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IMPLEMENTATION OF THE AASHTO MECHANISTIC EMPIRICAL PAVEMENT DESIGN GUIDE (AASHTOWare PAVEMENT-ME DESIGN) FOR FLEXIBLE PAVEMENTS – THE CASE OF UGANDA

by
RONALD MUKUNDE

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AN ABSTRACT OF THE THESIS OF

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Title: <u>Implementation of The Aashto Mechanistic-Empirical Pavement Design Guide (Aashtoware Pavement-Me Design) For Flexible Pavements – The Case of Uganda.</u>

For decades, the AASHTO 1993 method was relied on for pavement design in the U.S and is still being used in some U.S states, and in countries outside the U.S, including Uganda. In 2008, AASHTO published the Mechanistic-Empirical Pavement Design Guide (MEPDG): A Manual of Practice, and released the first version of the accompanying software program AASHTOWare Pavement-ME. The release of this Mechanistic Empirical (ME) design guide generated a paradigm shift for designing and analyzing pavement structures.

Moving from the previous empirically-based to ME-based design procedures provides a number of advantages, including the evaluation of a broader range of vehicle loadings, material properties, and climatic effects; improved characterization of the existing pavement layers; and improved reliability of pavement performance predictions. However, its implementation is presented with various challenges especially in data-scarce countries outside the U.S. This thesis serves as a guide for the implementation of Pavement-ME in Uganda, a country situated in the Great Lakes region of Africa. It comprises of a sensitivity analysis on how selected input parameters effect predicted results of pavement distresses in Uganda. It also proposes a framework for the calibration of Pavement-ME as well as exploring the challenges and opportunities for implementation of Pavement ME in Uganda.

Finally, the thesis recommends a roadmap for the implementation of Pavement ME in Uganda based on the findings of this study and the data available.

<u>Keywords:</u> Pavement design, Pavement ME, Great Lakes region of Africa, Sensitivity analysis, Local calibration.

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CHAPTER 1

INTRODUCTION

1.1. Introduction

For decades, the AASHTO 1993 method was relied on for pavement design in the U.S and is still being used in some U.S states, and in countries outside the U.S, including Uganda. It is an empirical design procedure that incorporates data from AASHO road test sections constructed in Ottawa, Illinois in the late 1950's (Huang, 2004). The design methodology incorporates statistical regression models, observations and performance measurements of the test sections. Material properties in addition to other critical parameters affecting pavement performance were not reliably identified and incorporated, specifically asphalt material properties and climate.

In 2008, AASHTO published the Mechanistic-Empirical Pavement Design Guide: A Manual of Practice (MEPDG) and released the first version of the accompanying software program AASHTOWare Pavement ME Design™ (formerly DARWin-ME) in 2011(Pierce & Ginger, 2014). The MEPDG and accompanying software are based on mechanistic-empirical (ME) principles and, as such, are a significant departure from the previous empirically based AASHTO pavement design procedures. The release of the Mechanistic–Empirical Design Guide for New and Rehabilitated Pavement Structures (M-E design guide) generated a new paradigm for designing and analyzing pavement structures (Nantung et al., 2005).

Moving from previous empirically based to ME-based design procedures provides a number of advantages, including the evaluation of a broader range of vehicle loadings, material properties, and climatic effects; improved characterization of the existing pavement layers; and improved reliability of pavement performance predictions (Pierce & Ginger, 2014). However, implementation of the ME guide may require a significant increase in the required time to conduct a pavement design, in the needed data (i.e., traffic, materials, and calibration and verification to local conditions), and in the knowledge and experience of the personnel conducting the pavement design or analysis (Pierce & Ginger, 2014).

The design guide differs from other previous design procedures in the way pavement thicknesses are obtained. The design is based on an iterative approach, where the designer inputs a trial design for a desired pavement type and the performance of the pavement section is checked against performance criteria, previously set according to the type and characteristics of the design road. If the design does not meet the criteria, another pavement configuration is checked until the criteria are met. After the criteria are met, predicted distress values and reliability achieved are to be observed in order to avoid over-designed pavement structures and reach an optimum design. It is worth noting that in its current format, Pavement ME default input values and performance prediction functions apply to the US and Canada.

The software has been updated several times, and the current software version in use is Pavement ME- version 2.5.

1.2. Background

1.2.1. Geography and Geology of Uganda

Uganda is a landlocked country located in the Great lakes region of Africa. It is in the Eastern Part of Africa and borders with Kenya to the East, Tanzania to the South, Congo to the West and Sudan to the North. It is divided in four major regions; - Western, Eastern, Northern and Central.

In terms of geology, Uganda's soils are characterized by either Silty-sandy or loamy-clay soils (Brown, 2007). The Silty-sandy soils occupy a big part of the country, while the loamy-clay soils are predominantly in the South - Western region of Uganda (Hilly area), and in some parts of the East and Central regions along major water bodies such as the Nile and Victoria basins.

1.2.2. The Road Network in Uganda

In Uganda, road infrastructure comprises a network of classified (national) roads of just fewer than 20,854 km, a district (feeder) road network of just over 38,603 km and, a network of urban and city roads of 19,959 km (UNRA, 2019). Currently, a significant amount of investment is being directed towards the construction of new Asphalt pavements in addition to maintenance and rehabilitation of existing roads (Ministry of Works and Transport, 2018).

The roads sector has a vital and supportive role to play in the development of a dynamic and robust private sector, and in the efficient delivery of social services. Hence an efficient road network is a critical element in sustaining high economic growth through its contribution to increased productivity.

The Ministry of Works and Transport in Uganda has been working towards the implementation of the National Development Plan which targets the construction of 6,000 km of newly paved roads by 2021. By the end of 2019, 4,971 km of paved road network had been constructed.

The different road categories in Uganda are defined as follows: -

a. Classified Roads

Classified (National) Roads are highways that connect district to district, bypasses and are managed by the Uganda National Roads Authority (UNRA) supervised by Ministry of Works and Transport.

b. District Roads

District roads are roads that are within the district boundaries and are managed by the District Local Governments.

c. City Roads

These are roads within Kampala capital city and are managed by the Kampala Capital City Authority (KCCA).

d. Urban Roads

Urban roads are those roads within the boundaries of Urban Councils (Municipal and Town Councils) but exclude links maintained by UNRA.

1.3. Research Problem

Preliminary studies, specification reviews, and contacts with relevant stakeholders indicated that the1993 AASHTO guide and the South African Pavement Engineering Manual (SAPEM) are the mostly used methods for pavement design in the Sub-Saharan Africa, including Uganda. The major limitation of the AASHTO-93 is that the correlations are derived considering single climatic and subgrade conditions. In addition, traffic loads have dramatically changed since the 1950s and 1960s; hence, there is a higher structural demand for today's pavement structures. Similarly, SAPEM uses a critical layer approach, which ignores the contribution of each individual layer to the total pavement deformation of the whole structure and relies on linear analysis models to calculate stresses and strains (Theyse et al., 1996). This may lead to overestimation of pavement performance. In addition to climatic changes, Uganda in general has seen an increase in traffic volumes over the past decade and is expected to further grow in order to leverage the population growth pressure and economic growth (Bishai et al., 2008). As such, currently adopted design methods will not be able to address the actual challenges faced by asphalt pavements in the country. Therefore, there is an urgent need to revise the current state of pavement design practice.

The AASHTO Mechanistic Empirical Design Guide (Pavement ME), when adopted, will be extremely useful in helping decision makers translate design alterations into a computer model that would predict, under real time conditions, the performance of the proposed pavement in a matter of minutes, and decrease the level of uncertainty when studying the cost benefit analysis of alternatives over their lifetime. Studies conducted by different highways agencies in the U.S and outside the U.S have pointed towards the importance of local calibration of Pavement ME in order to have a more accurate output for pavement design (Muthadi & Kim, 2008). Therefore, as part of this research study it is necessary to explore the challenges and opportunities of the implementation of Pavement ME in Uganda through the attempt to prepare the required input data, understand the ME guide's behavior and evaluate its pavement performance prediction for implementation purposes in the country.

1.4. Objectives of the Study

The main objectives of this research study are to: -

- Conduct preliminary evaluation of the suitability of flexible pavement designs provided in country-based catalogues in withstanding various levels of traffic loading,
- Evaluate the effect of different design input parameters on the performance of asphalt pavements with respect to Ugandan conditions,
- Explore the challenges and opportunities for implementation of Pavement-ME in Uganda.
- Create a framework for implementation of Pavement-ME in Uganda.

1.5. Research Approach

In order to achieve the above stated objectives, the methodology and research procedure will comprise of the following tasks:

Task 1: Conducting literature review to build a database of the:

a. Current pavement construction specifications and mix designs by relevant stakeholders.
b. State of practice in pavement road design and construction.
c. Historical climate data.
d. Traffic conditions, classification and growth.
e. Material selection and sub-grade conditions.

Task 2: Data acquisition from relevant stakeholders

Task 3: Conducting preliminary analysis through Pavement ME to check the appropriateness of the current state of practice in road construction under current and future conditions.

Task 4: Performing a sensitivity analysis of important input parameters using Pavement ME at Level 3.

Task 5: Validating the proposed road designs against current observed roads in Uganda.

Task 6: Creating a framework for Local calibration of Pavement -ME in Uganda.

This research study will focus on establishing and defining important input parameters required to predict Total pavement rutting, Asphalt rutting (AC layer only) and Bottom-Up (fatigue) cracking, as they are the most common asphalt pavement distresses in Uganda.

CHAPTER 2

OVERVIEW OF FLEXIBLE PAVEMENT DESIGN PROCEDURES

2.1.Introduction

Pavement design at its early beginnings before the 1920’s was dependent mainly on providing pavement layer thicknesses to protect subgrade against shear failure. Experience in previous projects played an important role in the evolution of several design procedures based on shear strength (Huang, 2004).

As time passed, important factors other than subgrade shear resistance came into the picture and their evaluation was essential for optimum pavement design. That’s when pavement performance was introduced through ride quality and the evaluation of other surface distresses that increase the rate of deterioration of a pavement structure. Serviceability of pavements became the focus for pavement design procedures where test tracks were used for experiments to quantify such measure. The AASHO road test conducted in the late 1950’s provided the basis for the evolution of the AASHTO design guide (Huang, 2004).

Empirical methods evolved from test track experiments to provide systematical methods for pavement design based on field testing and findings. Such methods provided results of good accuracy but are limited to the site conditions and the materials used for the derived equations (Carvalho & Schwartz, 2006).

With the advancement in materials' technology, new methods were introduced for protecting the subgrade. The newly introduced materials came with their own failure modes that had to be incorporated into the design procedures. The isotropic linear elastic theory was not appropriate in that case and the presence of nonlinearities, temperature and time dependency necessitated the introduction of advanced modeling to predict pavement performance mechanistically. The introduction of a new design procedure was of essence. A design procedure based on theories of mechanics that relates pavement structural behavior and performance to traffic loading and environmental conditions seemed to be the suitable approach. Despite the huge efforts exerted with regards to a fully mechanistic performance procedure, no such procedures are commercially available for application. However, a mechanistic-empirical approach was introduced to act as an intermediate stage towards a fully mechanistic design procedure (Hall et al., 2011). The Mechanistic-empirical approach is a hybrid approach that uses empirical models to fill the gaps between the mechanistic theory and the pavement performance. Mechanistic models function well in calculating pavement responses to loading such as stresses and strains but cannot predict pavement performance directly, and empirical transfer functions shall be used for appropriate correlation.

The objective of this chapter is to have an overview on advancements in flexible pavement design procedure where two major procedures are presented: empirical methods and mechanistic-empirical methods.

2.2.Empirical Methods

Empirical approaches are usually used when it is difficult to define the cause-and-effect relationships of a phenomenon. In an empirical design approach, observations are used to relate inputs and outputs of pavement design and performance. It is challenging to find a rational scientific basis for the developed relationships. However, engineering reasonableness and logic must be met.

In the mid-1920's, the first empirical methods were developed in conjunction with the first soil classification. In 1929, a method using the California Bearing Ratio (CBR) strength test was developed by the California Highway Department (Huang, 2004), where the material's CBR value was related to the required thickness to protect the subgrade against shear failure. Then, this method was developed by U.S. Corps of Engineers (USCE) in World War II and later became the most popular design method. The first soil classification system to be published was the Public Roads Authority (Huang, 2004). The Highway Research Board (HRB) modified this classification, where soils were grouped in 7 categories (A-1 to A-7) with indexes to differentiate soils within each group. The subbase quality and total pavement thickness were then estimated.

With the introduction of new materials to improve the pavement performance and smoothness and due to the increase in traffic loading and vehicle speed, shear failure was no longer the governing criterion.

Measuring pavement surface deflection was the first attempt to consider a structural response as a quantitative measure of pavement structural capacity. Later, other methods were developed that incorporated strength tests. Given that deflection is easy to measure in the field, it was attractive for practitioners to use it as a failure criterion although failures in pavement are more likely to happen due to excessive stresses and strains rather than deflection (Carvalho & Schwartz, 2006).

It was realized that pavement performance was of great importance to a pavement system and the link between it and design inputs must be investigated. In the 1950's, experiments conducted in tracks gave a better perspective for linking design inputs to performance data through regression models leading to the introduction of AASHTO - 93 design method.

The AASHTO - 93 method based on the AASHO road test is still the most widely used method today (Hamdar & Chehab, 2017).The AASHTO design equation is a regression relationship between the number of load cycles, pavement structural capacity and pavement performance measured in terms of serviceability. The serviceability index is based on surface distresses commonly found in pavements. In addition to test tracks, regression equations can also be developed using performance data from existing pavements (AASHTO, 1993).

The main disadvantage of regression models is that they are limited for application in conditions similar to those for which they were developed. Although they provide a fair understanding of pavement performance, their limited consideration of materials and construction data result in much uncertainty.

2.2.1. The AASHTO - 93 Guide

2.2.1.1.Method Description

Based on the results of the AASHO Road Test conducted in the late 1950's and early 1960s in Ottawa, Illinois, AASHTO published an interim design guide in 1961. The main objective of the test was to find a relation between the number of axle load repetitions and the performance of flexible and rigid pavement. It was revised several times until it was issued in 1986 and then in 1993. The empirical performance equations were developed under a given climatic setting with a specific set of pavement materials and subgrade soils. Design variables for the design method include:

- Time constraints
- Effective roadbed soil resilient modulus
- Structural number
- Traffic
- Reliability
- Environmental effects
- Serviceability

The selection of layer thicknesses is done by determining the structural number, which is function of layer thicknesses, layer coefficients and drainage coefficients. Using a nomograph developed for solution of the design equation with the available design variables, layer thicknesses are determined.

2.2.1.2. Major drawbacks of 1993 AASHTO Guide

i. Traffic Characterization

ESAL was used to characterize the traffic loading and the equivalency factors developed at the AASHO Road Test are highly doubtful to be applicable to today's traffic stream (combination of axle load, traffic levels and types of axles). The AASHO road test pavements carried approximately 1 million axle loads, while interstate roads in the US back then were designed for 5 to 10 million ESALs. Today, interstate pavements are designed for 50 to 200 million or more axle load applications. The original empirical pavement design models may not produce realistic designs.

ii. Materials Characterization

Reliable mix designs such as SuperPave™ and improved asphalt mixtures such as stone matrix asphalt, polymer-modified asphalt, natural fiber-asphalt etc. are not directly incorporated into the empirical design model. On the durability side, there were few material durability problems, such as asphalt stripping, over the 2-year AASHO Road Test period. Thus, the effect of long-term material durability on performance was not considered.

iii. Foundation Characterization

Pavements at the AASHO Road Test site were constructed over a single silty-clay (AASHTO A-6) subgrade. The effect of this single subgrade was “built into” the empirical design models.

iv. Empirical Nature of Pavement Design Equations

Using 2 years of pavement performance data, a combination of graphical techniques and least squares regression were used to develop the empirical pavement equations using. No field verification was performed for the original models being extended over time based on empirical methods. Serious design deficiencies are also found with respect to calculation of layer thicknesses since the procedure solves only for SN and not layer thickness.

v. Climate

Two major limitations are found with respect to climate. The first is the single environmental condition the empirical design equations were developed under, which was for the AASHO Road test site in northern Illinois. This implies that equations used are calibrated for just one climatic condition. The second limitation is the limited time interval for the road test, being 2 years. These 2 years provided only 2 annual climatic cycles, whereas pavement sections are normally designed for design lives up to 20 years or more.

2.3.Mechanistic Empirical Pavement Design Guide

Having recognized the need for a nationally developed and calibrated ME pavement design procedure, the AASHTO Joint Technical Committee on Pavements proposed a research effort to develop such a design procedure that would be based on current state-of-the-practice pavement design methods (AASHTO, 2008). This proposal led to the initiation of NCHRP Project 1-37, Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures, and subsequently, NCHRP Project 1-37A, Guide for the Design of New and Rehabilitated Pavement Structures, and NCHRP Project 1-40, Facilitating the Implementation of the Guide for the Design of New and Rehabilitated Pavement Structures. The products of these projects included an ME pavement design guide, rudimentary software, and a performance prediction model calibration guide.
The Mechanistic Empirical Pavement Design Guide (MEPDG) developed under NCHRP projec1-37A is based on mechanistic-empirical principles (Ayyala et al., 2018). It uses performance prediction models based on such principles to predict the performance of a

pavement over a specified design life. The ME guide differs from the previous AASHTO design guides in terms of inputs required, design procedure and output produced. It uses traffic, climate, pavement layers structure and material property data as input and returns the predicted performance of the pavement in terms of distress and roughness as output (Ayyala et al., 2018).

The design procedure is mechanistic-empirical in nature – the mechanistic component involves computation of stresses, strains and deflections which are fundamental responses of a pavement subjected to loading and temperature change, while the empirical component relates these fundamental responses to pavement distresses using empirical equations referred to as transfer functions to compute accumulated damage (Witczak et al., 2002). The objective of design is to minimize the predicted distresses: fatigue cracking, thermal cracking, rutting and the roughness (IRI) such that the performance of the pavement is maximized over its service life.

The design procedure begins with an initial set of values for the input variables often referred to as the control set of input values, which are varied by the user such that the predicted distresses are within specified limits and reliability. Therefore, an accurate knowledge of the input variables for which user-defined values are required, along with the tolerances and range by which they realistically vary is essential for reliable pavement design using the ME guide.

2.3.1. Interface and Design Input parameters

Various modules of the AASHTOware pavement ME design is listed below(AASHTO), 2014):

- General design inputs - Include information like pavement design type, pavement type, design life, and time of construction and opening to traffic.
- Performance criteria - Designer specified threshold value of performance prediction models and level of reliability.
- Traffic - Input data to determine the vehicle loadings on the pavement structure. These data can be derived from weigh-in-motion (WIM) sites, automatic vehicle classification (AVC) sites, statewide averages, or national averages. National default values are available for the majority of inputs.
- Climate - This type of inputs is required to assess the environmental effects on material responses and pavement performance. Besides the data from 1,083 US and Canadian weather stations in the software (AASHTO, 2011), virtual weather stations can also be created from existing weather stations and new weather stations can be added.
- Asphalt layer design properties - Comprises of surface shortwave absorptivity, fatigue endurance limit (if used), and the interface friction.
- Concrete layer design properties - for JPCP, this information includes, joint spacing and sealant type, dowel diameter and spacing, use of a widened lane or tied shoulders, and instruction related to the erodibility of the underlying layer. For CRCP, design properties include, percent steel, bar diameter, and bar placement depth.

- Pavement structure – This module allows the designer to enter the material types, asphalt mix volumetrics, concrete mix information, mechanical properties, strength properties, thermal properties, and thickness for each layer of the pavement section.
- Calibration factors – There are two options of specifying calibration coefficients of the performance prediction models. One is nationally calibrated program level calibration coefficients, and another is designer specified project-specific calibration coefficients. Unless otherwise mentioned, AASHTOWare will utilize the program-level calibration coefficients in the analysis.
- Sensitivity – This option allows the designer to define minimum and maximum values for different parameters like air voids, percent binder or layer modulus to determine the impact on the predicted condition.
- Optimization - This feature is utilized to determine the minimum layer thickness of a single layer that satisfies the performance criteria. In this mode, the designer inputs the minimum and maximum layer thickness for the layer to be analyzed. Then the software iterates the layer thickness within the specified range while all other inputs remain constant and the software determines the minimum layer thickness required to meet the selected performance criteria.
- Reports – The input summary, traffic loading prediction charts, climatic summary, material properties summary and other design tables and charts can be extracted as a PDF file and also in Microsoft Excel format.

A huge advantage of Pavement ME is the input approach that provides more realistic representation of the factors that act on a pavement. This is provided through the detailed level of inputs required. ME uses these inputs to represent the interaction that occurs between the various inputs which simulates the actual conditions that are expected to act on a pavement system throughout its life span. The ME software can be used to analyze a broad range of pavement design types, materials, traffic loadings, and climate regions. Described below are the major inputs required for pavement design using the AASHTOWare Pavement ME software:

i. Traffic :

Truck traffic is characterized according to the distribution of axle loads for a specific axle type (i.e., axle-load spectra), hourly and monthly distribution factors, and distribution of truck classifications (i.e., the number of truck applications by FHWA vehicle class). Truck traffic classification groups have been developed to provide default values for normalized axle-load spectra and truck volume distribution by functional classification. Pavement ME also provides the ability to analyze special axle configurations.

Traffic data is one of the key inputs for design of pavements structures. The ME guide adopts the use of load spectrum to encounter for traffic data acting on the pavement section. The required traffic data is categorized into four major groups:

Basic Information:

- Average Annual Daily Truck Traffic for the base year
- Percent truck in the design direction
- Percent Truck in the design lane
- Operational speed of vehicles

Traffic Volume Adjustment:

- Monthly adjustment factors
- Vehicle class distribution
- Hourly Truck traffic distribution
- Traffic growth factors

Axle Load Distribution Factors:

- Percent of the total axle applications within each load interval for
- Specific Axle Type: Single, Tandem, Tridem and Quad
- Specific Vehicle Class (Classes 4 to 13 of the FHWA Classification)

General Traffic Inputs:

- Mean Wheel Location
- Traffic Wander Standard Deviation
- Design Lane Width
- Number of Axles per truck class
- Axle configuration
- Wheelbase

The approach adopted in ME guide of using load spectra allows for the simulation of mixed traffic directly without the need of converting them to ESALs like the AASHTO method. This allows for special vehicle analysis, overloaded trucks analysis and weight limits analysis during critical environmental conditions (NCHRP, 2004).

ii. Materials :

Materials property characterization includes asphalt, concrete, cementitious and unbound granular materials, and subgrade soils. Laboratory and field testing are in accordance with AASHTO and ASTM test protocols and standards. The key layer property for all pavement layers is modulus (dynamic modulus for asphalt layers, elastic modulus for all concrete and chemically stabilized layers, and resilient modulus for unbound layers and subgrade soils). Three models in Pavement ME require material properties: climatic model, response model and distress model. The EICM uses the material properties along with the climate data to provide adjusted material properties taking into account the environmental effects. The adjusted material properties along with traffic loading are fed into the response models, where stresses and strains are calculated at critical locations. The calculated responses along with other material properties are fed into the distress models to predict the pavement performance.

The ME guide introduces a new philosophy in material input for flexible pavement design. This new approach is the account for the dynamic modulus for asphalt concrete and nonlinear stiffness model for unbound material. Time and temperature dependency of asphalt mixtures is modeled by dynamic modulus.

The dynamic modulus master curve models the variation of asphalt concrete stiffness due to rate of loading and temperature variation (hardening with low temperature/high frequency and softening with high temperature/low frequency). The nonlinear elastic behavior of unbound granular materials is modeled by a stress-dependent resilient modulus included as level 1 input (NCHRP, 2004);(Pierce & Ginger, 2014). Table 0-1 illustrates inputs required for material properties for different models within the MEPDG.

Table 0-1. Inputs requirements for Pavement ME models.

Material Type	Material properties required for different model input		
	Climatic Models	Response models	Distress Models
Asphalt concrete	**Mixture:** - Heat capacity - Thermal conductivity - Surface shortwave absorptivity	- Dynamic Modulus (E^*) of HMA - Poisson's ratio	- Tensile strength - Creep compliance - Coefficient of thermal expansion
	Asphalt Binder: - Viscosity (stiffness) characterization to account for aging.		
Unbound materials	- Plasticity index - Gradation parameters - Effective grain sizes - Specific gravity - Saturated hydraulic conductivity - Optimum moisture content - Parameters to define the soil-water characteristic curve	- Resilient modulus, $\mathbf{M_R}$ at optimum density and moisture content - Poisson's ratio - Unit Weight - Coefficient of lateral pressure	- Gradation parameters

iii. Climate :

Consideration of climate effects on material properties using the Integrated Climatic Model. This is used to model the effects of temperature, moisture, wind speed, cloud cover, and relative humidity in each pavement layer. These effects, for example, include aging in asphalt layers, curling and warping in concrete pavements, and moisture susceptibility of unbound materials and subgrade soils.

Incorporating detailed climatic data for the designed pavement section is another advantage. Data is fed into the Enhanced Integrated Climatic Model (EICM) embedded in Pavement ME. The input data is as follows:

- Hourly air temperature
- Hourly precipitation
- Hourly Wind speed
- Hourly percentage Sunshine
- Hourly relative humidity

Pavement ME contains a database of more than 1000 weather stations that contains the above-mentioned data covering the United States and Canada. This research uses climatic data extracted from the MERRA online platform, that is compatible with Pavement ME requirements. Additional climate data required are:

- Groundwater table depth
- Drainage/surface properties
- Surface shortwave absorptivity
- Infiltration
- Drainage path length
- Cross slope

The EICM uses the above-mentioned data to calculate moisture and temperature distributions within the pavement structure. Hence, variations of material properties can be calculated. Properties such as asphalt concrete and unbound material stiffness are sensitive to moisture variations (NCHRP, 2004).

iv. Performance prediction :

Pavement ME includes transfer functions and regression equations to predict pavement distress and smoothness, characterized by the International Roughness Index (IRI).

In order to calculate the structural responses generated in the pavement system, Pavement ME uses three models. Stresses, strains and displacements due to traffic loading are calculated using Multi-layer Elastic Theory (MLET) and the Finite Element Method (FEM). The FEM is used when a non-linear behavior of unbound material is desired through level 1 input; otherwise the load-related analysis is performed using MLET. The third model is the EICM, which is used for non-load-related temperature and moisture variations throughout the pavement system (NCHRP, 2004).

MLET is applied for multi-layered pavement systems of materials that have linear elastic properties. Burmister's layered theory is used for such materials following basic assumptions (Huang, 2004):

- Each layer is homogeneous, isotropic, and linearly elastic, characterized by Young's modulus of elasticity, E, and Poisson's ratio, ν.
- The material is weightless and horizontally infinite.
- The thickness of each layer is finite, and the subgrade is considered as infinite layer.
- The load is uniformly applied on the surface over a circular area.
- Continuity conditions are satisfied at the layer interfaces.

The main disadvantage of MLET is its inability to consider nonlinearities often exhibited by pavement materials.

The FEM is a multipurpose tool that has the capability of structural modeling multi-layer pavement systems having material properties that vary both vertically and horizontally. The concept of FEM is to subdivide an element into small discrete units forming a mesh, then calculating the stresses and strains across each unit. Equilibrium requirements are then applied to combine the individual units and get the formulation for the global problem in terms of a set of simultaneous linear equations. Although its suitability for pavement structural evaluation and response prediction, it requires longer computational time compared to MLET (NCHRP, 2004).

EICM is a mechanistic model of one-dimensional heat and moisture flow that simulates changes in the behavior and characteristics of pavement and subgrade materials induced by environmental factors. EICM represents a powerful tool in Pavement ME. The model takes into account the daily and seasonal variations of temperature and moisture within the pavement structure, which are induced by the location of the pavement system and produces the relevant material properties according to these factors. This is an important input for the structural response models since different materials have different responses to environmental variations. Asphalt concrete dynamic modulus varies with the variation of temperature as well as unbound material have different properties under different moisture content (NCHRP, 2004).

Structural responses are identified at critical locations in plan and at different depths of a pavement system based on maximum damage. Maximum responses calculated at each of these locations are used to predict the pavement performance through the calculation of various distresses. Variations in the material properties are tackled by subdividing each pavement layer into several sub-layers. Each of these sub-layers has its own properties. Critical pavement response variables include:

- Horizontal Tensile strain at the bottom/top of AC layer (for AC fatigue cracking)
- Vertical Compressive stresses and strains with AC layer (for AC rutting)
- Vertical Compressive stresses and strains within the base/subbase layer (for rutting of unbound layers)
- Vertical Compressive stresses and strains at the top of the subgrade (for subgrade rutting)

2.3.2. Design Criteria

The guide presents the design procedure in an iterative process. A design section is analyzed, and the predicted performance is checked against a previously set design criteria. The design criteria are one of the input values for the MEPDG. Limit values for the predicted distresses and reliability are inserted and checked against after analysis of the pavement section. These criteria include:

- Terminal IRI (in/Mile)
- AC Surface Down Cracking (Long. Cracking) (ft/mile)
- AC Bottom Up Cracking (Alligator Cracking) (%)
- AC Thermal Fracture (Transverse Cracking) (ft/mile)
- Chemically Stabilized Layer (Fatigue Fracture)
- Permanent Deformation (AC Only) (in)
- Permanent Deformation (Total Pavement) (in)

An initial value for IRI that defines the as-constructed smoothness of the pavement is also required. Typical values for these criteria shall be defined by agencies according to the road class, location, importance of the project, and economics (NCHRP, 2004). Another integral aspect of the MEPDG is the incorporation of input hierarchical levels. Although the analysis method is independent of the input level (i.e., regardless of the input level, the same analysis is conducted), the idea of including a hierarchical level for inputs is based on the concept that not all agencies will have detailed input data or that every pavement needs to be designed with a high level of input accuracy. For example, an agency would not necessarily use the same level of inputs for pavements on rural roads as they would for an urban interstate.

The input levels included in the MEPDG are as follows (AASHTO, 2008):

Level 1: Inputs are based on measured parameters (e.g., laboratory testing of materials, deflection testing) and site-specific traffic information. This level represents the greatest input parameter knowledge, but requires the highest investment of time, resources, and cost to obtain.

Level 2: Inputs are calculated from other site-specific data or parameters using correlation or regression equations. This level may also represent measured regional (non-site-specific) values.

Level 3: Inputs consist of default or user-selected values based on expert opinion, and global or regional averages such as LTPP sites. It represents the lowest level of the hierarchy system and provides the lowest level of reliability.

The ME guide recommends that the pavement designer use as high a level of input as available. Selecting the same hierarchical level for all inputs, however, is not required (AASHTO, 2008). Each agency is expected to determine the input level related to roadway importance, and data collection effort costs and time.

The MEPDG provides recommended input levels for site conditions and factors, rehabilitation design, and material properties (AASHTO, 2008).

National calibration of the pavement prediction models used in Pavement ME are based on the data included as part of the Long-Term Pavement Performance (LTPP) research program, and research studies from the Minnesota pavement test track (MnROAD) and the FHWA accelerated loading facility. Table 0-2 provides a list of pavement types that are included in the MEPDG.

Table 0-2. Pavement types included in the MEPDG

Asphalt Pavements	**Concrete Pavements**
➢ Conventional – 2 to 6in. asphalt layer over unbound aggregate and soil aggregate layers. ➢ Deep strength – thick asphalt layer(s) over an aggregate layer. ➢ Full-deep – asphalt layer(s) over stabilized layer or embarkment and foundation soil. ➢ Semi – rigid - asphalt layer(s) over cementitious stabilized materials. ➢ Cold In-place Recycle (CIR) – designed as a new flexible pavement. ➢ Hot In-place Recycle (HIR) – designed as mill and fill with asphalt overlay. ➢ Asphalt overlays (>2in.) – over existing asphalt pavement and	➢ JPCP – with or without dowel bars, over unbound aggregate, and/or stabilized layers. ➢ CRCP – over unbound aggregate, and/or stabilized layers. ➢ JPCP overlays (>6in.) – over existing concrete, composite, or asphalt pavements (minimum thickness of 6in. and 10 ft or greater joint spacing). ➢ CRCP overlays (7in.) – over existing concrete, composite or asphalt pavements (minimum thickness of 7 in.). ➢ JPCP restoration – diamond grinding, and a variety of pavement restoration treatments.

intact concrete pavements, with or without pre-overlay repairs, and milling.	

Source: (AASHTO, 2008).

JPCP = jointed plain concrete pavements; CRCP = continuously reinforced concrete pavements.

Performance prediction models included in the MEPDG are provided in Table 0-3. Since the release of the NCHRP 1-37A final report in 2004, a number of additional study efforts have been completed or are currently on-going to improve the MEPDG performance model prediction. These include:

• Reflective cracking model—NCHRP Report 669: Models for Predicting Reflection Cracking of Hot-Mix Asphalt Overlays (Lytton et al., 2010).

• Rutting models—NCHRP Report 719: Calibration of Rutting Models for Structural and Mix Design (Von Quintus et al., 2016).

• Longitudinal cracking model—NCHRP Project 1-52, A Mechanistic-Empirical Model for Top-Down Cracking of Asphalt Pavement Layers

Table 0-3. Performance prediction models included in the MEPDG

Asphalt Pavements	**Concrete Pavements**
➢ Rut depth – total, asphalt, unbound aggregate layers, and subgrade (inches). ➢ Transverse (thermal) cracking (non-load-related) (feet/mile). ➢ Alligator (bottom – up fatigue) cracking (percent lane area). ➢ Longitudinal cracking (top-down) (feet/mile).	➢ Transverse cracking (JPCP) (percent slabs). ➢ Mean joint faulting (JPCP) (inches). ➢ Punchouts (CRCP) (number per mile). ➢ IRI – predicted based on other distresses (JPCP and CRCP) (inches/mile).

➢ Reflective cracking of asphalt overlays over asphalt, semi-rigid, composite, and concrete pavements (percent lane area). ➢ IRI – predicted based on other distresses (inches/mile)	

Source: (AASHTO, 2008).

JPCP = jointed plain concrete pavements; CRCP = continuously reinforced concrete pavements.

2.3.3. How it works:

The general approach for conducting a pavement design and analysis is structured according to three major stages, each containing multiple steps; each stage of the MEPDG design process is summarized as follows (AASHTO, 2008):

• Stage 1—Determine materials, traffic, climate, and existing pavement evaluation (for overlay designs) input values for the trial design.

• Stage 2—Select threshold limits and reliability levels for each performance indicator to be evaluated for the trial design. Conduct the analysis on the trial design. If the predicted performance does not meet the criteria at the specified reliability level, the trial design is modified (e.g., thickness, material properties) and re-run until the performance indicator criteria is met.

• Stage 3—Evaluate pavement design alternatives. This analysis is conducted outside the MEPDG and may include an engineering analysis and life-cycle cost analysis of viable alternatives.

A summary of the MEPDG philosophy is summarized in Figure 0-1 below.

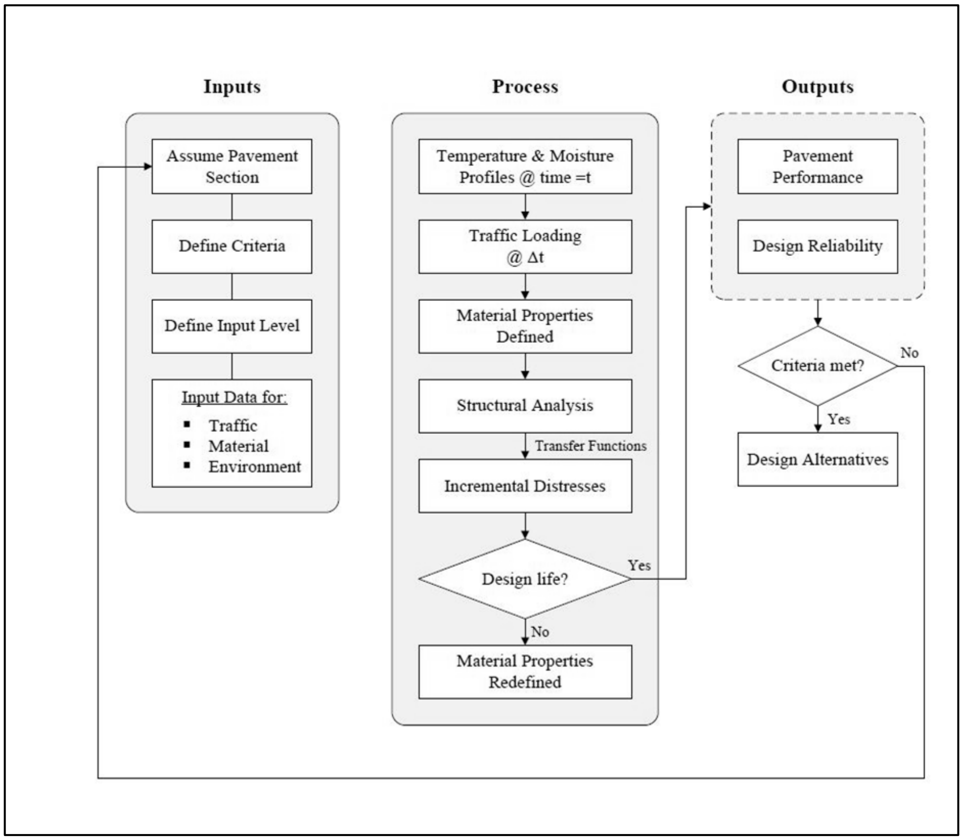

Figure 0-1. MEPDG Philosophy

During the process, the following sequence of operations is undertaken. At time = t,

i. The temperature and moisture profiles through the pavement are generated for the conditions at time = t (Environment)

ii. The spectrum of traffic loadings in the next time increment (Δt) are defined (Traffic)

iii. The elastic properties and thickness of each layer (E, μ and h) are defined from the initial input, the age since construction, the temperature and moisture profiles, and the speed (duration or frequency) of each load (Materials).

iv. The structural analysis is performed to estimate critical stresses and strains within the structure (Mechanistic)

v. An additional analysis is performed to determine the non-load-related stresses and strains (i.e., due to thermal conditions) (Mechanistic).

vi. The load-related and non-load-related critical stresses and strains are combined (Mechanistic).

vii. The incremental distresses are computed based on the critical stresses and strains (or their increments). These include the basic set of distresses and are computed based on calibrated empirical models (Empirical).

viii. Changes in initial material parameters (E, μ) resulting from the computed incremental damage are estimated.

ix. The time scale is incremented to $t = t_0 + \Delta t$, and the cycle is repeated

If the trial design satisfies agency-approved performance criteria at the specific design reliability, it becomes a candidate design structure and undergoes life cycle cost analysis (LCCA) for constructability (Kim et al., 2011). The Federal Highway Administration has listed the following advantages of the ME design approach over traditional empirical approaches (FHWA, 2017):

- Provision of agency-established performance criteria for design
- Ability to characterize material parameters to reflect pavement performance;
- Capability to evaluate pavement damage caused by unique loading configurations or increased axle loads;
- Inclusion of seasonal variation effects; and
- Capability to consider alternate design strategies and additional design features.

CHAPTER 3

LITERATURE REVIEW

3.1.Introduction

The topic of investigating and implementing the ME guide has attracted significant attention from researchers in the field of pavement design, in the United States and worldwide. Hundreds of publications, agency reports, as well as conference and journal papers have been published covering this topic from several aspects.

Since the issuance of MEPDG in mid-2000, some states committed to immediate implementation initiatives, such as new testing programs for developing material properties and traffic data, and establishment of permanent calibration test sections as part of partial calibration efforts of the current Pavement - ME software for local conditions. According to (Mallela et al., 2009), the most popular topics for researchers regarding the ME guide were found to be as follows:

- Characterization of input parameters such as traffic loading, layer material and subgrade foundation properties, climate, and other design features.
- Sensitivity of performance models to agency specific inputs.
- Validation and calibration of the pavement distress prediction models.
- Agency business plans and strategies for local implementation of the MEPDG.

Since the topic is relatively new, plenty of research is still being conducted and it would be an exhaustive task to cover each and every aspect of them within this research.

Therefore, this literature will only cover items of interest related to using the ME guide for flexible pavement design and related to implementing the design procedure in Uganda. This chapter discusses the development of ME guide, implementation and calibration efforts, and the current state of pavement design practice in Uganda.

3.2. The Development of the ME Guide

In recognition of the limitations of the AASHTO 1993 design procedures, the AASHTO Joint Task Force on Pavements initiated an effort to develop an improved pavement design guide in a workshop held in 1996 in Irvin, California (Coree et al., 2005). The purpose of the workshop was to develop a framework for improving the pavement design guide based on mechanistic- empirical principles with numerical models calibrated using pavement performance data from Long Term Pavement Program (LTPP). AASHTO then initiated two major projects NCHRP 1-37A and NCHRP 1-40 for the development of the guide. NCHRP project 1-37A was initiated in February 1998 and ended February 2004 (Ali, 2005). The project led to a Mechanistic Empirical Pavement Design Guide that included (*1*) a Guide for ME design and analysis, (*2*) companion software with documentation and a user manual, and (*3*) implementation and training materials (Ali, 2005). In June 2004, a research version of MEPDG was distributed for interested users for review and evaluation as a result of NCHRP project 1-40A. The MEPDG software has been subsequently updated under NCHRP Project 1-40D from the original version to MEPDG software Version 1.0. Several versions of the MEPDG software were released starting with the draft software Version 0.7 in June 2004, Version 0.9 in June 2006, Version 0.91 in September 2006, Version 1.00 in April 2007, Version 1.10 in August 2009, and DARWin-

ME which was released at the end of April 2011. Version 1.0 was balloted and approved by NCHRP, FHWA, and AASHTO as an interim AASHTO standard in October 2007 (Bayomy et al., 2012). In July 2008, AASHTO released an interim edition of Mechanistic-Empirical Pavement Design Guide: A manual of practice (AASHTO, 2008). In June 2012, AASHTO terminated the licensing and technical support for the AASHTO 1993 pavement design guide software DARWin and the transition of DARwin-ME to AASHTOWare Pavement ME Design took place in 2013. AASHTOWare Pavement ME version 2.5 is the latest version of the AASHTOWare series (Islam et al., 2019).

NCHRP, FHWA, and others have undertaken several research studies related to the MEPDG. Some examples are presented in Table 0-1. In addition to the projects shown in this table, hundreds of papers and reports have been published on various aspects of implementing the MEPDG (Mallela et al., 2009).

Table 0-1. Research studies for MEPDG review and Evaluation

Research Objective	**Research Title**	**Agency**
MEPDG review/ evaluation	➢ NCHRP 1-40A – Independent Review of the recommended Mechanistic – Empirical Design Guide and software	NCHRP
Improved performance modeling	➢ NCHRP 1-42 – Top-Down fatigue cracking of Hot Mix Asphalt layers	NCHRP
	➢ NCHRP 9-38 – Endurance Limit of Hot Mix Asphalt layers to prevent Fatigue Cracking in flexible pavements	NCHRP
	➢ NCHRP 1-41 – Selection, Calibration and Validation of a Reflective Cracking Model for Hot Mix Asphalt overlays	NCHRP
	➢ NCHRP 9-30 A – Rutting performance model for HMA mix and structural design	NCHRP

Development of Support tools	➢ NCHRP 1-39 – Traffic data collection, Analysis and forecasting for mechanistic pavement design	NCHRP
	➢ NCHRP 9-33 – A mixture design Manual for Hot Mix Asphalt	NCHRP
Implementation (into new or existing tools)	➢ NCHRP 9-30(01) – Expand Population of the M-E Database and conduct two pre-implementation studies	NCHRP
	➢ NCHRP 9-22 – Beta testing and validation of HMA performance related specifications	NCHRP
	➢ Modification of FHWA Highway performance data collection system and pavement performance models	FHWA
	➢ Adapting the improved models to NAPCOM	FHWA
	➢ Implementation and support of New Pavement Equations for Highway Economic requirements system	FHWA
	➢ Creation of reports for Pavement remaining Service Life using the Improved pavement performance models developed for HERS	FHWA
Technology transfer	➢ FHWA-NHI-131109 – Analysis of New and rehabilitated Pavements with M-E Design Guide	NHI
	➢ FHWA-NHI-131064 – Introduction to Mechanistic Design for New and Rehabilitated Pavements	NHI
	➢ FHWA Design Guide Implementation Team (DGIT) workshops on materials, climate, traffic, local calibration, etc.	FHWA

3.3. Implementation Initiatives of ME Design Guide

3.3.1. In the USA and Canada

After the release of Pavement ME, several states in the U.S have attempted implementation of the software for routine pavement design. NCHRP synthesis 457 conducted a survey in 2014 among fifty-seven highway transportation agencies across North America and reported that three agencies had implemented ME guide approaches and forty-six agencies were evaluating ME models. The technical report of the AASHTO Pavement ME national user group in 2017 stated that nine highway agencies (out of twenty-one responding) have successfully implemented the ME software for asphalt pavements. The report also listed several challenges faced by the state highway agencies in implementing the ME software. Local calibration and verification of Pavement ME performance models topped the list. Other challenges include availability of performance data, characterizing bound and unbound layer material properties, compatibility of performance measures and threshold criteria.

One of the prerequisites of implementing the software for routine design is to calibrate and validate the performance models to local conditions. In addition, truck-traffic characterization, developing a material inputs database, and establishing performance criteria and distress-wise reliability levels are also required. Nantung et al. proposed a six-step MEPDG implementation plan for the Indiana Department of Transportation. These steps include reviewing existing state-of-knowledge in pavement engineering and management, documenting hierarchical design input parameters, reviewing long-term pavement performance data pertinent to the agency, assessing laboratory and field investigation required for higher-level design inputs, executing local calibration and validation of MEPDG distress models, and providing necessary technology and training to implement ME design approaches at the district, local agency, and contractor levels

3.3.2. Outside the USA and Canada

Many states in the U.S. are far ahead globally in the implementation of Pavement ME design software and that is due to the availability of a reliable data base needed for input, in addition to hundreds of test sections that are used for the calibration of the distress models. Pavement ME software is globally calibrated under NCHRP 1-37A and 1-40 projects using a representative database of test sections monitored by the Long-Term Pavement Performance (LTPP) project only in North America. But due to the huge variation of the climate, materials, construction and maintenance practices across North America, local authorities have to do local software calibration of the transfer functions for the best prediction of the road performance. Even though this experimental procedure is identified as "local calibration", this process usually involves local verification, followed by calibration, followed by validation of the software.

Although implementation of the AASHTO MEPDG outside the U.S and Canada is still hindered by its complexity and large data input requirements, there have been efforts in some countries to take initiative towards transitioning from the previous AASHTO 1993 method to the current state- of-the-Art ME design guide.

In Europe, a study was conducted in Italy to address the implementation of MEDPG under the Italian conditions. The study emphasizes the need for updating the current Italian Pavement Design Catalog produced in 1993. The Catalog provides a series of standard pavement structures for 8 different types of road, in which the Italian road network is subdivided for Catalog purposes. The Catalog is mainly based on data from AASHTO 1972 and 1986 design procedures. Due to limitations of hypothetical assumptions for climate, traffic and material properties, the study concludes that the implementation of the ME guide is deemed necessary to overcome issues related to the uneconomic designs produced from the current Italian Catalog (Celauro, C., & Khazanovich, 2005). Another research was conducted in Italy in 2012 handling the calibration and implementation of the ME guide in Italy (Caliendo, 2012).

In Sweden, a research conducted in 2011 evaluated three mechanistic-empirical rutting models (MEPDG, CalME and PEDRO) under Swedish conditions with respect to traffic, climate and materials using accelerated pavement testing and long-term pavement performance studies. The study concluded that M-E PDG results were more accurate at the lowest material input data quality level (level 3) than at the highest (level 1). The main cause was probably the demonstrated inaccuracy of the predicted dynamic modulus at level 3 compared to the measured level 1 results, and the M-E PDG calibration at level 3. The CalME underestimated the permanent deformation in the semi-rigid section due to its response modeling sensitivity to overall pavement stiffness. Further, the results indicated that the relation between elastic and plastic material properties may change throughout the pavement life. The PEDRO model behavior due to lateral wander and observed field temperatures was reasonable. The zero-shear rate viscosity assessment method for asphalt concrete, utilized in PEDRO, should be further evaluated. All models produced reasonable permanent deformation results although further validation and calibration is recommended before employment for pavement design purposes in Sweden (Oscarsson, 2011).

A research in New Zealand describes efforts exerted in calibrating performance prediction models. The research presents the implication of such calibration on the materials and cost of construction. The calibrated models produced an average of 26% thinner asphalt layer thicknesses that resulted in an average of $70,000 cost saving for one lane width per kilometer (Saleh, 2011).

In 1999, a study was conducted in India investigating the development of a mechanistic-empirical pavement design procedures that correlate the performance data at some locations in India with stress-strain parameters in a pavement structure for flexible pavements. The study discussed proposed methods for analysis of pavement structure, material characterization, performance prediction criteria and climate incorporation according to Indian conditions. The study presents the development of a computer program IITPAVE for the design of bituminous pavements with granular bases incorporating the previous elements of research. The rutting and fatigue criteria calibrated from pavement performance data in India were used in thickness computation (Das, A., & Pandey, 1999).
In Latin America, preliminary studies towards implementation of the Mechanistic Empirical Pavement Design Guide were conducted in Chile (Delgadillo et al., 2011).
Similar initiatives towards implementation of the Mechanistic Empirical Pavement Design Guide have been conducted in the Arabian Peninsula with Lebanon (Hamdar et al., 2019; Hamdar & Chehab, 2017), KSA (Khattab et al., 2014) and Qatar (Sadek et al., 2014).
In Africa, efforts to collect data for critical input parameters to be used in implementation initiatives of ME design have so far been done in Egypt (Aguib & Khedr, 2016); and in South Africa (Theyse et al., 1996).

3.4. Validation and Calibration of ME guide Models

Local validation and calibration of models is identified to be one of the important challenges facing the implementation of the ME guide. Plenty of efforts are exerted by State Highway Agencies (SHA) in the US, and Internationally to tackle this issue.

Performing sensitivity analysis on certain variables to quantify which parameters are worth building a database for is of high importance in local calibration.

(Mallela et al., 2013) provided a description of work done to verify and calibrate, if found required, distress prediction and smoothness models within the MEPDG for Colorado. The criteria for performing local calibration were based on the following parameters;

a) whether the given global model exhibited a reasonable goodness of fit (between measured and predicted outputs) and

b) whether distresses/IRI were predicted without significant bias.

Based on selection criteria, projects within the states were identified and data for these road projects were used in the validation process. Alligator Cracking, Rutting, Thermal Cracking and Smoothness prediction models were investigated. The study identified three major steps for the verification process:

i. Predict performance using ME guide global models for the distress under study for flexible pavement,

ii. Perform statistical analysis to determine goodness of fit using field-measured total distress and bias in estimated distress,

iii. Evaluate goodness of fit and bias statistics and determine any need for local calibration to Colorado conditions.

Results for the verification process showed that for the four models under investigation, none predicted the performance to an acceptable level with respect to Colorado conditions and local calibration was deemed necessary. Using data from Colorado DOT (CDOT), calibration coefficients were developed for these models using nonlinear model optimization tools available in the SAS statistical software and tested for goodness of function and bias (Mallela et al., 2013).

(Caliendo, 2012) in Italy conducted research efforts in investigating MEPDG performance prediction models in comparison with local practices. Due to the unavailability of pavement performance data in Italy, results of MEDPG performance prediction models were compared to results of theoretical equations and/or assumptions widely used in Italy. Performance was predicted initially using MEPDG. Performance was firstly predicted using MEPDG and calibration coefficients within the MEPDG were adjusted to reduce the differences in the predictions. The study revealed that for fatigue cracking, the MEPDG underestimates the distresses predicted and calibration coefficients were then adjusted. As for rutting and due to the absence of a rutting model calibrated to Italy local conditions, recalibration of MEPDG models was done in such a way that calibration coefficients found in literature were used to limit the rut depth to values accepted in Italy based on experience. The smoothness prediction model in the MEPDG was found satisfactory to Italy's conditions and no calibration was then required. Top-down cracking and thermal cracking were found to be of minute importance given the very small value of the first compared to fatigue cracking and the infrequent occurrences of the second due to Mediterranean moderate non-severe climatic conditions (Caliendo, 2012).

North Carolina presented a three-step local calibration plan for performance prediction models in MEPDG that includes 1) verification, 2) calibration and 3) validation. Verification runs were performed on local pavement sections and a null hypothesis test was performed on the results. Predicted performance data were compared to those of the local chosen sections and average of the residual errors between them was identified. The test was done for fatigue cracking and rutting. Results for the test indicated that performance prediction for both distresses failed the test and recalibration effort must be done. Calibration coefficients for both distresses were varied using Microsoft Excel Solver to minimize the bias between predicted and measured data. Using road sections data not used in calibration, the calibrated performance prediction models were validated and checked for bias using a Chi-square test (Muthadi & Kim, 2008).

In 2009, Ohio conducted its own calibration research. The study's purpose was to validate and recalibrate MEPDG performance prediction models to best suit Ohio's local conditions. The IRI smoothness model was tested statistically and non-statistically and found to be inadequate to implementation in Ohio. Recalibration of the model was deemed necessary. On the other hand, the transverse thermal cracking model was found adequate and results of predicted performance were satisfactory compared to test sections data. However, it was expressed in the study that literature showed that the MEPDG default creep compliance and tensile strength estimates overestimated the true creep compliance of HMA mixes and underestimated thermal cracking. Reassessment from Ohio DOT is recommended before closing this issue. HMA rutting model was also validated in the study and results revealed that MEPDG over-estimates rutting values. Recalibration was conducted to eliminate the existing bias. The study also mentioned that the fatigue cracking model was not validated due to presence of noise data where it was difficult to separate the bottom-up fatigue cracking from top-down construction cracking (Mallela et al., 2009) .
In a seven-step procedure, a study conducted by Iowa State University (Kim et al., 2014) identified the way towards calibration. The 7 steps are:

Step 1: Select typical pavement sections around the state.

Step 2: Identify available sources to gather input data and determine the desired level for obtaining each set of input data.

Step 3: Prepare an MEPDG input database from available sources including the Iowa DOT Pavement Management Information System (PMIS), material testing records, design database, and research project reports relevant to MEPDG implementation in Iowa.

Step 4: Prepare a database of performance data for the selected Iowa pavement sections from the Iowa DOT PMIS.

Step 5: Assess local bias from national calibration factors.

Step 6: Identify local calibration factors (sensitivity analysis and optimization of calibration factors).

Step 7: Determine adequacy of local calibration factors.

Following the above seven-step approach, the key findings for flexible pavements were as follows:

- The identified local calibration factors increase the accuracy of rutting predictions and longitudinal (top-down) cracking predictions for Iowa HMA,
- The nationally-calibrated alligator (bottom-up) cracking model provides acceptable predictions for new Iowa HMA pavement,
- Little or no thermal cracking is predicted when using the proper binder grade for Iowa climatic conditions, but significant thermal cracking is observed in Iowa HMA,

- Good agreement is observed between the IRI measures for Iowa HMA pavement and the MEPDG predictions from the nationally-calibrated IRI model as well as the IRI model of locally-calibrated distress inputs with nationally-calibrated coefficients.

3.5.Sensitivity Analyses and Parametric studies

A powerful tool to understand Pavement ME is sensitivity analysis. Sensitivity analysis is the process of varying model input parameters over a practical range and observing the relative change in model response. Two main methodologies for sensitivity analyses are Local Sensitivity Analysis (LSA) and Global Sensitivity Analysis (GSA). The one-at-time (OAT) method is the most common type of LSA. In OATs, baseline cases are identified, and each input is varied independently in turn. In the GSA approach, all input parameters are varied simultaneously, and sensitivity is assessed over the entire parameter space (Schwartz et al., 2013).

The significance of sensitivity analysis is to understand the level of importance of each data item needed for pavement design and analysis using Pavement ME and to develop strategies for data collection activities. A successful implementation plan usually contains a sensitivity analysis. Enormous number of implementation research studies conducted includes sensitivity analysis of which the OAT was the most popular due to its simplicity. A selective number of studies were chosen to be included in this section. (Mallela et al., 2009) conducted a sensitivity analysis for a typical flexible pavement section. A baseline section is design according to the state’s specifications and manuals, as well as LTTP data. Input parameters are varied to values above and below their corresponding baseline values and the effect on pavement performance is observed. Performance is observed through monitoring the following:

- Longitudinal “top down” fatigue cracking,
- Alligator “bottom-up” fatigue cracking,
- Rutting,
- Transverse “low temperature” cracking,
- Smoothness (IRI).

By keeping all inputs constant and varying each parameter at a time, conclusions of MEPDG performance are represented in Table 0-2 as follows:

Table 0-2. Summary of sensitivity analyses – Ohio 2009

Distress/IRI	**Effect of parameter under study**							
	Base type	**Climate**	**HMA thickness**	**Subgrade type**	**HMA %Air voids**	**HMA volumetric binder content**	**Vehicle class and ALS**	**Mix Type**
Longitudinal fatigue cracking	None	None	High for thickness < 8in.	None	None	None	None	None
Transverse cracking	High	High	Moderate	None	Low to Moderate	Moderate to High	None	Low
Alligator fatigue cracking	High	Low	High	Moderate	Moderate	High	Moderate	Low
Rutting	High	Moderate	High	Moderate	Low	High	High	Low

IRI	Moderate	Low	High	Low	Low	Low	Low	Low

The study concludes that MEPDG predictions for rutting and fatigue cracking are the most sensitive to inputs and HMA thickness is the input parameter of the highest influence on MEPDG predictions. According to the study, inputs of the least influence are subgrade type, HMA % air voids and HMA mix type. Longitudinal cracking, according to the study, was the most insensitive distress to any of the input parameters.

In a sensitivity analysis study in Iowa, two flexible pavement structures were analyzed for the sensitivity of the performance predictions to design inputs (Kim, *et al.*, 2005). Five performance measures were evaluated for sensitivity:

- Longitudinal cracking,
- Alligator cracking,
- Thermal cracking,
- Rutting,
- Smoothness

Twenty-three input parameters were investigated in the study. The sensitivity was conducted by either changing one or two input parameters at a time. Sensitivities were graded in three levels: very sensitive, sensitive and insensitive. Two climate conditions were chosen for the investigation representing locations of the two pavement sections in the study and accordingly two types of PG binder grades were used with respect to the location. The sensitivity results show that longitudinal and thermal cracking as well as rutting and smoothness are sensitive to climatic factors. However, alligator cracking showed to be insensitive to climatic factors. A general conclusion was drawn out of the study that none of the input parameters was sensitive to all performance measures which indicates that an optimum pavement structure that resists all distress will be a difficult goal. (Bayomy et al., 2012) conducted a sensitivity analysis to evaluate the use of MEDPG in Idaho. A pavement section representing medium conditions of Idaho was studied using the Idaho Transportation Department (ITD) design procedures. Key variables identified for investigation in the study are:

- HMA and base layer thicknesses,
- HMA material properties,
- Subgrade soils properties,
- Traffic,
- Environment

The sensitivity runs were conducted by varying one input at a time while keeping all other inputs at the medium level.

Table 0-3 summarizes the outcomes of the analyses.

Table 0-3. Summary of sensitivity analyses – Idaho 2012

Input parameter	Performance Models						
	Cracking		Rutting				IRI
	Longitudinal	Alligator	Ac	Base	Subgrade	Overall	
AC thickness	ES	ES	VS	ES	VS	ES	LS
AC mix stiffness	ES	S	ES	LS	I	S	I
Effective Binder Content	ES	ES	LS	I	I	I	I
Mix Air Voids	ES	ES	S	LS	VS	S	LS
Base layer thickness	ES	ES	I	ES	ES	LS	I
Subgrade modulus	ES	LS	LS	I	VS	VS	LS
Climate	VS	S	ES	LS	I	LS	I

Ground Water Table (GWT) level	VS	I	I	I	I	I	I
ALS	ES	ES	LS	LS	LS	LS	I
Truck traffic volume	ES	ES	ES	ES	ES	ES	VS
Traffic speed	LS	LS	LS	I	I	I	I

- ES : Extremely Sensitive
- VS : Very Sensitive
- S : Sensitive
- LS : Low Sensitivity
- I : Insensitive

The sensitivity level of each distress was evaluated according to Distress Ratio (DS) which is the ratio of the highest to the smallest distress or IRI. The criteria used for defining the level of sensitivity are shown in Table 0-4 below.

Table 0-4. Level of sensitivity criteria

Sensitivity Level	**Criteria**
ES: Extremely Sensitive	$DS \geq 2.0$
VS: Very Sensitive	$1.6 \leq DS < 2.0$
S: Sensitive	$1.3 \leq DS < 1.6$
LS: Low Sensitive	$1.10 \leq DS < 1.3$

I: Insensitive	DS<1.1

Results from the above summary are in some part in contrary to those of the study conducted in Ohio. Results show that longitudinal cracking is the most sensitive distress to inputs parameters followed by alligator cracking. IRI was found to be of very low sensitivity to any of the input parameters. The input parameter that was found to have the highest influence on performance predicted and smoothness is truck traffic volume, while the least was found to be GWT level.

In a research conducted by The University of Maryland (Carvalho & Schwartz, 2006), a sensitivity analysis is presented. Variables selected for analysis were:

- Asphalt Layer thickness,
- Base Layer thickness,
- Traffic,
- Climate,
- Material properties,
- Performance model parameters,
- Design criteria

Methodology for the analysis was typical, where a typical pavement section for Maryland for low traffic was designed and values for parameters under study were varied by percentage above and below their reference values. When percentage variation was not possible, distinct cases were chosen. Only Alligator “bottom-up” fatigue cracking and permanent deformation were evaluated under this study. Below is a summary of findings of the study.

- The thickness analysis for AC and base layer showed that the MEPDG emphasizes the structural contribution of the asphalt layer, a direct consequence of the multilayer linear elastic theory analysis. It also shows that large granular base layer thickness did not allow for much reduction in the asphalt layer thickness to meet the same performance criterion.
- Pavement performance evaluated by MEPDG was very sensitive to traffic input data and the use of equivalent traffic is not adequate for presenting traffic load for pavement analysis and design.
- MEPDG performance was very sensitive to environmental changes expected for GWT table level.
- The MEPDG predicted performance trends agreed with expected fatigue cracking performance for the variations in input parameters considered and the expected trends for permanent deformations could not be clearly observed in the MEPDG predictions.
- The parametric study of unbound material properties showed that the MEPDG performance predictions are generally consistent with expectations. The results are also consistent with the implications of multi-layer linear elastic theory for pavement responses. The MEPDG performance predictions are sensitive to basic unbound material properties.

The literature conducted within this research presents the impact of different input parameters on performance predictions of Pavement ME. Different conclusions presented in literature imply that the impact of different input parameters on performance predictions is location specific and that evaluation studies should be conducted for each region to identify the behavior of pavement performance predictions in this region. Another important conclusion was the importance of the quality of the input data which was proven to have great influence on the performance prediction using MEPDG.

It is evident from sensitivity analyses and parametric studies presented here and through literature that the trends of Pavement ME prediction are not always constant and exhibits high variability. This variability is a direct consequence of variety of inputs required by Pavement ME. Thus, it was found necessary to conduct such analyses to Ugandan local conditions, where possible, to have a better understanding of Pavement ME prior to implementation in Uganda.

CHAPTER 4

TEMPERATURE ZONING MAP OF UGANDA BASED ON SUPERPAVE SYSTEM

4.1.Introduction

Super pave binder specification is based on the performance of asphalt pavement, which is the main difference between this system and other earlier approaches such as Penetration and Viscosity grading (ASTM D 6373, 1999). The physical and mechanical properties requirements of asphalt binders are fixed for all performance grades, but in the Superpave method, the maximum and minimum temperatures at which the binder shall meet the requirements are the basis of various grades. AASHTO M320 contains a listing of the more commonly used performance grades (PG), but the PG grades are not limited to those given classifications because the specification temperatures are unlimited (Azari et al., 2003). The high and low temperatures extended as far as necessary in the standard six increments. This chapter explains the steps and criteria of selecting performance grade (PG) with reference to the Strategic Highway Research Program (SHRP) Specification and determination of Uganda Temperature Zoning map. The currently used Penetration grading approach in Uganda is empirical and suffers the limitation of accuracy in determining the full effects of variations in environmental and loading conditions on the pavement performance. Registered historical temperature data for ten years was obtained from various weather stations to cover different regions of the country. The selection of performance grade based on SHRP specifications includes three important factors which are: historical temperature data, traffic conditions, and the desired reliability factor. The desired reliability and historical temperature data are of high importance in selecting the temperature zone for the selected country or region. Although other factors such as the traffic condition should be considered when selecting binder grade for the asphalt mix, the Superpave system facilitates the knowledge of the base PG of the asphalt binder to be used in the project area directly from the temperature zoning map.

4.2.Methodology

4.2.1. High pavement design temperature

SHRP developed two models for determining high pavement temperature for PG grading purposes; one based on the rutting damage concept (the rutting damage model) and the other based on adjusting the PG with depth into the pavement (the LTPP High pavement temperature model). The latter was used in this study and is a function of air temperature, latitude and depth (Mohseni et al., 2005) as shown in **Equation 1.**

$$T_{H.pav} = 54.32 + 0.78\, T_{air} - 0.0025\, Lat^2 - 15.14 \log(H+25) - Z\, (9+0.61\, \sigma^2_{Tair})^{0.5}$$

…Equation 1

Where,

$T_{H.pav}$ = High pavement design temperature.

T_{air} = Average seven-day high air temperature, ˚C.

Lat = The Geographical latitude of the project.

H = Depth to surface, mm.

σ^2_{Tair} = Standard deviation of the mean low air temperature, °C.

Z = Standard normal distribution value 2.055 for 98% reliability.

4.2.2. Low pavement design temperature

Low-temperature (LT) performance grade is selected using the algorithm developed from LTPP climatic data. LT algorithm relates minimum pavement temperature to minimum air temperature, latitude, and depth (Mohseni et al., 2005) as shown in **Equation 2.**

$$T_{L.pav} = -1.56 + 0.72\ T_{air} - 0.004\ Lat^2 + 6.26\ \log(H+25) - Z\ (4.4 + 0.52\ \sigma^2_{T\ air})^{0.5}$$

…Equation 2

Where:

$\mathbf{T_{L.pav}}$ = Low pavement temperature at surface, °C.

$\mathbf{T_{air}}$ = Annual average low air temperature, °C.

4.3.Results

The mean and standard deviation for maximum and minimum air temperatures during the specified period (ten years) for all the meteorological stations representing all the regions in Uganda are tabulated in Table 0-1.

Table 0-1. The mean and standard deviations for 7-day high and annual average low air temperatures in Uganda

Region	**Station**	**Mean**		**Standard deviation (sd)**		**+2 sd (98% reliability)**	
		Highest °C	**Lowest °C**	**Highest °C**	**Lowest °C**	**Highest °C**	**Lowest °C**
North	Gulu	34.56	16.67	1.27	0.52	37.09	15.63
	Lira	34.11	16.22	1.14	0.63	36.38	14.96
	Moroto	32.00	16.89	0.88	0.63	33.75	15.62
	Arua	33.78	16.78	0.92	0.67	35.62	15.43
West	Kabale	27.44	14.00	1.42	0.47	30.28	13.06
	Mbarara	27.56	14.00	1.26	0.47	30.09	13.06
	Kasese	27.78	13.11	0.99	0.95	29.77	11.21
	Fort Portal	27.78	13.11	0.99	0.95	29.77	11.21
	Hoima	33.22	16.22	0.82	0.63	34.87	14.96
East & Central	Entebbe	25.11	17.33	0.79	1.48	26.69	14.38
	Kampala	28.22	15.44	1.03	1.07	30.29	13.29
	Jinja	29.89	14.44	1.62	1.08	33.13	12.28
	Kalangala	25.11	17.33	0.79	1.48	26.69	14.38
	Tororo	31.33	14.67	1.07	0.67	33.48	13.32
	Mbale	29.78	14.11	0.94	0.42	31.66	13.27
	Soroti	34.33	16.11	1.27	0.74	36.87	14.64

SHRP performance-based binder and mixture specifications are developed based on the tests related to the pavement performance under different climatic conditions. **Equation 1** is used to transfer the highest air temperature to high design pavement temperature.

Similarly, **Equation 2** is used to determine the low design pavement temperature. The results of design pavement temperatures were used to determine the PG grades for different regions in Uganda.

4.4.Ugandan Temperature Zoning Map

Uganda is sub-divided into three major climatic zones, Western, Northern, and East & Central. The country experiences predominantly tropical climate which is characterized by high precipitation throughout the year. Moreover, the air temperatures rarely fall below 10 ºC yet do not go beyond 37 ºC. In this study, sixteen weather stations were selected as representatives of different regions due to their strategic location and availability of historical climatic data. As a result of this study, the Uganda Temperature Zoning map was established, to serve as reference for asphalt PG grade selection in this region according to the Superpave system. The hottest areas reported high pavement design temperature not more than 64 ºC and the lowest reported design temperature was 58 ºC.

The following assumptions were used to establish the Ugandan Temperature Zoning Map:

i. The station in any region should be taken as a reference for the whole region;

ii. If there are many stations in one region, then the high recorded reading is used; and

iii. The regional/ provincial borders are used as separators between zones.

The results from sixteen (16) weather stations for the three main climatic zones in Uganda are illustrated in Figure 0-1 below;

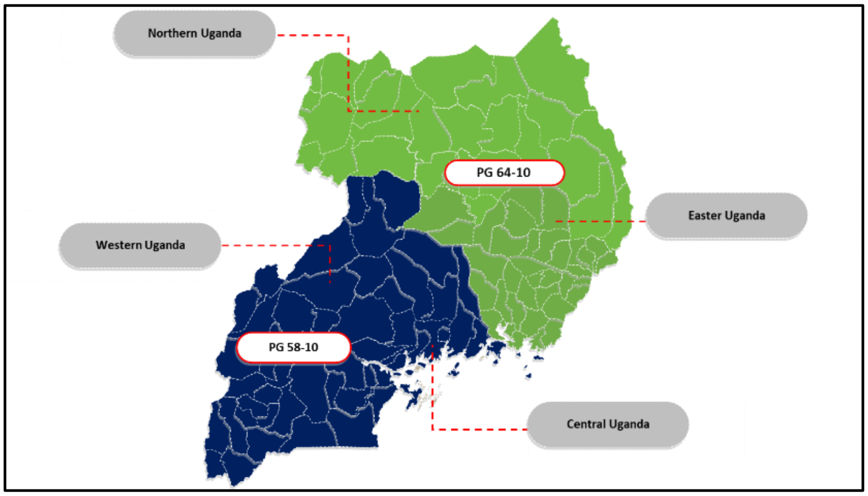

Figure 0-1. Uganda's PG zoning map

4.5.Incorporating Effects of Traffic Conditions

Final selection of required Performance Grade (PG) for a given project depends on adjusting the base binder grade selected according to climatic criteria to account for desired reliability and traffic conditions. Table 0-2 includes the Superpave specifications for high-temperature design grade adjustment (grade bump) based on traffic volume and speed. One major drawback is that SHRP does not provide any guidelines regarding the depth of the asphalt layer up to which this bumping should be applied (Chehab et al., 2019).

Table 0-2. Adjustment to the high-temperature grade of Binder based on traffic speed and traffic level

Adjustment to Binder PG grade [2]					
Traffic loading rate (design speed)	**Base Grade**	**Design ESALs (millions) [1]**			
		< 3	**3 - 10**	**10 - 30**	**>30**
Standing (Av. Speed < 20Km/hr.)	**52**	2	2	2	2
	58	2	2	2	2
	64	2	2	2	2
	70	2	2	2	2
Slow (Av. Speed 20 to 70 Km/hr.)	**52**	1	1	1	1 [(3)]
	58	1	1	1	1
	64	1	1	1	1
	70	1	1	1	1
Fast (Av. Speed > 70 Km/hr.)	**52**	-	-	-	1
	58	-	-	-	1
	64	-	-	-	1
	70	- [(3)]	-	-	1

[1.] Design ESALs is the anticipated project traffic level expected on the design lane over 20 years.

[2.] Increase the high-temperature grade by the number of grade equivalents indicated (one grade equivalent to 6°C). The low-temperature grade is not adjusted.

[3.] Consideration should be given to increasing the high-temperature grade by one grade equivalent.

4.6. Conclusion

Based on the comprehensive collected data from sixteen meteorological weather stations in Uganda and the analysis of the data based on the SHRP Superpave system procedure, the following conclusion can be stated:

1. The range of average lowest and highest air temperatures during the specified period is between (13.11 ºC to 17.33 ºC) and (25.11 ºC to 34.56°C) respectively.
2. Based on the weather data analysis, the maximum and minimum air temperatures registered are in Northern and Western regions of Uganda respectively.
3. The study also showed that the base maximum pavement design temperature is 64 ºC and the conservative minimum pavement design temperature is -10 ºC.
4. The temperature zoning in Uganda is thus distributed into two zones which are PG 64-10 and PG 58-10.
5. The PG grade map acts as a reference for pavement design in Uganda. The design traffic conditions should be taken into consideration for pavement design in any of the regions, to know the need for improving the PG grade according to Table 0-2.

CHAPTER 5

STATE OF PRACTICE AND PARAMETRIC INVESTIGATIONS

5.1. Introduction

Based on findings from literature and knowledge of flexible pavement design and performance, this thesis proposes a roadmap for the implementation of Pavement-ME by authorities in Uganda based on their current state of pavement design and construction practice. This chapter will define the research scope and exhaustively investigate the required input parameters.

5.2. Research Scope

This research will explore the current state of pavement design practice in Uganda and utilize Pavement-ME to perform simulations that will be subjected to sensitivity analysis, to assess the variation of the predicted distresses as a function of Pavement-ME input parameters. In addition to that, the need for the calibration of certain distress prediction models will be verified.

5.3. Current state of practice

This section presents the findings regarding the current state of practice of pavement industry in Uganda. This is based on thorough review of documentation such as Specifications, reports from different consultants as well as on interviews conducted with relevant stakeholders in the Uganda road sector.

5.3.1. General

Uganda uses the Ministry of Works and Transport (MoWT) Road Design Manual Volume III as reference for flexible pavement design. This is one of a series of Engineering Specifications, standards, manuals and Guidelines issued by the Ministry to give guidance and recommendations for road design in Uganda. Other volumes include:

a. Volume 1 (Geometric design),
b. Volume II (Drainage design),
c. Volume IV (Bridge design)

The manual provides a simple and easily applied method for determining an appropriate pavement structure for a given design criteria. It is based on the use of a design catalogue which enables the pavement designer to select possible structural configurations that should meet the design criteria.

However, this catalogue-based manual is presented with certain limitations:

a. The manual does not apply for concrete pavements,
b. The manual does not cover special considerations for urban pavements,
c. The manual is not for design trafficking of more than 30 million Equivalent Standard Axles,
d. The manual does not specifically cover existing subgrade conditions for which the nominal California Bearing Ratio (CBR) is less than 2%.

Therefore, suggested designs should be checked against current Mechanistic-Empirical and Mechanistic methods for suitability.

5.3.2. Design Process

The design process in the Uganda pavement design guide is defined in five steps:

a. Estimating the cumulative traffic loading expected during the design life,

b. Defining the strength of the subgrade over which the road will be built,

c. Defining nominal operating climate (wet or dry),

d. Determining any practical aspects which will influence the design selection,

e. Selecting possible pavement structures.

5.3.2.1. Estimating Design traffic loading

The design life of a pavement is the period during which the road is expected to carry traffic at a satisfactory level of service, without requiring major rehabilitation or repair work. It is implicit however that certain maintenance work will be carried out throughout this period in order to meet the expected design life. This maintenance is primarily to keep the pavement in a satisfactory serviceable condition and would include routine maintenance tasks. Absence of this type of maintenance would almost certainly lead to premature pavement failure and significant loss of initial investment.

According to MoWT, a minimum of 10 years and a maximum of 20 years are recommended for design life consideration of a flexible pavement. The selection of design life depends mainly on design data reliability and the importance of the road as shown in Table 0-1.

Table 0-1.Pavement design life selection guidance

Design data reliability	Importance/Level of service	
	Low	High
Low	10 – 15 years	15 years
High	10 - 20 years	15 – 20 years

Design Traffic Loading

Uganda still uses the EASLs approach, where the estimation of the average daily number of ESALs is projected and cumulated over the design period to give the design traffic loading.

The following formula, using the average daily traffic flow for the first year (not the value at opening to traffic, but the projected average for the year), is used to calculate the cumulative totals:

$$D_T = \frac{T*365*[1+r/100]^{n} -1}{r/100}$$ ………………………………. Equation 3

where:

D_T is the cumulative design traffic in a vehicle category, for one direction, and

T = average daily traffic in a vehicle category in the first year (one direction)

r = average assumed growth rate, percent per annum

n = design period in years.

The steps taken in principal to determine the average daily traffic are defined in the MoWT manual. These include acquisition of traffic count data and static vehicle axle load survey data, defining the design traffic class, and determining the probability distribution of traffic, axle and wheel load distribution, lateral wonder, directional factor and lane factor.

Table 0-2 & Table 0-3 below shows vehicle classifications and different traffic classes according to the Uganda road design manual

Table 0-2.Uganda vehicle classification

Class	Vehicle Type
Class 1	Motocycles
Class 2	Saloon Cars
Class 3	Light goods Pickups/Vans/4WD
Class 4	Small buses/Mini-buses
Class 5	Medium Buses
Class 6	Buses
Class 7	Light Single Unit Trucks
Class 8	Medium – Large Single Unit Trucks, Lorries
Class 9	Truck Trailers and Semi - Trailers

Source: Ministry of Works and Transport (MoWT)

Table 0-3.Traffic classes according to the Uganda road design manual

Traffic Class	ESAL (Millions)
T1	<0.3
T2	0.3 – 0.7

T3	0.7 – 1.5
T4	1.5 – 3
T5	3 – 6
T6	6 – 10
T7	10 – 17
T8	17 – 30

Source: Ministry of Works and Transport (MoWT)

5.3.2.2. Determining Subgrade strength

Subgrade is classified in terms of the California Bearing Ratio (CBR) to represent realistic conditions for design. In practice, CBR strength is determined for the wettest moisture condition likely to occur during the design life, at the density expected to be achieved in the field.

The classification of subgrade condition in Uganda is shown in Table 0-4

Table 0-4. Subgrade classification in Uganda

Subgrade Class designation						
Subgrade classes	S1	S2	S3	S4	S5	S6
CBR ranges (%)	2	3 - 4	5 - 7	8 - 14	15 - 29	30+

Source: Ministry of Works and Transport (MoWT)

5.3.2.3. Defining nominal operating climate

The design catalogue in the manual includes specific pavement structures for either nominally wet or nominally dry regions, in order to simplify the selection of appropriate pavements. While some consideration to the prevailing conditions has already been given in the selection of appropriate subgrade classification, the manual defines the wet and dry regions as follows:

- **i.** **Predominantly Dry Regions:** regions where annual rainfall is less than 250 mm and there is no likelihood of moisture ingress due to factors such as significant flooding (in low-lying flood plain areas, or in tidal basins, for example), underground springs or wells, or any other detrimental conditions.
- **ii.** **Predominantly Wet Regions:** Any regions which do not comply with (ii) above must be regarded as being predominantly wet.

While climate is a key factor that affects the performance of a pavement structure, it is not comprehensively addressed in the current road manual used in Uganda. Therefore, as part of this research, a temperature zoning map is determined in the previous chapter, based on the Superpave^{TM} system to serve as reference for selecting PG grade in different climatic zones of Uganda.

5.3.2.4. Practical considerations that influence the design selection

The primary factors used in the design catalogue to determine appropriate structures, are traffic class, subgrade support classification and nominal climate conditions. However, consideration is also given to other factors which will have a practical influence on finalizing possible pavement structures. Most significant of these is the availability, in terms of both quantity and quality, of materials for road construction. Other factors include the general topography, and the use of established local methods for road layer construction. Each of these will affect the final selection of a pavement.

5.3.2.5. Selecting possible pavement structures.

The design catalogue comprises two distinct sets of structures for nominally dry and nominally wet conditions**.** The appropriate design set(s) are then accessed based on design trafficking class and design subgrade condition, and the designer can review the alternatives to finalize the selection.

a. Typical Pavement layers

Typical pavement layers are described in the Uganda Pavement design catalogue as shown in Figure 0-1. It can be seen from the figure that there are five different layers for paved roads, namely Wearing course, surfacing layers, Base, sub-base and subgrade. Details for each of these layers are described in the following sections.

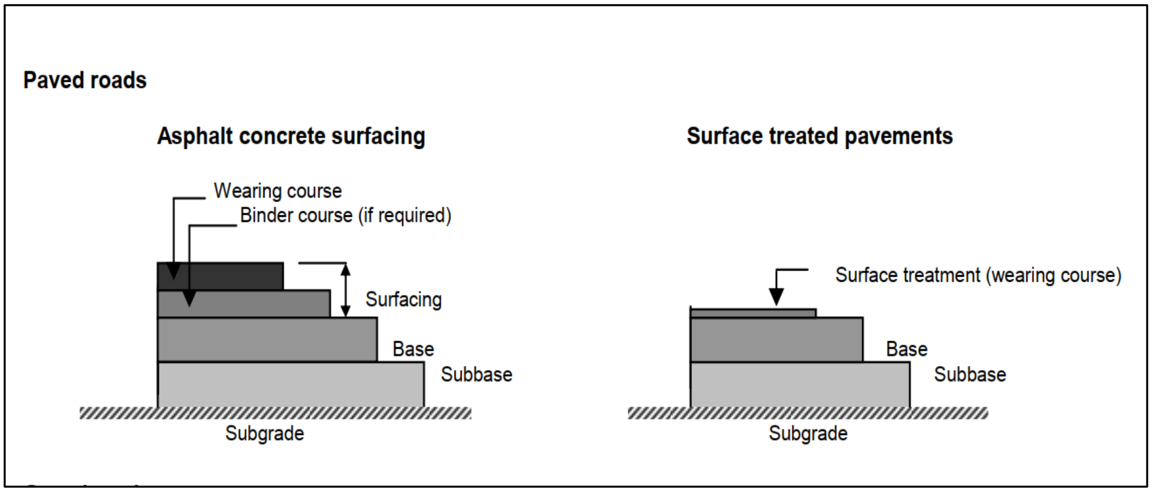

Figure 0-1.Typical pavement structures in Uganda. Source: Ministry of Works and Transport (MoWT)

b. Pavement materials.

The properties of the materials used in the various pavement layers were determined based on the guidance provided in the Uganda pavement design guide manual.

c. Subgrade

Uganda generally has a wide range of natural soils consisting of weathered sands and mainly clay soils. Subgrade soils according to Uganda's pavement design guide are categorized from S_1 which is the weakest with CBR of 2% to S_6 which is the strongest, with CBR of 30%. The present Uganda pavement design guide has 6 classes of subgrade defined by CBR values as already define above.

These CBR values are measured using the BS 1377 method, on soaked subgrade samples compacted to 95% of maximum dry density (MDD) which must be the same as in situ.

d. Granular Sub-base

Based on the Ugandan road design manual, the granular material may consist of either crushed stone or gravel with CBR value not less than 30% for gravel CBR value not less than 45% for crushed stone when compacted to 95% of MDD using the BS 1377 4.5 kg rammer method. In this study, crushed stone was selected for the Sub-base layer due to its common use in Uganda.

e. Granular Base

According to the Uganda road design catalogue, the thickness of a single crushed stone road base layer varies between 100 and 250 mm. This material must comply with a given grading envelope.

f. Wearing Course

A standard surfacing of asphalt concrete, laid as a 40 mm course, is used on all asphalt concrete pavement designs according to the Uganda road design manual. The Manual specifies Pen 50-70, to be assigned for the wearing course with bitumen content typically between 4 and 5% by mass.

5.4. Research Methodology

This part describes the procedure used for sensitivity analysis of important design input parameters of AASHTOWare Pavement ME Design software version 2.3. Screen shots of the Pavement ME software interface in detail are attached in the appendix.

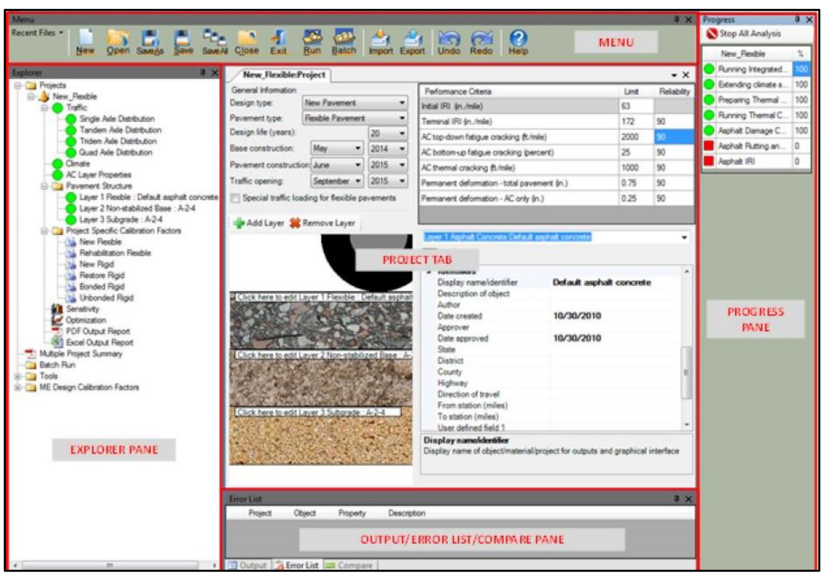

Figure 0-2. ME Design main window

The process involves selection of the hierarchical input level, as it is one of the most important steps towards achieving accurate results. Because this selection is primarily an agency decision, Uganda National Roads Authority (UNRA)-provided inputs were mostly used in this study; therefore, the inputs were coherent with Uganda's current field and laboratory testing capabilities, material and construction specifications, and traffic data collection procedures. The input selection procedure is divided into three categories: traffic, climate, and materials.

5.4.1. Performance Criteria

Performance verification forms the basis of the acceptance or rejection of a trial design evaluated using ME Design (AASHTO, 2015). The design procedure is based on pavement performance, and therefore, the critical levels of pavement distresses that can be tolerated by the agency at the selected level of reliability needs to be specified by the user. If the simulation process shows that the trial design produces excessive amount of distresses, then the trial design must be modified accordingly to produce a feasible design in the future trials.

The distress types considered in the design of a new flexible pavement are total rutting (all layers and subgrade), AC rutting, load-related top-down cracking (longitudinal cracking in the wheel path) and bottom-up fatigue cracking (alligator cracking), and thermal cracking (transverse cracking). In addition, pavement smoothness is considered for performance verification and is characterized using the International Roughness Index (IRI).

Table 0-5 shows the performance criteria adopted for this research using Pavement ME version 2.3.

Table 0-5. Performance criteria for flexible pavements

Distress type	Limit	Reliability
Initial IRI (in/mile)	63	90
Terminal IRI (in/mile)	172	90
AC top-down fatigue (ft/mile)	2000	90
AC bottom – up fatigue (% lane area)	25	90
AC thermal cracking (ft/mile)	1000	90
Permanent deformation – Total pavement (in)	0.75	90
Permanent deformation – AC only (in)	0.25	90

This research study will focus on bottom – up fatigue cracking and total pavement rutting as they are the most common distresses identified in Uganda. Moreover, flexible pavements in Uganda do not experience AC thermal cracking due to the nature of climate.

5.4.2. Traffic Input

The AASHTOWare pavement ME design software allows a designer to input traffic parameters at three levels. Level 1 is project-specific with extensive traffic volume and load data, Level 2 is regional input parameters derived from weigh-in motion (WIM) and automatic vehicle classifier (AVC) stations across the state, and Level 3 is based on global default values already included in the software.

This study used site-specific traffic inputs such as AADTT data, operational speed, number of lanes, and the percentage of trucks in a design direction or lane. Details of site-specific inputs are shown in Table 0-6. The AASHTOWare Pavement-ME default values are used for the remaining required design traffic inputs.

Table 0-6. General traffic inputs

Input parameters		Source	Value used	Input level
AADTT	Two-way AADTT	Actual Project Data (APD)	As in APD	1
	Number of lanes	APD	As in APD	1
	Percent trucks in design direction	Default	50	3
	Percent trucks in design lane	APD	95 (for 2 lanes)	1
	Operation speed	APD	As in APD	1
Axle Configuration	Average axle width (ft.)	Default	8.5	3
	Dual tire spacing (in.)	Default	12	3
	Tire pressure (psi)	Default	120	3
	Tandem axle spacing (in.)	Default	51.6	3
	Tridem axle spacing (in.)	Default	49.2	3

	Quad axle spacing (in.)	Default	49.2	3
Lateral Wander	Mean wheel location (in.)	Default	18	3
	Traffic wander standard deviation (in.)	Default	10	3
	Design lane width (ft)	Default	12	3
Wheelbase	Average spacing of short axles (ft)	Default	12	3
	Average spacing of medium axles (ft)	Default	15	3
	Average spacing of long axles (ft)	Default	18	3
	Percent trucks with short axles	Default	17	3
	Percent trucks with medium axles	Default	22	3
	Percent trucks with long axles	Default	61	
Growth factor (%)		APD	3%	3

Three different traffic conditions are selected for this study, which are low, medium and high. The different traffic levels represent different classes according to the Ugandan road design manual and the purpose of the road classified as local, collector and arterials roads.

Other traffic Input parameters that can be derived from WIM and AVC stations are described below:

a) Vehicle Class Factor

VCFs determine the frequency of trucks in each vehicle class from FHWA vehicle Class 4 to Class 13. FHWA vehicle category classification is shown in Level 3 VCFs in the AASHTOWare pavement ME design software, based on roadway function, classification, and truck traffic classification (TTC) groups for a particular roadway. In this study level 3 VCFs were used.

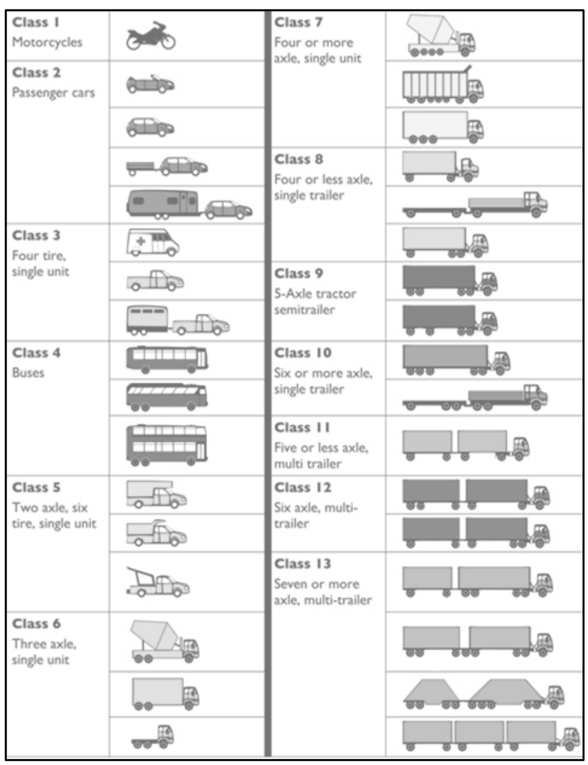

Figure 0-3. FHWA Vehicle Classification (FHWA, 2013)

b) Monthly Adjustment Factors

Truck traffic MAFs are defined as the proportion of annual truck traffic for a particular truck class for a specific month (NCHRP, 2004). These factors are used to determine monthly variation in truck traffic within a base year. MAFs are influenced by factors such as adjacent lane use, location of industries, and roadway location (Chhade et al., 2018). Default MAFs were used for this study.

c) Hourly Distribution Factors

HDFs, which are required only for rigid pavements, are derived from the percentage of AADTT at each hour of the day (NCHRP, 2004). The hourly distribution of truck traffic is required to compute incremental damage of the pavement structure for various thermal gradients during a 24-hour period (Chhade et al., 2018). In this study, default ME values were used for HDFs.

d) Axle Group per Vehicle

AASHTOWare pavement ME design software requires average number of axle group per vehicle (AGPV) and axle load spectra in order to compute average damage imposed on the pavement structure by truck traffic in each vehicle class (Romanoschi et al., 2011). AGPV is obtained for each vehicle class by dividing the total number of axles (single/tandem/tridem/quad) by the number of trucks. Default values were used in this study.

e) Axle Load Spectra

AASHTOWare pavement ME design software requires the frequency of total axle load applications within each load class interval for a specific axle type (single, tandem, tridem, and quad) and vehicle class (Classes 4 through 13) for each month of the year. For single axles, load distribution ranges from 3,000 to 40,000 lbs at 1,000-lb intervals; for tandem axle, distribution ranges from 6,000 to 80,000 lbs at 2,000-lb intervals; and for tridem and quad axles, distribution ranges from 12,000 to 102,000 lbs at 3,000-lb intervals. Although AASHTOWare ME requires normalized axle load distribution for each month of the year, truck weight data were not available at any site for all months of the year. However, previous researchers have suggested that variations in axle load spectra across the months within a year and across the years are not significant (Tam, W., & Von Quintus, 2003). Therefore, this study used default axle load spectra.

5.4.3. Climate

Pavement – ME takes into consideration the effect of climate on the performance of the pavement over its design life. Previous studies indicated that Pavement-ME is sensitive to climatic in put data (Breakah et al., 2010). In order to carry out the performance analysis using the Pavement - ME software, it is necessary to use the climate data base available in the software. To use the Pavement – ME software with the weather data from Uganda, one of the existing U.S stations needed to be modified. The MERRA climate data incorporated in LTPP infopave online tool enables users to download climate data in the Hourly Climatic Database (HCD) format, which is used as an input in the AASHTOWare Pavement ME Design software.

The climatic data in ".hcd" file format are logged by weather stations mainly at airports all around Northern America. Those files hold the following information in the following format:

YYYYMMDDHH, Temperature (F), Wind Speed (mph), % Sunshine, Precipitation, Relative Humidity.

An example of that is:

2017021012,60,4,100,0.1,100: which indicates that on 2017, February 10th at 12:00 P.M., air temperature was 60°F, wind speed was 4 mph, percent sunshine was 100%, precipitation was at 0.1 inches and relative humidity was at 100%.

In this study, the climatic data was obtained from a globally recognized source for properly logged and readily available ". hcd" files - MERRA (Modern-Era Retrospective Analysis for Research and Applications) database by NASA.

(Schwartz et al., 2015) recommends MERRA data as an acceptable source for climate data. Therefore, the procedure was used to prepare the climatic file for research purposes only but compromises accuracy of the software to a certain degree, due to its shortcomings, if it is to be applied to more routine design unless the MEPDG software adds the flexibility for it to be used in countries outside U.S and Canada.

Among all climatic inputs, the geo coordinates (latitude and longitude) and elevations for all pavement sections were given input as site-specific values, or Level 1 inputs. Depth of the water table for all segments was set at 12 ft because this value does not affect performance predicted by Pavement ME. Climatic stations used in this study were chosen from the default locations in the software and data replaced by the area specific historical climatic data (1985 to 2017) obtained from MERRA database, to more accurately depict the effect of climate on pavement structure. Brief descriptions of the climatic stations, as well as other inputs used in this study for flexible pavements, are presented in

Table 0-7.

Table 0-7. Site-specific climatic inputs for flexible pavement sections

Project Name	Climate Station	Latitude (deg)	Longitude (deg)	Elevation (ft)	Mean annual air temperature (F)	Mean annual precipitation(in.)
Mbarara Northern Bypass road	UG_WESTERN_REGION_US_97682	-0.6072	30.6545	3763	69.26	46.3
Kampala Northern Bypass	UG_CENTRAL_REGION_US_98837	0.3476	32.5825	3937	71.70	68.61
Gulu Highway	UG_NORTHERN_REGION_US_101717	2.7724	32.2881	3608	77.33	65.12

5.4.4. Materials Input

This study utilized site-specific inputs such as volumetric data of the asphalt concrete mixture. MoWT specifications-suggested values and MEPDG default values were also provided as material inputs in the ME design software.

a) Asphalt Concrete Properties

AASHTOWare pavement ME design software requires dynamic modulus (E*), creep compliance, and indirect tensile strength of the asphalt mix for Level 1 input. Dynamic shear modulus (G*) and phase angle (δ) values of the asphalt binder are also required to generate dynamic modulus master curves for asphalt mixes. Since these data were not available for the selected projects, Level 3 input values (aggregate gradation, binder grade, mix volumetric properties) obtained from Actual Project Data (APD) were used in this research project. A brief descriptions of AC property inputs used in ME simulations are given in Table 0-8.

Table 0-8. Inputs of AC properties

Input Parameters		**Source**	**Value Used**	**Input level**
Mixture Volumetrics	Thickness (in.)	APD	6.6 inches	1
	Unit weight (pcf)	Default	150	3
	Poisson's ratio	Default	0.35	3
	Air voids (%)	APD	4	1
	Effective Binder content (%)	APD	11.6	1
Mechanical properties	Reference temperature (°F)	Default	70	3
	Indirect tensile strength at 14°F (psi)	Default	Default value	3
	Creep compliance (1/psi)	Default	Default value	3
Thermal properties	Thermal conductivity (BTU/hr-ft-*F)	Default	0.67	3
	Heat capacity (BTU/hr-ft-*F)	Default	0.23	3
	Thermal contraction	Default	1.30E-05	3
	AC surface shortwave absorptivity	Default	0.85	3

AC layer properties	Is endurance limit applied?	Default	No	3
	Endurance limit (microstrain)	Default	100	3
	Layer interface	Default	Full friction interface	3

g. Base Course Material Inputs

Pavement sections used in this study were all constructed with crushed stone base, granular sub-base and granular subgrade. Brief descriptions of all base course property inputs are given in Table 0-9 &

Table 0-10.

Table 0-9. Inputs of base course materials

Input parameters		Source	Value used	Input Level
Crushed stone/aggregate base	Thickness	APD	12.6 inches	1
	Poisson's ratio	Default	0.35	3
	Elastic/resilient modulus	APD	35,000	1
	Coefficient of lateral earth Pressure (ko)	Default	0.5	3
	Gradation	Default	Default values	

Table 0-10. Inputs of subbase layer materials

Input parameters		Source	Value used	Input Level
Granular subbase layer	Thickness	APD	15.8 inches	1
	Poisson's ratio	Default	0.35	3
	Elastic/resilient modulus (psi)	APD	25,000	1
	Coefficient of lateral earth Pressure (ko)	Default	0.5	3
	Gradation	Default	Default values	3

h. Subgrade Soil Inputs

The subgrade soil of all pavement sections was classified as A-4. The values used in the study are shown in Table 0-11.

Table 0-11. Inputs of subgrade layer materials

Input parameters		Source	Value used	Input Level
Subgrade layer	Thickness	Default	Semi - infinite	3
	Poisson's ratio	Default	0.35	3
	Coefficient of lateral earth Pressure (ko)	Default	0.5	3
	Resilient modulus (psi)	APD	15,000	1
	Gradation	Default	Default values	3

CHAPTER 6

RESULTS AND ANALYSIS

6.1.Introduction

Understanding the behavior of Pavement ME predictions is identified from literature to be a very important step in adopting its implementation according to local conditions. A powerful tool to achieve such goal is conducting sensitivity analyses. This study comprises results of investigating Pavement ME and the objective of this study is to understand its behavior in Uganda with respect to measures of effectiveness identified from literature.

In the subsequent runs, the values of the HMA material inputs were varied to represent the Actual project data used for flexible pavement design in different regions of Uganda. The Design Level of inputs was preselected as level 3 for all parameter based on data available and varied to level 1 where actual project data was obtained in Uganda. The variation in the distresses at the end of a 20-year design life with the change in design input is based on the comparison of distresses resulting from the design inputs in the reference file. It is assumed that no maintenance or rehabilitation occurs within the 20-years of service life.

6.2.Methodology

According to literature and previous studies, several inputs were identified to be measures of effectiveness for MEPDG performance. Although not all studies reached the same conclusions for some of these measures, a common trend can be identified. Measures of effectiveness chosen to be included in this sensitivity analysis are those with which consensus was found among different studies on their high influence on performance predictions of Pavement ME. These measures are:

1. Traffic volume

2. Traffic growth factor

3. Design operation speed

4. % Air Voids

5. Effective Binder Content

6. Subgrade type

7. Performance Grade (PG) Binder

8. Climate

The sensitivity analysis conducted in this study is performed on the One-At-Time (OAT) basis, in which for each measure under investigation, all other measures are kept constant while the measure under study is varied above and below its reference value. Pavement ME performance predictions are then observed with these variations in terms of Terminal IRI, fatigue cracking (AC bottom-up & Top-down) and rutting (AC rutting & Total pavement rutting). Reference values of all the design inputs (traffic, climate, structure, and materials) for this study are chosen to be typical in designing pavement structures in central Uganda under medium traffic conditions. AASHTOWare Pavement-

ME version 2.3 is used for evaluation in this research at 90% reliability. Table 0-1 shows the reference values as well as variations above and below.

Table 0-1: Reference values for Sensitivity Analyses.

Measure of effectiveness	Reference value	Variation
Initial two-way AADTT (Traffic Volume)	Medium (10,000)	Low (4000), High (25,000)
Traffic growth rate (%)	3	0, 5, 7, 9
Design operation speed (mph)	60	25, 40, 75
%Air voids	4	3, 5, 7, 9
Effective Binder content (% by Volume)	11.6	9.6, 10.6, 12.6, 13.6
Subgrade type	A-4	A-2-4, A-6, A-7-6
PG grade	PG 70-10	PG 64-10, PG 76-10
Climate	Central Uganda	Western Uganda, Northern Uganda

Sensitivity evaluation criteria adopted by (Bayomy et al., 2012) is used in this study. The sensitivity of measures of effectiveness for each parameter is evaluated according to the Distress Ratio (DS) between the highest and lowest value of the evaluated distress. DS = Highest Distress Value/ Lowest Distress value. Table 0-2 shows the evaluation criteria.

Table 0-2: Sensitivity Evaluation Criteria

Sensitivity Level	Criteria
Extreme Sensitive	DS $\geq$2.0
Very Sensitive	1.6 $\leq$ DS < 2.0
Sensitive	1.3 $\leq$ DS < 1.6
Low Sensitivity	1.10 $\leq$ DS < 1.3
Insensitive	DS < 1.1

6.3.Pavement ME results and Sensitivity Analysis

This section presents results of the of predicted performance using Pavement ME version 2.3, and their sensitivity to different design input parameters under study including traffic loading conditions, AC layer properties, subgrade type and climatic conditions as compared to the reference value.

6.3.1. Traffic Volume

Results for pavement performance and distress ratio are summarized in

Table 0-3 and Table 0-4.

Table 0-3: Predicted pavement performance under different Traffic loading conditions

	Initial two-way AADTT		
	Low Traffic (4,000 AADTT)	**Medium Traffic (10,000 AADTT)**	**High Traffic (25,000 AADTT)**
IRI (in/mile)	139.71	145.82	160.59
Permanent deformation - total pavement (in)	0.59	0.7	0.9
Bottom-up cracking (% lane area)	2.09	8.46	25
Top-down fatigue(ft/mile)	2437.3	3591.2	7491.21
Permanent deformation - AC only (in)	0.15	0.23	0.41

Table 0-4: Distress Ratio for predicted performance under different traffic loading conditions

	DS	**Sensitivity**
IRI (in/mile)	1.1	**LS**
Permanent deformation - total pavement (in)	1.5	**S**
Bottom-up cracking (% lane area)	12.0	**ES**
Top-down fatigue(ft/mile)	3.1	**ES**

Permanent deformation - AC only (in)	2.7	**ES**

Results show that there is increase in pavement deterioration with increase in traffic loading. It is highly noticeable for fatigue cracking (Bottom-up & Top-down) under high traffic loading compared to low and medium traffic loading. This is expected since the same pavement structure experiences a high traffic volume load (25,000 AADTT), which is more than twice that experienced under medium traffic loading (10,000 AADTT) and more than six times experienced under low traffic loading (4,000 AADTT).

While asphalt rutting is equally sensitive to traffic loading, total pavement deformation shows less sensitivity to traffic loading conditions since it factors in the effect on unbound layers.

Therefore, fatigue cracking and AC rutting are reported to be significantly sensitive to traffic loading conditions.

6.3.2. Traffic growth rate

Results for pavement performance and distress ratio are summarized in Table 0-5 and Table 0-6.

Table 0-5: Predicted pavement performance for different traffic growth rates

	Traffic growth rate (%)				
	0%	**3%**	**5%**	**7%**	**9%**
IRI (in/mile)	143.93	145.82	147.05	148.24	149.36

Permanent deformation - total pavement (in)	0.66	0.7	0.71	0.73	0.75
Bottom-up cracking (% lane area)	4.26	8.46	12.32	15.69	18.04
Top-down fatigue(ft/mile)	3260.25	3591.2	3804.9	4013.14	4224.11
Permanent deformation - AC only (in)	0.21	0.23	0.25	0.26	0.27

Table 0-6: Distress Ratio for predicted performance of different traffic growth rates.

	DS	**Sensitivity**
IRI (in/mile)	1.0	I
Permanent deformation - total pavement (in)	1.1	LS
Bottom-up cracking (% lane area)	4.2	ES
Top-down fatigue(ft/mile)	1.3	S
Permanent deformation - AC only (in)	1.3	S

Results show that predicted performance deteriorated with the increase of traffic growth rate. Bottom-up fatigue cracking is reported to be significantly sensitive to traffic growth rate while Rutting values show less sensitivity to traffic growth rate.

6.3.3. Operational Speed

Results for pavement performance and distress ratio are summarized in Table 0-7 and

Table 0-8.

Table 0-7: Predicted pavement performance for different design Operational Speeds.

	Operational Speed (mph)			
	25	**40**	**60**	**75**
IRI (in/mile)	148.33	146.94	145.82	145.23
Permanent deformation - total pavement (in)	0.74	0.71	0.70	0.68
Bottom-up cracking (% lane area)	13.91	10.93	8.46	7.27
Top-down fatigue(ft/mile)	3953.5	3760.12	3591.2	3497.66
Permanent deformation - AC only (in)	0.27	0.25	0.23	0.22

Table 0-8: Distress Ratio of predicted performance for different design Operational Speeds

	DS	**Sensitivity**
IRI (in/mile)	1.0	**I**
Permanent deformation - total pavement (in)	1.1	**LS**
Bottom-up cracking (% lane area)	1.9	**VS**
Top-down fatigue(ft/mile)	1.1	**LS**
Permanent deformation – AC only (in)	1.2	**LS**

Results show that predicted pavement performance deteriorated with the decrease in the design Operational Speed. Bottom-up fatigue cracking is reported to be very sensitive to design Operational Speed. This is because at lower operational speed, asphalt is more viscous and thus the pavement will experience more stress for a longer loading time (time for which the pavement feels the load) compared to a high Operation speed where the loading time is relatively shorter and asphalt exhibits elastic behavior. Rutting values indicated less sensitivity.

6.3.4. Air voids

Results for pavement performance and distress ratio are summarized in Table 0-9 and
Table 0-10.

Table 0-9: Predicted pavement performance for different % Air Voids values.

	% Air Voids				
	3%	**4%**	**5%**	**7%**	**9%**
IRI (in/mile)	144.43	145.82	148.48	155.77	165.74
Permanent deformation - total pavement (in)	0.69	0.7	0.71	0.74	0.78
Bottom-up cracking (% lane area)	2.52	8.46	20.71	27.57	39.01
Top-down fatigue(ft/mile)	2823.24	3591.2	4719.59	8231.42	11321.97
Permanent deformation - AC only (in)	0.22	0.23	0.24	0.27	0.3

Table 0-10: Distress Ratio of predicted performance for different % Air Voids values.

	DS	**Sensitivity**
IRI (in/mile)	1.1	**LS**

Permanent deformation - total pavement (in)	1.1	**LS**
Bottom-up cracking (% lane area)	15.5	**ES**
Top-down fatigue(ft/mile)	4.0	**ES**
Permanent deformation - AC only (in)	1.4	**S**

Results for variations in % air voids show that fatigue cracking is extremely sensitive to such variations. On the other hand, rutting is observed to be sensitive. Results are expected for fatigue cracking since the increase of % air voids increases the likelihood of initiation of bottom-up cracking under horizontal tensile stresses at the bottom of the AC layer and its propagation to the surface. Although results show a slight increase in rutting for % air voids greater than 7%, further investigation is found necessary.

6.3.5. Bitumen Content

Results for pavement performance and distress ratio are summarized in Table 0-11 and Table 0-12.

Table 0-11: Predicted pavement performance for different effective binder content values

	Effective Binder Content (% by Volume)				
	9.6%	**10.6%**	**11.6%**	**12.6%**	**13.6%**

IRI (in/mile)	146.26	145.97	145.82	145.51	145.86
Permanent deformation - total pavement (in)	0.68	0.69	0.7	0.7	0.71
Bottom-up cracking (% lane area)	17.59	12.99	8.46	5.57	4.05
Top-down fatigue(ft/mile)	4119.88	3821.14	3591.2	3406.6	3255.6
Permanent deformation - AC only (in)	0.22	0.23	0.23	0.24	0.24

Table 0-12: Distress Ratio of predicted performance for different effective binder content values

	DS	**Sensitivity**
IRI (in/mile)	1.0	**I**
Permanent deformation - total pavement (in)	1.0	**I**
Bottom-up cracking (% lane area)	4.3	**ES**
Top-down fatigue(ft/mile)	1.3	**S**
Permanent deformation - AC only (in)	1.0	**I**

Effective binder content is identified to be a significant property that affects the estimated dynamic modulus |E*| of an AC mixture (Schwartz & Carvalho, 2007) (Hamdar

& Chehab, 2017). Results show that higher effective binder content enhances the performance of the flexible pavement with respect to resistance to fatigue cracking due to better tensile strength achieved. On the other hand, poor rutting resistance is observed with the increase of effective binder content. A suggestion here is that higher binder content reduces |E*| resulting in higher compressive strain values and consequently leading to more rutting.

6.3.6. Subgrade Type

Results for pavement performance and distress ratio are summarized in Table 0-13 and Table 0-14.

Table 0-13: Predicted pavement performance for different types of subgrade.

	Subgrade Type			
	A-7-6	**A-6**	**A-4**	**A-2-4**
IRI (in/mile)	153.26	152.59	145.82	146.17
Permanent deformation - total pavement (in)	0.65	0.66	0.7	0.62
Bottom-up cracking (% lane area)	11.03	19.54	8.46	7.36
Top-down fatigue(ft/mile)	2658.27	2859.1	3591.2	4224.11
Permanent deformation - AC only (in)	0.23	0.23	0.23	0.23

Table 0-14: Distress Ratio of predicted performance for different types of subgrade

	DS	Sensitivity
IRI (in/mile)	1.1	**LS**
Permanent deformation - total pavement (in)	1.1	**LS**
Bottom-up cracking (% lane area)	2.7	**ES**
Top-down fatigue(ft/mile)	1.6	**VS**
Permanent deformation - AC only (in)	1.0	**I**

Results for predicted performance of different subgrade types indicate that weaker subgrade experience higher distress levels. It can be noticed that rutting in AC layer is not sensitive to subgrade strength unlike rutting in the total pavement. This is expected since rutting in AC is mainly affected by AC and Granular Base layer properties rather than subgrade properties. It was unexpected though to observe higher sensitivity of fatigue cracking than that of rutting, to different subgrade types.

6.3.7. Performance Grade (PG) of Asphalt

Results for pavement performance and distress ratio are summarized inTable 0-15 and

Table 0-16.

Table 0-15: Predicted pavement performance for different asphalt PG grades.

	PG Grade

	PG 64-10	PG 70-10	PG 76-10
IRI (in/mile)	147.16	145.82	144.58
Permanent deformation - total pavement (in)	0.72	0.7	0.67
Bottom-up cracking (% lane area)	11.13	8.46	6.29
Top-down fatigue(ft/mile)	3779.56	3591.2	3396.39
Permanent deformation - AC only (in)	0.25	0.23	0.21

Table 0-16: Distress Ratio of predicted performance for different asphalt PG grades.

	DS	Sensitivity
IRI (in/mile)	1.0	**I**
Permanent deformation - total pavement (in)	1.1	**LS**
Bottom-up cracking (% lane area)	1.8	**VS**
Top-down fatigue(ft/mile)	1.1	**LS**

Permanent deformation - AC only (in)	1.2	**LS**

According to the above results, although variation in PG grade does not exhibit much significant impact on the investigated HMA pavement performance parameters, it is noticeable that higher PG grade translates into higher fatigue resistance and vice versa. Bottom-Up fatigue cracking is found to be very sensitive to PG grade while rutting is found to be less sensitive. This is because PG grade influences the stiffness of HMA mixture. It is important to note that Binder selection is affected by different variables such as Geographical location (Air temperature & pavement temperature), Traffic volume, traffic speed and Pavement structure.

6.3.8. Climatic Regions

Results for pavement performance and distress ratio are summarized in

Table 0-17 and

Table 0-18.

Table 0-17: Predicted pavement performance for different climatic conditions.

	Climatic Regions		
	Western	**Central**	**Northern**
IRI (in/mile)	145.82	171.27	189.13

Permanent deformation - total pavement (in)	0.7	1.21	1.61
Bottom-up cracking (% lane area)	8.46	4.31	9.72
Top-down fatigue(ft/mile)	3591.2	2603.25	2968.95
Permanent deformation - AC only (in)	0.23	0.77	1.16

Table 0-18: Distress Ratio of predicted performance for different climatic conditions.

	DS	**Sensitivity**
IRI (in/mile)	1.3	**S**
Permanent deformation - total pavement (in)	2.3	**ES**
Bottom-up cracking (% lane area)	2.3	**ES**
Top-down fatigue(ft/mile)	1.4	**S**
Permanent deformation - AC only (in)	5.0	**ES**

Although variations in climatic conditions in Uganda are not as significant as those of the US due to difference in climatic seasons, results in this study indicate that both rutting and fatigue cracking are significantly sensitive with respect to changes in climatic conditions. This is in line with similar studies conducted in the U.S.

6.4.Summary of Results

Table 0-19: Summary of results for sensitivity analyses.

Distress	Effect of parameter under study							
	Traffic			Material properties			Climate	
	Volume	Growth factor	Operation speed	Air voids	Effective Binder content	Subgrade type	PG grade	climate
Terminal IRI (in./mile)	**LS**	**I**	**I**	**LS**	**I**	**LS**	**I**	**S**
Permanent deformation - Total pavement (in.)	**S**	**LS**	**LS**	**LS**	**I**	**LS**	**LS**	**ES**
AC bottom - up fatigue (% lane area)	**ES**	**ES**	**VS**	**ES**	**ES**	**ES**	**VS**	**ES**
AC top - down fatigue (ft/mile)	**ES**	**LS**	**LS**	**ES**	**S**	**VS**	**LS**	**S**
AC rutting (in.)	**ES**	**LS**	**LS**	**S**	**LS**	**I**	**LS**	**ES**

Results of the sensitivity analysis show that Bottom-Up fatigue cracking is the most sensitive distress to design input parameters under study while Top-Down fatigue cracking is significantly sensitive to traffic volume and % Air voids in the asphalt mix. Rutting is generally less significantly sensitive compared to fatigue. Total pavement Rutting is mostly significant to climatic conditions while rutting in AC only is significantly sensitive, to both climatic conditions and Traffic volume. However, there is need for local calibration to better understand the behavior of these performance prediction models.

Based on the sensitivity analysis conducted in the case study for this thesis, the following observations were made: -

- Sections designed using current design procedures in Uganda for the same serviceability and life span do not perform equally, being evaluated by Pavement ME.
- Current Uganda design procedures produce over-designed pavement sections.
- The current Uganda design procedure under-estimates fatigue cracking for high traffic loadings.
- Bottom-up fatigue cracking is reported to be the most sensitive distress to variations in investigated design inputs for Pavement ME.
- Climate and traffic loading are reported to be the most influential design inputs for pavement performance for inputs under investigation

- Pavement ME is a comprehensive pavement design procedure that addresses comprehensively factors acting on pavements sections and is expected to produce more economic/cost effective pavement sections.

- Pavement ME is not yet recommended for design use in Uganda and further research related to pavement material properties and input data preparation is required prior to its implementation.

CHAPTER 7

FRAMEWORK FOR LOCAL CALIBRATION OF DISTRESS PREDICTION MODELS

7.1.Introduction

Since there are no LTPP sites to be used for implementation in Uganda, the only available data on pavement distresses is from the Uganda National Roads Authority (UNRA) and other stake holders in the road design and construction sector in the country. UNRA is attempting to build a pavement management system in order to regularly perform the needed observations and consequently, timely rehabilitation interventions for their road networks.

However, to achieve reasonable and accurate results using Pavement ME, there is need to follow the calibration procedure recommended by the proceedings of project

NCHRP 1-40b, but with some variation as to suit the specific conditions and data available in Uganda.

7.2.Local Calibration

This chapter describes the procedure for the local calibration of performance prediction models of AASHTOWare Pavement ME Design software version 2.3. The calibration process is based on guidelines for local calibration in the MEPDG guide (AASHTO, 2010) developed under NCHRP Research Project 1-40B. The 10-step procedure for local calibration followed in this study is listed below.

1. Select hierarchical level of input for each parameter.
2. Develop local experimental plan and sampling technique.
3. Estimate sample size for various distress prediction models.
4. Select roadway segments.
5. Extract and evaluate distress and project data.
6. Conduct field and forensic investigations.
7. Assess local bias by validating globally calibrated values to local conditions, policies, and materials.
8. Eliminate local bias of distress and IRI prediction models.
9. Assess the standard error of estimate (Se).
10. Reduce the standard error of estimate (Se).

The following sections describe the local calibration methodology by briefly discussing these 10 steps.

7.2.1. Select Hierarchical Input Level

The selection of the hierarchical input level is one of the most important steps in local calibration. Because this selection is primarily an agency decision, inputs provided by UNRA or MoWT can be used. The inputs should be coherent with the agency's current field and laboratory testing capabilities, material and construction specifications, and traffic data collection procedures and equipment. The input selection procedure can be broadly divided into three categories: traffic, climate, and materials.

7.2.2. Develop Local Experimental Plan and Sampling Technique

A sampling template usually is created in order to select projects that representatively reflect current and future agency practices for the design and construction of pavements.

7.2.3. Estimate Sample Size

Sample size estimation approximates the minimum sample size or the minimum number of pavement projects needed for local calibration and validation of MEPDG distress prediction models depending on the model error or standard error of estimate (SEE), the confidence level for statistical analysis, and the threshold value of performance indicators at an agency's design reliability level. The required number of pavement projects for local calibration and validation of MEPDG models as recommended by (AASHTO, 2008b) are presented in Table 0-1.

Table 0-1. Estimated number of pavement projects required for the local calibration and validation.

Pavement Type	Performance Indicators	Performance Indicator Threshold (at 90% reliability)	Standard Error of Estimate (SEE)	Minimum number of Projects required for local calibration and validation	Minimum number of Projects required for each pavement type
New HMA	IRI	169 inches/mi	18.8 inches/mi	80	18
	Rutting	0.4 inches	0.107 inches	14	
	Alligator cracking	20% lane area	5.01%	16	
	Transverse thermal cracking	Crack spacing > 100ft of 630 ft/mi.	N/A	18	
New JPCP	Faulting	<0.15 inches	0.033 inches	21	21
	Transverse cracking	<10% slabs	4.52%	5	
	IRI	169 inches/mi.	17.1 inches/mi.	98	

Source: (AASHTO, 2010)

The sample size, or number of pavement projects, was less than the minimum recommended number of projects. However, since accuracy of the IRI model depends on the accuracy of other distress prediction models of MEPDG, a large number of projects do not need to be adopted in order to calibrate and validate IRI models if other models are accurately calibrated and reliable.

7.2.4. Select Pavement projects

Pavement projects are selected based on the availability of information in terms of design in put requirements for Pavement ME, in order to accurately calibrate and validate the intended pavement distress models.

7.2.5. Extract and Evaluate Measured Distress and Project Data

Extraction and evaluation of measured distress and project data requires four activities, as described in the following sections (AASHTO, 2010).

a) Extract and Convert Measured data

Measured distress data must first be extracted, reviewed, and, if necessary, converted into values predicted by the MEPDG. AASHTO suggests that a consistent definition and a measurement protocol of surface distress should be used throughout the calibration and validation process. Agencies can use LTPP-measured distress data or their own PMS database for local calibration. When using PMS data, a minimum of three observations per project should be taken to use it for calibration or using the PMS condition survey data from the established PMS segments is recommended. In this study measured pavement distress and IRI data were collected from the UNRA database.

b) Comparison of distress values

Comparing of distress values includes comparison of maximum measured distress values with trigger values or design criteria specified by an agency. According to AASHTO, the average maximum measured distress values from samples should exceed 50% of the design criteria as a minimum because if maximum distress values are significantly lower than the agency's design criteria for that distress (less than 50% of the design criteria), the accuracy and bias of the transfer function may not be well-defined at the values that trigger major rehabilitation (AASHTO, 2010).

c) Checking Anomalies and Outliers

Measured distress data of all pavement sections should be evaluated and checked for outliers and anomalies using a thorough visual inspection of data with time to ensure reasonability of the distress data or using a detailed statistical comparison of measured performance data. In this study statistical analysis was performed to find the outliers and some explicit outliers and anomalies were excluded from the measured data set.

d) Determination of all MEPDG Inputs

In this stage all MEPDG inputs, including site-specific values, Agency-suggested input values and default values are determined.

7.2.6. Conduct Field and Forensic Investigations

Study inputs can be collected from various sources, including actual project data, Agency-suggested input values, and MEPDG default values. In case no field or forensic investigations are conducted, these evaluations are necessary when any data element is missing, or key inputs must be validated. Field or forensic investigations are recommended because they improve the reliability of the calibrated models.

7.2.7. Assess Local Bias from Global Calibration Factors

In this step global calibration values of MEPDG can be used to calculate the performance indicator for each roadway section. The predicted values may then be compared to the measured values to determine the bias and SEE to validate each distress prediction model for local conditions, policies, specifications, and materials (AASHTO, 2010). Bias is the difference between MEPDG-predicted output values and field-observed distresses. If the software-predicted distress is systemically different from the field-measured distress, then statistical bias exists in the model and it must be calibrated, requiring the standard error of sampling distribution which is also known as SEE. AASHTO defines the SEE as the standard deviation of residual errors for pavement sections included in the calibration data set for each prediction model (AASHTO, 2010). In addition to bias and SEE, an attempt should be made to quantify utility of the global and calibrated performance models using the coefficient of determination (R^2) parameter. The purpose of this parameter is to represent the proportion of the sum of squares of deviations of AASHTOWare predicted values about their mean that can be attributed to a linear relationship between predicted and field measured data (Mendenhall, W., Sincich, T., & Boudreau, 1996).

7.2.8. Eliminate Local Bias

If bias is found to be significant from global calibration coefficients, it must be eliminated by local calibration. The calibration process usually depends on the cause of bias and accuracy desired by the agency. The following three approaches generally are followed in order to eliminate bias (AASHTO, 2010):

- If residuals errors are always positive or negative with a low SEE compared to the limiting value and the slope of residual errors versus predicted values is relatively constant and close to zero, then the precision of the prediction model is reasonable, but the accuracy is poor. This situation generally requires the least level of effort, and most of the time few runs or iterations are enough to reduce the bias.

- If the bias is low and relatively constant with time but the residual errors have wide variation from positive to negative values, then the accuracy of the model is reasonable, but the precision is poor. The coefficients of the prediction equation can be used to reduce the bias, but the value of the local calibration coefficients may be dependent on site features, material properties, or design features in the sampling template. This condition usually requires an increased number of MEPDG runs and higher effort to reduce the bias.
- If the residual error versus the predicted values show a significant and variable slope that appears to be dependent on the predicted value, then the correlation between the predicted and measured values is very poor and the precision of the prediction model is also poor. This is the most complex condition for local calibration because the exponents of the number of loading cycles must be considered. This condition also requires the highest level of effort and much more MEPDG runs to reduce the bias.

7.2.9. Assess the Standard Error of Estimate

In this step SEE derived from locally calibrated models is compared to the SEE of the nationally calibrated distress prediction models of MEPDG, and the reasonability is checked. Reasonable values of the SEE of nationally calibrated models are provided in Table 0-1.

7.2.10. Reduce the Standard Error of Estimate

If the user agency determines that the standard error is too large, resulting in overly conservative design in higher reliability levels, then local calibration values of the transfer function or statistical model may need to be revised. An agency must decide about that, however, because the process can be very complicated and potentially require external revisions to local calibration parameters or agency-specific input values in order to improve precision of the prediction model (AASHTO, 2010).

Two types of errors commonly constitute the standard error: lack-of-fit, or model error and measurement error. Local calibration can reduce only lack-of-fit portion of the standard error. The measurement error is the larger of the error components, and changes only to values of local calibration coefficients will not change the magnitude of this error. Therefore, the agency must decide whether or not to spend additional money and effort on reducing measurement errors. If the determination is made that the extra effort will significantly reduce the SEE of the specific distress or IRI prediction models, they can revise the local calibration process.

The standard error of each cell of the matrix must be determined in order to establish whether the local standard error term is dependent on any primary or secondary input parameter of the sampling matrix. The local standard error results can be used to make necessary revisions to specific local calibration parameters. After the revisions are complete, the local calibration values are adjusted to reduce the standard error of the recalibrated data set. Based on goodness-of- fit criteria, a fitting process of the model constants are evaluated on the best set of values for the coefficients of the model. The analytical approaches used are based on least squares using multiple regression analysis, stepwise regression analysis, principal components analysis, or principal components regression analysis.

7.3. Challenges and opportunities of Implementing ME guide in Uganda

7.3.1. Introduction

The AASHTO 1993 guide and the South African Pavement Engineering Manual (SAPEM) are the current major references being used for pavement design in Uganda. The national road design catalogue issued by the Ministry of Works and Transport (MoWT) in accordance to the South African code of practice, provides the designs for various types of pavement construction in a series of charts. Based on the subgrade types, $\mathbf{S_i}$ and traffic classes, $\mathbf{T_i}$ the thicknesses of the construction layers are assigned for each pavement type. However, stakeholders such as MOWT and the Uganda National Roads Authority (UNRA) realize the shortcomings and limitations of the existing methods and have shown interest in implementing the mechanistic-empirical pavement design approach. As previously mentioned, various challenges present themselves in trying to implement Pavement – ME by any highway agency.

Such challenges include:

1. Inadequate pavement design inputs database that is necessary for ensuring accurate and realistic results for this new design methodology,
2. Insufficient road performance data to aid in the calibration of the performance prediction functions in the software.

3. Absence of equipment and technologies that enable measurement of critical input parameters related to material characterization and traffic characterization.
4. The financial cost incurred with the efforts of implementation, which will require utilization of huge monetary resources.
5. Absence or deficiency of documented historical climatic data in the required format.
6. Absence of human resources that are knowledgeable and trained on Pavement-ME.
7. Need for validation check to conditions of Uganda and local calibration.

Therefore, there is an urgent need for research and technology transfer that is focused on ways to render the adoption of mechanistic-empirical pavement design tools more appealing to countries other than the U.S.A and Canada.
Understanding these limitations is a fundamental component of this research study. This understanding will help in defining the requirements of implementing Pavement-ME in Uganda and will act as a reference for future research in Uganda and other neighboring countries.

7.3.2. Loopholes of the Current Design Approach in Uganda

Pavements in Uganda are commonly designed based on the catalogs for typical pavement structures in Uganda. These Catalogs that are derived from the AASHTO 1993 guide and SAPEM. They assume that after the design period, pavements will reach critical conditions, (Theyse, H. L., & Muthen, 2000) and rehabilitation treatment will be required. Such an approach is, in fact, incompatible with Pavement Management Systems, as it does not deliver information about the scale and progress of deterioration. The Mechanistic-Empirical Pavement Design Guide M-EPDG (Pavement-ME) has provision for pavement performance, extent, and intensity of pavement distress. Despite its introduction in the USA, there have been limited trials of implementing ME guide in Africa.

Table 0-2 presents the conceptual difference between AASHTO-93 guide, AASHTO ME guide and the Ugandan pavement design catalog

Table 0-2: Conceptual comparison between AASHTO-93, Uganda pavement design catalog and Pavement ME

Parameter	**AASHTO 1993**	**Pavement – ME**	**Uganda pavement design Catalog**
Comprehensive software	No	Yes	No
Pavement Type			
New pavement Design (Flexible or Rigid)	Yes	Yes	Yes
Rehabilitation AC over Fractured PCC slab	No	Yes	No
Inputs			
Hierarchical Input levels	No	Yes	No
Traffic			
Load Spectra	No	Yes	No
Hourly, Daily, Monthly Traffic Distribution	No	Yes	No
Traffic Lateral Displacement	No	Yes	No
Traffic Speed (Rate of Loading)	No	Yes	No
Special Vehicle Damage Analysis	No	Yes	No
Climate			

Historical data record	No	Yes	No
Dry or Wet warm climate	No	Yes	Yes
High Elevation Climate	No	Yes	No
Coastal climate	No	Yes	No
Material Characterization			
Non-linear inbound Material characterization	No	Yes	No
Considers short- and Long-term Age Hardening	No	Yes	No
Hot Mix Asphalt Modulus at different Temperatures and Loading frequencies	No	Yes	No
Unbound Material Resilient Modulus Adjusted for Moisture Variation During pavement life	No (only seasonal variations of the modulus are considered)	Yes	No
Binder characterization	No	Yes	No
Distress Predictions			
AC and inbound materials rutting	No	Yes	No (only subgrade terminal rutting of 20mm)
Alligator and Longitudinal Fatigue cracking	No	Yes	No
Transverse cracking	No	Yes	No
Smoothness	No	Yes	No
Allows different Design reliability for Each Distress	No	Yes	No
Models Calibration			

Nationally calibrated/Validated models	No (only data from AASHO road test)	Yes	No
Traffic repetition used in Calibration	Only 1.1 million ESALs	Up to 27 years	Considers daily repetition of a specific axle type

7.3.3. Challenges for Implementation of Pavement-Me in Uganda.

There are numerous challenges for implementing Pavement-ME in Africa, and particularly in Uganda. These challenges primarily relate to the high level of detail of the various inputs required for accurate and reliable pavement design. In this case study, the research done at the Uganda National Roads Authority identified the following as significant deterrents:

- **Climate files:** All climatic files embedded in Pavement-ME are particular to regions in the United States and Canada. Although MERRA provides historic climatic data whose format matches that required by the ME software, implementing Pavement-ME in Uganda would require compiling the proper hourly climatic databases for accuracy.
- **Traffic level and vehicle classification:** A comprehensive traffic database is not readily available in Uganda and could hinder adopting the Pavement-ME until the required records are obtained.
- **Material properties and testing:** Uganda, in its entirety, has not yet adopted certain advanced asphalt materials testing, such as the simple performance test, and

Superpave binder grading tests. Moreover, material testing standards used could be different than those required for the Pavement-ME, such as British Standards, and country-specific standards.

- **Model calibration:** The performance prediction models and the Witczak models used in the Pavement-ME have been regressed based on data that is specific to the USA. Although the Pavement-ME offers the option of inputting local calibration coefficients, Uganda does not have the means to calibrate these equations locally. Perhaps the biggest challenge and obstacle towards implementing Pavement-ME in Uganda would be the lack of means and the required test sections to calibrate the prediction models.

In addition to the above limitations, design firms in Uganda, may often lack the financial resources required to purchase and maintain annual Pavement-ME software licenses. Moreover, education, awareness, training, and willingness are crucial. Pavement designers and Engineers in Uganda need to gain enough familiarity with the new Pavement M-E software and knowledge of the incorporated design methodology, design inputs and levels, and their relationships to key distresses and performance.

7.3.4. Opportunities for Implementation of Pavement-Me in Uganda

There are many opportunities for adopting the Mechanistic-Empirical Pavement Design Guide (MEPDG) in Uganda and the neighboring countries as well. Such opportunities can be attributed to using a more comprehensive guide and Pavement-ME too. The tool has provision for designing both new pavements and existing pavements for rehabilitation, different traffic loading types, material characterization at various input levels (Ayyala et al., 2018), and incorporates the effect of the environment to the service life of the pavement.

7.3.5. Recommendations and Benchmark for Future Research

Pavement-ME utilizes an entirely different and significantly improved approach (Daniel, J. S., & Chehab, 2008) than current design methods, and research identifies what needs to be done to switch to it from the currently adopted Guides. Additionally, this thesis project provides guidelines for identifying test sites to be used as calibration sections in the future to perform local and regional calibration of the models in the Pavement-ME.

The primary focus for future research should be on the evaluation of the currently used Design methods and Pavement - ME, with emphasis on Materials in consideration to related factors such as climatic conditions and traffic characterization in Uganda.

The challenges of adopting the new AASHTO – pavement-ME are highly attributed to inadequate input data (Chehab et al., 2017; Nantung et al., 2005).

As part of this thesis recommendations, future research should look at addressing these challenges to facilitate the transition from the current empirical design practices to the new ME guide.

Some of the main items that should be considered include:

For pavement design:

- Who performs pavement designs: in-house, contractors, division/main offices?
- What is the current design methodology?
- What information and data are used in the current pavement design?

- What are the major distresses and issues of concern: fatigue cracking, fitting, smoothness?
- What are the failure criteria: % cracking, rut depth, IRI?
- Reliability level in design (error tolerance)
- Classification of roads: low volume vs. high volume roads?
- Where are material properties measured (main lab, division labs, research labs)
- Are materials, design, and construction specifications detailed and appropriate for use in developing default input data?

For data collection and management:

- List of all input data collected by the road's authority for pavement design and management;
- Units of measurements, testing protocols, protocols for data storage: electronic media, paper;
- Link and communication between the various pavement related databases;
- Existence of instrumented sites with construction and performance records.

CHAPTER 8

CONCLUSION

8.1. Introduction

The methodology that the Mechanistic Empirical Pavement Design is based on requires a lot of data to be collected. Some of this data is acquired by lab testing and site visits, which requires proper lab equipment and expertise, other type of data is collected by road observations which also requires technologies, expertise and proper management. This thesis was able through sensitivity analysis, to identify the most significant inputs for certain parameters (
Table 0-19) under study. The thesis also recommended the need for local calibration of the distress prediction models used in Pavement ME and proposed a framework for such calibration based on current pavement design and management practices.

Based on the research conducted in the case study for this thesis, the following conclusions can be made: -

- As much as Pavement-ME is the currently recognized design guide by AASHTO, many countries outside the US and Canada still use empirical design approaches that are derived from the AASHTO 1993 guide.
- Uganda and other neighboring countries in the Great Lakes region of Africa are hesitant to adopt MEPDG (Pavement-ME) due to lack of sufficient Input data required for Pavement-ME. Therefore, region-specific customization of such input files like Climate files can be helpful in the transition process from the currently used design methods to the new pavement-ME.

- Further research is needed to spearhead local calibration efforts in Uganda and other neighboring countries.

8.2. Recommendations for future Research

The following 10-point recommendations are identified for future research based on the findings of this thesis: -

I. Calibration of the distress prediction models adopted by ME guide to best represent the Ugandan conditions.

II. Investigation of other design inputs such as AC mix properties, input level, and Granular base properties

III. Preparation of a comprehensive climatic database for Uganda as well as the development of a tool for interpolating climatic conditions at locations that don't have weather data in the country. Such a feature is available in Pavement ME, only for locations in the US and Canada.

IV. Conducting Statistical Analysis for the ME design inputs to understand the interaction between design inputs and their effect on predicted performance.

V. Preparation of a pavement management system on spatial basis to be used for further research and study of ME design.

VI. Development of design failure criteria for ME design according to Ugandan conditions for design and calibration purposes.

VII. Conduct comparative study between vehicles classification in Uganda and that of FHWA used in Pavement ME, for accurate traffic input.

VIII. Develop Truck traffic class distributions with respect to various road classifications in Uganda as well as hourly and monthly adjustment factors.

IX. Develop optimization tools to assist in the iterative design approach adopted by MEPDG for designing optimum pavement sections.

X. Investigate drainage inputs since most parts of the Uganda experiences high precipitations, use DRIP (drainage design software embedded in Pavement ME).

8.3. Recommendations for Implementation

Implementation plans set by each agency differ according to the level of preparation, data availability, budget and knowledge of the ME guide. The details of each implementation procedure vary accordingly. Based on the findings of this study, the following Implementation procedure is recommended for the case study:

1. Formation of a local ME Implementation Team to develop and implement a communication plan with various agencies in order to raise the awareness towards implementing MEPDG. Activities conducted by implementation team may include:
    - Conduct workshops and training for agency staff.
    - Create user groups for sharing experiences and feedback issues with practice.
    - Identify knowledge gaps and research needs.

- Share successes and challenges of implementation with highways agencies and communities.

2. Development of an implementation plan, that includes:
   - Responsibilities, timelines,
   - Resource allocation (people, lab and field equipment, computers and software, training, etc.) and cost estimates/budgets,
   - Calibration tasks/activities and schedule
3. Development of implementation criteria, that includes:
   - Objectively based performance indicators,
   - Oversight or steering committee,
   - Audit process,
   - Update and/or improvement needs assessment

4. Preparation of detailed traffic data to be used for Pavement ME input through the development of monthly, vehicle class, and axle load distributions data using WIM/AVC collected data and other data such as lane distributions, volume variations and tire pressures.
5. Preparation of a local pavement materials database to be used in Pavement ME and calibration. This is to be done through:
   - Identification of specific material properties to be measured,
   - Decide on the most effective testing protocols for measuring material properties,

- Execution of material properties testing programs to generate the information necessary to create a local material properties database.

6. According to literature, high priority efforts should be focused on testing to determine HMA properties such as Dynamic modulus (E^*) and Shear modulus (G^*).
7. Investigation of local subgrade properties to determine in-situ modulus values for local subgrade soils and to provide comprehensive information on seasonal variations.
8. Preparation of accurate site-specific climatic database for weather data to be used for Pavement ME inputs with continuous update and refinement.
9. Establishment of in-service pavement test sections to locally calibrate and validate Pavement ME performance predictions. In situ instrumentation, periodic testing and pavement performance surveys are recommended to provide sufficient data for local calibration and validation.
10. Creation of pavement performance criteria for each distress type and functional class of highway using measured performance data from test sections an evaluation.
11. Verification, validation, and calibration of Pavement ME performance prediction models with local collected performance data to remove bias (consistent over- or under-prediction) and improve accuracy of prediction of distress and IRI models.
12. Development of local Pavement Design Manual procedures for using the Pavement ME to design new, reconstructed, and rehabilitated pavement structures in order to provide guidance on design procedures, obtaining proper design inputs, validation and calibration procedures and provide a set of recommendations to enable the

pavement designer to make adjustments to the design to meet performance criteria. The manual should also include a catalog of representative traffic load spectra for different road functional classes and climatic regions for use in routine design.

Figure 0-1 illustrates a proposed Mechanistic Empirical pavement design implementation framework for Uganda based on the findings of this thesis.

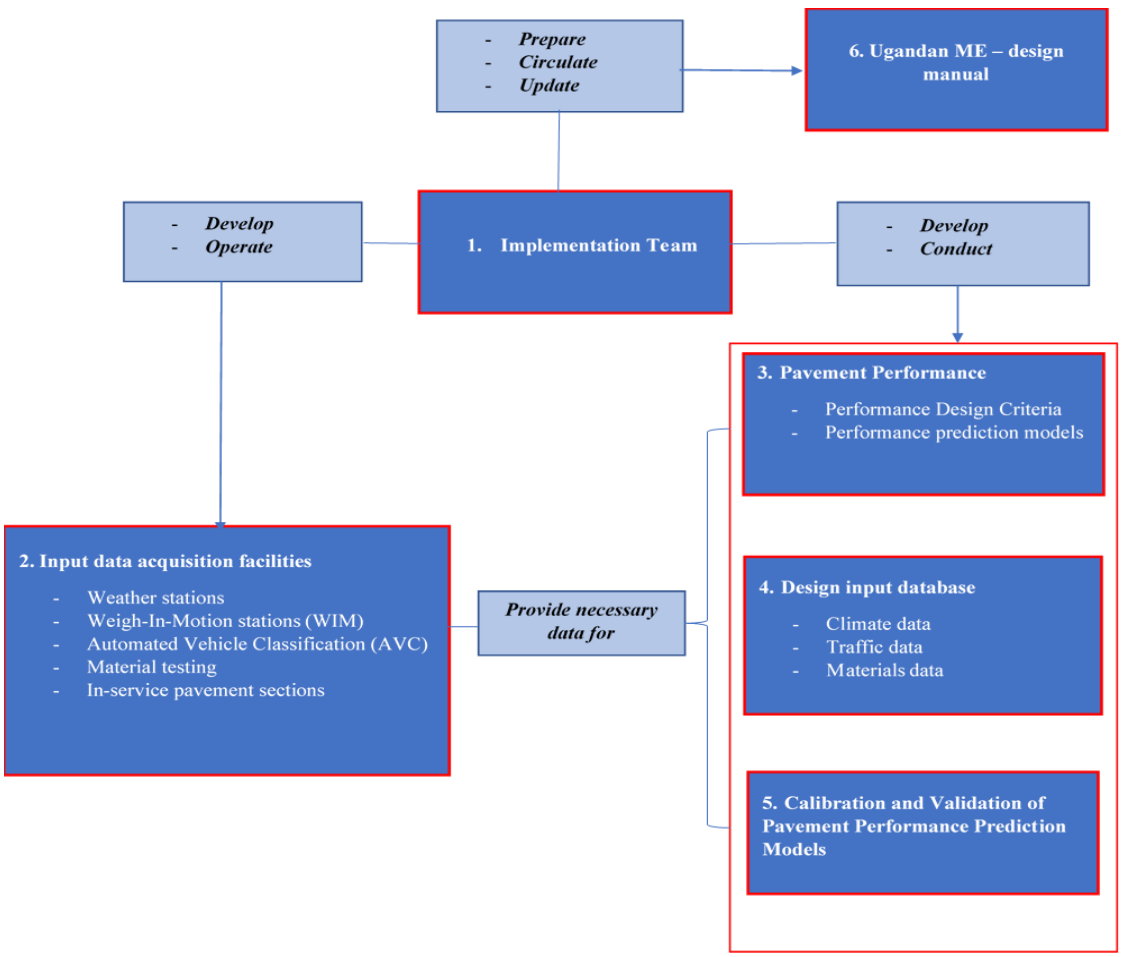

Figure 0-1. Mechanistic Empirical Pavement Design Implementation Framework for Uganda

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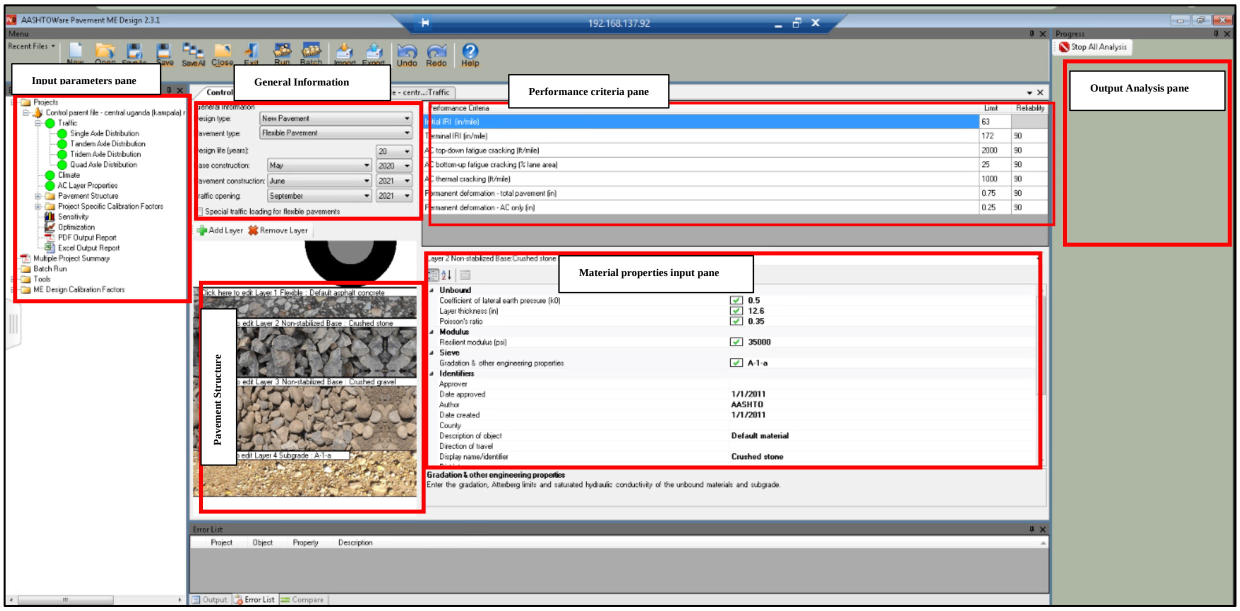
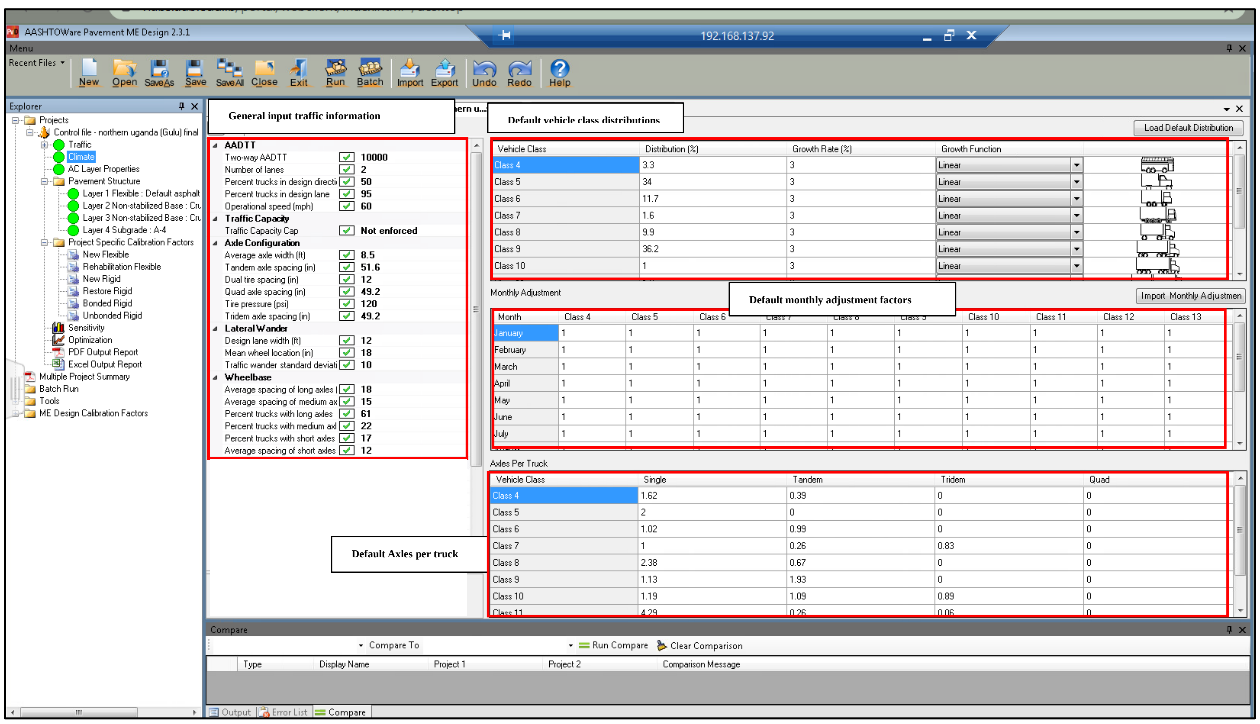
APPENDIX

AASHTOWare Pavement ME Input screen shots

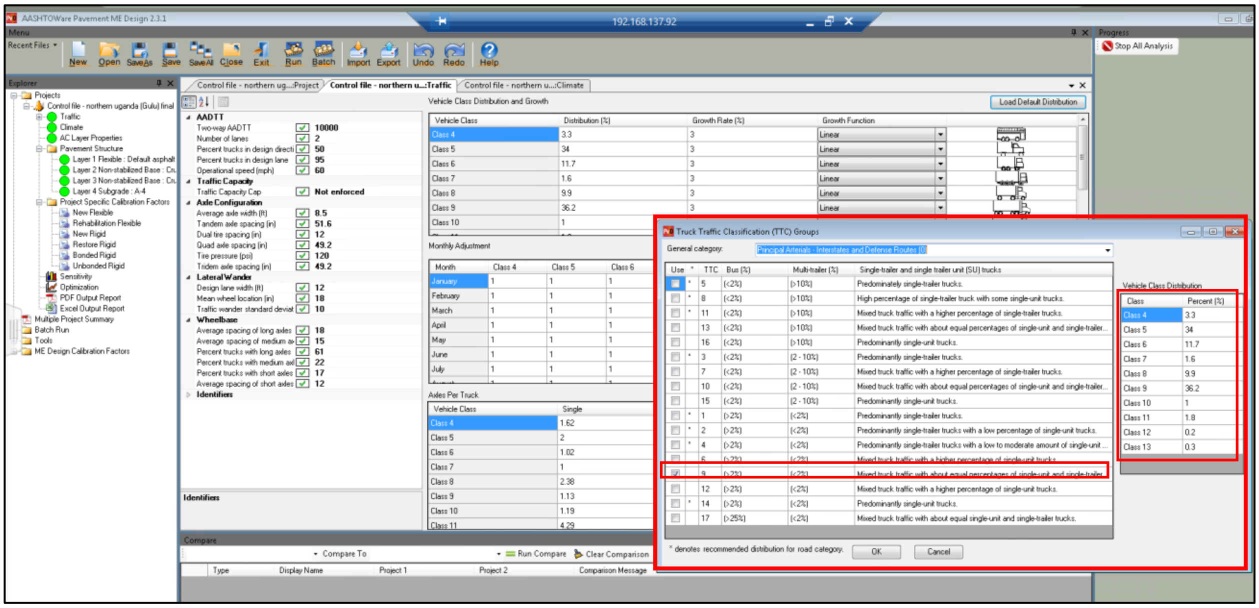
Software welcome Screen



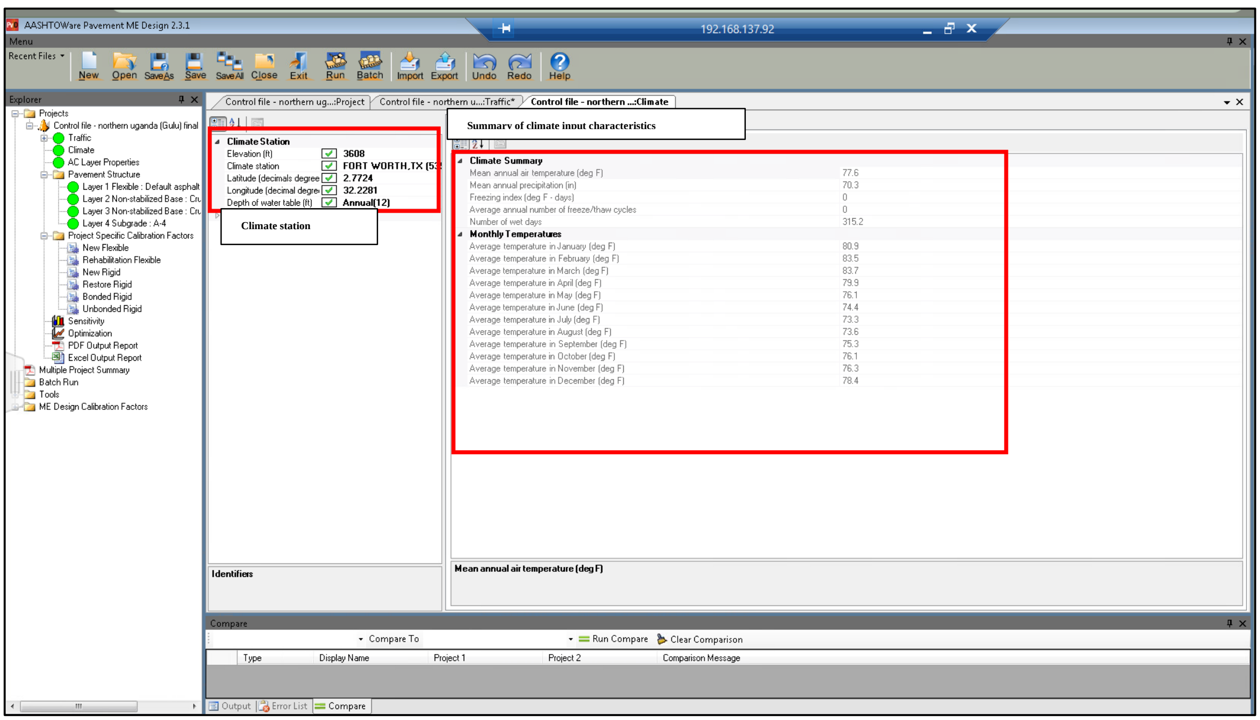
Home Screen



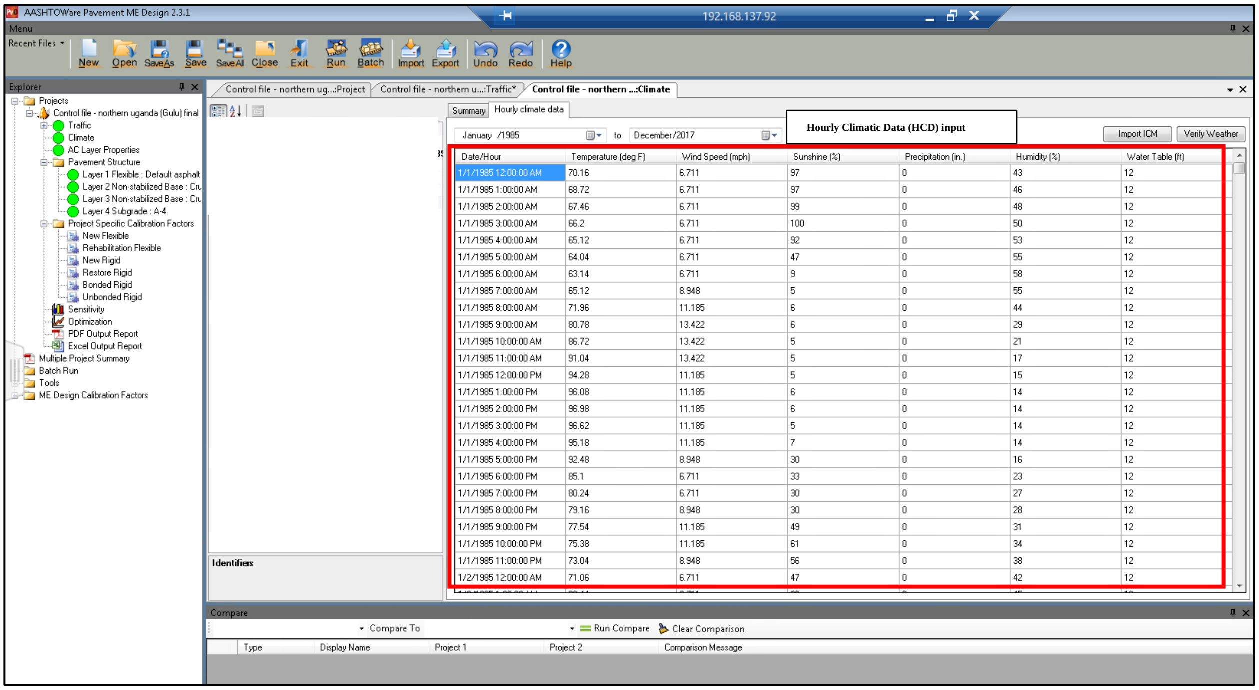
Traffic data input pane



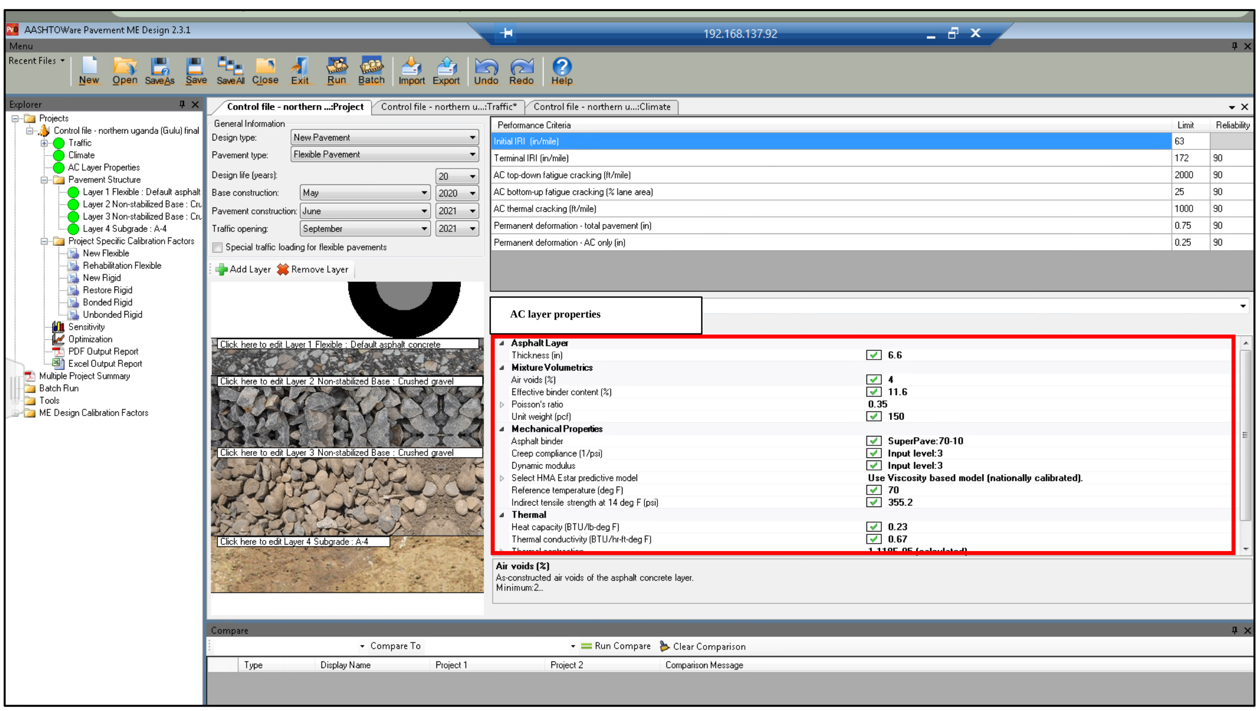
Truck Traffic Classification (TTC) and Vehicle distribution factors pane



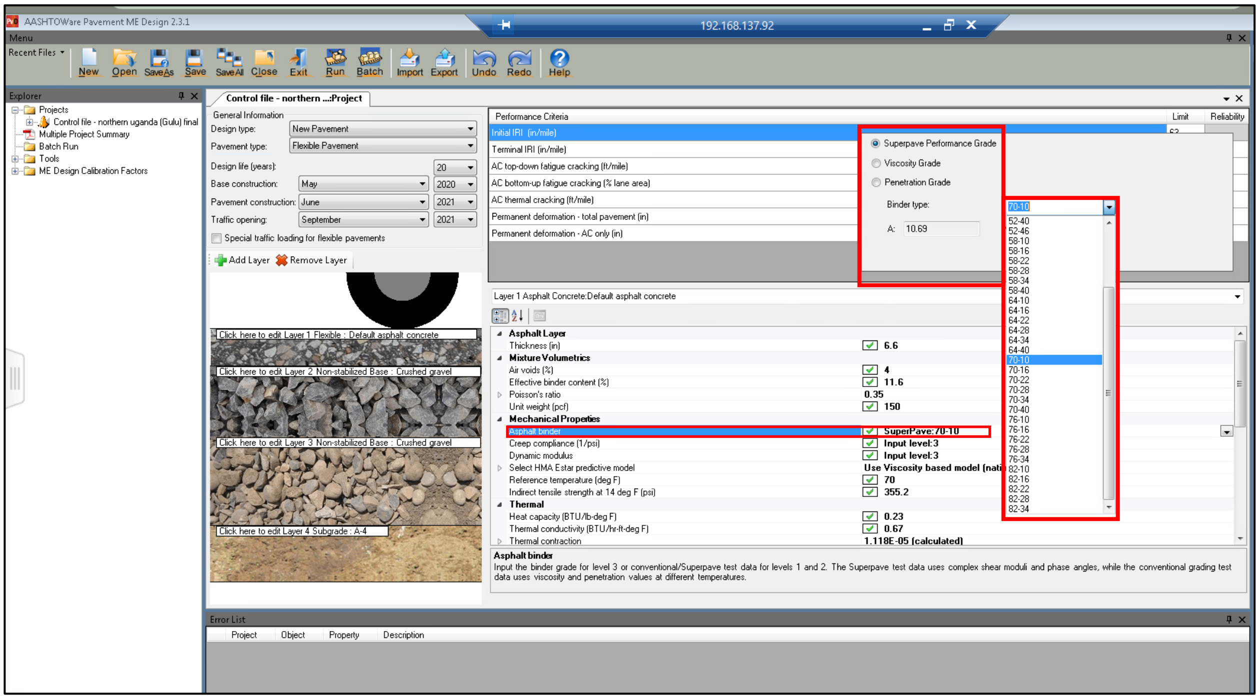
Climate data input pane



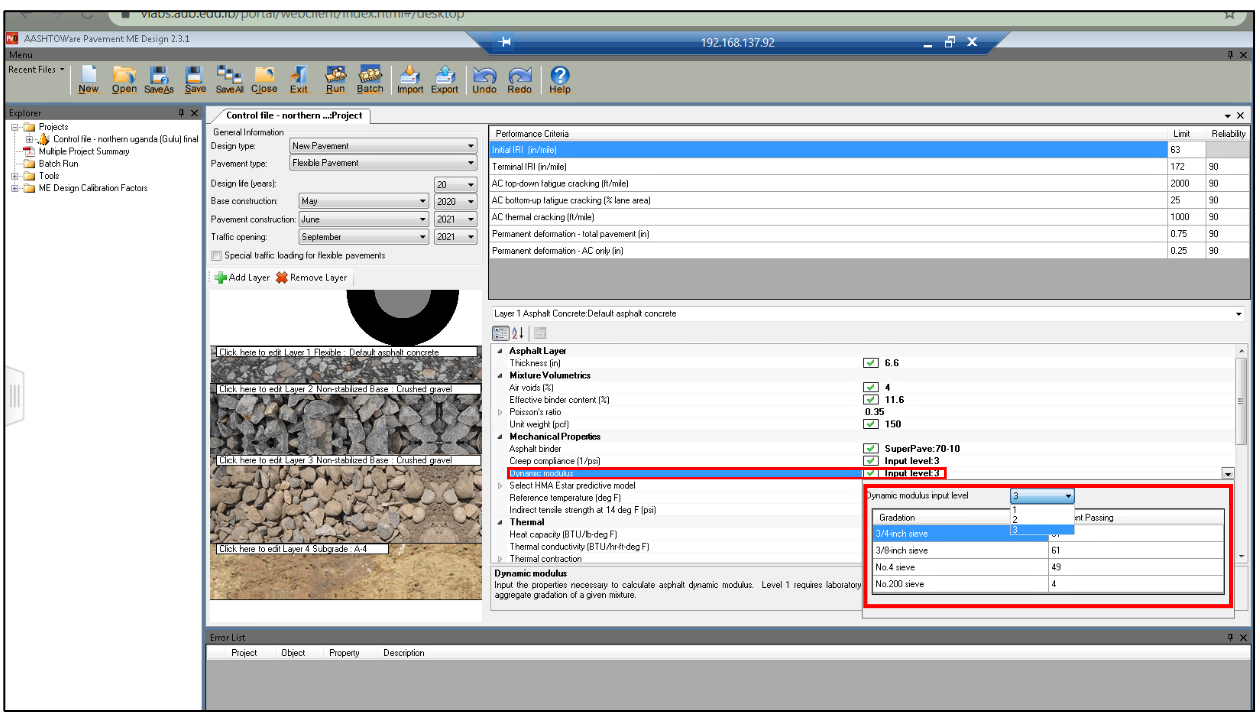
Hourly Climatic Data input pane



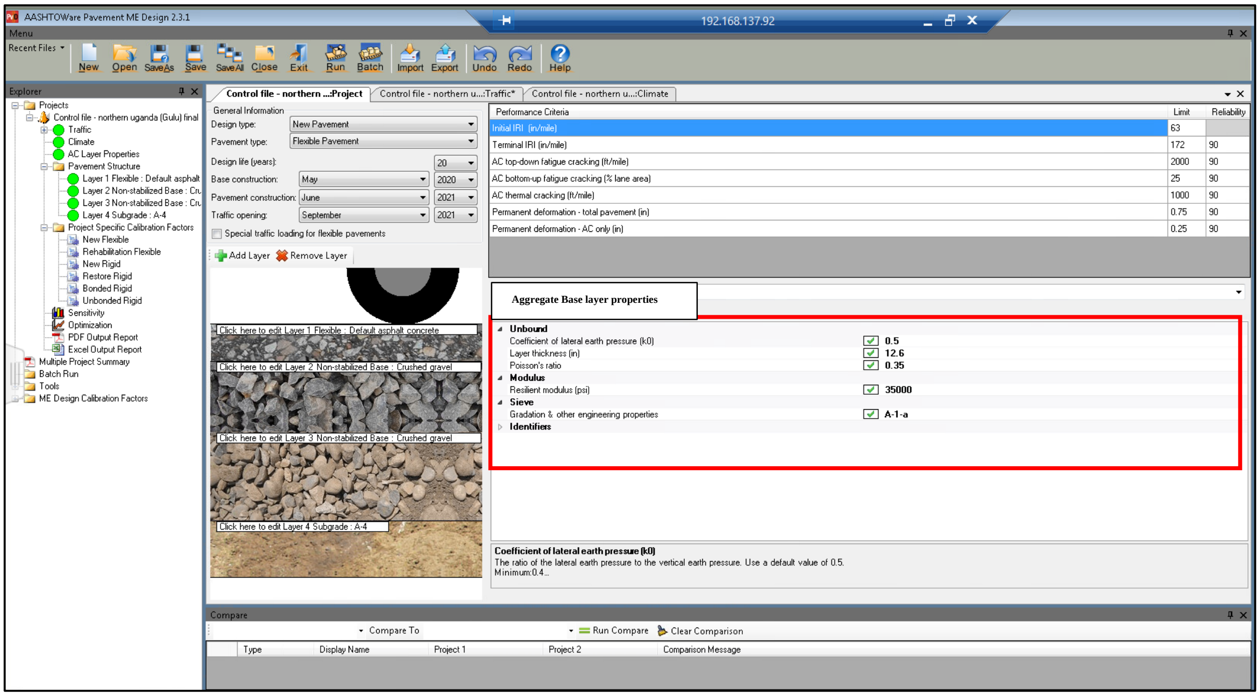
AC layer properties input pane



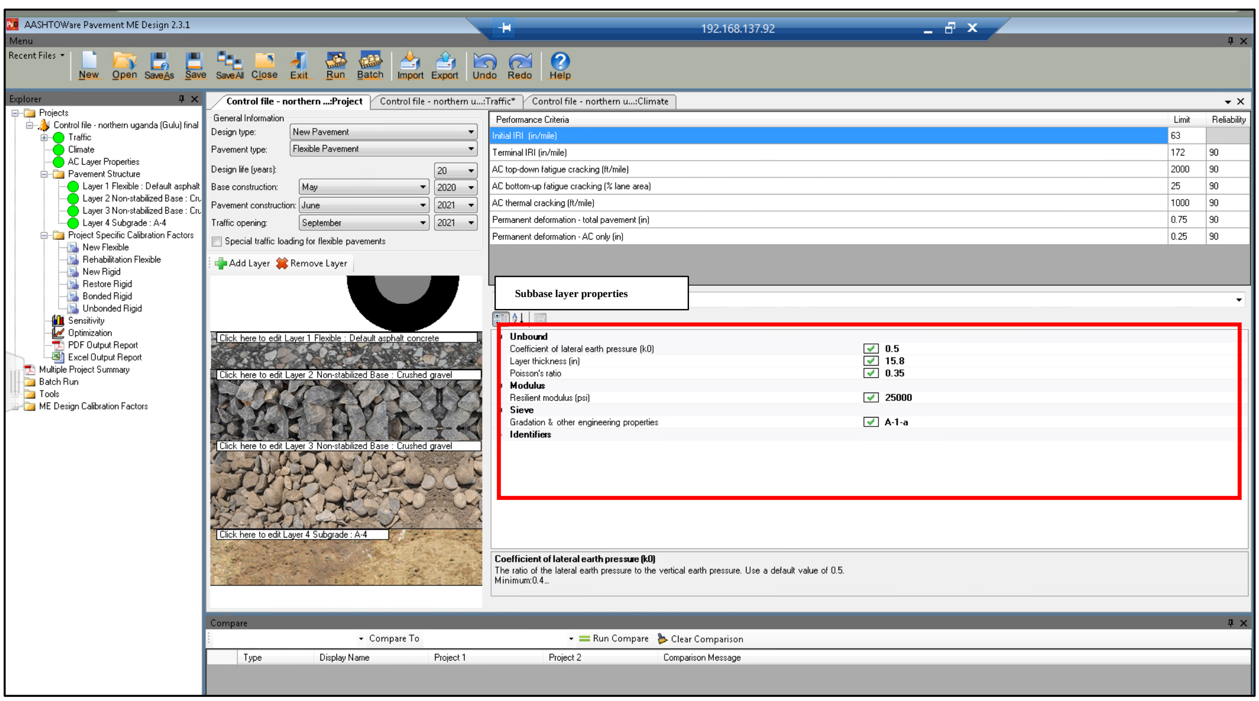
Asphalt Binder selection pane



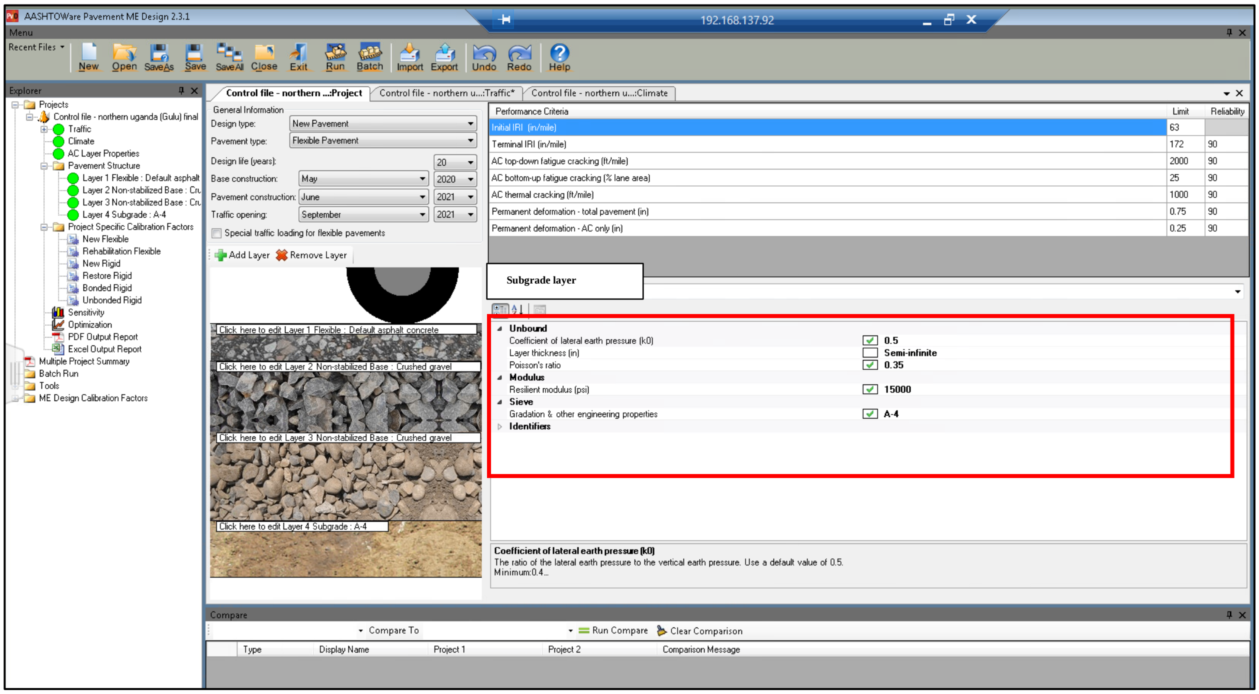
Dynamic modulus input level pane



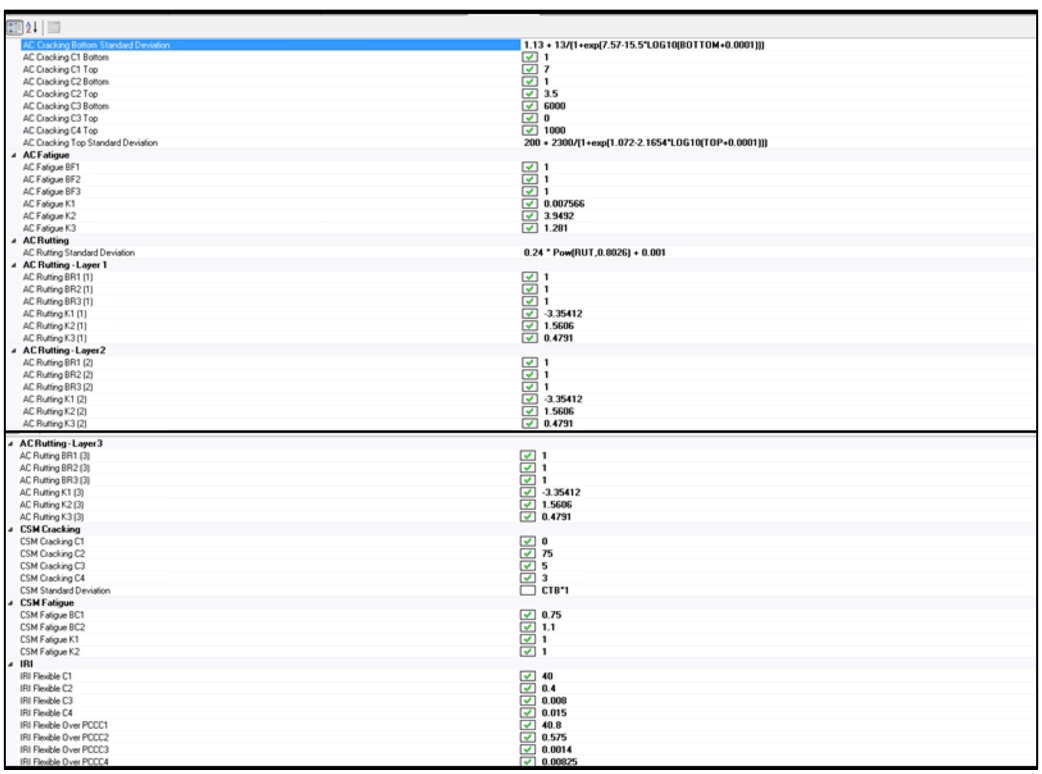
Unbound Layer properties pane (Aggregate Base)



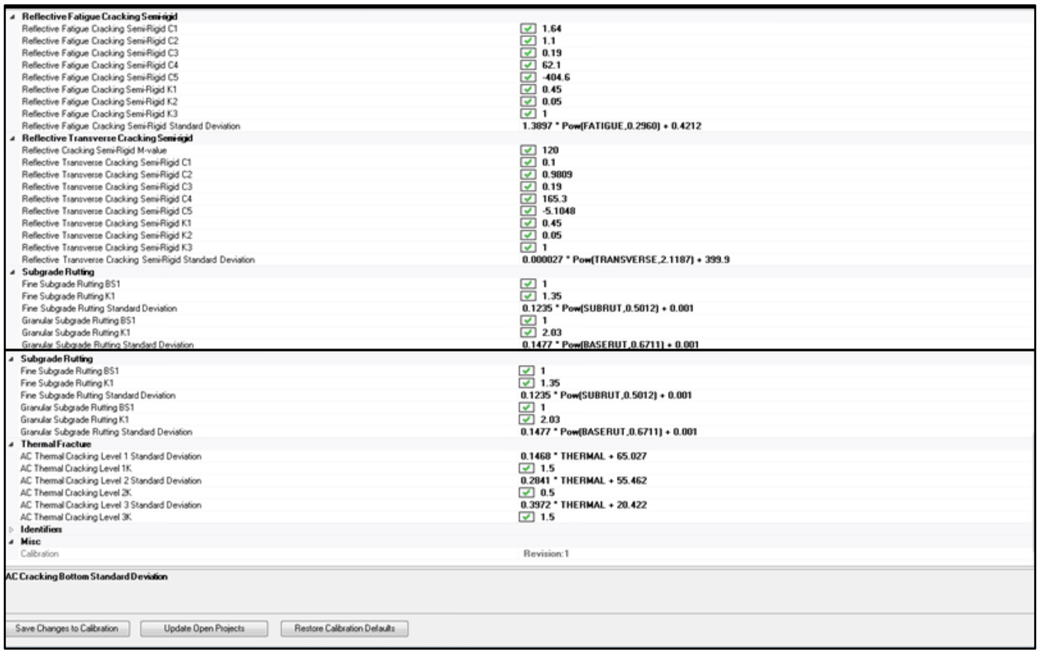
Unbound layer properties pane (Subbase layer)



Unbound layer properties pane (Subgrade layer)



Distress Model calibration factors



Distress Model calibration factors