.97E-04	0.054	0.055	0.055	3.35E-04	0.041	0.042	0.042	3.78E+06	27,280
250	-1.82E-04	0.052	0.053	0.053	3.25E-04	0.040	0.041	0.041	5.60E+06	36,420
300	-1.69E-04	0.051	0.052	0.052	3.15E-04	0.040	0.040	0.040	8.00E+06	48,060
350	-1.59E-04	0.050	0.051	0.051	3.07E-04	0.039	0.040	0.040	1.11E+07	62,780

Note: 1. Response location at center of one of the wheels

2. Response location at corner of one of the wheels contact area

3. Response location between dual wheels

4.1.4 Modulus of subbase

Resilient modulus of subbase course layers varies according to material properties and moisture conditions of the environment. For this sensitivity analysis a range from 100-250MPa is used and other parameters are kept constant. The hypothetical pavement structure is the same as used in Section 4.1.1.

Pattern of pavement response to change of subbase modulus is different from that of change of base modulus in that Increase in allowable repetitions of standard axles per unit increase of modulus decreases as subbase modulus increases but the reverse is true for change in base modulus. As shown in Table 4.7, over the range of 100-250 MPa subbase modulus, decrease in critical horizontal tensile Stain is 9% but 17% for vertical strain at top of subgrade. Change in pavement deflection is 15% and 10% at a depth of 15 cm and 65.0001cm respectively. Increase in allowable repetitions of standard axles is 64%.

Table 4. 7 Effect of modulus of subbase on pavement response and performance

Modulus of Subbase	Pavement Response at bottom of AC				Pavement responses at Top of Subgrade				allowable repetitions of standard axles	Increase in allowable repetitions of standard axles per unit increase of modulus (in MPa)
	H. strain	Deflection (cm)			V. Strain	Deflection (cm)				
		1	2	3		1	2	3		

100	-1.80E-04	0.056	0.057	0.057	3.46E-04	0.042	0.043	0.043	5.95E+06	-
125	-1.76E-04	0.054	0.055	0.055	3.36E-04	0.041	0.042	0.042	6.67E+06	28,920
150	-1.72E-04	0.052	0.053	0.053	3.26E-04	0.040	0.041	0.041	7.36E+06	27,280
175	-1.69E-04	0.051	0.052	0.052	3.15E-04	0.040	0.040	0.040	8.00E+06	25,800
200	-1.67E-04	0.050	0.051	0.051	3.05E-04	0.039	0.040	0.040	8.61E+06	24,400
250	-1.63E-04	0.048	0.049	0.049	2.88E-04	0.038	0.038	0.039	9.74E+06	22,560

Note: 1. Response location at center of one of the wheels

2. Response location at corner of one of the wheels contact area

3. Response location between dual wheels

4.1.5 Modulus of subgrade

As give in ERA Design Manual Modulus of a subgrade material varies as shown in Table 4.8.

Table 4. 8 Range of modulus of subgrade for mechanistic analysis [**8**]

Material	Parameter	Value	Comment
Subgrades	Elastic modulus in MPa		Poisson's ratio for all Subgrades was assumed to be 0.4
S1		28	
S2		37	
S3		53	
S4		73	
S5		112	
S6		175	

Therefore, here a pavement response analysis for change in subgrade resilient modulus from 28-175mpa is presented in Table 4.9.

Pattern of pavement response to change of subgrade modulus is also different from that of change of subbase modulus in that Increase in allowable repetitions of standard axles per unit increase of modulus decreases rapidly as subgrade modulus increases. But the reverse is true for change in base modulus. Over the range of 28-175 MPa subgrade modulus, decrease in critical horizontal tensile Stain is 4% but 62% for vertical strain at top of subgrade. Change in pavement deflection is 60% and 74% at a depth of 15 cm and 65.0001cm respectively. Increase in allowable repetitions of standard axles is 39%. The rapid decrement in allowable repetitions of standard axles per unit increase of modulus is a clear indication of further increase in modulus of subgrade is inefficient.

Table 4. 9 Effect of subgrade modulus on pavement response and performance

Modulus of Subgrade	Pavement Response at bottom of AC				Pavement responses at Top of Subgrade				Allowable repetitions of standard axles	Increase in allowable repetitions of standard axles per unit increase of modulus (in MPa)
	H. strain	Deflection (cm)			V. Strain	Deflection (cm)				
		1	2	3		1	2	3		
28	-1.70E-04	0.060	0.060	0.061	3.52E-04	0.048	0.049	0.049	6.86E+06	-
37	-1.69E-04	0.051	0.052	0.052	3.15E-04	0.040	0.040	0.040	8.00E+06	127,333
53	-1.68E-04	0.042	0.043	0.043	2.69E-04	0.031	0.031	0.032	8.35E+06	21,750
73	-1.67E-04	0.035	0.036	0.036	2.30E-04	0.025	0.025	0.025	8.68E+06	16,300
112	-1.65E-04	0.029	0.029	0.029	1.80E-04	0.018	0.018	0.018	9.12E+06	11,385
175	-1.63E-04	0.024	0.024	0.024	1.35E-04	0.013	0.013	0.013	9.56E+06	7,032

Note: 1. Response location at center of one of the wheels

2. Response location at corner of one of the wheels contact area

3. Response location between dual wheels

4.1.6 Temperature

Responses and in turn performances of Pavement structures containing bituminous layers are affected by pavement temperature and loading time (in other words speed of vehicles) because of visco-elasticity. Other layers are assumed to be independent of this temperature factors (but for freeze and thaw condition which is inappropriate for our case). The effect of temperature is considered in pavement design and reflected in its modulus by using predicting models as mentioned in literatures above.

Among many predicting models, the AI (Hwang and Witczak (1979)) model which is also used in DAMA software [**5**], is used here just for simplicity.

$$E * = 100,000 \text{ x } 10^{\beta 1}$$

$$\beta_1 = \beta_3 + 0.000005\ \beta_2 - 0.0018913\ \beta_2\ f^{-1.1}$$

$$\beta_2 = \beta_4^{0.5} * T^{0.5}$$

$$\beta_3 = 0.553833 + 0.028829\ (P_{200}\ f^{-0.1703}) - 0\ .03476Va + 0.070377\lambda + 0.931757f^{-0.02774}$$

$$\beta_4 = 0\ .483Vb$$

$$\beta_5 = 1.3 + 0\ .49825 \log f$$

$\beta_1$toβ_5 are temporary constants in the formula,

f load frequency in Hz,

T temperature in °F,

P_{200} percentage by weight of aggregate passing through a No. 200 sieve,

Va volume of air void in %,

λ asphalt viscosity at 70°F in 10^6 poise, and

Vb volume of bitumen in %.

If sufficient viscosity data are not available to estimate λ at 70°F, the following equation could be used

$$\lambda = 29{,}508\ .2\ (P_{77}°F)^{-2.1939} \tag{4.1}$$

Where P_{77}°F is the penetration at 77°F (25°C)

Mean pavement temperature at specified depth is given by the following equation,

$$M_p = M_a\left(1+\frac{1}{z+4}\right)-\frac{34}{z+4}+\ 6 \tag{4.2}$$

Z = depth below the pavement surface in inches

M_p= mean monthly pavement temperature

M_a= mean monthly air temperature

The temperature on the upper 1/3 of the pavement layer is used. The climate of Ethiopia varies according to elevation with an annual average air temperature range of 16 ^{o} C -27 ^{o} C. for example monthly air temperature of Jimma.
As we can see from the appendix, the mean monthly temperature varies from 16 (°C)-24 (°C) in these two years (2018 & 2019). Using MS-office excel and the above formula for mean monthly pavement temperature, the MMPT at 1/3rd of the 15cm AC is predicted as given below.

Mean Monthly Air Temperature T (°C)	Mean Monthly Pavement Temperature T (°C)	Mean Monthly Pavement Temperature T (°F)
16	21.83	71.29
18	24.16	75.49
20	26.50	79.70
22	28.83	83.90
24	31.17	88.10

Using a standard material of AC, which has constituents as shown in Table below, and

Vb	Va	p200	f	Penn @ 77
11.0	5.0	4.0	8.0	70.0

Where

f load frequency in Hz, which is equivalent to a speed of 35-40 mph

P_{200} percentage by weight of aggregate passing through a No. 200 sieve,

Va volume of air void in %,

Vb volume of bitumen in %.

Penn @ 77is the penetration at 77°F (25°C)

Using Microsoft excels and AI (Hwang and Witczak (1979)) equations, the modulus of AC is determined as in Table 4.10.

Table 4. 10 Predicted modulus using Hwang and Witczak equation

Temperature		Dynamic Modulus	
T (°F)	T (°C)	PSI	MPa
71.3	16	592,483.76	4085
75.5	18	493,395.04	3402
79.7	20	407,683.35	2811
83.9	22	334,277.15	2305
88.1	24	272,012.35	1876

The analysis result is presented in Table 4.11 and Figure 4.3. As Mean Monthly Air Temperature T (°C) increases modulus of asphalt layer decreases, which in turn results in increase in critical pavement responses as was expected.

Table 4. 11 Effect of Temperature on Pavement response and performance

Mean Monthly Air Temperature T (°C)	At bottom of AC Layer (15 cm)			At top of Subgrade (65.0001cm depth)		
	Maximum Horizontal strain	Allowable Load repetition	Damage Ratio	Maximum Vertical Strain	Allowable Load repetition	Damage Ratio
16	-1.47E-04	9.20E+06	1.09E-07	2.96E-04	1.37E+07	7.32E-08
18	-1.61E-04	8.15E+06	1.23E-07	3.08E-04	1.17E+07	8.52E-08
20	-1.76E-04	7.38E+06	1.36E-07	3.19E-04	1.01E+07	9.89E-08
22	-1.92E-04	6.88E+06	1.45E-07	3.31E-04	8.75E+06	1.14E-07
24	-2.08E-04	6.64E+06	1.51E-07	3.43E-04	7.59E+06	1.32E-07

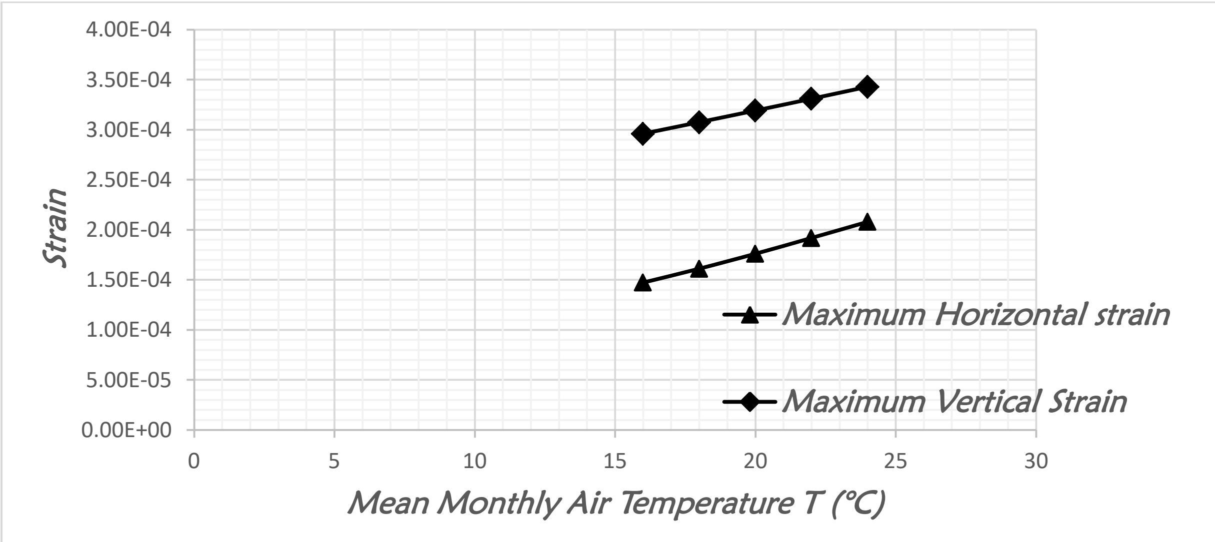


Figure 4. 3 Effect of temperature on pavement response

4.1.7 Traffic load

a. Axle load magnitude

A single axle with Single wheel load of 2.5-11.5 tone is reported on Jimma weight station, which is equivalent to 24.5-112.8 KN. For purpose of this sensitivity analysis 50,60,70,80,100 KN are taken.

As can be shown in Figure 4.4 EALF for fatigue cracking and EALF for deformation are overlapped but EALF from 4th power rule is greater than both from EALF for fatigue cracking and EALF for deformation especially on higher axle loads. But for axle loads below the standard axle, it is relatively lower and insignificant. Therefore, using 4th power rule is conservative for higher axle loads and may underestimate for lower axle loads.

Table 4. 12 Effect of axle load magnitude on pavement response and performance

Axle Load (kN)	At bottom of AC Layer (15 cm)				At top of Subgrade (65.0001cm depth)				EALF (4thpower rule)
	Maximum Horizontal strain	Allowable Load repetition	Damage Ratio	EALF(for fatigue cracking)	Maximum Vertical Strain	Allowable Load repetition	Damage Ratio	EALF (for deformation)	
50	-2.02E-04	3.35E+06	2.99E-07	0.17	3.93E-04	4.41E+06	2.27E-07	0.16	0.12
60	-2.29E-04	1.76E+06	5.70E-07	0.32	4.70E-04	2.15E+06	4.65E-07	0.32	0.27
70	-2.59E-04	9.52E+05	1.05E-06	0.58	5.46E-04	1.18E+06	8.48E-07	0.59	0.55
80	-2.89E-04	5.55E+05	1.80E-06	1.00	6.23E-04	6.96E+05	1.44E-06	1.00	1.00
100	-3.45E-04	2.29E+05	4.38E-06	2.43	7.74E-04	2.92E+05	3.43E-06	2.39	2.73
120	-3.96E-04	1.15E+05	8.72E-06	4.84	9.23E-04	1.45E+05	6.91E-06	4.81	6.20

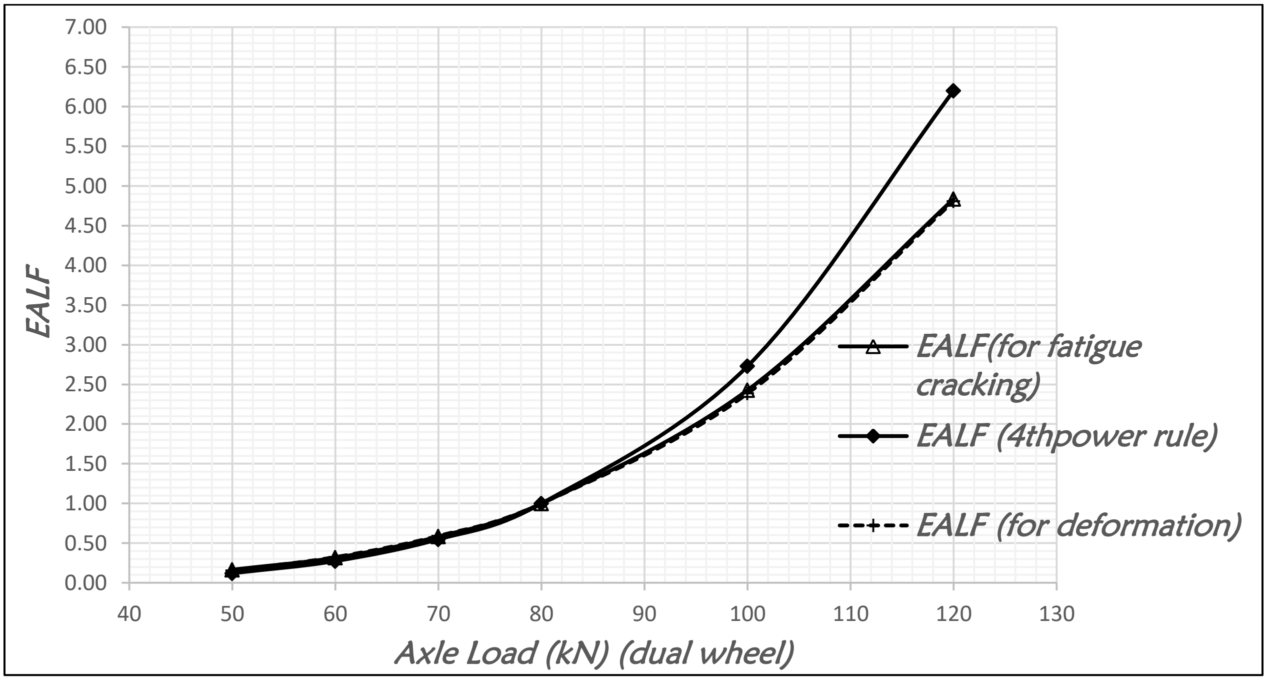


Figure 4. 4 Equivalent axle load factor for fatigue and rutting

b. Axle load configuration

Axle Configuration used in Ethiopian is from ORN40 Modified. The following configuration of axle loads have considered with axle load of 80, 80,160,240 KN respectively.

1. Single wheel single axle 80
2. Dual wheel single axle 80
3. Dual wheel dual axle (tandem) 160
4. Dual wheel tri axle (tridem) 240

Table 4. 13 Axle Configuration

			Radius (m)	Radius (cm)	Dual Spacing (cm)	Axle Spacing (cm)
Single	0.055	0.017	0.1321	13.21	-	-
Dual	0.027	0.009	0.0934	9.34	34.00	-
Tandem	0.027	0.009	0.0934	9.34	34.00	121.00
Tridem	0.027	0.009	0.0934	9.34	34.00	121.00

As can be seen from the results the magnitude and location of maximum pavement responses is different for the different types of axle configurations. For single axle (single wheel) the maximum pavement deflection at a depth of 15 cm is 0.05655cm and located at the center of the wheel while for the single axle (dual wheel) it is 0.05183cm and located between the dual wheels. The location of maximum horizontal strain is at the center of the wheels with magnitude -1.71x10-4 and -2.16x10-4 for the dual wheel and single wheel axles respectively.

For singe axle (both single and dual wheel), the location of maximum pavement deflection at a depth of 65.0001 cm and 15cm is the same.

For tandem axle (160kN) the location of maximum pavement deflection at a depth of 15 cm is the same as that of the dual wheel & single axle (80) (i.e. between the dual wheels) and its magnitude is 0.0754cm. As the magnitude of axle load increases by two fold, the pavement response i.e. deflection increases only by 45.5%.

Maximum horizontal Strain is located at the center of one of the wheels for all cases of axle load configuration and its magnitude is -2.16x10-4, -1.71x10-4 -1.65x10-4and -1.64x10-4 for single axle with single wheel, single axle with dual wheel, tandem and tridem respectively. for single axle with single wheel and single axle with dual wheel the load magnitude is kept constant and the response i.e. horizontal Strain increases by 21.65% due to change only in axle load configuration. When the axle load configuration is changed from single axle with dual wheel to tandem axle horizontal strain decreases by only 3.68% and change in horizontal strain is 0.67% which is insignificant when the axle load configuration is changed from tandem to tridem.

Maximum vertical compressive strain at the top of subgrade is located between the dual wheels for the later three cases of axle load configuration and their magnitude is -3.15x10-4, -3.48x10-4 and -3.80x10-4 respectively for single axle with dual wheel, tandem and tridem axles. It has to be noted here that vertical strain increases while horizontal Strain decreases as axle load configuration changes from single axle with to dual wheel tandem and then tridem axle.

As shown in Table 4.18 single axle with single wheel has 3.18 more damage to asphalt fatigue than the standard axle. Tandem and tridem axles have 1.17 and 1.45 EALF for asphalt fatigue respectively. EALF for permanent deformation is 1.39, 1.68 and 2.82 for single axle with single wheel, tandem and tridem axles respective. Note here that EALF is different for asphalt fatigue and permanent deformation for one type of axle load configuration. Therefore, using a single EALF for flexible pavement is highly empirical.

Table 4. 14 Pavement responses for single Axle with single wheel load configuration

	Pavement response at depth of 65.0001cm (Top of Subgrade)		Pavement response at depth of 15cm (Bottom of AC Layer)	
Radial Coordinate	**Vertical Disp.**	**Vertical Strain**	**Vertical Disp.**	**Radial Strain**
0	0.04128	3.42E-04	0.05655	-2.16E-04
13.21	0.04071	3.23E-04	0.05326	-8.47E-05

Table 4. 15 Pavement responses for single Axle with single wheel load configuration

Coordinate	Point No.	Pavement Response At Depth Of 65.0001cm (Top Of Subgrade)		Pavement Response At Depth Of 15cm (Bottom Of Ac Layer)	
		Vertical Disp.	Vertical Strain	Vertical Disp.	Horizontal Strain
0,0	1	0.03963	2.96E-04	0.05097	-1.71E-04
0,9.34	2	0.04023	3.11E-04	0.05177	-1.65E-04
0,17	3	0.04048	3.15E-04	0.05183	-1.56E-04
60.5,0	4	0.03224	1.26E-04	0.0345	-1.95E-05
60.5,9.34	5	0.03255	1.30E-04	0.0349	-1.66E-05
60.5,17	6	0.03263	1.31E-04	0.03501	-1.57E-05
121,0	7	0.02297	3.14E-05	0.02341	-3.27E-06
121,9.34	8	0.02308	3.21E-05	0.02354	-2.98E-06
121,17	9	0.02311	3.23E-05	0.02358	-2.90E-06

Table 4. 16 Pavement responses for single Axle with single wheel load configuration

Coordinate	**Point No.**	Pavement response at depth of 65.0001cm (Top of Subgrade)		Pavement response at depth of 15cm (Bottom of AC Layer)	
		Vertical Disp.	**Vertical Strain**	**Vertical Disp.**	**Horizontal. Strain**
0,0	1	0.0626	3.27E-04	0.07439	-1.65E-04
0,9.34	2	0.06331	3.44E-04	0.07532	-1.59E-04
0,17	3	0.0636	3.48E-04	0.0754	-1.49E-04
60.5,0	4	0.06448	2.51E-04	0.06899	-2.68E-05
60.5,9.34	5	0.0651	2.60E-04	0.0698	-3.04E-05
60.5,17	6	0.06526	2.63E-04	0.07001	-3.13E-05
121,0	7	0.0626	3.27E-04	0.07439	-1.65E-04
121,9.34	8	0.06331	3.44E-04	0.07532	-1.59E-04
121,17	9	0.0636	3.48E-04	0.0754	-1.49E-04

Table 4. 17 Pavement responses for single Axle with single wheel load configuration

Coordinate	Point No.	Pavement response at depth of 65.0001cm (Top of Subgrade)		Pavement response at depth of 15cm (Bottom of AC Layer)	
		Vertical Disp.	Vertical Strain	Horizontal. Strain	Vertical Disp.
0,0	1	0.07512	3.25E-04	-1.64E-04	0.08682
0,9.34	2	0.07586	3.41E-04	-1.58E-04	0.08777
0,17	3	0.07615	3.45E-04	-1.48E-04	0.08787
60.5,0	4	0.08114	2.56E-04	-2.73E-05	0.08567
60.5,9.34	5	0.08181	2.66E-04	-3.10E-05	0.08654
60.5,17	6	0.08199	2.68E-04	-3.19E-05	0.08676
121,0	7	0.08556	3.59E-04	-1.58E-04	0.0978
121,9.34	8	0.08639	3.76E-04	-1.52E-04	0.09886
121,17	9	0.08671	3.80E-04	-1.43E-04	0.09898

Table 4. 18 Damage comparison of axle load configuration types

Type	Pavement response and damage ratio at depth of 15cm (Bottom of AC Layer)				Pavement response and damage ratio at depth of 65.0001cm (Top of Subgrade)				
	Horizontal Strain at Bottom of AC Layer	Allowable Load Repetition	Damage Ratio	EALF	Compressive strain at Top of Subgrade	Allowable Load Repetitions	Damage Ratio	Minimum Allowable Load Repetitions	EAL F
Standard Axle	-2.16E-04	2.39E+06	4.18E-07	3.18	3.42E-04	7.67E+06	1.3E-07	2.39E+06	1.38
Standard (but with single wheel)	-1.71E-04	7.61E+06	1.32E-07		3.15E-04	1.06E+07	9.41E-08	7.61E+06	
Primary Pavement responses due to 1st axle	-1.65E-04	9.19E+06	1.09E-07		3.48E-04	7.20E+06	1.39E-07	6.34E+06	
Differential Pavement responses due to 2nd axle	-1.38E-04	2.23E+07	4.48E-08		2.11E-04	5.30E+07	1.89E-08		
Sum	-3.03E-04	6.51E+06	1.54E-07	1.17	5.59E-04	6.34E+06	1.58E-07		1.68
Primary Pavement responses due to 1st axle	-1.64E-04	9.50E+06	1.05E-07		3.80E-04	5.05E+06	1.98E-07	3.78E+06	
Differential Pavement responses	-1.37E-04	2.32E+07	8.61E-08		2.43E-04	3.00E+07	6.67E-08		
Sum	-3.00E-04	5.23E+06	1.91E-07	1.45	6.23E-04	3.78E+06	2.65E-07		2.82

c. Tire pressure

In mechanistic pavement design, it is important to know tire pressure which is assumed to be equal to contact pressure. The variation of tire or contact pressure is also considered in determination of contact area. As per the tire pressure survey conducted in Jimma city along major federal roads, tire pressure varies from 500-1000 kpa. However, this variation is not for constant axle load. But for purpose of sensitivity analysis a standard axle load of 80kN and circular contact area is used

As shown in Table 4.19 and Figure 4.5, as tire pressure and in turn contact pressure increases, maximum horizontal strain at bottom of asphalt layer increases by 19% and allowable load repetition decreases by 58% over a range of 500-1000kpa contact pressure. But increases by only 1% for max vertical strain which is insignificant. Therefore, increase in tire pressure and in turn contact pressure affects only the asphalt layer and has no relation with deformation of asphalt layer.

Table 4. 19 Effect of tire/contact pressure on pavement response

Tire Pressure	At bottom of AC Layer (15 cm)			At top of Subgrade (65.0001cm depth)		
	Maximum Horizontal strain	Allowable Load repetition	Damage Ratio	Maximum Vertical Strain	Allowable Load repetition	Damage Ratio
500	-1.54E-04	1.28E+07	7.79E-08	3.13E-04	1.09E+07	9.17E-08
600	-1.62E-04	9.84E+06	1.02E-07	3.14E-04	1.08E+07	9.29E-08
700	-1.69E-04	8.00E+06	1.25E-07	3.15E-04	1.06E+07	9.40E-08
800	-1.75E-04	6.83E+06	1.47E-07	3.15E-04	1.06E+07	9.43E-08
900	-1.79E-04	5.98E+06	1.67E-07	3.16E-04	1.06E+07	9.48E-08
1000	-1.83E-04	5.35E+06	1.87E-07	3.16E-04	1.05E+07	9.53E-08

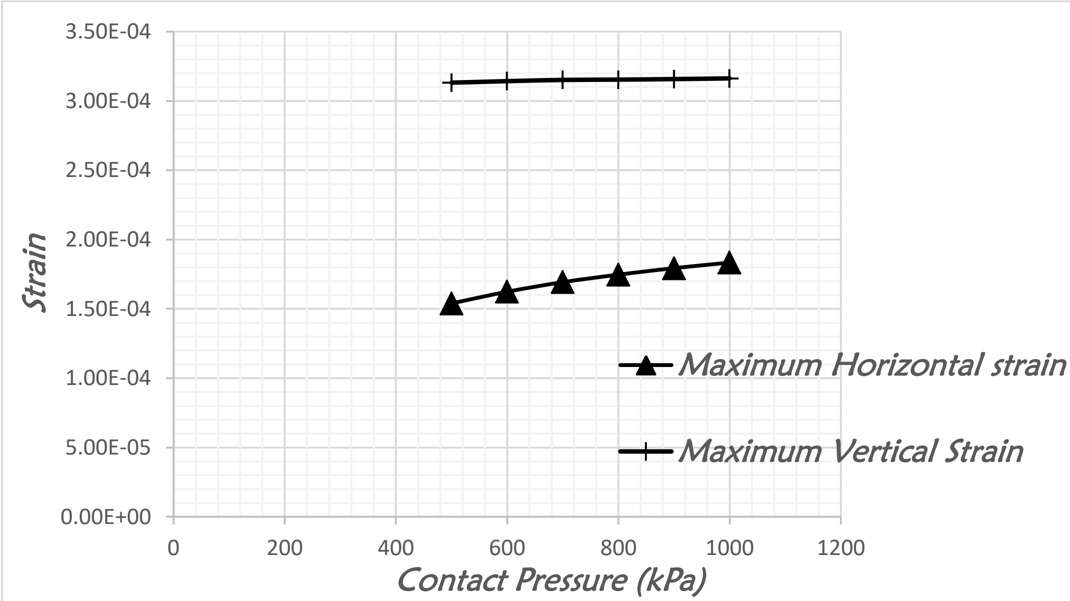


Figure 4. 5 Effect of contact pressure on critical pavement responses

4.1.8 Shape of Tire Print Area/Contact Area

Representing the actual contact area between tires and the pavement surface is very difficult. Therefore, in pavement design, Contact area is approximated by different shapes for simplicity. Shape of a contact area, which is relatively accurate approximation, is represented by two semi-circles and a rectangle combined together with a length L and width 0.6L.

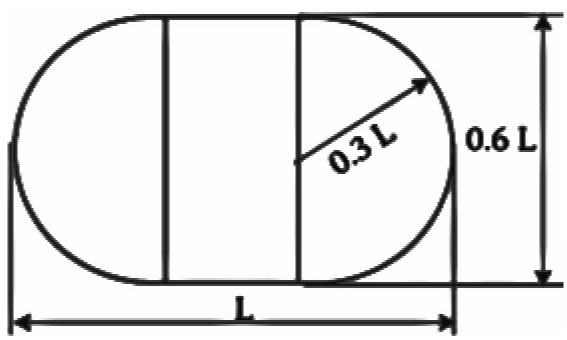


Figure 4. 6 Approximate tire contact area

$$Ac= \pi\ (0.6L)^2+0.6L*0.4L=0.5227L^2 \tag{4-2}$$

$$L=\left(\frac{Ac}{0.5227}\right)^{\frac{1}{2}} \tag{4-3}$$

Where Ac is contact area which is obtained by dividing the load on each tire to contact pressure by assuming that contact pressure and tire pressure/inflation pressure are equal. The following are, therefore, alternative tire imprint areas as shown in Figure 4.7.

1. Two semi-circles and one rectangle: - approximate tire contact area

2. Rectangular: - rarely used and
3. Circular: -the most commonly used in asphalt pavement design

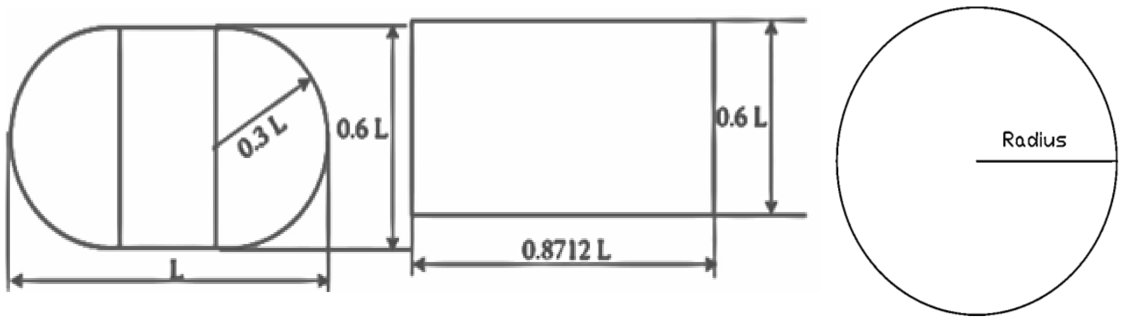


Figure 4. 7 Alternative tire imprint areas

The 1st two shapes cann't be modeled in KENLAYER and therefore here both shapes has been modeled in abaqus and 3D Move and analysis has been undertaken. Properties of pavement layers used in this analysis are as given in Table 4.20 below

Table 4. 20 Properties of pavement layers used for analysis of effect of contact area

AC layer	Modulus (MPa)	Poisson's Ratio (μ or ν)
AC layer	3000	0.3
Base course	300	0.35
Subbase	175	0.4
Subgrade	28	0.45

Pavement Contact pressure and the pavement structure used here is the same as in section 4.1.1. all pavement responses at each level of the pavement structure are maximum for the circular intermediate for semicircular and relatively low for rectangular area for 3D-Move analysis. The same is true for ABAQUS analysis. However, there are Some inconsistencies at subgrade level i.e. the vertical compressive strain is maximum for rectangular area intermediate for semicircular area and low for the case of circular contact area. The effect is more pronounced for tensile strain at bottom of AC with the value at the circular shape is greater than that of the rectangular and semicircular by 10.71% and 9.67% respectively. Inversely, the rate of stress distribution is low for circular, intermediate for semicircular and high for rectangular. Close investigation of the numbers reveals that using the circular contact area will overestimate pavement responses and in turn underestimates pavements life prediction which results in conservative design. The reverse is true for the rectangular one. The semicircular contact area is intermediate in all respects

Table 4. 21 Effect of contact area on pavement response

Pavement response	3D-Move Analysis_V2.1			ABAQUS		
	Shapes of contact area			Shapes of contact area		
	S-1	S-2	S-3	S-1	S-2	S-3
Horizontal tensile strain at bottom of AC (@ center of one Wheel) (in 10^{-6})	-155.27	-153.81	-170.28	114.51	91.41	125.49
Horizontal tensile strain at bottom of AC (b/n dual wheels)(in 10^{-6})	-143.52	-142.34	-156.92	104.40	76.59	113.86
Vertical compressive strain on top of subgrade (@ center of one Wheel) (in 10^{-6})	318.84	318.63	319.76	-205.45	-126.77	-202.35
Vertical compressive strain on top of subgrade (b/n dual wheels)(in 10^{-6})	339.77	339.54	340.70	-214.53	-130.64	-211.94
Deflection (@ center of one Wheel) (10^{-6}m)	594.45	594.06	596.93	-516.00	-390.60	-539.68
Deflection (b/n dual wheels) (10^{-6}m)	584.18	583.89	587.74	-510.15	385.91	-530.47
Stress Just below Top Surface	705.00	705.00	699.98	701.63	716.05	689.32
Stress Just on Top of Subgrade	95.97	95.92	96.20	5.99	5.17	5.92
Rate of Stress distribution	9.37	9.37	9.29	10.70	10.94	10.51

4.2 Sensitivity of rutting and fatigue

In section 4.1, the effect of physical factors on response and in turn pavement performance is studied. However, in this section separate sensitivity analysis of fatigue and rutting to changes in the following properties/parameters of materials has been made.

1. Thickness of bituminous layer
2. Modulus of bituminous layer
3. Thickness of base and subbase
4. Modulus of base, subbase or subgrade

4.2.1 Thickness of bituminous layer

Sensitivity analysis of fatigue cracking and permanent deformation to changes of Thickness of bituminous layer is presented in Table 4.22 and Figure 4.8. At lower thickness of AC layer the horizontal strain is compressive i.e. it is positive and as thickness increased horizontal strain decreases in magnitude from a high positive value at very low asphalt thickness to low positive values until it becomes zero at about 1.98 cm AC thickness. After then, horizontal strain (tensile) starts to increase in magnitude as asphalt thickness increases. As shown in Figure 4.18 and 4.9, in a range of 2-6.3 cm thickness, the greater the depth of asphalt, the greater the magnitude of tensile strain induced at its underside. Further increases in thickness, horizontal strain declines in magnitude again. At this point the AC layer starts to act structurally and distributes load to other sub layers. A peak strain level was reached at 6.3cm which confirms as stated in literatures which are in the range of 40–80 mm for highway traffic loading. Further increases in asphalt thickness reduce the flexure of the structure and the resulting strain in the asphalt. As shown in Figure 4.8 and 4.9, there are two asphalt thickness which results in the same micro strain and in turn the same theoretical fatigue performance. From 6.5cm onwards vertical strain at top of subgrade and horizontal strain at bottom of strain decreases similarly. But increase in allowable repetition of standard axle loads is rapid for the fatigue cracking than for deformation/rutting. Allowable repetition of standard axle loads was increased from $3.17x10^{05}$ to $1.35x10^{07}$ and $32.48x10^{06}$ to $5.45x10^{07}$ for fatigue cracking and deformation/rutting respectively.

Table 4. 22 Sensitivity analysis of fatigue cracking and deformation/rutting to change in thickness of bituminous layer

H(cm)	Et	Nt	ec	Nr
4	2.59E-04	9.53E+05	4.59E-04	1.20E+06
5	3.06E-04	4.12E+05	4.36E-04	1.52E+06
6	3.23E-04	3.16E+05	4.13E-04	1.94E+06
7	3.23E-04	3.18E+05	3.90E-04	2.48E+06
8	3.13E-04	3.68E+05	3.69E-04	3.19E+06
9	3.00E-04	4.59E+05	3.49E-04	4.10E+06
10	2.85E-04	5.96E+05	3.30E-04	5.27E+06
11	2.69E-04	7.96E+05	3.12E-04	6.77E+06
12	2.53E-04	1.08E+06	2.95E-04	8.67E+06
13	2.37E-04	1.48E+06	2.80E-04	1.11E+07
14	2.23E-04	2.03E+06	2.65E-04	1.41E+07
15	2.09E-04	2.80E+06	2.51E-04	1.78E+07
16	1.96E-04	3.85E+06	2.39E-04	2.25E+07
17	1.84E-04	5.30E+06	2.27E-04	2.83E+07
18	1.73E-04	7.27E+06	2.16E-04	3.53E+07
19	1.62E-04	9.94E+06	2.06E-04	4.40E+07
20	1.52E-04	1.35E+07	1.96E-04	5.45E+07

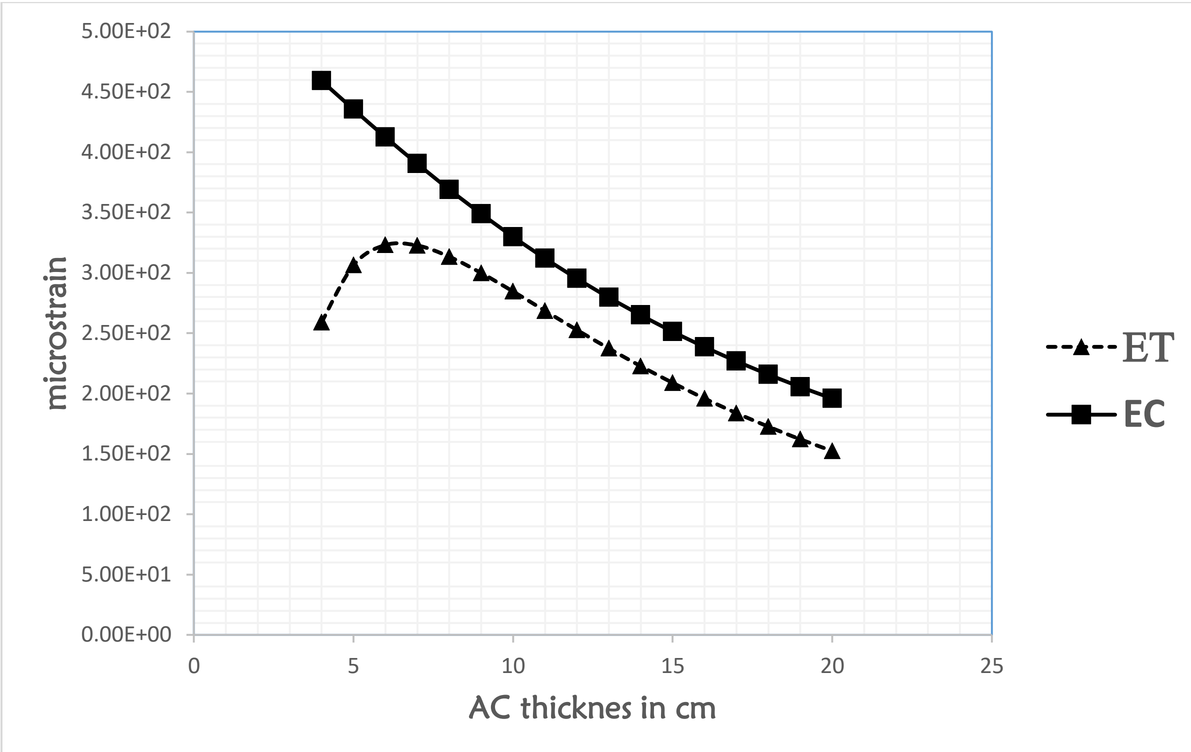


Figure 4. 8 Change critical pavement responses against change in thickness of bituminous layer

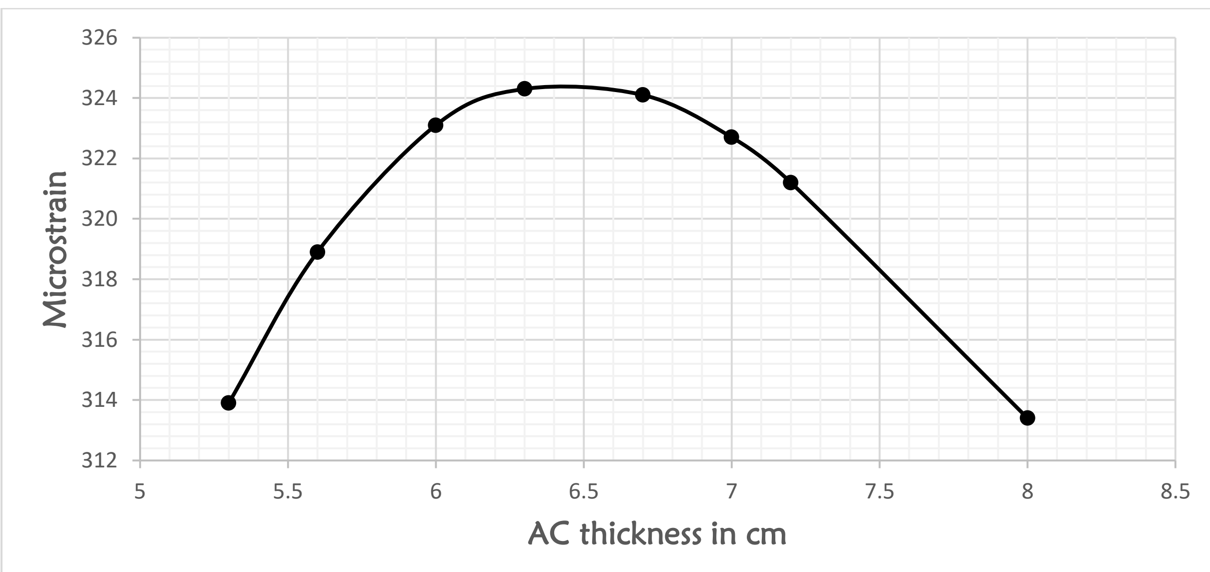


Figure 4. 9 Two theoretical AC thickness for a single pavement response

4.2.2 Modulus of bituminous layer

As shown in Figure 4.10 and Table 4.23, allowable repetition of standard axle loads for deformation/rutting increases from 9.77 x10^{06} to 2.08x10^{07} which is 113% over the range of 1500-3500mpa AC modulus. However, allowable repetition of standard axle loads for fatigue cracking increases only by 64% over the same range of AC modulus.

Table 4. 23 Sensitivity analysis of fatigue cracking and deformation/rutting to change in modulus of bituminous layer

modulus	horizontal strain at bottom of AC	allowable repetition of load (Nc)	Vertical strain on top of subgrade	allowable repetition of load (Nr)
1500	2.80E-04	1.09E+06	2.88E-04	9.77E+06
1800	2.62E-04	1.16E+06	2.78E-04	1.13E+07
2100	2.46E-04	1.25E+06	2.70E-04	1.29E+07
2400	2.32E-04	1.35E+06	2.63E-04	1.45E+07
2700	2.20E-04	1.46E+06	2.57E-04	1.61E+07
3000	2.09E-04	1.57E+06	2.51E-04	1.78E+07
3300	1.99E-04	1.70E+06	2.46E-04	1.96E+07
3500	1.93E-04	1.78E+06	2.43E-04	2.08E+07

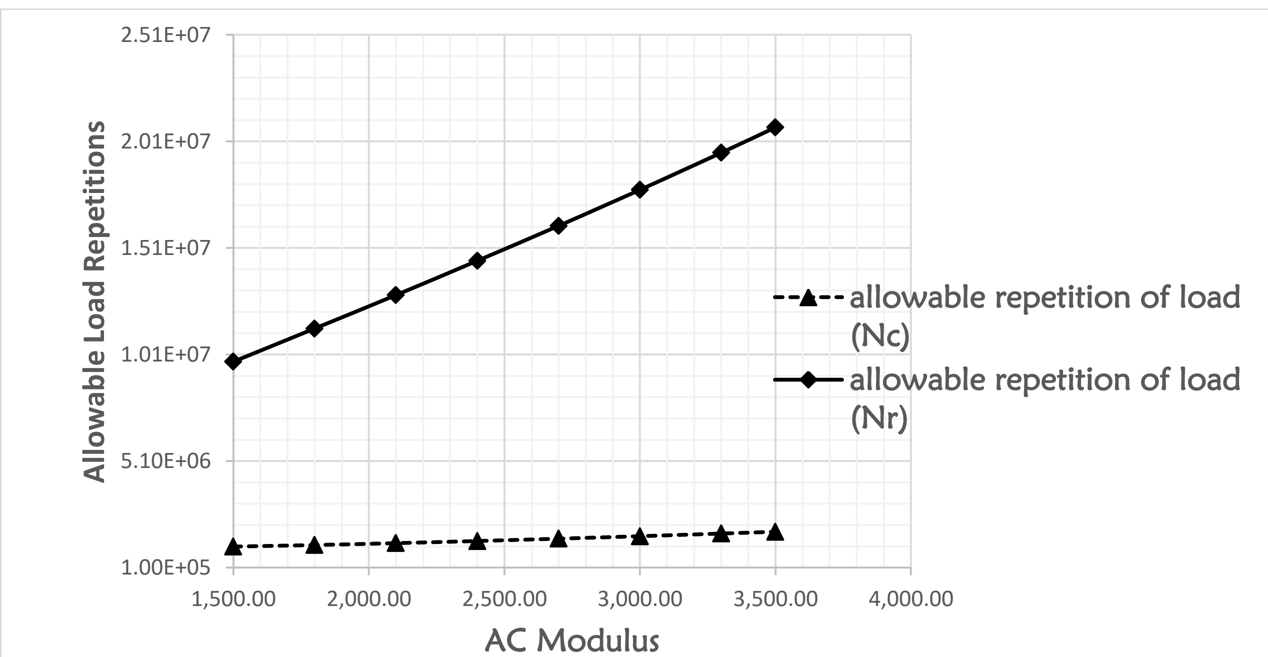


Figure 4. 10 Sensitivity analysis of rutting and fatigue cracking to changes in Ac modulus

4.2.3 Modulus of base course layer

Tensile strain at bottom of Asphalt layer decreases rapidly as modulus of base increases when compared to compressive strain at top of subgrade. It is illustrated in Table 4.24 and Figure 4.11. Allowable load repetition for fatigue cracking (N_t) increases from $5.21x10^{05}$ to $3.87x10^{06}$ which is 7.44 times greater. However, Allowable load repetition for rutting (Nr) increases only by 75%. This indicates that fatigue cracking is more sensitive to changes in Base modulus than rutting.

Table 4. 24 Sensitivity analysis of rutting and fatigue cracking to changes in modulus of base course layer

E(MPa)	εt	Nt	εc	Nr
100	-2.92E-04	5.21E+05	2.78E-04	1.14E+07
150	-2.64E-04	8.72E+05	2.72E-04	1.25E+07
200	-2.42E-04	1.34E+06	2.65E-04	1.40E+07
250	-2.24E-04	1.97E+06	2.58E-04	1.58E+07
300	-2.09E-04	2.80E+06	2.51E-04	1.78E+07
350	-1.96E-04	3.87E+06	2.45E-04	2.00E+07

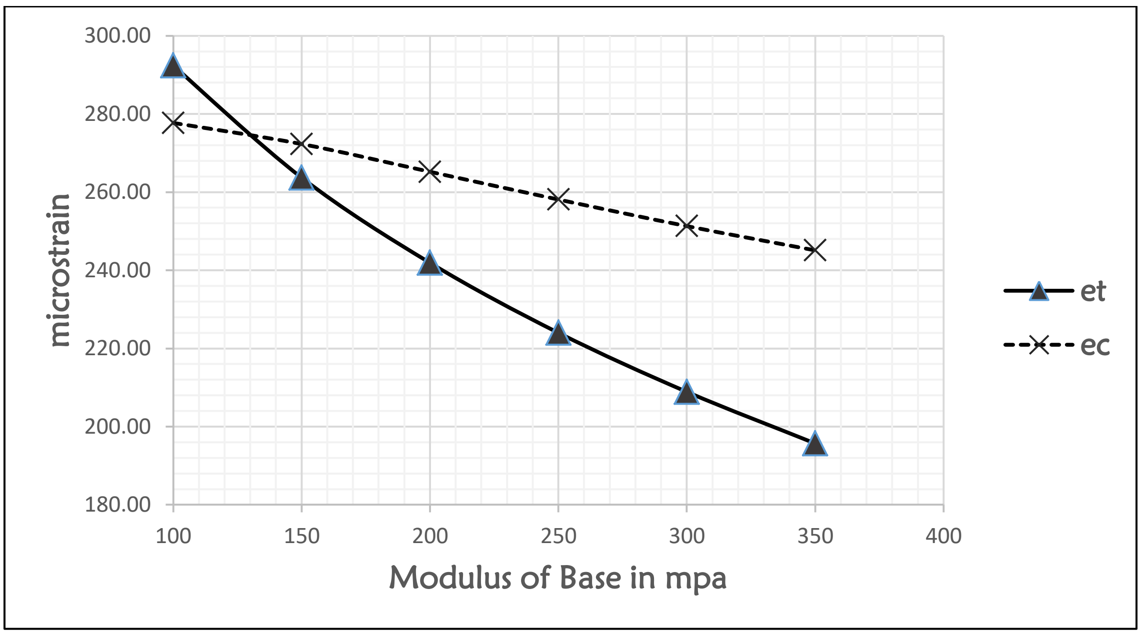


Figure 4. 11 Sensitivity analysis of rutting and fatigue cracking to changes in modulus of base course layer

4.2.4 Thickness of Base Course Layer

As shown in Figure 4.12 and Table 4.25, horizontal strain at bottom of asphalt layer(et) decreases slowly as base thickness increases from 10-30cm. Compressive strain at top of subgrade increases rapidly (from $3.25x10^{-04}$ to $3.25x10^{-04}$) over the same range of base thickness. Allowable load repetition for fatigue cracking (N_t) increases from $1.99x10^{06}$ to $3.31\ x10^{06}$ which is 66% increase. However, Allowable load repetition for rutting (Nr) increases more than 8.9 times. This indicates that rutting is more sensitive to changes in Base thickness than fatigue cracking.

Table 4. 25 Sensitivity analysis of rutting and fatigue cracking to changes in thickness of base course layer

h(cm)	εt	Nt	εc	Nr
10	-2.24E-04	1.99E+06	3.25E-04	5.66E+06
15	-2.15E-04	2.44E+06	2.85E-04	1.02E+07
20	-2.09E-04	2.80E+06	2.51E-04	1.78E+07
25	-2.05E-04	3.08E+06	2.23E-04	3.04E+07
30	-2.02E-04	3.31E+06	1.99E-04	5.04E+07

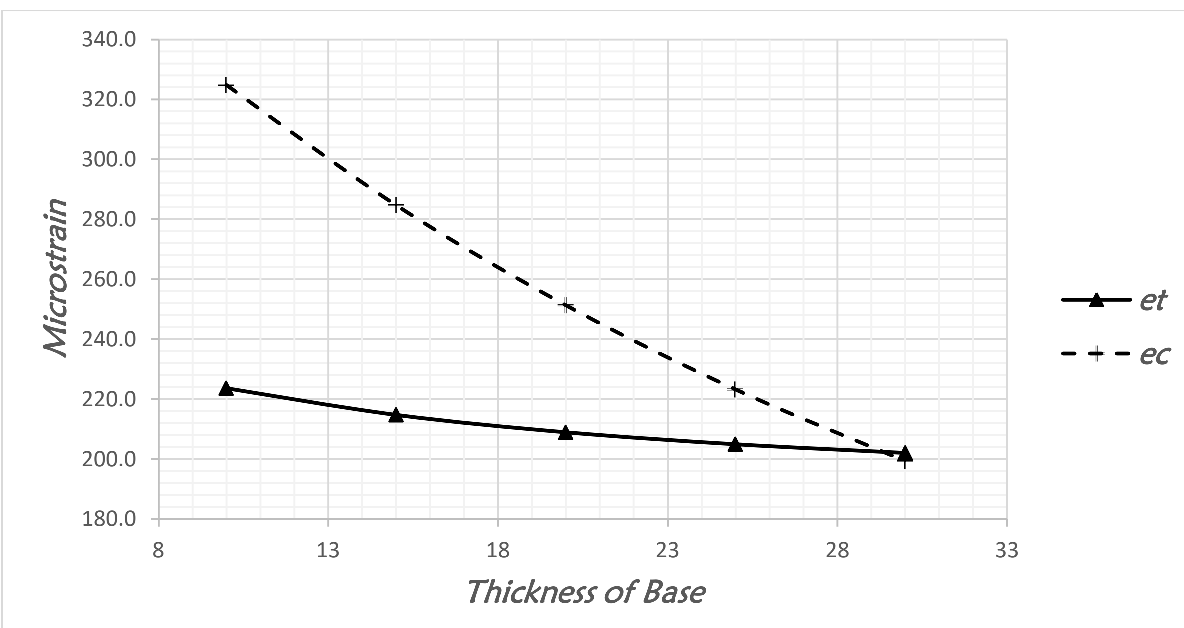


Figure 4. 12 Sensitivity of rutting and fatigue cracking to changes in thickness of base layer

4.2.5 Thickness of subbase layer

As shown in Figure 4.13 and Table 4.26, change in horizontal strain at bottom of asphalt layer (ε_t) and Allowable load repetition for fatigue cracking (N_t) is insignificant. However, Allowable load repetition for rutting (Nr) increases more than 14 times over the range of 10-35cm subbase thickness. This indicates that rutting is more sensitive to changes in Subbase thickness than fatigue cracking. It has to be noted here that rutting is more sensitive to changes in subbase thickness than base course thickness.

Table 4. 26 Sensitivity of rutting and fatigue cracking to changes in thickness of subbase layer

h(cm)	εt	Nt	εc	Nr
10	-2.16E-04	2.35E+06	4.12E-04	1.96E+06
15	-2.14E-04	2.50E+06	3.60E-04	3.58E+06
20	-2.12E-04	2.62E+06	3.17E-04	6.31E+06
25	-2.10E-04	2.72E+06	2.81E-04	1.08E+07
30	-2.09E-04	2.80E+06	2.51E-04	1.78E+07
35	-2.08E-04	2.86E+06	2.26E-04	2.88E+07

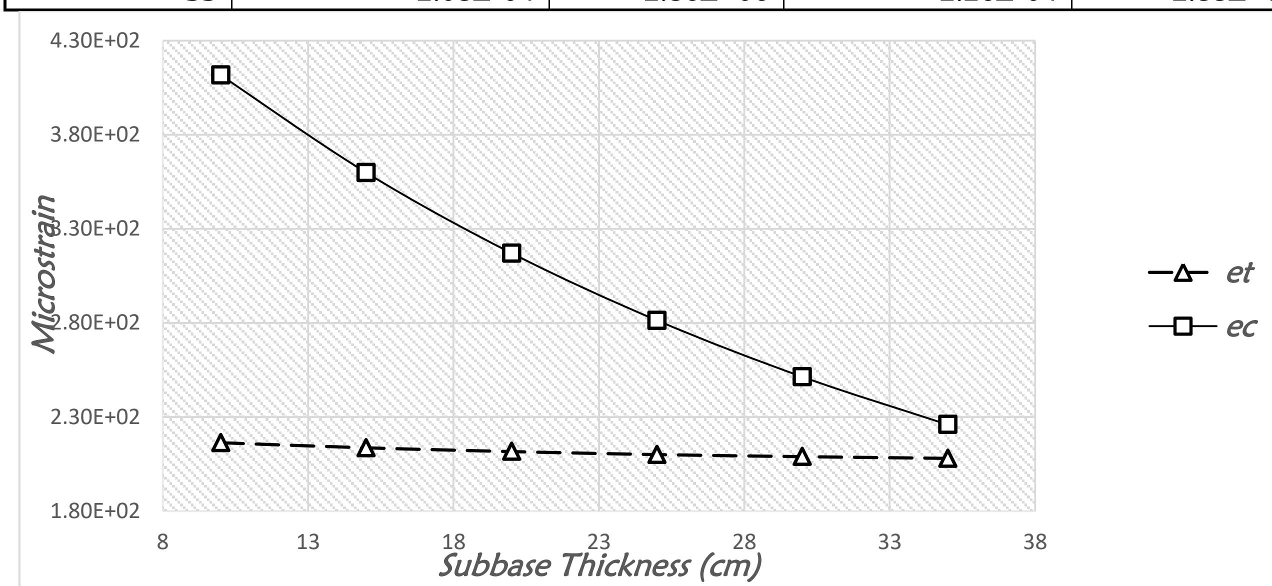


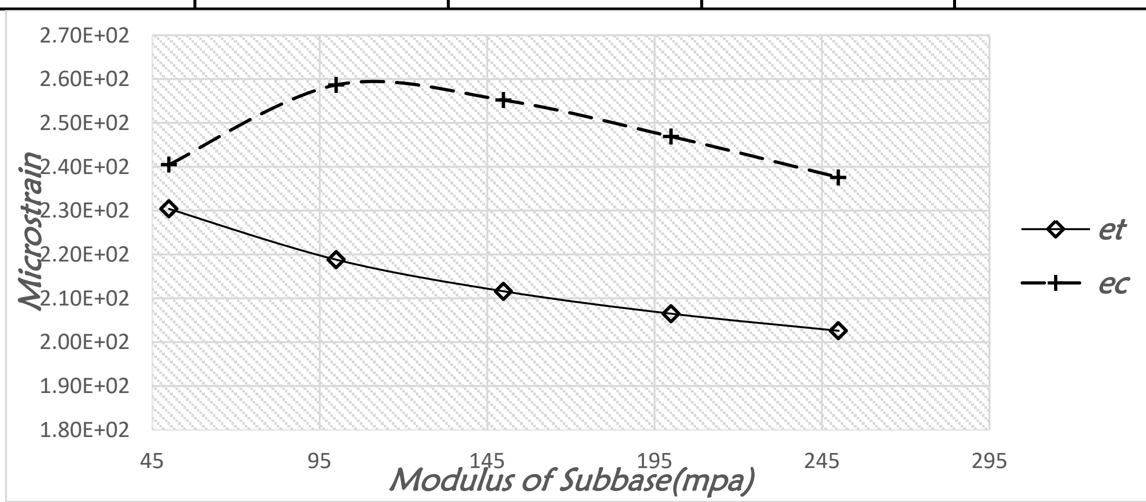
Figure 4. 13 Sensitivity of rutting and fatigue cracking to changes in thickness of subbase layer

4.2.6 Modulus of Subbase layer

At low value of subbase modulus (when modulus of subbase is less than modulus of subgrade), vertical compressive strain at top of subgrade (ε_c) increases and allowable load repetition for deformation/rutting decreases. After a modulus value of about 105 MPa, the reverse is true i.e. Vertical compressive strain at top of subgrade (ε_c) decreases and allowable load repetition for deformation/rutting increases. Horizontal strain at bottom of asphalt layer (ε_t) decreases and allowable load repetition for fatigue cracking increases over the whole range of subbase modulus.

Table 4. 27 Sensitivity of rutting and fatigue cracking to changes in thickness of subbase layer

E (MPa)	εt	Nt	εc	Nr
50	-2.30E-04	1.71E+06	2.41E-04	2.17E+07
100	-2.19E-04	2.22E+06	2.59E-04	1.57E+07
150	-2.12E-04	2.62E+06	2.55E-04	1.66E+07
200	-2.07E-04	2.96E+06	2.47E-04	1.93E+07
250	-2.03E-04	3.25E+06	2.38E-04	2.29E+07



4.2.7 Modulus and CBR of Subgrade layer

Over the range of 28-175 MPa, horizontal strain at bottom of asphalt layer (ε_t) decreases only by 3% and allowable load repetition for fatigue cracking increases by 19%. Vertical compressive strain at top of subgrade (ε_c) decreases by 61% and allowable load repetition for deformation/rutting increases about 66 times over the same range of subgrade modulus. As shown in Figure 4.14 and Table 4.28 the trend of response to changes in CBR of subgrade is the same as explained for change in subgrade modulus. Among all parameters, deformation/rutting is more sensitive to subgrade modulus and CBR.

Table 4. 28 Sensitivity of rutting and fatigue cracking to changes in modulus of subgrade

E (MPa)	εt	Nt	εc	Nr
28	-2.13E-04	2.55E+06	3.80E-04	2.79E+06
37	-2.12E-04	2.62E+06	3.42E-04	4.49E+06
53	-2.10E-04	2.71E+06	2.93E-04	8.92E+06
73	-2.09E-04	2.80E+06	2.51E-04	1.78E+07
112	-2.07E-04	2.92E+06	1.98E-04	5.15E+07
175	-2.06E-04	3.04E+06	1.49E-04	1.85E+08

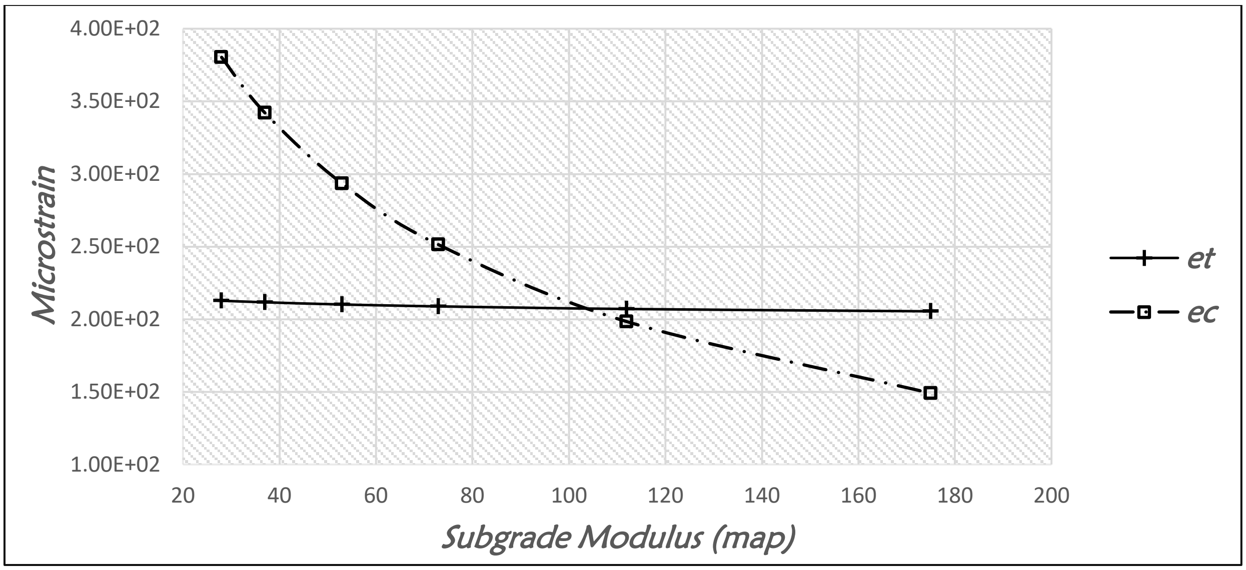


Figure 4. 14 Sensitivity of rutting and fatigue cracking to changes in modulus of Subgrade

Table 4. 29 Sensitivity analysis of rutting and fatigue cracking to changes in subgrade CBR (%)

CBR (%)	εt	Nt	εc	Nr
2	-2.129E-04	2.54E+06	3.85E-04	2.63E+06
5	-2.105E-04	2.69E+06	3.04E-04	7.62E+06
10	-2.087E-04	2.81E+06	2.45E-04	2.02E+07
15	-2.076E-04	2.89E+06	2.12E-04	3.83E+07
20	-2.069E-04	2.94E+06	1.90E-04	6.20E+07
30	-2.059E-04	3.00E+06	1.62E-04	1.28E+08
40	-2.053E-04	3.05E+06	1.42E-04	2.27E+08
50	-2.048E-04	3.09E+06	1.29E-04	3.54E+08
60	-2.045E-04	3.11E+06	1.18E-04	5.22E+08
65	-2.043E-04	3.13E+06	1.14E-04	6.22E+08

4.3 Performance Evaluation of pavement structures designed using ERA guide

As discussed in chapter 2, Pavement performance evaluation or prediction is important in road asset management activities such as in selection of new pavement options and appropriate maintenance and rehabilitation techniques. In doing so, pavement performance models are extremely important. Different approaches have been used in performance prediction of pavements. However, here we have used mechanistic-empirical models which are deterministic (i.e. predicts a single value of dependent variable from pavement performance prediction models) approaches. The following procedure has been used for the performance evaluation of pavement structures designed as per ERA design guideline.

Procedures for performance evaluation

1. Determine layer modulus & Poisson's ratio values
2. Model the pavement structure, which is empirically designed as per ERA guidelines using KENLAYER, a MLE software, and Assign appropriate layer modulus and Poisson's ratio for each pavement structure whose performance is being evaluated for the average climatic condition.
3. Calculate the maximum tensile strain at the bottom of the bituminous layer and the maximum compressive strain at top of subgrade by applying Equivalent standard axle load.
4. For the max strain magnitudes determined in step 3 (for a standard axle load), use empirical-mechanistic models (selected fatigue models) to determine performance of the pavement structure in terms of standard axle load repetitions.
5. Repeat step 4 for rutting (permanent deformation) life under the maximum subgrade compressive strain determined in step 3.
6. Take the minimum of step 4 & 5 as the allowable Repetition of Equivalent standard axle load that the pavement structure will serve and beyond which the pavement structure fails under the respective distress type.
7. Compare the allowable Repetition of Equivalent standard axle load to the No of Design ESA which is expressed as a range in ERA design charts. The design ESA in the design charts is the Repetition of Equivalent standard axle load which the pavement structure under consideration is believed to serve in its service life.

In this section performance of pavement structures in ERA design charts are evaluated using KENLAYER, a mechanistic empirical pavement analysis and design software. The empirical parts of the software are its distress models used, which are manually inputted as appropriate. In doing so distress models, which are deemed appropriate for our particular condition, for rutting and fatigue cracking are used. Austroads criteria and shell 95% criteria are used for fatigue cracking and performance deformation respectively. In ERA design manual Design ESA (106) is in a range as shown in Table 4.30.

Performance evaluation results of ERA catalogue structures are presented in Table 4.31. In this table different abbreviated terms are used in code column to indicate the location of the modeled pavement structure in ERA design charts. For example C1T6S5, is to indicate the location of this pavement structure is in chart C1, traffic class 6 and subgrade class 5.

In this analysis, Allowable Load Repetitions (N_f) is lower than the lower bound of Design ESA (10^6) for Design Charts A1-C1. Allowable Load Repetitions (Nr) for 25 % of Chart C1 catalogue structures is also lower than the lower bound of Design ESA (10^6) and 33% is between the average & the lower bound and 42% are above the upper bound. Chart C2 performs beyond the design period for permanent deformation and 50% for fatigue cracking. 50% of Chart D is falls b/n the average and the lower bound of Design ESA (10^6) for permanent deformation/rutting and all structures perform beyond the Design ESA (106) for fatigue cracking. Among all catalogue structures analyzed here, 54% performs below the design ESA when the Austroads criteria and shell 95% criteria are used for fatigue cracking and performance deformation respectively. Of which 80% are due to fatigue cracking. Therefore, it is necessary for Ethiopia to Adapt mechanistic empirical design approach as Allowable Load Repetitions predicted by KENPAVE is less almost by 71% and 54% from the no of standard design Load Repetitions expected by the ERA method for charts C1 and C2 respectively. It has to be noted here that performance evaluation is presented only for structures with Ac thickness 10 cm and above i.e. for charts C1 - D.

Table 4. 30 Traffic class and corresponding range of Design ESA (10^6) [**8**]

Traffic class	ESA(10^6)
T1	< 0.3
T2	0.3 – 0.7
T3	0.7 – 1.5
T4	1.5 – 3.0
T5	3.0 – 6.0
T6	6.0 – 10
T7	10 – 17
T8	17 – 30
T9	30 – 50
T10	50 – 80*

Table 4. 31 Performance analysis results of ERA catalogue structures

No.	total thickness (cm)	Code	Minor Horizontal P. Strain at Bottom of AC	Max. Vertical Strain on Top of Subgrade	Allowable Load Repetitions (N_f)	Allowable Load Repetitions (Nr)			Design Load Repetitions (10^6)	Average ESA(106)
			e_t	e_z	N_f	AI	shell 85%	shell 95%		
1	875	C1T6S1	-2.34E-04	3.01E-04	1.59E+06	6.23E+06	2.363E+07	1.279E+07	6.0 – 10	8
2	725	C1T6S2	-2.35E-04	3.47E-04	1.54E+06	3.30E+06	1.333E+07	7.217E+06	6.0 – 10	
3	550	C1T6S3	-2.37E-04	4.07E-04	1.48E+06	1.63E+06	7.084E+06	3.834E+06	6.0 – 10	
4	475	C1T6S4	-2.39E-04	4.14E-04	1.42E+06	1.52E+06	6.623E+06	3.585E+06	6.0 – 10	
5	400	C1T6S5	-2.45E-04	3.92E-04	1.26E+06	1.94E+06	8.258E+06	4.470E+06	6.0 – 10	
6	300	C1T6S6	-2.37E-04	3.72E-04	1.49E+06	2.43E+06	1.011E+07	5.471E+06	6.0 – 10	
8	1000	C1T8S1	-1.63E-04	2.11E-04	9.68E+06	3.06E+07	9.881E+07	5.348E+07	17 - 30	23.5
9	850	C1T8S2	-1.64E-04	2.46E-04	9.28E+06	1.54E+07	5.332E+07	2.886E+07	17 - 30	
10	675	C1T8S3	-1.65E-04	2.71E-04	9.05E+06	9.99E+06	3.613E+07	1.955E+07	17 - 30	
11	575	C1T8S4	-1.66E-04	2.82E-04	8.73E+06	8.36E+06	3.076E+07	1.665E+07	17 - 30	
12	500	C1T8S5	-1.65E-04	2.58E-04	9.02E+06	1.25E+07	4.413E+07	2.388E+07	17 - 30	
13	400	C1T8S6	-1.60E-04	2.42E-04	1.06E+07	1.64E+07	5.647E+07	3.056E+07	17 - 30	
14	800	C2T6S1	-1.95E-04	2.27E-04	3.95E+06	2.21E+07	7.358E+07	3.982E+07	6.0 – 10	8
15	650	C2T6S2	-1.96E-04	2.41E-04	3.86E+06	1.68E+07	5.751E+07	3.113E+07	6.0 – 10	
16	550	C2T6S3	-1.99E-04	2.50E-04	3.56E+06	1.42E+07	4.958E+07	2.684E+07	6.0 – 10	
17	425	C2T6S4	-2.01E-04	2.42E-04	3.42E+06	1.63E+07	5.619E+07	3.041E+07	6.0 – 10	
18	400	C2T6S5	-2.04E-04	2.30E-04	3.13E+06	2.08E+07	6.981E+07	3.778E+07	6.0 – 10	
19	350	C2T6S6	-1.92E-04	2.15E-04	4.26E+06	2.77E+07	9.029E+07	4.887E+07	6.0 – 10	
20	950	C2T8S1	-1.29E-04	1.57E-04	3.07E+07	1.12E+08	3.177E+08	1.719E+08	17 - 30	23.5

21	800	C2T8S2	-1.30E-04	1.62E-04	3.00E+07	9.69E+07	2.789E+08	1.510E+08	17 - 30	
22	650	C2T8S3	-1.33E-04	1.68E-04	2.69E+07	8.38E+07	2.447E+08	1.324E+08	17 - 30	
23	525	C2T8S4	-1.34E-04	1.61E-04	2.62E+07	1.00E+08	2.873E+08	1.555E+08	17 - 30	
24	450	C2T8S5	-1.39E-04	1.80E-04	2.15E+07	6.12E+07	1.844E+08	9.980E+07	17 - 30	
25	400	C2T8S6	-1.30E-04	1.69E-04	2.96E+07	8.19E+07	2.395E+08	1.296E+08	17 - 30	
26	765	D1T7S1	-1.26E-04	2.84E-04	3.46E+07	8.07E+06	2.982E+07	1.614E+07	10 – 17	13.5
27	640	D1T7S2	-1.28E-04	2.99E-04	3.20E+07	8.17E+06	2.421E+07	1.310E+07	10 – 17	
28	540	D1T7S3	-1.29E-04	2.89E-04	3.14E+07	9.55E+06	2.781E+07	1.505E+07	10 – 17	
29	390	D1T7S4	-1.28E-04	3.12E-04	3.29E+07	6.73E+06	2.037E+07	1.102E+07	10 – 17	
30	340	D1T7S5	-1.21E-04	2.67E-04	4.34E+07	1.36E+07	3.829E+07	2.072E+07	10 – 17	
31	220	D1T7S6	-1.25E-04	2.92E-04	3.61E+07	9.07E+06	2.658E+07	1.438E+07	10 – 17	
32	830	D1T9S1	-1.03E-04	2.31E-04	9.85E+07	2.03E+07	6.837E+07	3.700E+07	30 - 50	40
33	705	DT9S2	-1.04E-04	2.39E-04	9.33E+07	1.75E+07	5.966E+07	3.229E+07	30 - 50	
34	605	D1T9S3	-1.03E-04	2.28E-04	9.42E+07	2.15E+07	7.192E+07	3.892E+07	30 - 50	
35	455	DT9S4	-1.02E-04	2.43E-04	1.01E+08	1.62E+07	5.582E+07	3.021E+07	30 - 50	
36	380	D1T9S5	-9.76E-05	2.17E-04	1.26E+08	2.65E+07	8.685E+07	4.701E+07	30 - 50	
37	260	D1T9S6	-9.98E-05	2.27E-04	1.12E+08	2.79E+07	7.255E+07	3.927E+07	30 - 50	

4.4 Economical pavement layer composition and new Design Framework

In this section a new design framework is developed and then an economical section is designed as discussed below.

4.4.1 New design Framework and design of Economical and safe pavement structure

Before selection of an economical pavement layer composition, a new design framework is developed. Various research results have been used in the design method development to flexible pavement design which is mechanistic Empirical. The problem while developing this method was unavailability of database of materials properties and criteria of performance. However, the developed method is based on a simplified approach to the problem. The basis of the method is the Kenlayer computer program for the linear elastic stress analysis of the structure. Database of materials properties in ERA manual and tire pressure survey results conducted for this particular study has been used in the design method development. Austroads and shell 95% reliability has been used as criteria of performance for fatigue cracking and rutting respectively.

The following procedures has been used in the design method development

1. Using KENPAVE and by varying material properties alternatively a database of pavement responses in terms of allowable load repetitions with corresponding material properties has been established.
2. Then using this database, a set of regression models are formulated using CurveExpert pro
3. Using excel VBA user interface, UI is developed
4. A program is written in VBA which integrates the UI and the formulated regression model
5. By Integrating the UI and the formulated regression model the design method is finalized

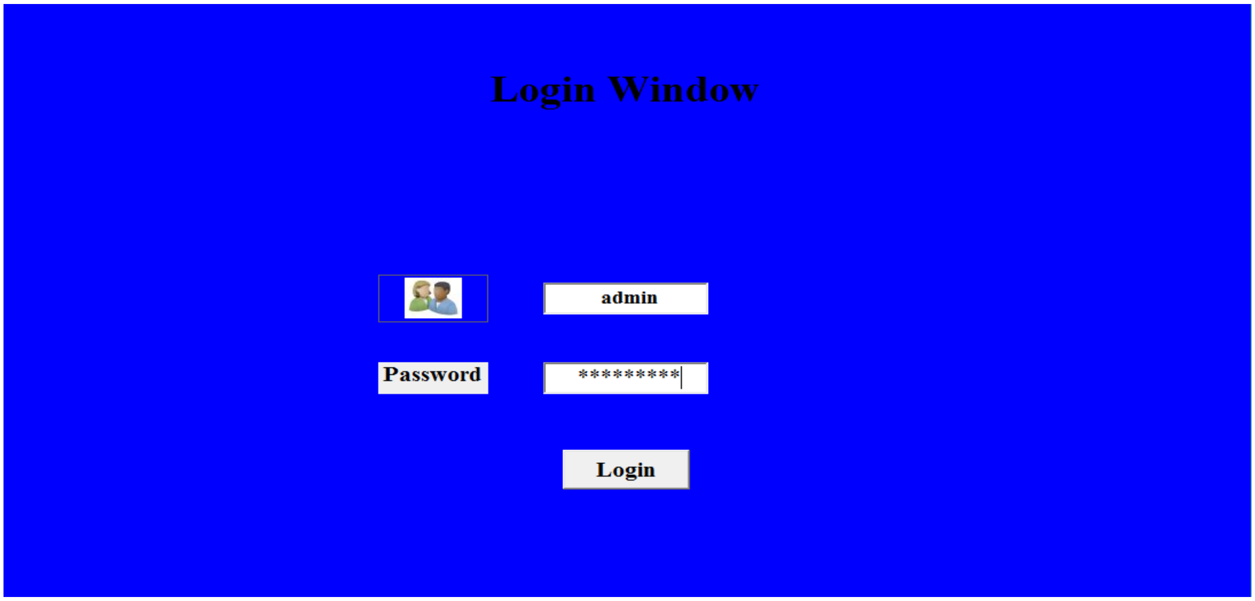


Figure 4. 15 Login window for the design tool

Figure 4. 16 The 1st User interface window

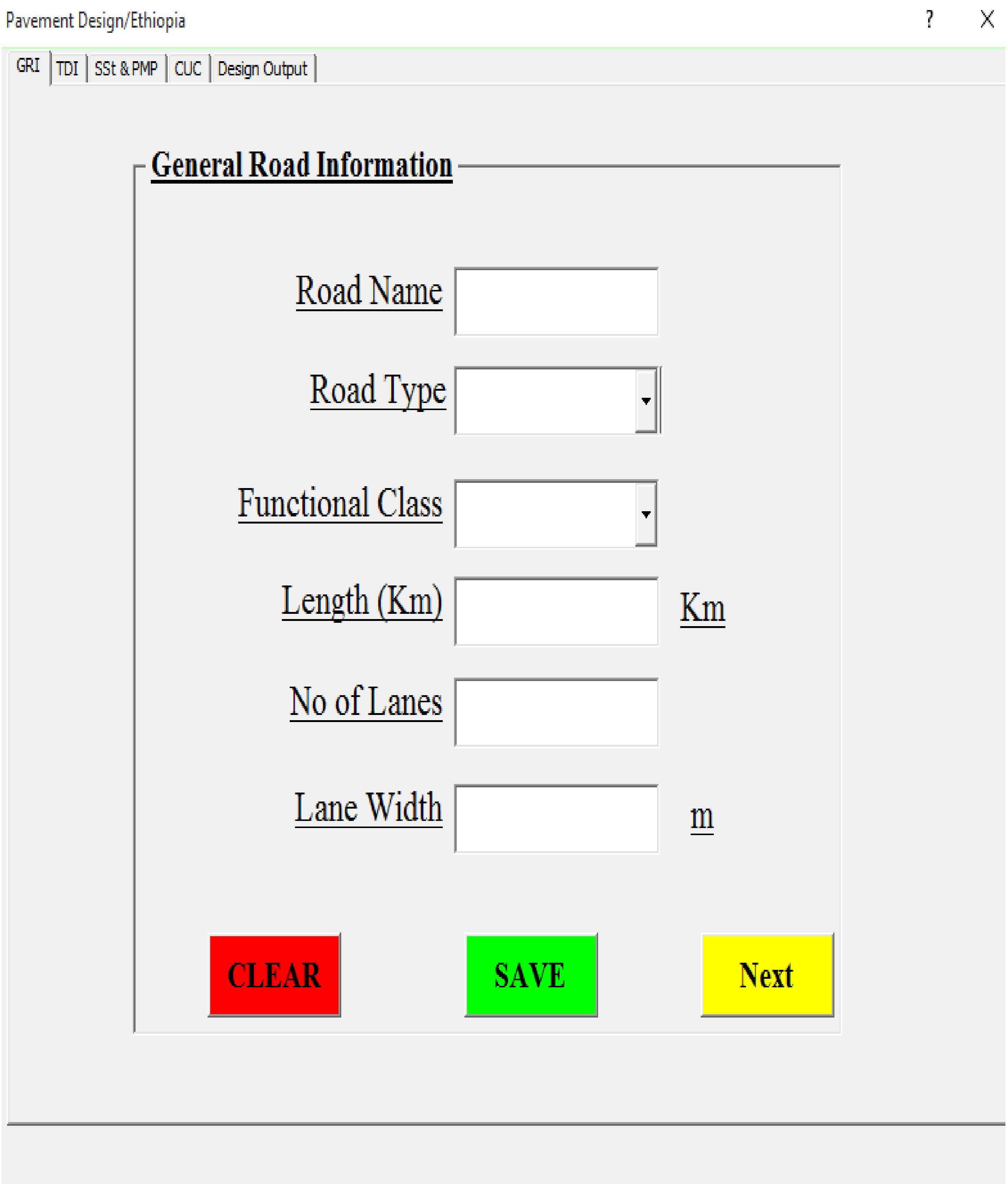


Figure 4. 17 General Road information

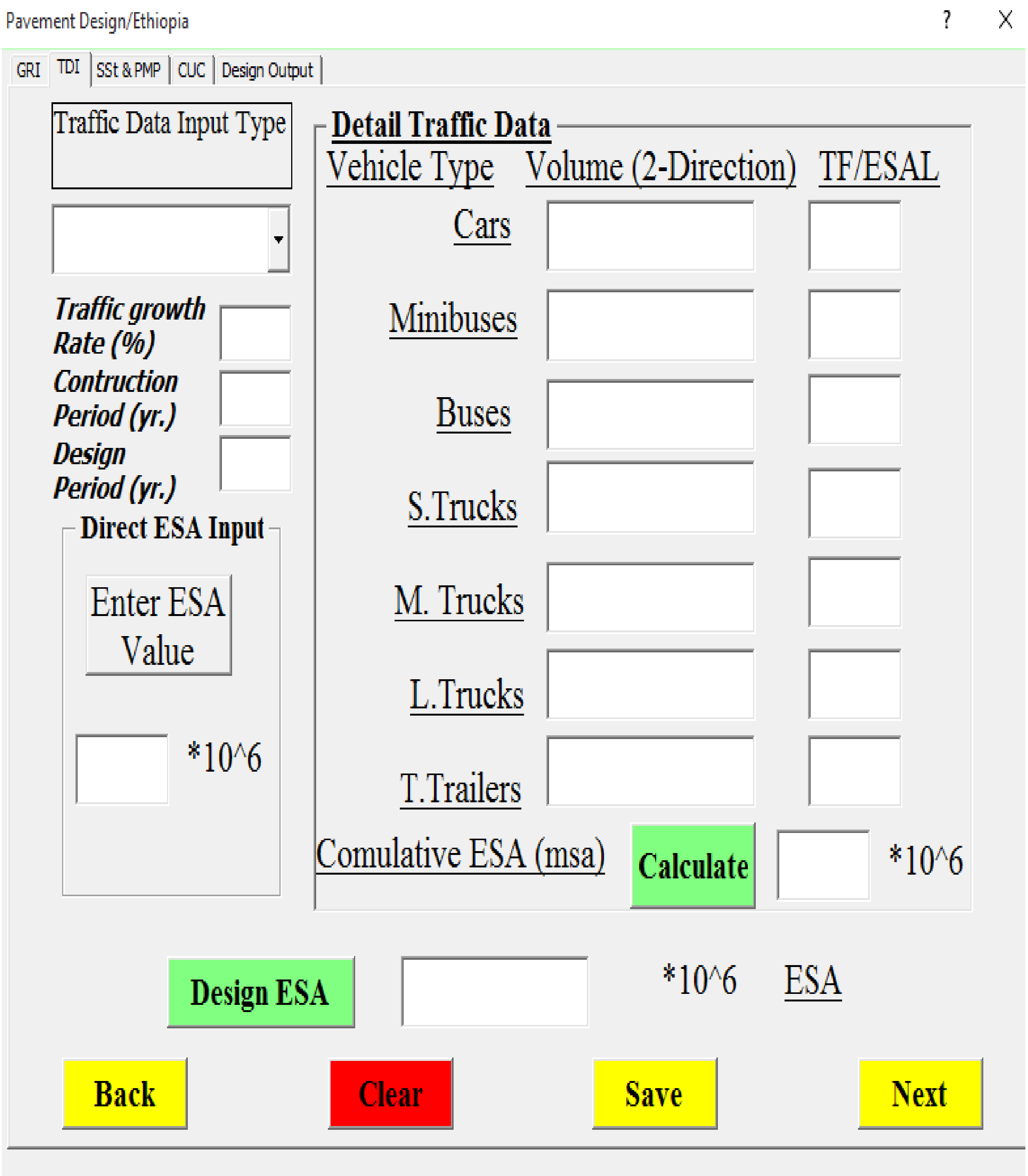


Figure 4. 18 Traffic Data inputs window

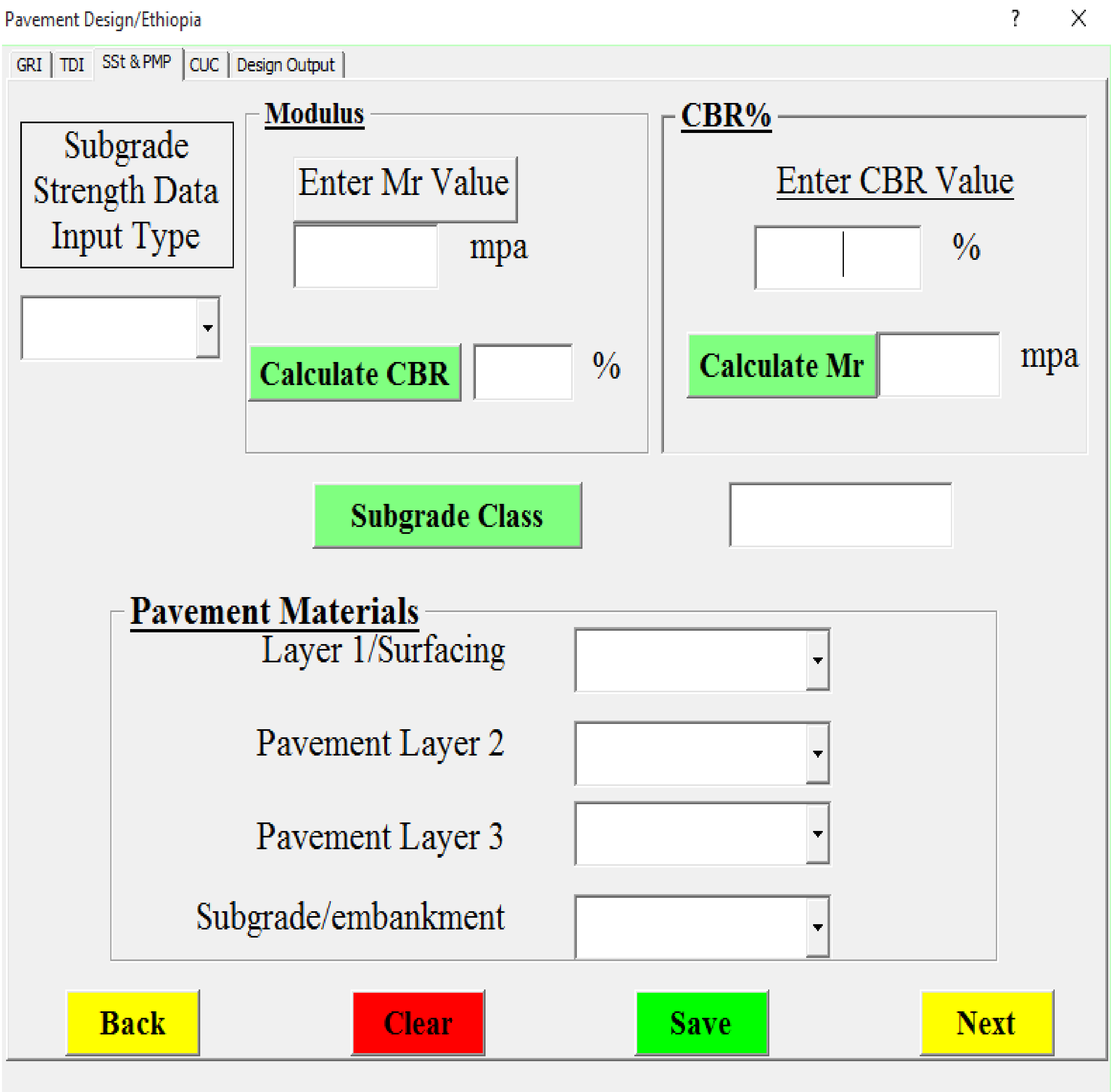


Figure 4. 19 Subgrade Strength and pavement layer material types

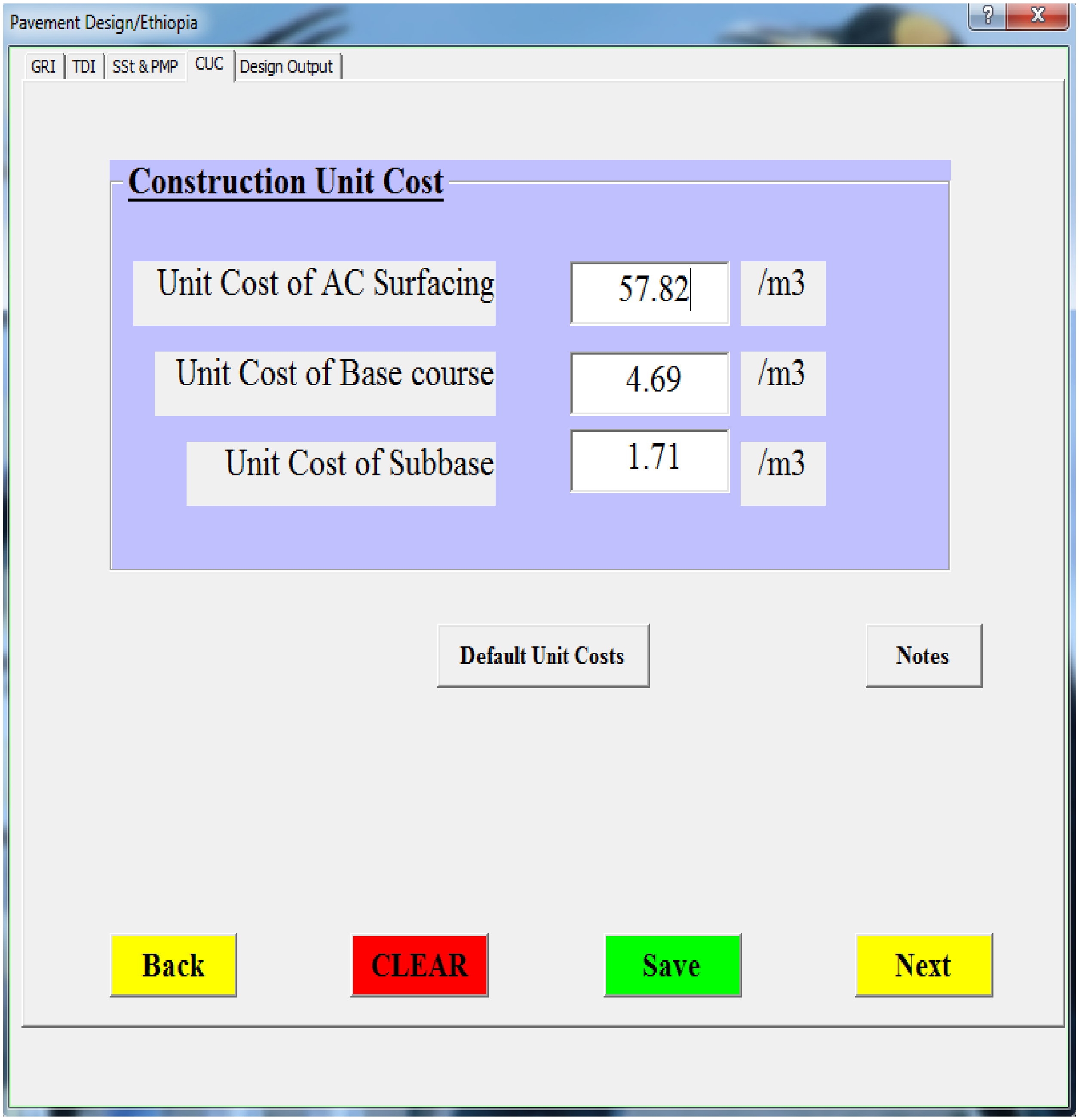


Figure 4. 20 Construction Unit coast of pavement layer materials

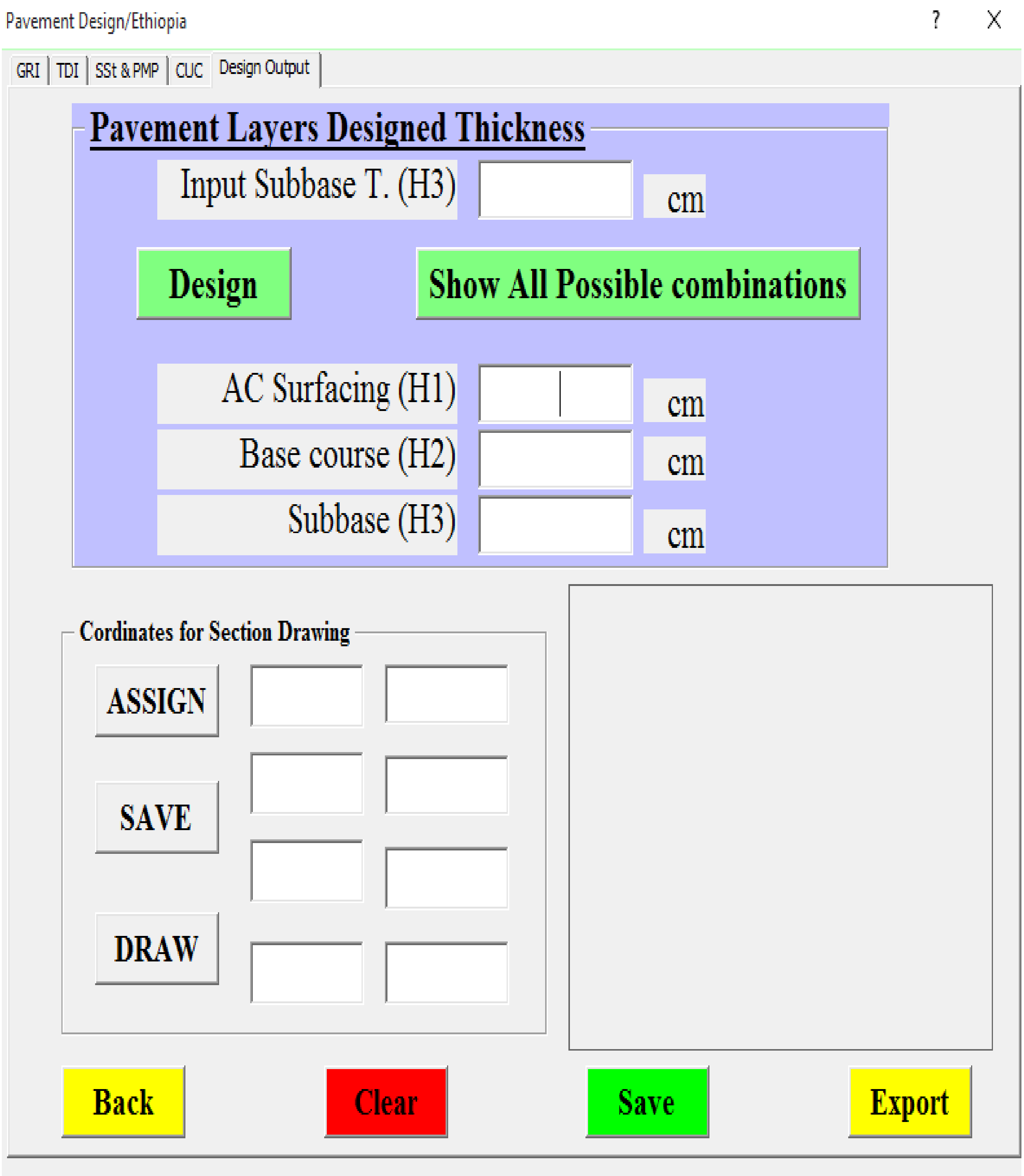
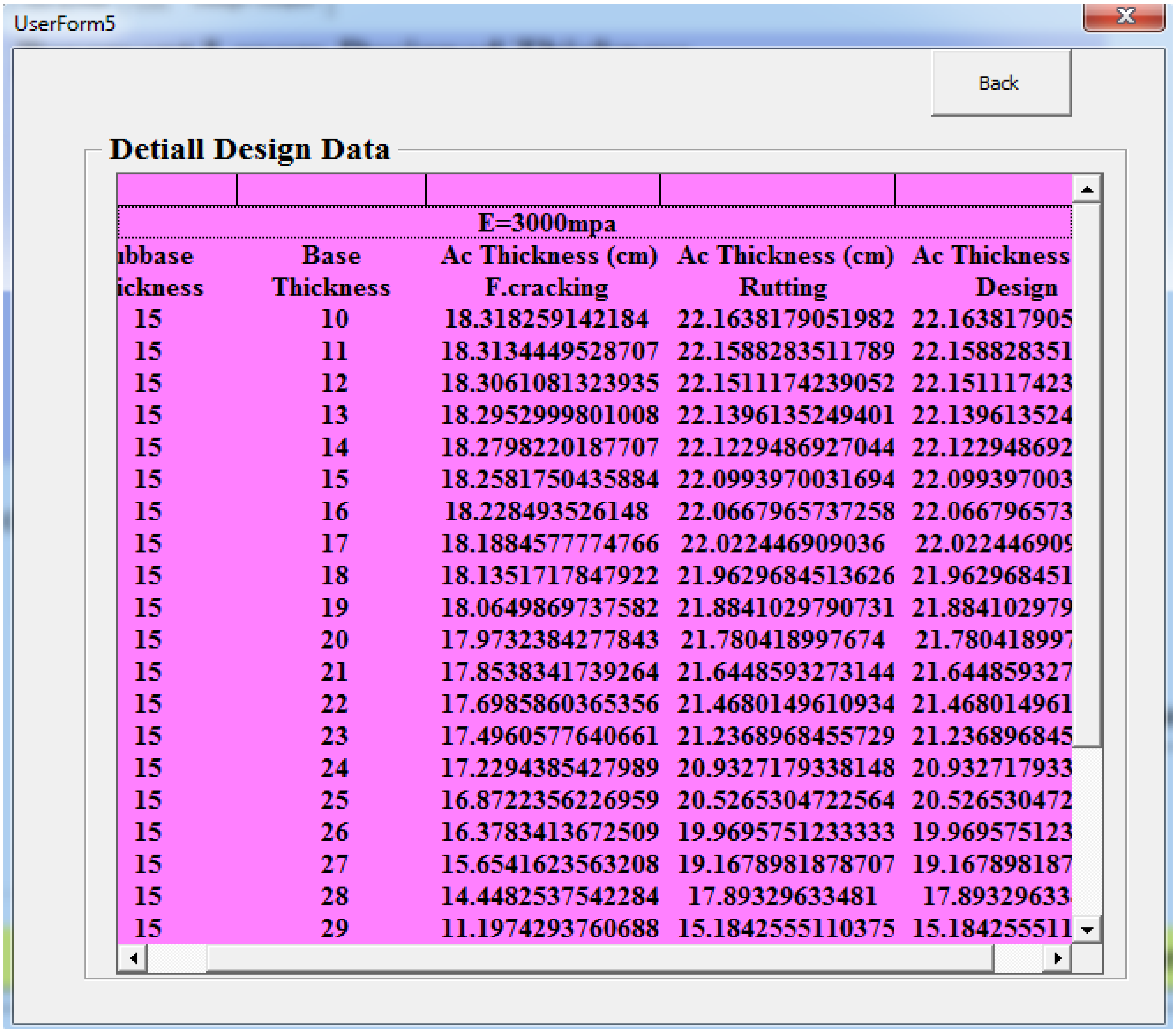


Figure 4. 21 Design output window



E=3000mpa				
ubbase ickness	Base Thickness	Ac Thickness (cm) F.cracking	Ac Thickness (cm) Rutting	Ac Thickness Design
15	10	18.318259142184	22.1638179051982	22.163817905
15	11	18.3134449528707	22.1588283511789	22.158828351
15	12	18.3061081323935	22.1511174239052	22.151117423
15	13	18.2952999801008	22.1396135249401	22.139613524
15	14	18.2798220187707	22.1229486927044	22.122948692
15	15	18.2581750435884	22.0993970031694	22.099397003
15	16	18.228493526148	22.0667965737258	22.066796573
15	17	18.1884577774766	22.022446909036	22.02244690
15	18	18.1351717847922	21.9629684513626	21.962968451
15	19	18.0649869737582	21.8841029790731	21.884102979
15	20	17.9732384277843	21.780418997674	21.78041899
15	21	17.8538341739264	21.6448593273144	21.644859327
15	22	17.6985860365356	21.4680149610934	21.468014961
15	23	17.4960577640661	21.2368968455729	21.236896845
15	24	17.2294385427989	20.9327179338148	20.932717933
15	25	16.8722356226959	20.5265304722564	20.526530472
15	26	16.3783413672509	19.9695751233333	19.969575123
15	27	15.6541623563208	19.1678981878707	19.167898187
15	28	14.4482537542284	17.89329633481	17.89329633
15	29	11.1974293760688	15.1842555110375	15.184255511

Figure 4. 22 Sample detail output for subbase thickness of 15cm

An economical pavement layer composition can be established using the design tool as follows

1) For predetermined design life, traffic is forecasted in terms of ESA, or
2) Direct traffic data in two direction with corresponding EALF can be used and
3) ESA is directly predicted by the developed design tool
4) Material property Inputs (such as resilience modulus and CBR (%)) are determined from material specifications and used as input in the design tool

In doing so the following objective function has to be optimized. The Objective function can be expressed in the following form

$$C=H_{ac}*U_{ac} + H_b*U_b + H_{sb}*U_{sb} =\text{minimum}$$

Where

H_{ac} is thickness of asphalt concrete layer (cm)

H_b is thickness of granular base course layer (cm)

H_{sb} is thickness of subbase layer (cm)

U_{ac} is construction cost of one square meter of asphalt concrete material having a thickness of 1 cm

U_b is construction cost of one square meter of granular base material having a thickness of 1 cm

U_{sb} is construction cost of one square meter of granular subbase material having a thickness of 1 cm

Among alternative set of structures, the economical one is selected manually or directly by the design tool. Composition of the pavement layer is therefore optimized for the objective function and constraints. For our case, lower bound constraints are minimum allowable thickness of pavement layers and allowable repetition of standard axles

4.4.2 An outline of a design procedure with example

a. Design procedures

1. Login
2. Input General road information (optional)
3. Select Traffic data input type from the dropdown menus, may be direct ESA or Detail traffic count data according to the available data in hand, and then the design ESA is calculated directly.
4. Select the Subgrade strength data input type from the dropdown menu according to the available data in hand, in terms of CBR or Mr., then subgrade class automatically determined by the design tool according to ERA classification
5. Pavement layer material types (optional).
6. Construction unit cost of pavement layers (should be per m^3) for each unit thickness (cm) of pavement layers, which is used to select the economical section.
7. Initial subbase thickness in intervals of 5 cm such as 10, 15, 20, 25, 30,and 35
8. Then possible combination of pavement layer thicknesses are calculated by the tool, of these possible combinations, the economical section is selected automatically.
9. Finally exporting results graphically and in text form.

b. Design Example

Design a pavement structure with the following data

Subgrade strength 35 MPa
Design period 20 years
Construction period 3 years
Growth rate 5%

Table 4. 32 Traffic count data and ESALF

Traffic type	Two direction traffic volume	Average ESA per vehicle
Cars	400	0
Minibuses	350	0.5
Buses	160	2
Small T	155	1.5
Medium T	240	5
Large T	100	15
Truck Trailers	60	12

Construction unit costs

Unit cost of AC 57.82 ETB/m^3 for 1 cm thickness

Unit cost of base course 4.69 ETB/m^3 for 1 cm thickness

Unit cost of subbase 1.71 ETB/m^3 for 1 cm thickness

Subbase thickness 25 cm

Table 4. 33 Outputs of the design tool

Design ESA(10^6)	**29.06**	
CBR/Mr.	3/35	
Subgrade Class	S2	
Counted Traffic		
Vehicle Type	No	EALF
Cars	400	0
Minibuses	350	0.5
Buses	160	2
Small Trucks	155	1.5
Medium Trucks	240	5
Large Trucks	100	15
Truck Trailers	60	12
Growth Rate (%)	5	
Construction Period (years)	3	

Opening Traffic		
Vehicle Type	No	EALF
Cars	464	0
Minibuses	406	0.5
Buses	186	2
Small Trucks	180	1.5
Medium Trucks	278	5
Large Trucks	116	15
Truck Trailers	70	12
Design period (years)	20	
Design ESA(10^6)		**29.06**

Table 4. 34 Economical and safe pavement structure

Design outputs	
Thickness of AC Surfacing (H_1)	17.78 cm
Base Thickness (H_2)	29 cm
Subbase Thickness (H_3)	25 cm

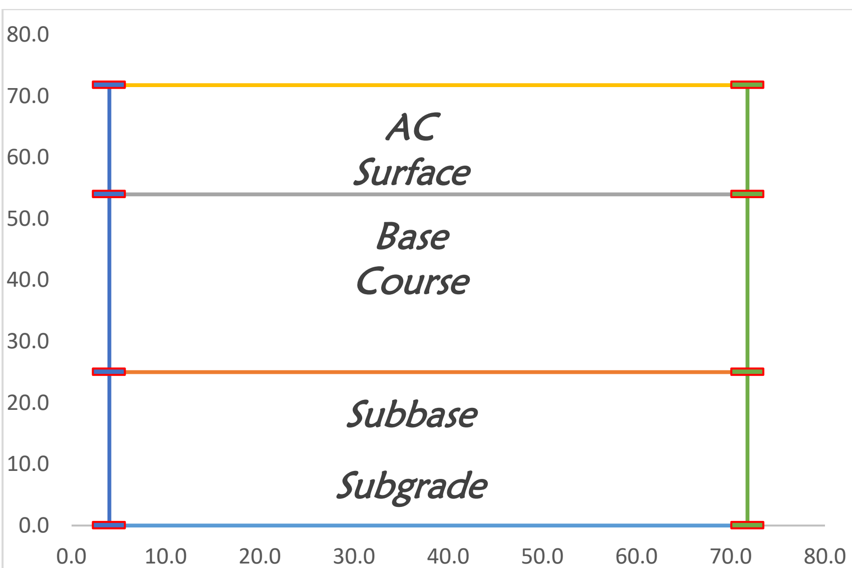


Figure 4. 23 Economical and safe pavement structure

Table 4. 35 Safe Possible combinations of a pavement structure

Subbase Thickness	Base Thickness	Ac Thickness (cm) F. Cracking	Ac Thickness (cm) Rutting	Ac Thickness (cm) Design	Construction cost of pavement/m2
25	10	19.68	22.05	22.05	1364.70
25	11	19.67	22.04	22.04	1368.92
25	12	19.67	22.03	22.03	1372.91
25	13	19.66	22.01	22.01	1376.60
25	14	19.65	21.99	21.99	1379.90
25	15	19.64	21.96	21.96	1382.69
25	16	19.62	21.91	21.91	1384.83
25	17	19.60	21.85	21.85	1386.14
25	18	19.57	21.78	21.78	1386.43
25	19	19.53	21.68	21.68	1385.42
25	20	19.48	21.55	21.55	1382.79
25	21	19.42	21.39	21.39	1378.12
25	22	19.34	21.18	21.18	1370.85
25	23	19.24	20.92	20.92	1360.21
25	24	19.11	20.58	20.58	1345.10
25	25	18.96	20.13	20.13	1323.85
25	26	18.76	19.52	19.52	1293.64
25	27	18.51	18.67	18.67	1249.17
25	28	18.20	17.36	18.20	1226.21
25	29	17.78	14.74	17.78	1206.99

Chapter Five

Conclusion and Recommendation

Analysis of results of this research and synthesis of literatures, the following conclusion and recommendations has been drawn.

5.1 Conclusion

1. Pavement responses (all strains and deflection) have decreased when AC modulus increases and in turn increases allowable repetitions of standard axles. Increase in poisson's ration of AC results in decrease of pavement responses and in turn increase in allowable repetitions of standard axle loads but the rate of increase (sensitivity) is less compared with the sensitivity to modulus. Increase in allowable repetitions of standard axles per unit increase of modulus decreases rapidly as subgrade modulus increases. This rapid decrement in allowable repetitions of standard axles per unit increase of modulus is an indication that further increase in modulus of subgrade is inefficient. As Mean Monthly Air Temperature T (°C) increases modulus of asphalt layer decreases, which in turn results in increase in critical pavement responses.
2. EALF is different for asphalt fatigue and permanent deformation for one type of axle load configuration. Therefore, using a single EALF for flexible pavement is highly empirical. EALF from 4th power rule is greater than both from EALF for fatigue cracking and EALF for deformation especially on higher axle loads. But for axle loads below the standard axle, it is relatively lower and insignificant. Therefore, using 4th power rule is conservative for higher axle loads and may underestimate for lower axle loads. Maximum horizontal strain at bottom of asphalt layer increases by 19% over a range of 500-1000kpa contact pressure. But increases by only 1% for max vertical strain which is insignificant. Therefore, increase in tire pressure and in turn contact pressure affects only the asphalt layer and has insignificant effect on rutting of flexible pavement.
3. Using the circular contact area overestimate pavement responses and in turn underestimates pavements life prediction which results in conservative design. The reverse is true for the rectangular one. The semicircular contact area is intermediate in all respects.
4. At low thickness of AC layer, horizontal strain increases as thickness increases. A peak strain level was reached at 6.3cm thickness of AC. Beyond this point the AC layer starts to act structurally and distributes load to other sub layers. Further increase in asphalt thickness reduces the flexure of the structure and the resulting strain in the asphalt.
5. Allowable repetition of standard axle loads for deformation/rutting increases by 113% over the range of 1500-3500mpa AC modulus. However, allowable repetition of standard axle loads for fatigue cracking increases only by 64% over the same range of AC modulus. This indicates that rutting sensitive to AC modulus than fatigue cracking. Tensile strain at bottom of Asphalt layer decreases rapidly as modulus of base increases

when compared to compressive strain at top of subgrade. This indicates that fatigue cracking is more sensitive to changes in Base modulus than rutting.

6. Allowable load repetition for fatigue cracking (N_t) increases only by 66% over a range of 10cm-30cm base course thickness. However, Allowable load repetition for rutting (Nr) increases more than 8.9 times. This indicates that rutting is more sensitive to changes in Base thickness than fatigue cracking. As subbase thickness increases, change in horizontal strain at bottom of asphalt layer (ε_t) and Allowable load repetition for fatigue cracking (Nt) is insignificant. However, Allowable load repetition for rutting (Nr) increases more than 14 times over the range of 10-35cm subbase thickness. This indicates that rutting is more sensitive to changes in Subbase thickness than fatigue cracking. It has to be noted here that rutting is more sensitive to changes in subbase thickness than base course thickness. Among all parameters, deformation/rutting are more sensitive to subgrade modulus and CBR.
7. Allowable Load Repetitions and in turn performance period predicted by Kenlayer is less almost by 71% and 54% from the number of standard design Load Repetitions expected by the ERA method for charts C1 and C2 respectively. Therefore, it is necessary for Ethiopia to adapt mechanistic empirical design approach. Accordingly, an effort has been made to develop a new framework for design of flexible pavements.
8. A new flexible pavement design tool was developed which is mechanistic Empirical based. The basis of the method is the Kenlayer computer program for the linear elastic stress analysis of the structure. The developed method treats the pavement structure as system of pavement layers and designing it accordingly. An outline of the design procedure is described in detail in section 4.5 with examples

5.2 Recommendations from the results of this research

It is necessary for Ethiopia to adapt mechanistic empirical design approach.

To protect deformation/rutting it is better to improve subgrade modulus than any other measures

Fatigue Performance of pavements can be improved by increasing the thickness of bituminous layer and having higher elastic modulus value (more viscous).

Using the semicircular contact area in pavement modeling and analysis is intermediate in all respects.

Using a single EALF for flexible pavement is highly empirical. Therefore, different EALF is required for fatigue cracking and rutting.

5.3 Recommendation for further work

Data inputs for mechanistic analysis in ERA design manual are limited. Therefore, an effort has to be made to characterization of material properties to set a database for data input to design softwares to our particular condition.

Axle load Data is collected by ERA on major roads. However this data couldn't be used for mechanistic design because of limitation of types loading configuration data. Loading configuration i.e. single tire, dual tire, single axle, dual axle, and tridem axle types should be included in axle load survey.

In the new design framework developed here, is only for ac modulus of 3000mpa therefore it needs further development for other modulus values of AC

Effect of nonlinearity for all materials and visco-elasticity of bituminous layers is not considered in this research therefore it needs further work.

References

[1] Eastcoast Pavement Services. (2017, November) Is Traffic and Vehicle Loads affect Pavement? [Online]. https://eastcoatpavement.com/can-traffic-vehicle-loads-affect-pavement/

[2] Ahmad Kamil bin A., "Flexible Pavement Design," Transitioning from Empirical to Mechanistic-Based Design Methods, July 2007.

[3] Newcomb David E. and Timm David H., "Mechanistic Pavement Design: The Next Wave," Hot Mix Asphalt Technology, pp. 49-51, September/October 2001.

[4] pavementinteractive. (2019, june) pavement Interactive. [Online]. https://www.pavementinteractive.org

[5] ABAQUS, "Finite Element Computer Program:Theory and User's Manual ,Version 6.14.," Hibbitt, Karlsson, and Sorensen, Inc, Pawtucket, Rhode Island, 2002.

[6] Haung Y. H., Pavement Analysis and Design., 2004.

[7] A. Dione, M. Fall, Berthaud Yves, Benboudjema, F. Yves, and A3. Michou, "Implementation of Resilient Modulus - CBR relationship in Mechanistic-Empirical (M. -E) Pavement Design," Revue du CAMES – Sciences Appliquées et de l'Ingénieur, vol. 1, 2014.

[8] Ethiopian Roads Authority (ERA), Pavement Design Manual Volume1, 2002.

[9] Ethiopian Roads Authority (ERA), pavement design manual. Addis Ababa, 2013, vol. I.

[10] David E Newcomb and David H. Timm, "Wanted: Transfer Functions, Experience Needed," Hot Mix Asphalt Technology, pp. 22-25, September/October 2001.

[11] (2019, March 21) Mechanistic pavement design concepts. [Online]. http://www.zdruzenjezas.si/Material/Delavnica_Projektiranje_Voziscnih_Konstrukcij/Michael_Wistuba.pdf

[12] Michael S.Mamlouk, "Design of Flexible Pavements," in Handbook of Highway Engineering, T.F. Fwa, Ed.: CRC Press:Taylor & Francis Group, 2006, ch. 8, pp. 8.1-8.34.

[13] McElvaney J. Snaith M.S., "Analytical design of flexible pavements," in Highways: The location, design, construction and maintenance of road pavements, 4th ed., A.M. Emeritus C.A. O'Flaherty, Ed. Burlington, MA 01803, USA: Elsevier Ltd, 2002, ch. 15, pp. 396-422.

[14] AASHTO, AASHTO Guide for Design of Pavement Structures. Washington, DC, 1993.

[15] "COST 33: Development of New Bituminous Pavement Design Method," European Commission Directorate General Transport, Brussels, Final report of the action ISBN 92-828-6796-X, 1999.

[16] De Beer M. Theyse H. L. and Rust F. C, "Overview of South African Mechanistic Pavement Design

Method," Transportation Research , Record 1539,.

[17] Powell W.D., Potter J. F, Mayhew H. C, and Nunn M. E., "The Structural Design of Bituminous Roads," Transport and Road Research Laboratory, Crowthorne, United Kingdom, TRRL Laboratory Report LR 1132, 1984.

[18] AUSTROADS, A guide to the structural design of road pavements. Sydney, 1997.

[19] AUSTROADS, A guide to the structural design of road pavements. Sydney, Australia, 2004.

[20] AUSTROADS, Guide to Pavement Technology Part 2:Pavement Structural Design, 4th ed. Sydney NSW 2000 , Australia: Austroads Ltd, 2017.

[21] Baburamani P., "Asphalt Fatigue Life Prediction Models -A Literature Review," ARRB Transport Research Ltd, Vermont south, Victoria, Research Report ARR 334, 1999.

[22] Gebremedhin Kibret, "Truck Overload Fine Rationalization OnEthiopian Primary Roads a Case Study on Addis Ababa -Adama Expressway," AAU, AA, MSc Thesis Report 2015.

[23] R.B. Mallick and T. El-Korchi, pavement engineering principles and practice, 2nd ed. Boca Raton London New York: CRC Press, 2013.

[24] David E. Newcomb and David H. Timm, "Lies Damned Lies, and traffic forcasting," Hot Mix Asphalt Technology, pp. 14-21, July/August 2002.

[25] Roman N., Pawe£ T., and Magdalena Z., "Defining The Domain and Boundary Conditions for Finite Element Model of Flexible Road Pavement," Roads and Bridges - Drogi i Mosty, no. 16, pp. 265 – 277, 2017.

[26] Zaghloul S. and White T., "Use of a Three-Dimensional, Dynamic Finite Element Program for Analysis of Flexible Pavement," TRB, National Research Council, Washington, D.C, 1993.

[27] DAR-HAO C., MUSHARRAF Z., JOAKIM L., and ALAN S., "Assessment of computer programs for analysis of flexible pavement structure," Transportation Research Record 1482, pp. 123-133, 1995.

[28] Kim Minkwan, "Three-Dimensional Finite Element Analysis of Flexible Pavements Considering Nonlinear Pavement Foundation Behavior," University of Illinois at Urbana-Champaign, PhD dissertation 2007.

[29] Thom Nick, Principles of pavement engineering, 2nd ed. London, UK: ICE Publishing, 2014.

[30] Mamlouk. M. S and SAROFIM R.T, "Modulus of Asphalt Mixtures, An Unresolved Dilemma," TRANSPORTATION RESEARCH RECORD 1171,.

[31] Kumlai Sarayoot, Jitsangiam Peerapong, and Nikraz Hamid, "Comparison Between Resilient Modulus and Dynamic Modulus of Western Australian Hot Mix Asphalt Based On Flexible Pavement Design Perspectives," in 26th ARRB Conference-Research Driving Efficiency, Sydney, New South Wales, 2014.

[32] Stempihar Jeff, "Resilient Modulus to Dynamic Modulus Relationship and Pavement Analysis with the Mechanistic-Empirical Pavement Design Guide," Arizona State University, 2015.

[33] Huang Y. H., Kenpave Software User Manual, University of Kentucky, 2004.

[34] Christopher Barry R., Schwartz Charles, and Boudreau Richard, "Geotechnical Aspects of Pavements.," National Highway Institute, Federal Highway Administration and U.S. Department of Transportation, Washington, D.C., Publication No. FHWA NHI-05-037, May 2006.

[35] Ayed Amr, "Development of Empirical and Mechanistic Empirical Performance Models at Project and Network Levels," Waterloo, Ontario, Canada, 2016.

[36] Martin T.C., "A Review Of Existing Pavement Performance Relationships," AKRB Transport Research Ltd., Research Report No. 282, 1996.

[37] Saba R.G, Huvstig A., and Hildebrand G., "Performance Prediction Models for FlexiblePavements: A State-of-the-art Report," Teknologirapport: 2477, 2006.

[38] A.W. Ahmed, "Mechanistic-Empirical Modeling of Flexible Pavement Performance: Verifications Using APT Measurements," Royal Institute of Technology, Stockholm, Sweden, 2014.

[39] Robinson R. and Thagesen B., Road Engineering for Development , 2nd ed. New Fetter Lane, London, UK: Spon Press, 2004.

[40] David E Newcomb and David H. Timm, "Mechanistic Pavement Design: A Homogenized, Icey tropic half what? ," Hot Mix Asphalt Technology, pp. 26-30, January/February 2002.

[41] TRRL., A guide to the structural design of bitumen surfaced roads in tropical and sub-tropical countries. Overseas Road Note 31 , 4th ed. Crowthorne, UK.: TRL, 1992.

[42] Hesham A. Ali and Shiraz D. Tayabji, "Mechanistic Evaluation of Test Data from LTPP Flexible Pavement Test Sections," Federal Highway Administration, Final Report FHW A-RD-98-012,.

[43] David E Newcomb and David H. Timm, "It is still Dirt: Rocks, and Asphalt-right? ," Hot Mix Asphalt Technology, pp. 16-22, June 2002.

[44] Sanjay G. (2019, March, 28) Flexible Pavement Design in India. [Online]. https://www.academia.edu/12745242/Flexible_Pavement_Design_in_India

[45] Abu El-Maaty Ahmed Ebrahim, "Temperature Change Implications for Flexible Pavement Performance and Life," International Journal of Transportation Engineering and Technology, vol. 3, no. 1, pp. 1-11, 2017.

[46] KUMAR H., "Analysis of Flexible Pavement Using Kenlayer," National Institute of Technology Kurukshetra, open access MSc thesis 2013.

[47] Transportation Research Board of the national academes. (2019, April,4) Mechanical Models for Asphalt

Behavior and Performance:Transportation Research Board Webinar. [Online]. http://onlinepubs.trb.org/onlinepubs/webinars/121018.pdf

[48] Pawe£ T. Roman N. and Magdalena Z., "Defining The Domain and Boundary Conditions for Finite Element Model of Flexible Road Pavement," Roads and Bridges - Drogi i Mosty, vol. 16, pp. 265 – 277, 2017.

[49] Kim M., Tutumluer E., and J.Kwon, "Nonlinear pavement foundation modeling for three dimensional finite-element analysis of flexible pavements," International Journal of Geomechanics , no. 9, pp. 195-208, 2009.

[50] Ghadimi Behzad, Nikraz Hamid, and Leek Colin, "A Comparison of Different Approaches in Numerical Modeling of Pavement Granular Material ," Journal of Civil Engineering and Architecture, vol. 8, no. 4, pp. 446-455, April 2014.

[51] Duncan J.M., Monismith C.L., and Wilson E.L., "Finite element analyses of pavements," Highway Research Record Highway Research Board , no. 228, pp. 18-33, 1968.

Appendix

7.1 AADT and axle load data

Table 7. 1 Annual Average Daily Traffic by Road Section

YEAR	ROAD NO	ROUTE NO	ROUTE NAME	Length	Cars	Land Rover	Small Buses	Large Bus	Small Truck	Medium Truck	Heavy Truck	Truck& Trailer	Total
2016	6	1	ADDIS ABABA GHION	116	803	956	1227	653	825	1059	912	803	7238
		2	GHION WELKITE	42	78	322	808	66	226	390	353	85	2328
		3	WELKITE JIMMA	188	59	192	462	106	244	324	283	127	1797
2017	6	1	ADIS ABABA GHION	116	990	1120	1412	863	989	1228	936	1049	8587
		2	GHION WELKITE	42	92	337	775	72	207	479	338	102	2402
		3	WELKITE JIMMA	188	82	182	446	97	201	273	248	145	1674
2018	6	1	ADIS ABABA GHION	116	1250	1370	1740	1117	1309	1604	1374	1272	11036
		2	GHION WELKITE	42	79	271	739	60	177	392	341	108	2167
		3	WELKITE JIMMA	188	126	212	476	104	261	314	273	154	1920

Am19

Jimma University

Jimma Institute of Technology

Faculty of Civil & Environmental Engineering

Highway Engineering Chair

Ref. No: - FCEE/HEC/ 192 /2019

Date: - 05 / 08 / 2019

12/08/2019

Am19/38/10-115

To: ERA Office

Addis Ababa, Ethiopia

Letter of recommendation for data collection

With reference to the above matter, I am writing to certify that **Mr. Wubetu Yenegeta** (ID No.: RM0566/10) who is a registered student of Master of Science in **Highway Engineering** at Jimma Institute of Technology, Jimma University. He is collecting materials for the Final MSc thesis on the title: **Pavement performance evaluation under traffic load using mechanistic empirical approach.** In this regard he needs to collect the input data's and materials from your side for conducting his final thesis.

Therefore, this is to kindly request your good office to provide him the necessary materials in advance.

Kind regards

Oluma Gudina
Highway Engineering
Chair Holder

School of Civil & Environmental Engineering
Jimma Institute of Technology

12/08/2019

Figure 7. 1 Official letter of recommendation for data collection

Table 7. 2 Axle load

Form Ax-001

ETHIOPIAN ROADS AUTHORITY

Size and Weight Control Station

Jimma		Axel Load Report
PERIOD :From July 1, 2015-June 30,2016		

AXLE LOAD IN Ton	AXLE DISTRUBUTION OF VEHECLIES									TOTAL AXLE WEIGHET: NO OF VEHICLES			
										FRONT		REAR	
	F1	F2	R1	R2	R3	R4	R5	R6	R7	LEGAL	ILLEGAL	LEGAL	ILLEGAL
1.0-2	0	0	0	0	0	0	0			0		0	
2.1-3	0	0	0	1	0	0	0			0		1	
3.1-4	0	0	0	1	0	0	0			0		1	
4.1-5	14	0	2	4	6	6	5			14		23	
5.1-6	275	2	2	4	8	22	33			277		69	
6.1-7	1175	0	12	17	50	172	174			1175		425	
7.1-8	3101	0	73	75	188	862	511			3101		1709	
8.1-9	236	0	136	170	669	806	1124				236	2905	

9.1-10	5	0	3032	2981	1993	1098	1173				5	10277	
10.1-11	2	0	1160	1120	176	90	15				2		2561
11.1-12	0	0	19	17	11	8	0				0		55
12.1-13	0	0	67	61	1	5	1						135
13.1-14	0	0	247	242	0	0	1						490
14.1-15	0	0	37	47	1	0	0						85
15.1-16	0	0	10	6	0	0	0						16
16.1-17	0	0	8	1	0	0	0						9
17.1-18	0	0	3	0	0	0	0						3
18.1-19	0	0	0	0	0	0	0						0
19.1-20	0	0	0	0	0	0	0						0
20.1-21	0	0	0	0	0	0	0						0
21.1-22	0	0	0	0	0	0	0						0
TOTAL	4808	2	4808	4747	3103	3069	3037	0	0	4567	243	15410	3354
prepared by:										Approved by:			
Title:										Title:			
Signature:										Signature:			

7.2 Tire Pressure Data

Table 7. 3 Addis-Jimma Agaro -Bedele #**1**

Vehicle Type	Tire Size	Tire Type	Status	Tire Pressure (Bar)			Interviewee	Remark
				Rear		Front		
Issus	8.25x16		New	5-6		4.5-5	Garage Worker /Tire Inflator Operator	
			Old					
FSR	9x20		New	8		7		
			Old					
Sino Truck	12x20		New	8.5-9		8.5		
			Old					
Dangote	12x20		New	9		8		
			Old					
Truck Trailer/Mercedes			New	9		8		
			Old					
Truck Trailer/Low Bed			New	9		8		
			Old					
Loader			New					
			Old					
Full Trailer/Djibouti Long Vehicle			New	8-12		8-10		
			Old					

The maximum tire pressure you have noticed/inflated to? 12 bar

Table 7. 4 Addis-Jimma: common for all segments (beyond the junction i.e. to Addis side) **#2**

Vehicle Type	Tire Size	Tire Type	Status	Tire Pressure (Bar)			Interviewee	Remark
				Rear		Front		
Issus	8.25x 16		New	6-7		6-7	Garage Worker/ Tire Inflator Operator	
			Old					
FSR	9x20		New	6-7		6		
			Old					
Sino Truck	12x2 0		New	8-8.5		8		
			Old					
Dangote	12x2 0		New	9		8		
			Old					
Truck Trailer/Mercedes	12x2 4		New	9		8		
			Old					
Truck Trailer/Low Bed			New	9		8		
			Old					
Loader			New	4		4		
			Old					
Grader			New	3		3		
Full Trailer/Djibouti Long Vehicle			Old	9	9	8		

The maximum tire pressure you have noticed/inflated to? 9 bar

Table 7. 5 Addis-Jimma-Bonga-Mizan **#3**

Vehicle Type	Tire size	Tire type	status	Tire pressure (bar)			interviewee	remark
				rear		front		
Issus	8.25x16		New	7		7	Garage worker/ tire inflator operator	
			old					
	7.5x16		New	7		7		
			Old	5-6.5		6		
FSR	9x20		New	8		8		
		wire	Old	<7				
		string		<6				
Sino Truck	12x20		New	7-8		7-7.5		
			Old					
Dangote/Sino	12x20		New	9		8		
			Old					
Truck Trailer/Mercedes	12x20		New	9		8		
			Old					
Truck Trailer/Low Bed			New	9		8		
			Old					
Loader			New					
			Old					
Grader			New					
Full Trailer/Djibouti Long Vehicle				9	9	8		

Table 7. 6 Addis-Jimma-Bonga-Mizan **#4**

Vehicle type	Tire size	Tire type	status	Tire pressure (bar)			interviewee	remark
				Rear	Middle	Front		
Issus	8.25x16		New	6		5.5	Garage worker/ tire inflator operator	
			Old					
	7.5x16		New	6		5.5		
			Old	5-6.5		6		
FSR	9x20		New	7		7		
		wire	Old	6-6.5				
		string		<6				
Sino Truck	12x20		New	8		7		
			Old					
Dangote/Sino	12x20		New	9		8		
			Old					
Truck trailer/Mercedes	12x24		New	8		8		
			Old					
Truck trailer/low bed			New	8		8		
			Old					
loader			New					
			Old					
grader			New					
Full trailer/Djibouti long vehicle				8	9	8		

Table 7. 7 Drivers **#5**

Vehicle type	Tire size	Tire type	status	Tire pressure (bar)			interviewee	remark
				rear	middle	front		
Issus	8.25x16		New	6.5-7		5.5-6	Driver	
			Old	<6		<5		
	7.5x16		New	6		5.5		
			Old					
FSR	9x20		New	8		7		
		wire	Old	<7				
		string		<6				
Sino Truck	12x20		New	8-9		7-8		Spec.8.5
			Old	<8		<7		

Summary of axle load survey

Vehicle type	Tire pressure		remark
	Bar	kpa	
Isuzu	5-7	500-700	
FSR	6-8	600-800	
Sino Truck	7-9	700-900	
Dangote/Sino	8-9	800-900	
Truck trailer/Mercedes	8-9	800-900	
Truck trailer/low bed	8-9	800-900	
Loader	3	300	
Grader	3-4	300-400	
Truck trailer/Djibouti long vehicle	8-12	800-1200	

Weighted Average Tire Pressure Using No of axles surveyed

																	Total
Average Tire Pressure	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5	11.0	
No of each axle type surveyed	2.0	0.0	2.0	0.0	2.0	4.0	12.0	3.0	15.0	2.0	25.0	2.0	17.0	0.0	1.0	0.0	0.0	87.0
Weighted Average Tire Pressure																		7.33

Weighted Average Tire Pressure using AADT

2013 Cycle 3 AADT

	Small T.	Medium T.	Heavy T.	T. Trailer	Total AADT
2013 Cycle 3 AADT	177	358	195	91	821
Average Tire Pressure	6	7	8	8.8	
Weighted Average Tire Pressure					7.22

AADT (Average of 2016,17&18)

	Small T.	Medium T.	Heavy T.	T. Trailer	Total AADT
AADT (Average of 2016,17&18)	235	304	268	142	949
Average Tire Pressure	6	7	8	8.8	
Weighted Average Tire Pressure			7.30		

For computational simplicity, the weighted average tire pressure should be taken as 730 kpa.

7.3 Temperature Data of Jimma Area

Monthly Temperature data of Jimma (2018)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Min (°C)	13	14	14	13	13	12	12	12	12	10	11	12
Max (°C)	28	29	29	28	27	23	22	23	24	23	24	26

Monthly Temperature data of Jimma (2019)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Min (°C)	13	14	14	13	12							
Max (°C)	30	30	30	26	25							

Mean monthly Temperature data of Jimma (2018)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Aver T (°C)	20	21	21	20	19	17	16	16	17	19	19	21

Mean monthly Temperature data of Jimma (2019)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Aver T (°C)	24	24	23	21	21							

Source https://www.Worldweatheronline.com/
title=Historical average weather
data provided by worldweatheronline.com

7.4 Kenlayer Sample Output for Performance Evaluation

INPUT FILE NAME -D:\Current For Research1\KENPAVE\552 Tire Pressure\730_CHART C2 from T6S4_T8S2.DAT

NUMBER OF PROBLEMS TO BE SOLVED = 5

TITLE -C2T6S4

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=2, SO DAMAGE ANALYSIS WITH DETAILED PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 4
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 10 15 17.5
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.25 0.3 0.25 0.4
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.000E+06 2 3.000E+05
3 2.500E+06 4 7.300E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 9.34
CONTACT PRESSURE (CP)------------- = 730
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 3
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 0
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 34

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 9.340
3 0.000 17.000

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 1
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 4

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 1

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 1 ARE: 0.507 5 1.8

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 4 ARE: 1.365E-09 4.447

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

```
 POINT    VERTICAL   VERTICALVERTICAL    MAJOR      MINOR  INTERMEDIATE
                          PRINCIPAL    PRINCIAL  P. STRESS
  NO.    COORDINATE    DISP.    STRESS     STRESSSTRESS  (HORIZONTAL
                    (STRAIN)   (STRAIN)   (STRAIN)  P. STRAIN)
  1     10.00000   0.03444    233.145    233.414    -686.505    -571.102
        (STRAIN)          1.825E-04  1.826E-04  -2.007E-04  -2.007E-04
  1     42.50010   0.02695     19.913     20.347      4.026      4.833
        (STRAIN)          2.219E-04  2.302E-04  -8.283E-05  -8.283E-05

  2     10.00000   0.03406    178.946    182.516    -489.747     -67.519
        (STRAIN)          1.058E-04  1.073E-04  -1.728E-04  -1.728E-04
  2     42.50010   0.02764     21.221     21.308      4.216      5.226
        (STRAIN)          2.385E-04  2.402E-04  -8.764E-05  -8.764E-05

  3     10.00000   0.03334    136.145    268.088    -328.187    136.145
        (STRAIN)          5.039E-05  1.054E-04  -1.431E-04  -1.431E-04
  3     42.50010   0.02785     21.544     21.544      4.272      5.345
        (STRAIN)          2.424E-04  2.424E-04  -8.881E-05  -8.881E-05
```

AT BOTTOM OF LAYER 1 TENSILE STRAIN = -2.007E-04
ALLOWABLE LOAD REPETITIONS = 3.416E+06 DAMAGE RATIO = 2.927E-07

AT TOP OF LAYER 4 COMPRESSIVE STRAIN = 2.424E-04
ALLOWABLE LOAD REPETITIONS = 1.633E+07 DAMAGE RATIO = 6.125E-08

****
* SUMMARY OF DAMAGE ANALYSIS *
****
AT BOTTOM OF LAYER 1 SUM OF DAMAGE RATIO = 2.927E-07
AT TOP OF LAYER 4 SUM OF DAMAGE RATIO = 6.125E-08

MAXIMUM DAMAGE RATO = 2.927E-07 DESIGN LIFE IN YEARS = 3415975.

PROBLRM NO. 5

TITLE -C2T8S2

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=2, SO DAMAGE ANALYSIS WITH DETAILED PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1

TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 6
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 15 15 12.5 12.5 20
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.25 0.3 0.25 0.25 0.3 0.4
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.000E+06 2 3.000E+05
3 3.500E+06 4 2.500E+06 5 1.000E+05 6 3.700E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 9.34
CONTACT PRESSURE (CP)------------- = 730
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 3
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 0
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 34

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 9.340
3 0.000 17.000

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 1
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 6

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 1

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 1 ARE: 0.507 5 1.8

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 6 ARE: 1.365E-09 4.447

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

```
 POINT    VERTICAL   VERTICALVERTICAL     MAJOR      MINOR  INTERMEDIATE
                              PRINCIPAL    PRINCIAL  P. STRESS
  NO.    COORDINATE    DISP.     STRESS      STRESSSTRESS  (HORIZONTAL
                        (STRAIN)    (STRAIN)    (STRAIN)  P. STRAIN)
  1      15.00000    0.03570     147.129     147.453    -436.921    -335.333
         (STRAIN)            1.134E-04   1.135E-04  -1.300E-04  -1.300E-04
  1      75.00010    0.02857      6.010       6.095       0.318       0.446
         (STRAIN)            1.532E-04   1.565E-04  -6.213E-05  -6.213E-05
```

```
 2    15.00000   0.03590    131.627    132.008    -361.860    -112.598
      (STRAIN)         8.338E-05  8.354E-05  -1.222E-04  -1.222E-04
 2    75.00010   0.02884      6.210      6.228       0.307       0.439
      (STRAIN)         1.596E-04  1.603E-04  -6.378E-05  -6.378E-05

 3    15.00000   0.03574    116.354    116.354    -296.725      40.396
      (STRAIN)         6.015E-05  6.015E-05  -1.120E-04  -1.120E-04
 3    75.00010   0.02893      6.320      6.320       0.320       0.453
      (STRAIN)         1.624E-04  1.624E-04  -6.458E-05  -6.458E-05

AT BOTTOM OF LAYER  1   TENSILE STRAIN = -1.300E-04
ALLOWABLE LOAD REPETITIONS =   2.997E+07 DAMAGE RATIO =   3.336E-08

AT TOP OF LAYER  6   COMPRESSIVE STRAIN =   1.624E-04
ALLOWABLE LOAD REPETITIONS =   9.686E+07 DAMAGE RATIO =   1.032E-08

******************************
* SUMMARY OF DAMAGE ANALYSIS *
******************************
AT BOTTOM OF LAYER  1     SUM OF DAMAGE RATIO = 3.336E-08
AT TOP OF LAYER  6        SUM OF DAMAGE RATIO = 1.032E-08

MAXIMUM DAMAGE RATO =   3.336E-08   DESIGN LIFE IN YEARS = 29974010.
```

7.5 Pavement layer composition and loading arrangement for performance analysis

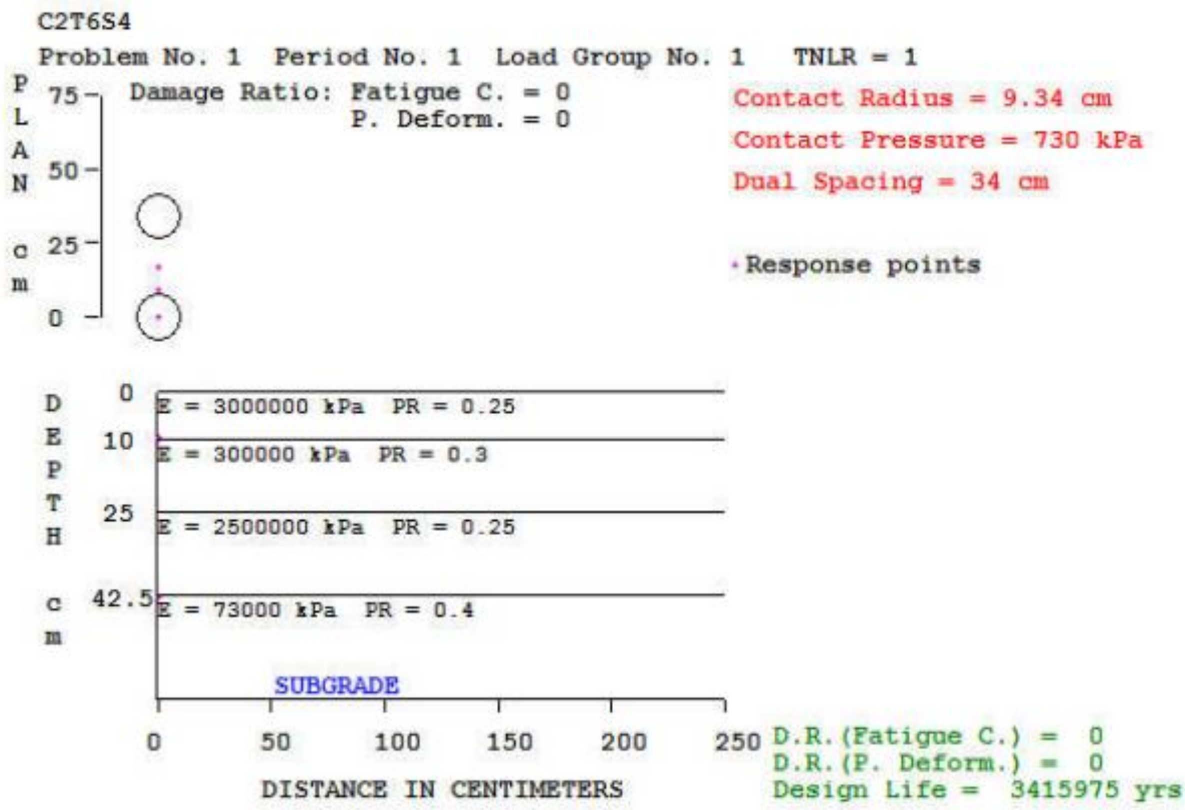


Figure 7. 2 Pavement layer composition for C2T6S4

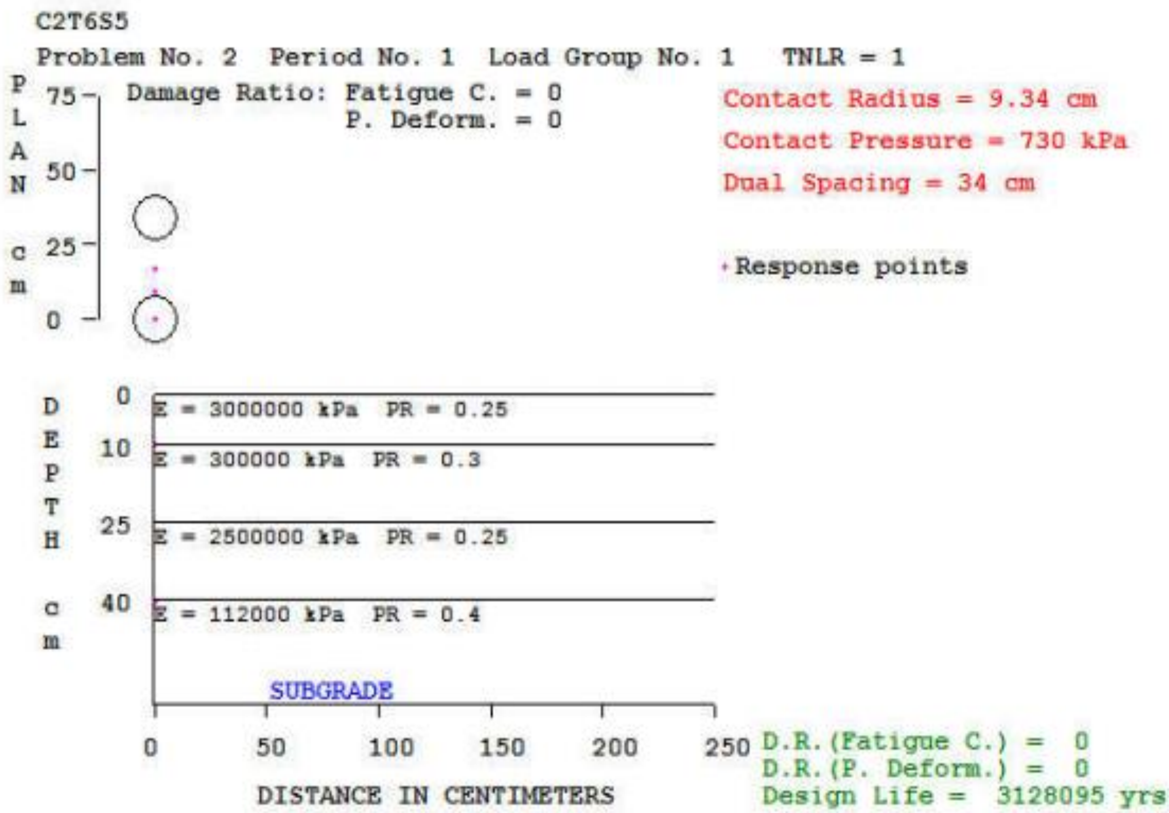


Figure 7. 3 Pavement layer composition and loading arrangement for C2T6S5

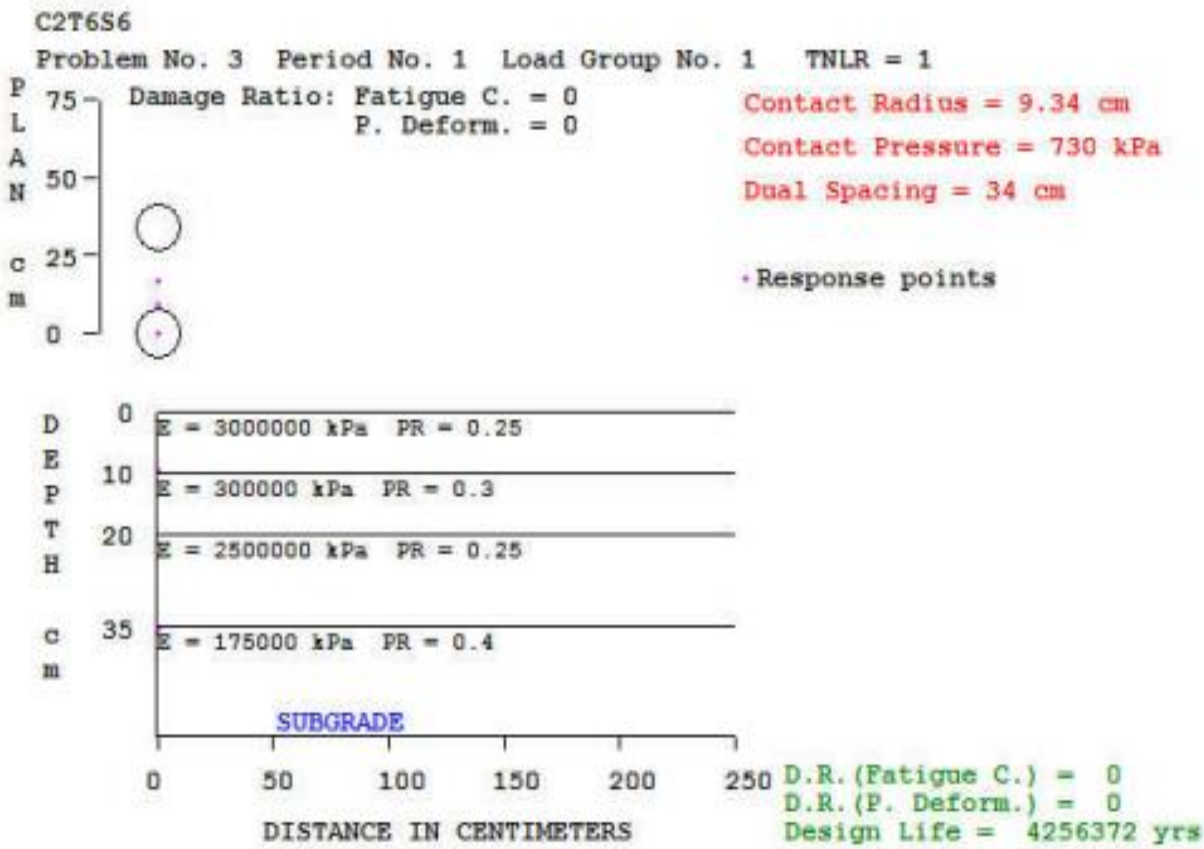


Figure 7. 4 Pavement layer composition and loading arrangement for C2T6S6

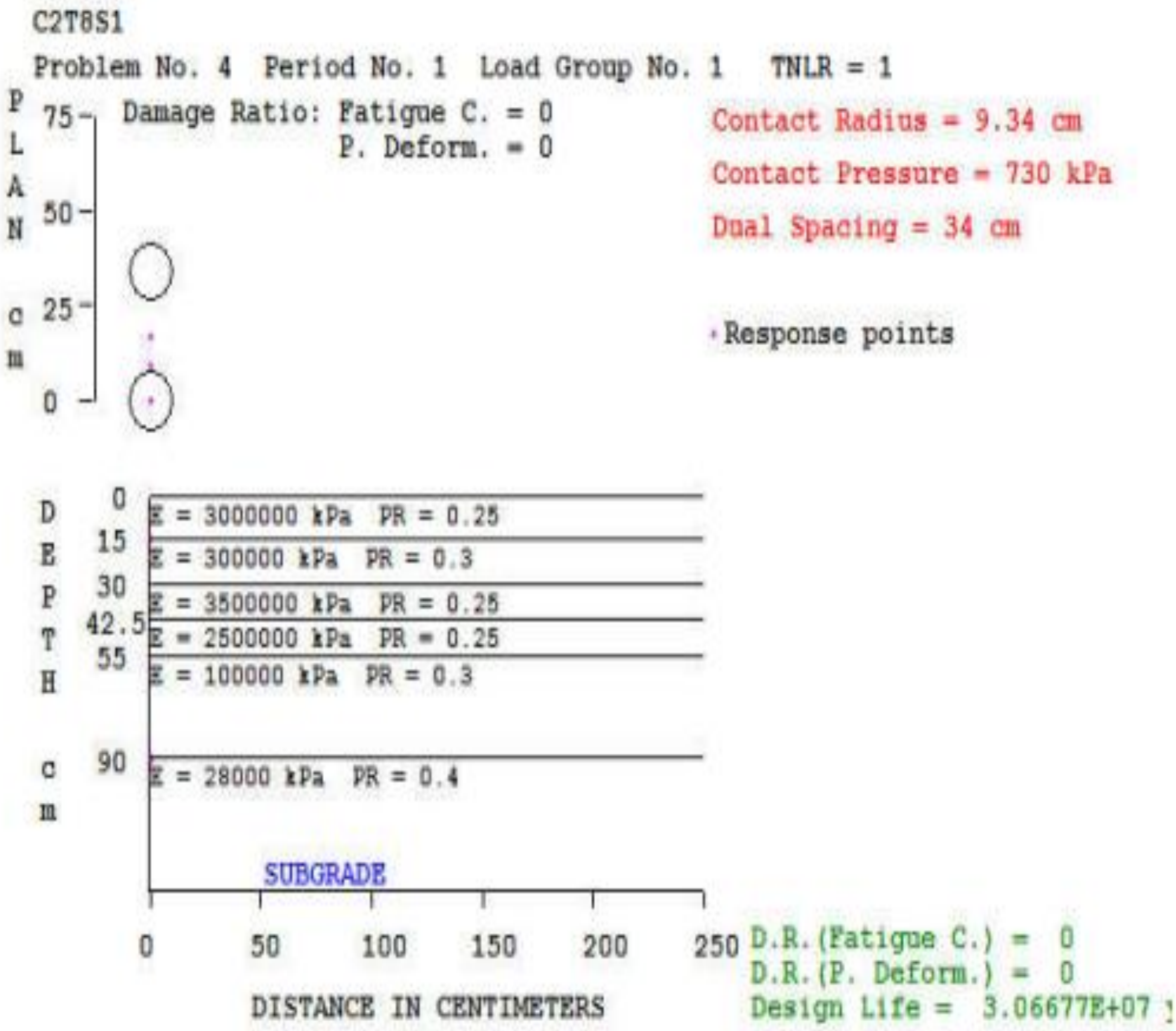


Figure 7. 5 Pavement layer composition and loading arrangement for C2T8S1

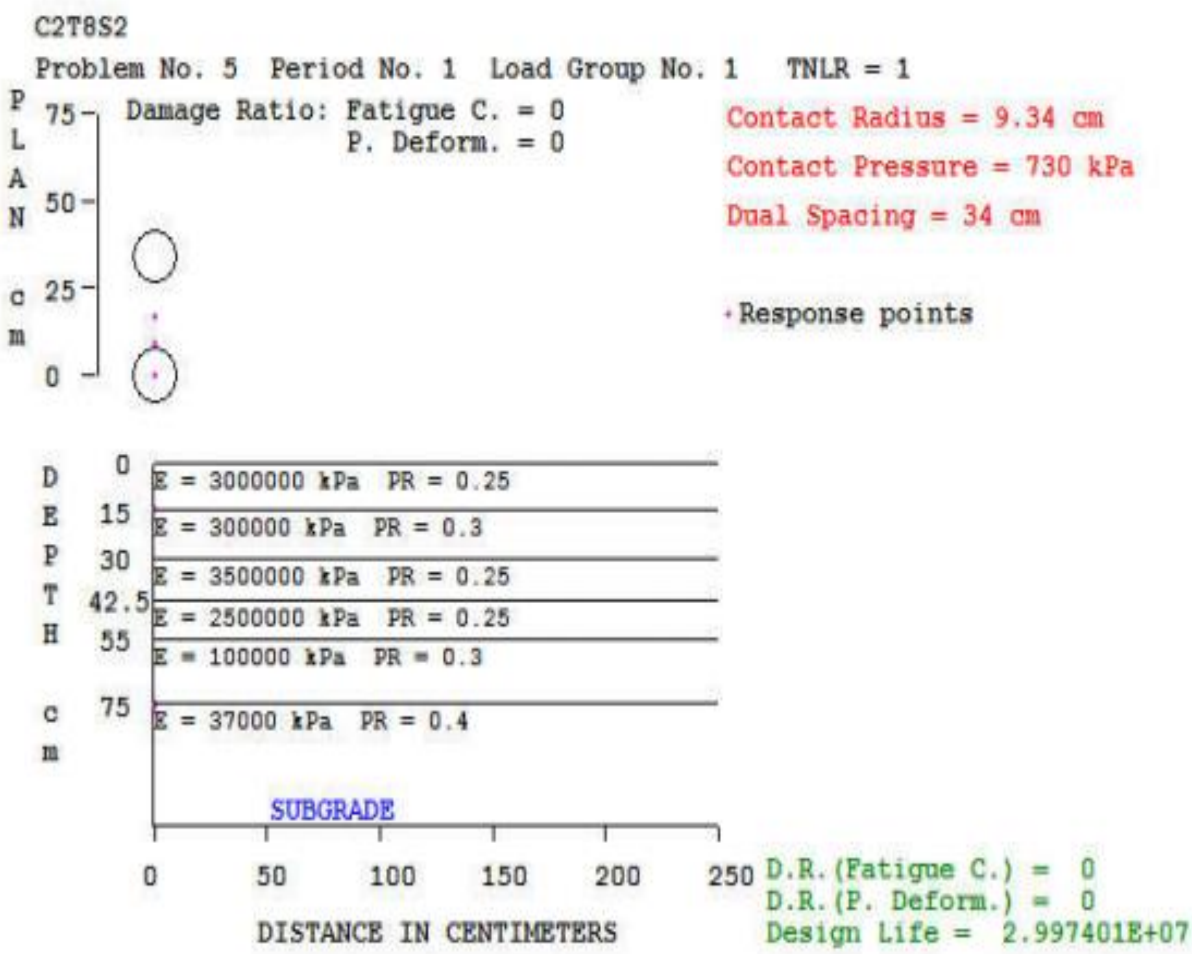


Figure 7. 6 Pavement layer composition and loading arrangement for C2T6S2

7.6 Kenlayer Sample output for design method development

INPUT FILE NAME -D:\Current For Research\KENPAVE\1_CBR@s2_SUBBASE_25cm & E_3000mpa.DAT

NUMBER OF PROBLEMS TO BE SOLVED = 5

TITLE -NPROB†7

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=2, SO DAMAGE ANALYSIS WITH DETAILED PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001

NUMBER OF LAYERS (NL)------------- = 4
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 5 10 25
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.3 0.35 0.4 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.000E+06 2 3.000E+05
3 1.750E+05 4 3.700E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 9.34
CONTACT PRESSURE (CP)------------- = 730
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 3
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 0
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 34

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 9.340
3 0.000 17.000

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 1
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 4

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 1

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 1 ARE: 0.507 5 1.8

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 4 ARE: 1.940E-07 4

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

POINT NO.	VERTICAL COORDINATE	VERTICAL DISP.	VERTICAL STRESS (STRAIN)	MAJOR PRINCIPAL STRESS (STRAIN)	MINOR PRINCIAL STRESS (STRAIN)	INTERMEDIATE P. STRESS (HORIZONTAL P. STRAIN)
1	5.00000	0.08926	429.096	429.361	-1261.976	-1157.481
	(STRAIN)		3.850E-04	3.851E-04	-3.478E-04	-3.478E-04
1	40.00010	0.06515	30.263	31.204	0.646	2.796
	(STRAIN)		7.646E-04	8.015E-04	-3.961E-04	-3.961E-04

```
 2     5.00000   0.08716    239.751    273.589   -763.264    -37.077
      (STRAIN)           1.566E-04  1.712E-04  -2.781E-04  -2.781E-04
 2    40.00010   0.06721     32.229     32.399      0.583      3.417
      (STRAIN)           8.204E-04  8.270E-04  -4.198E-04  -4.198E-04

 3     5.00000   0.08423     99.587    747.898   -297.870     99.587
      (STRAIN)          -1.181E-05  2.691E-04  -1.840E-04  -1.840E-04
 3    40.00010   0.06773     32.625     32.625      0.564      3.623
      (STRAIN)           8.308E-04  8.308E-04  -4.256E-04  -4.256E-04
```

AT BOTTOM OF LAYER 1 TENSILE STRAIN = -3.478E-04
ALLOWABLE LOAD REPETITIONS = 2.184E+05 DAMAGE RATIO = 4.579E-06

AT TOP OF LAYER 4 COMPRESSIVE STRAIN = 8.308E-04
ALLOWABLE LOAD REPETITIONS = 4.071E+05 DAMAGE RATIO = 2.456E-06

****
* SUMMARY OF DAMAGE ANALYSIS *
****
AT BOTTOM OF LAYER 1 SUM OF DAMAGE RATIO = 4.579E-06
AT TOP OF LAYER 4 SUM OF DAMAGE RATIO = 2.456E-06

MAXIMUM DAMAGE RATO = 4.579E-06 DESIGN LIFE IN YEARS = 218400.

PROBLRM NO. 2

TITLE -NPROB†7

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=2, SO DAMAGE ANALYSIS WITH DETAILED PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 4
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 6 10 25
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.3 0.35 0.4 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.000E+06 2 3.000E+05
3 1.750E+05 4 3.700E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 9.34
CONTACT PRESSURE (CP)------------- = 730
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 3
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 0
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 34

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 9.340
3 0.000 17.000

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 1
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 4

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 1

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 1 ARE: 0.507 5 1.8

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 4 ARE: 1.940E-07 4

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

POINT NO.	VERTICAL COORDINATE	VERTICAL DISP.	VERTICAL STRESS (STRAIN)	MAJOR PRINCIPAL STRESS (STRAIN)	MINOR PRINCIAL STRESS (STRAIN)	INTERMEDIATE P. STRESS (HORIZONTAL P. STRAIN)
1	6.00000	0.08475	357.418	357.753	-1248.706	-1113.152
	(STRAIN)		3.553E-04	3.554E-04	-3.407E-04	-3.407E-04
1	41.00010	0.06323	28.404	29.285	0.615	2.514
	(STRAIN)		7.189E-04	7.534E-04	-3.701E-04	-3.701E-04
2	6.00000	0.08389	216.363	232.069	-819.375	-130.268
	(STRAIN)		1.655E-04	1.723E-04	-2.833E-04	-2.833E-04
2	41.00010	0.06516	30.280	30.445	0.559	3.006
	(STRAIN)		7.730E-04	7.795E-04	-3.917E-04	-3.917E-04
3	6.00000	0.08190	111.570	540.386	-434.028	111.571
	(STRAIN)		2.655E-05	2.124E-04	-2.099E-04	-2.099E-04
3	41.00010	0.06565	30.685	30.685	0.542	3.167
	(STRAIN)		7.842E-04	7.842E-04	-3.971E-04	-3.971E-04

AT BOTTOM OF LAYER 1 TENSILE STRAIN = -3.407E-04
ALLOWABLE LOAD REPETITIONS = 2.423E+05 DAMAGE RATIO = 4.127E-06

AT TOP OF LAYER 4 COMPRESSIVE STRAIN = 7.842E-04
ALLOWABLE LOAD REPETITIONS = 5.130E+05 DAMAGE RATIO = 1.950E-06

```
******************************
* SUMMARY OF DAMAGE ANALYSIS *
******************************
AT BOTTOM OF LAYER  1     SUM OF DAMAGE RATIO = 4.127E-06
AT TOP OF LAYER  4        SUM OF DAMAGE RATIO = 1.950E-06

MAXIMUM DAMAGE RATO =   4.127E-06   DESIGN LIFE IN YEARS = 242302.5

PROBLRM NO. 5

TITLE -NPROB†7

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=2, SO DAMAGE ANALYSIS WITH DETAILED PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  4
NUMBER OF Z COORDINATES (NZ)------ =  0
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 7  12  25
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.3  0.35  0.4  0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  3.000E+06   2  3.000E+05
3  1.750E+05   4  3.700E+04

LOAD GROUP NO. 1  HAS 2  CONTACT AREAS
CONTACT RADIUS (CR)--------------- =  9.34
CONTACT PRESSURE (CP)------------- =  730
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- =  3
WHEEL SPACING ALONG X-AXIS (XW)------------------- =  0
WHEEL SPACING ALONG Y-AXIS (YW)------------------- =  34

RESPONSE PT. NO. AND (XPT, YPT) ARE:  1   0.000   0.000  2   0.000   9.340
  3   0.000  17.000

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- =  1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- =  1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 1
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 4
```

```
LOAD REPETITIONS (TNLR) IN PERIOD 1  FOR EACH LOAD GROUP ARE : 1

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 1  ARE: 0.507  5  1.8

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 4  ARE:  1.940E-07 4

DAMAGE ANALYSIS OF PERIOD NO.  1  LOAD GROUP NO.  1

 POINT    VERTICAL   VERTICALVERTICAL     MAJOR       MINOR  INTERMEDIATE
                              PRINCIPAL    PRINCIAL  P. STRESS
  NO.    COORDINATE    DISP.     STRESS      STRESSSTRESS  (HORIZONTAL
                        (STRAIN)    (STRAIN)    (STRAIN)  P. STRAIN)
  1      7.00000    0.07829    306.769    307.171   -1163.807   -1017.222
        (STRAIN)            3.203E-04   3.205E-04  -3.169E-04  -3.169E-04
  1     44.00010    0.05890     24.701     25.445       0.462       1.948
        (STRAIN)            6.292E-04   6.584E-04  -3.207E-04  -3.207E-04

  2      7.00000    0.07809    197.835    205.900    -802.861    -181.523
        (STRAIN)            1.636E-04   1.671E-04  -2.701E-04  -2.701E-04
  2     44.00010    0.06055     26.311     26.457       0.409       2.259
        (STRAIN)            6.769E-04   6.826E-04  -3.382E-04  -3.382E-04

  3      7.00000    0.07671    116.562    382.640    -487.691     116.563
        (STRAIN)            4.936E-05   1.647E-04  -2.125E-04  -2.125E-04
  3     44.00010    0.06098     26.682     26.682       0.394       2.360
        (STRAIN)            6.876E-04   6.876E-04  -3.426E-04  -3.426E-04

AT BOTTOM OF LAYER  1   TENSILE STRAIN =  -3.169E-04
ALLOWABLE LOAD REPETITIONS =   3.478E+05  DAMAGE RATIO =   2.875E-06

AT TOP OF LAYER  4   COMPRESSIVE STRAIN =   6.876E-04
ALLOWABLE LOAD REPETITIONS =   8.676E+05  DAMAGE RATIO =   1.153E-06

*******************************
* SUMMARY OF DAMAGE ANALYSIS *
*******************************
AT BOTTOM OF LAYER  1     SUM OF DAMAGE RATIO = 2.875E-06
AT TOP OF LAYER  4        SUM OF DAMAGE RATIO = 1.153E-06

MAXIMUM DAMAGE RATO =   2.875E-06   DESIGN LIFE IN YEARS = 347833.4
```

7.7 Sample VBA Code Used to display design results in userforms listbox

```
Private Sub CommandButton18_Click()
'variable declaration
```

```
    Dim Sh1 As Worksheet
    Dim Sh2 As Worksheet
    Dim Sh3 As Worksheet
    Dim Sh4 As Worksheet
    Dim Sh5 As Worksheet
    Dim Sh6 As Worksheet
    Dim i As Long
'variable assignment
    Set Sh1 = ThisWorkbook.Sheets("OUTPUT_DATA(S1)")
    Set Sh2 = ThisWorkbook.Sheets("OUTPUT_DATA(S2)")
    Set Sh3 = ThisWorkbook.Sheets("OUTPUT_DATA(S3)")
    Set Sh4 = ThisWorkbook.Sheets("OUTPUT_DATA(S4)")
    Set Sh5 = ThisWorkbook.Sheets("OUTPUT_DATA(S5)")
    Set Sh6 = ThisWorkbook.Sheets("OUTPUT_DATA(S6)")
'looping to select while a particular condition meets & display results
    Select Case sclass.Value
    Case "s1"
        With MainUserForm
        If TextBox64.Value = 10 Then
           For i = 11 To Application.WorksheetFunction.CountA(Sh1.Range("B:B"))
               UserForm5.ListBox1.AddItem Sh1.Cells(i, 1).Value
               UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 1) = Sh1.Cells(i, 2).Value
               UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 2) = Sh1.Cells(i, 3).Value
               UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 3) = Sh1.Cells(i, 4).Value
               UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 4) = Sh1.Cells(i, 5).Value
           Next i
            ElseIf TextBox64.Value = 15 Then
              For i = 11 To Application.WorksheetFunction.CountA(Sh1.Range("B:B"))
                  UserForm5.ListBox1.AddItem Sh1.Cells(i, 6).Value
                  UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 1) = Sh1.Cells(i, 7).Value
                  UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 2) = Sh1.Cells(i, 8).Value
                  UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 3) = Sh1.Cells(i, 9).Value
                  UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 4) = Sh1.Cells(i, 10).Value
              Next i
            ElseIf TextBox64.Value = 20 Then
               For i = 11 To Application.WorksheetFunction.CountA(Sh1.Range("B:B"))
                   UserForm5.ListBox1.AddItem Sh1.Cells(i, 11).Value
                   UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 1) = Sh1.Cells(i, 12).Value
                   UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 2) = Sh1.Cells(i, 13).Value
                   UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 3) = Sh1.Cells(i, 14).Value
                   UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 4) = Sh1.Cells(i, 15).Value
                   'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 5) = Sh1.Cells(i, 6).Value
                   'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 6) = Sh1.Cells(i, 7).Value
                   'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 7) = Sh1.Cells(i, 8).Value
                   'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 8) = Sh1.Cells(i, 9).Value
                   'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 9) = Sh1.Cells(i, 10).Value
               Next i
            ElseIf TextBox64.Value = 25 Then
               For i = 11 To Application.WorksheetFunction.CountA(Sh1.Range("B:B"))
```

```
                UserForm5.ListBox1.AddItem Sh1.Cells(i, 16).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 1) = Sh1.Cells(i, 17).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 2) = Sh1.Cells(i, 18).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 3) = Sh1.Cells(i, 19).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 4) = Sh1.Cells(i, 20).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 5) = Sh1.Cells(i, 6).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 6) = Sh1.Cells(i, 7).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 7) = Sh1.Cells(i, 8).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 8) = Sh1.Cells(i, 9).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 9) = Sh1.Cells(i, 10).Value
            Next i
          ElseIf TextBox64.Value = 30 Then
            For i = 11 To Application.WorksheetFunction.CountA(Sh1.Range("B:B"))
                UserForm5.ListBox1.AddItem Sh1.Cells(i, 21).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 1) = Sh1.Cells(i, 22).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 2) = Sh1.Cells(i, 23).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 3) = Sh1.Cells(i, 24).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 4) = Sh1.Cells(i, 25).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 5) = Sh1.Cells(i,26).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 6) = Sh1.Cells(i, 7).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 7) = Sh1.Cells(i, 8).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 8) = Sh1.Cells(i, 9).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 9) = Sh1.Cells(i, 10).Value
            Next i
          ElseIf TextBox64.Value = 35 Then
            For i = 11 To Application.WorksheetFunction.CountA(Sh1.Range("B:B"))
                UserForm5.ListBox1.AddItem Sh1.Cells(i, 26).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 1) = Sh1.Cells(i, 27).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 2) = Sh1.Cells(i, 28).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 3) = Sh1.Cells(i, 29).Value
                UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 4) = Sh1.Cells(i, 30).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 5) = Sh1.Cells(i, 6).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 6) = Sh1.Cells(i, 7).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 7) = Sh1.Cells(i, 8).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 8) = Sh1.Cells(i, 9).Value
                'UserForm5.ListBox1.List(UserForm5.ListBox1.ListCount - 1, 9) = Sh1.Cells(i, 10).Value
            Next i
        End If
      End With
    End Select
    UserForm5.Show
End Sub
```