CALIBRATION OF MEPDG PERFORMANCE MODELS FOR FLEXIBLE PAVEMENT DISTRESSES TO LOCAL CONDITIONS OF ONTARIO

by

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Presented to the Faculty of the Graduate School of

The University of Texas at Arlington in Partial Fulfillment

of the Requirements

for the Degree of

MASTER OF CIVIL ENGINEERING

THE UNIVERSITY OF TEXAS AT ARLINGTON

July 2016

Acknowledgements

I would like to endorse my sincere gratitude to my Supervisor, Dr. Stefan Romanoschi, for his interest and enthusiasm in the topic, continued guidance and help to overcome barriers. His subjects and methods motivated and attracted me to this domain of civil engineering.

I would like to express my gratitude to the members of my committee, Dr. James Williams and Dr. Stephen Mattingly for their valuable time to review my thesis and contributions of great value. I would like to extend my appreciation to Dr. Sia Ardekani and Dr. Victoria Chen for their subjects and teaching.

I would like to thank Fouad Tannous and Warren Lee from the Ministry of Transportation Ontario (MTO) for their contribution in obtaining pavement structure data and pavement performance information.

At last, I would like to express my gratitude to my fiancé for her unceasing encouragement and support. One final acknowledgement goes to my parents and sisters for their education and care.

July 18, 2016

Abstract

CALIBRATION OF MEPDG PERFORMANCE MODELS FOR FLEXIBLE PAVEMENT DISTRESSES TO LOCAL CONDITIONS OF ONTARIO

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The implementation of the American Association of State Highway and Transportation Officials (AASHTO) Mechanistical-Empirical Pavement Design requires the development of a design procedure that can be used by the agencies and engineering consultants to design new and reconstructed rigid and flexible pavements. To calibrate the design procedure for a region, a large dataset representing the particular local conditions is needed. It includes traffic, climate, site material characteristics, performance requirements and historical data. The performance models were calibrated in North America using the Long Term Pavement Database Program (LTPP), therefore, the models must be calibrated to local conditions in order to obtain more suitable parameters, formulas and predictions. It is expected that calibrated performance models using site-specific data will predict pavement performance approximated to the performance measured in the field. Gathering data related with observed distresses is essential for subsequent comparison with predicted distresses.

The primary objective of this project is to calibrate the performance models of flexible pavement distresses, including total rutting (permanent deformation) and asphalt concrete (AC) bottom-up fatigue cracking, to the local conditions of new flexible pavement in Ontario, Canada.
Sixteen (16) representative pavement sections from widening and reconfiguration highway projects were selected. Performance data, traffic data, structure information, materials properties

and performance data were obtained from site-specific investigation and pavement design reports provided by the Ministry of Transportation Ontario (MTO).

The AASHTOWare Pavement ME Design^{TM} was used to run the initial predictions using the global calibration coefficients. Then, the obtained predicted distresses were compared with the measured distresses to assess for local bias and goodness of fit. The analysis showed that, using the global calibration coefficients, the AASHTOWare model under predicted alligator cracking and over predicted total rutting. Statistical analysis, such as, Regression Analysis and the Microsoft Solver numerical optimization routine were used to find the regression coefficients, using the approach of minimizing the sum of squared error (SSE).

Concerning alligator cracking, the local calibration factors have improved the bias and standard error of the estimate (SEE). Plots also showed that points are randomly scattered along equality line and predicted values closer to the measured values.

Regarding permanent deformation (rutting), the local calibration factors have improved the bias and standard error of the estimate. The accuracy of the transfer function has increased in comparison to the use of the global calibration values, suggesting that the local calibration procedure has improved the rutting model. Analyzing the plots measured versus predicted, points are better scattered and a shift is clearly noted in the chart from global to local calibration, indicating that local calibration coefficients improved distress estimations.

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1 Introduction

The design of a pavement structure encompasses multiple modeling steps and the combination of many variables. Traffic load is a heterogeneous conjunction of vehicles, axle types and configurations that fluctuate throughout the day, season and life of the pavement structure. The pavement materials perform differently and the performance is influenced by temperature, moisture, magnitude, stress and other factors. More complications are added when they are exposed to severe environmental conditions including temperature differentials ranging from torrid heat to freezing and from dry to saturated moisture states.

Numerous developments over the last decades have provided and improved the accuracy and rationality of pavement design methods. The American Association of State Highway Officials (AASHO), financed by State highway agencies, the U.S. Bureau of Public Roads and other agencies and countries, constructed in 1958 a test site, called the AASHO Road Test, to study the pavement performance under specific conditions (uniform mixtures, on a single type of subgrade soil and under specific environmental conditions – Central Illinois). The information obtained was used to develop an empirical method, the AASHTO Pavement Design Guide (1972), to design flexible and rigid pavement structures. Over the years, refined versions have been published, (1986, 1993, 1998) which improve the original empirical equations to address shortcomings in the design procedure. The 1993 AASHTO Pavement Design Guide was broadly adopted by states, countries and transportation agencies.

Over the latest decades, pavement design methods advanced from empirical procedures (based on pavement performance observation) to mechanistic-empirical procedures (incorporating pavement response, damage accumulation and distresses). This evolution is due to advanced computational mechanics and computers that allow performing calculations, which significantly improved the predictions of pavement responses to load and climate effects. Improved characterization of traffic and materials, study and comprehension of climate effects on materials, greater knowledge of pavement performance and large databases are also outcomes of this evolution.

AASHTO has released the Mechanistic-Empirical Pavement Design Guide (MEPDG) – Manual of Practice in 2008. It provides a methodology to design pavements based on engineering models calibrated at the national level using a large amount of pavement performance information from the LTPP database. In 2011, AASHTO lunched the associated software AASHTOWare Pavement ME DesignTM, which is a software tool that develops and optimizes the models of the computational project carried out as part of National Cooperative Highway Research Program (NCHRP) Project 1-37A.

The current AASHTO Pavement Mechanistic-Empirical Design is based on the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed by the NCHRP, which works with the fundamental properties of the pavement, base and subgrade materials. The design and analysis procedure estimates pavement responses (stresses, strains, and deflections) and applies those responses to calculate incremental damage. In this procedure, the cumulative damage is empirically related to observed pavement distresses (AASHTO, 2008). In pavement design, the Mechanistic-Empirical approach integrates two principles. Firstly, the “mechanistic” approach is focused on the study of physical causes (loads and material properties of the pavement structure) seeking to explain the stresses, strains and deflections. The correlations between physical causes and phenomena are depicted using mathematical models, being the most common the layered elastic model. Secondly, the “empirical” approach deals with the observed performance to determine relationships. The estimated responses (stresses, strains and deflections) of a selected pavement segment will be tied with the observed performance International Roughness Index (IRI) and surface distresses (permanent deformation and cracking) under diverse traffic loading and climatic conditions in order to define what values result in pavement failure. Correlations between pavement failure and physical phenomena are represented by derived equations that calculate the number of loading cycles to failure.

The MEPDG design procedure is an iterative process. The procedure does not provide thickness. Instead, the outputs from the procedure are pavement distresses and smoothness. Site conditions, namely, traffic, material and subgrade properties, climate and pavement condition are considered for a trial design for a new pavement or rehabilitation project. The adequacy of the trial design is then assessed when compared with performance criteria and reliability through the prediction of pavement distresses and IRI. If the design fails to meet the required performance criteria at a specified reliability, the trial design must be revised and the process must be repeated until compliance (AASHTO, 2008).

Currently, the Ministry of Transportation Ontario (MTO) has implemented the pavement design methodologies based on the AASHTO 1993 method that have been validated and adapted to represent the conditions of the Province of Ontario. Lately, MTO is working to validate and implement of the AASHTOWare Pavement ME Design™. Accordingly, MTO issued the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report in 2012, which incorporates MEPDG current practices and default parameters for Ontario conditions (Ontario's Interim Report, 2012). Although the current guide provides reliable information based on engineering mechanics, transportation agencies in Ontario must conduct implementation with prudence. Note that the empirical models that correlate mechanistic pavement structural responses to predicted pavement distresses have been calibrated using information from the LTPP database. Given that these pavement sections are mostly in the United States, the calibrated distress models may not reflect the traffic and climatic conditions and material characteristics of Ontario. MTO is continually promoting, supporting and financing studies and surveys in order to obtain more data to refine and calibrate the models and predict with consistency and accuracy the pavement distresses.

1.1 Importance of Research

The MTO has undertaken ample investigation in an effort to validate and establish the AASHTOWare Pavement ME Design method for Ontario conditions. In 2012, MTO issued the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report (Ontario's Interim Report, 2012).

This is a reference document that provides default parameters for Level 3 analysis for Ontario conditions, which is very useful for designers when not all data from field investigation is available.

Recent studies (Jannat, G. 2012) conducted in Ontario concluded that the predicted values for International Roughness Index (IRI) and permanent deformation or rutting are greater than the measured values, which means that the existing calibrated models are conservative. These studies were conducted using traffic data from the Pavement Management System (PMS-2) database and general information about materials and performance.

In order to obtain more accurate calibration models, this thesis will seek to improve the pavement performance models using updated traffic information and performance information from MTO's Pavement Management System (PMS) and layers thicknesses and materials properties information from site-specific laboratory and field-testing presented in the pavement design reports. This calibration procedure does not consider rehabilitation and will focus only on new flexible pavement segments.

AASHTOWare requires a number of input parameters for analysis and allows for different levels of design input based on the available resources. Level 1 parameters provide higher accuracy, and thereby, this thesis will pursue site-specific laboratory and field-testing results related to traffic, materials, pavement structure, environmental conditions and pavement performance.

A calibration procedure encompasses the establishment of a parallelism between predicted and measured distresses. For flexible pavements, the calibration procedure will be carried out for total rutting (permanent deformation), and AC bottom-up fatigue cracking.

Figure 1-1 Alligator Cracking (Asphalt Institute, 2016)

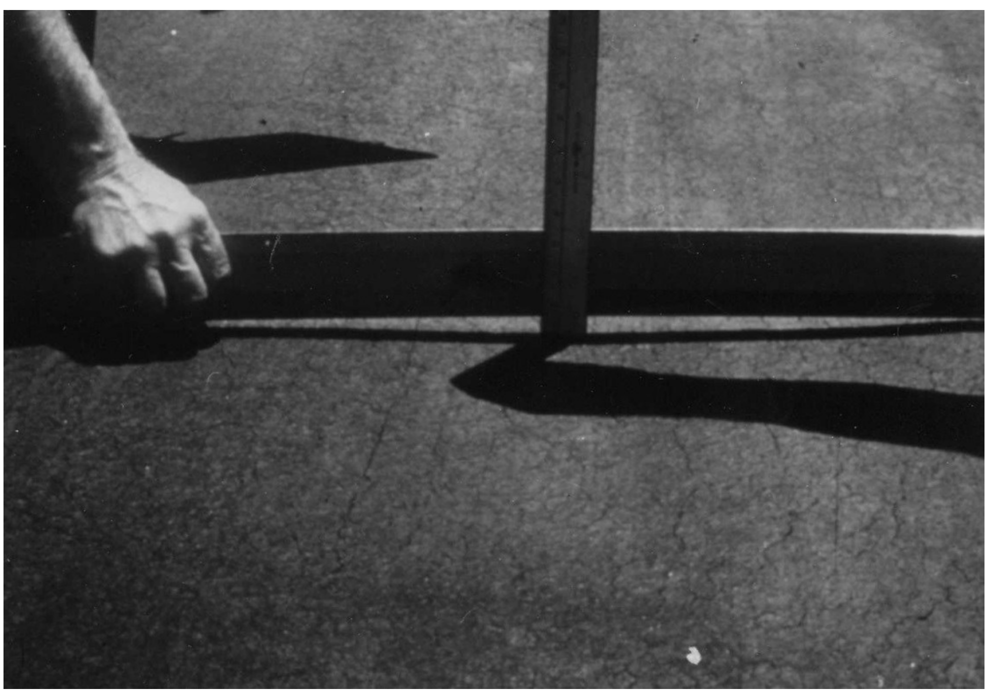

Figure 1-2 Rutting (Asphalt Institute, 2016)

1.2 Research Objectives

The primary objective of this project is to calibrate the performance models of flexible pavements distresses to the local conditions of Ontario, including total rutting (permanent deformation) and AC bottom-up fatigue cracking, using traffic information from MTO PMS and materials information from site-specific laboratory and field-testing.

1.3 Methodology

The methodology used consists of collecting, analyzing and processing data obtained from pavement databases and laboratory and field testing. The MTO pavement database is the major source for traffic, field-testing structural information and performance data. In addition, some traffic data, structural information and Hot Mix Asphalt (HMA) design reports were gathered from the Highway 407 Project situated in the Greater Toronto Area. For climatic data, weather stations embedded in the MEPDG software will be used.

The calibration methodology will follow the Guide for Mechanistic-Empirical Design (AASHTO, 2008) and other guidelines in the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report (Ontario's Interim Report, 2012). The AASHTOWare Pavement ME Design™ software will be used to iterate, calibrate and design pavement performance models.

1.4 Thesis Structure

This thesis is composed by five chapters considering the Introduction in Chapter 1. Chapter 2 offers a broad review of applicable literature related to Mechanistic-Empirical Pavement Design. It covers pavement design methods background, the AASHTO ME Pavement Design Method and relative design software AASHTOWare Pavement ME DesignTM, hierarchical design input levels, performance models for flexible pavements, Ontario's state of practice for local calibration and former calibration research.

Chapter 3 provides the database assembly, starting with the local calibration guide, followed by the selection of the hierarchical input level, selection of pavement segments, sample size computation, extraction and evaluation of data and lastly the data inputs for design.

Chapter 4 assesses the local bias and the goodness of fit of the models and presents the development of the calibration models, containing the flexible pavement models decomposition and the outcomes for local calibration parameters.

Chapter 5 summarizes the project findings and discusses the conclusions and recommendations for forthcoming research.

2 Literature Review

2.1 Pavement Design Methods Background

Empirical design methods are completely based on the experience of evaluating the pavement performance of specific test sites constructed, such as the AASHO Road Test. The most common empirical method used is the CBR method developed in the 1920´s by the California Department of Highways. It was demonstrated with field testing and refined during World War II for airfields and is still in use. The AASHTO Guide (1993) was developed using performance data from the AASHO Road Test of the 1950´s. The pavement sections of the AASHO Road Test were constructed with uniform mixes, on a single type of subgrade soil and under specific environmental conditions (Central Illinois).

Empirical methods are restricted by the range of the data set that they were developed from. For instance, HMA stiffness varies with pavement temperature, and the CBR method does not take into account HMA stiffness variation due to seasonal climate variation. Similarly, the AASHTO method reveals the same problem in transferring the experience from one climate zone to another.

From 1978 to 1998, the Shell International Petroleum Company edited several editions of The Shell Design Method (Shell Bitumen, 2003). This method uses the elastic theory in multi-layer systems and differentiates up to five different layers, a) bituminous layer (which can be subdivided into two layers, b) two unbounded layers with different moduli, and c) subgrade. The design criteria is fatigue cracking of the asphalt. The method allows for the estimation of the cracking damage at the bottom of the asphalt layers under the cumulative traffic for the entire design period.

Mathematical equations have been used to determine the critical strains in the asphalt layer and subgrade. The pavement design for a given traffic, subgrade strength and average monthly environmental temperatures is performed using appropriate charts, nomographs and diagrams. BISAR, a computer program, calculates the maximum horizontal tensile stains at the bottom of the last asphaltic layer and the maximum compressive strain at the top surface of the subgrade. If the calculated compressive strain at the top of the subgrade soil is excessive, permanent deformation will occur at the top of the subgrade and it will cause rutting at the pavement surface. Likewise, cracking of the asphaltic layer will occur (and it will be visible in the near future), if the calculated horizontal tensile strain is excessive.

This method uses some assumptions: 1) based on materials testing, Poisson's ratio does not vary significantly and a value of 0.35 can be assumed for the asphalt layer, sub-base and subgrade, 2) the subgrade thickness is infinite.

Since 1950, the Asphalt Institute launched various editions of the Thickness Design Manual for flexible pavements. This method seeks to optimize the quantity and type of material used to handle the projected traffic, on a specific subgrade, under particular climate conditions, achieving a balance between structural capacity and life-cycle cost.

The state-of-art in flexible pavement design is the Mechanistic-Empirical (M-E) method. The ME method typically models the pavement as a multilayered elastic system and uses the principles of mechanics to calculate deflections, stresses and strains at any point in the structure in response to an external wheel load. Calculated strains are then compared with observed pavement performance to determine the required pavement thickness (The Asphalt Handbook, 2007).

Limiting the horizontal tensile strain at the bottom of the HMA layer to avoid fatigue cracking and limiting the vertical compressive strain at the top of the subgrade to avoid rutting are the two critical performance criteria.

Since the 1950's, pavement design procedures evolved from empirical based methods to mechanistic-empirical based methods. This evolution occurred mainly because of improved characterization of materials and classification of traffic, enhanced understanding of climate effects on materials, refined pavement performance and more developed computational capabilities.

The Mechanistic-Empirical approach integrates mechanistic and the empirical principles. On one side, the "mechanistic" approach to pavement design is focused on the calculation of the pavement responses (stresses, strains and deflections), while the “empirical” part deals with the estimation of performance from the computed responses.

In traditional design methods, various elements are used to generate the design requirements for the pavement structure. In the ME design, the pavement structural design is initially assumed, along with inputs for traffic and climate.

In flexible pavements, ME design software can estimate the response to the load and environmental stresses produced by the traffic and climate inputs. This results in the calculation of the level of damage sustained by the pavement over time, in terms of pavement distresses and deterioration in ride quality.

The mechanistic procedure requires calibration and verification of the distresses models to ensure accuracy between actual and predicted distresses. The calibration focuses on computing functions relating responses estimated mechanistically and physical distresses.

In rigid pavement analysis, the structural models are more advanced than the distress models. The distress models include major types of distresses such as fatigue cracking, pumping, faulting, joint deterioration for jointed concrete pavement and punchouts for CRCP.

2.2 Ontario’s State of Practice

Since the early 90’s, the Ministry of Transportation Ontario (MTO) has been using a modified version of the AASHTO Guide for Design of Pavement Structures along with attached design software. This is an empirical-based design procedure with limited features (MTO, 2012).

The MEPDG calibration program in Canada started through pooled fund studies, sponsored by the Transportation Association of Canada. The introduction of the AASHTO ME-Design software program has been a slow process mainly due to the complexity of the program and staff training. Other factors have decelerated the implementation, such as, understanding the input requirements, developing accurate models to predict distresses and time and resources needed.

In the last decades, the MTO has been following the evolution of the mechanistic-empirical pavement design methodologies and has taken the lead towards the calibration, validation and adoption of the AASHTO ME-Design procedure. In 2012, the Ministry of Transportation issued a report that presents the default parameters for level 3 analysis in AASHTOWare ME-Design for Ontario conditions and includes province practices. In addition to that, a web application (*iCorridor*) has been launched to allocate and share traffic data essential for mechanistic-empirical design.

Although some calibration work has been undertaken, the transfer functions need to be enhanced in order to improve accuracy. The models must be refined using traffic data from *iCorridor* and materials information from field-testing and project pavement design reports.

2.3 MEPDG Method

The Mechanistic-Empirical method developed by NCHRP (Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, 2004) is a method for the design and evaluation of pavement structures. Deflections, stresses and strains (structural responses) are mechanistically determined based on loading characteristics, environmental conditions and material properties. Distress performance predictions are computed through empirical models using the responses as inputs.

The MEPDG is dependent on empirical models to predict pavement performance distresses from calculated responses and material properties. Calibration of empirical distress models to observed performance and quality of the input information are essential to ensure the accuracy of the models.

The MEPDG deals with two types of empirical models: those that predict the distresses directly (e.g. faulting for rigid pavements, rutting for flexible pavements) and the others that predict damage which is then related to field distresses (e.g. fatigue cracking for flexible pavements, and punchout for rigid pavements).

In contrast to AASHTO'93, the MEPDG is not a straightforward method which determines the thickness of a pavement structure based on a design equation. In ME Design, the design of the pavement structure is initially assumed, in conjunction with traffic and climate inputs. Then, the software computes the response to the load and environmental stresses created by these inputs and estimates the level of damage that the pavement will tolerate over time, with respect to pavement distresses and deterioration in quality of ride. At this stage, one must assess if the predicted performance of the pavement satisfies the criteria for design and if reasonable alternatives to the initial assumptions could improve the predicted performance or lower life cycle costs (Pavement Interactive, 2012).

The design approach comprises two main stages, evaluation and analysis (Figure 2-1). The evaluation stage consists of the preparation of the input values for the analysis. Foundation analysis is a fundamental step in this process, consisting of stiffness determination and assessment of volume change, frost-heave, thaw weakening and drainage issues. For rehabilitation projects, subgrade analysis is also included, as well as the investigation of the distress types and extent occurring in the existing pavements and the distresses underlying causes. Deflection testing and backcalculation procedures are used to evaluate the overall strength/stiffness of the existing pavements. Traffic loading and pavement material characterization data are developed in this stage, also. The Enhanced Integrated Climate Model (EICM) is used to model environmental conditions within each pavement layer and the subgrade.

This model retains hourly climatic data from weather stations (temperature, precipitation, wind speed, solar radiation and cloud cover). The design criteria boundaries for acceptable pavement performance at the end of the design period (acceptable levels of rutting, roughness, fatigue cracking and thermal cracking) and the selection of reliability levels for each distress considered in the design are also defined in this stage.

The next stage, structure/performance analysis, is an iterative process. Selection of an initial trial design is the first step. Initial estimates of layer thickness, initial smoothness, pavement materials characteristics and other inputs are required for the trial design. The trial design section is analyzed incrementally throughout the design period by applying the pavement response and distress models. The accumulated damage, distress amount and smoothness over time are the outputs from analysis.

The predicted performance evaluation of the trial design is compared with the specified reliability level and performance criteria. If the trial design does not meet the performance criteria, the material selection and/or thickness must be adjusted and process repeated until the design is acceptable.

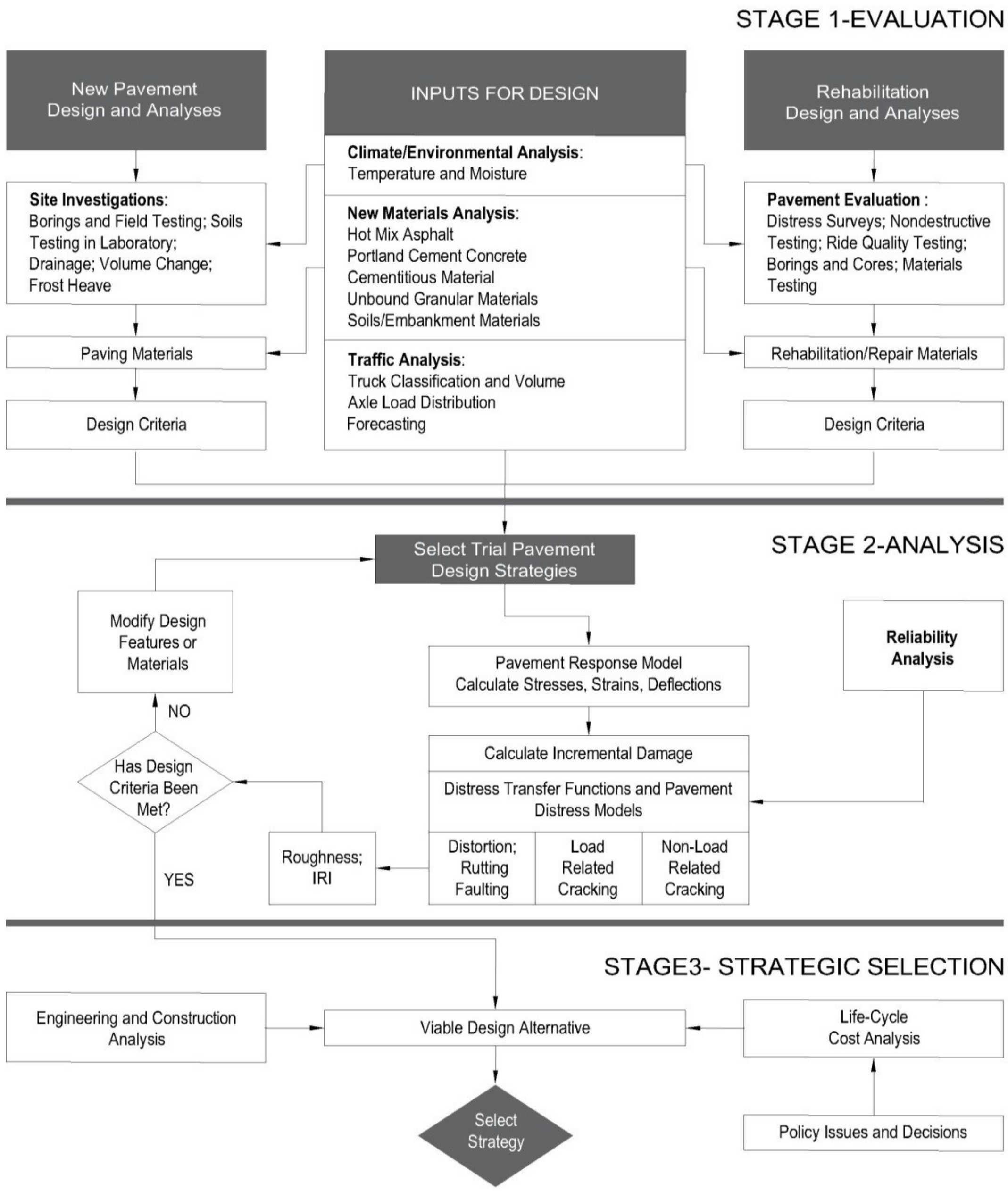

Figure 2-1: Conceptual Flow Chart of the Three-Stage Design/Analysis Process for the MEPDG

(AASHTO, 2008)

2.3.1 Hierarchical Design Input Levels

The hierarchical approach to design inputs allows greater flexibility in obtaining the design inputs for a design project based on the resources available and required accuracy for the project. The hierarchical approach encompasses three levels of inputs and is used for the parameters related to traffic, materials and climate.

Level 1 input parameters are directly measured from project specific sites. This level reflects the highest knowledge and level of accuracy of the parameter. Level 1 material input involves laboratory, field testing and data collection (e.g., dynamic modulus testing of hot-mix asphalt concrete, axle load spectra data collections, nondestructive deflection testing) to determine the input values and therefore require more time and resources than other levels. This level of accuracy should be used for pavement designs with very specific site characteristics, materials or traffic conditions that are outside of the range used to develop the correlations and default values established for levels 2 and 3.

Level 2 input parameters offer an intermediate level of accuracy. This input level commonly represents an agency database and/or regional values and are derived from a limited testing program and estimated from regression equations and correlations. The input values are determined from data from other similar projects and the parameters are less expensive and easier to obtain. Some examples would be determining asphalt concrete dynamic modulus from binder, aggregate and mix properties, using site-specific traffic classification data and traffic volume combined with agency-specific axle load spectra.

Level 3 input parameters show the lowest level of accuracy and may be selected for design where the consequences of an earlier failure are negligible (e.g., roads with low traffic volume). Inputs are typically user-selected parameters and are estimated and based on national/regional default values.

The input matrix can comprise a mix of levels. An important note is that regardless of the design level input, the computational algorithm for damage is the same. The distresses and smoothness prediction is performed using the same procedures and models, regardless of the input level.

2.3.2 Pavement Performance Models for Flexible Pavements

The MEPDG relies on three models to predict pavement responses (strains, stresses and displacements). Finite Element Model (FEM) and Multi-Layer Elastic Theory (MLET) are utilized to compute responses due to traffic loading. To predict temperature and moisture (climatic factors) through the pavement structure, the Enhanced Integrated Climate Model (EICM) is used. FEM is selected when non-linear behavior of unbound material is intended, otherwise MLET is used to perform load-related analysis. The structural responses due to traffic loading are estimated at critical locations based on maximum damage. The response is evaluated at each point, at varying depths, and afterward the more severe is used to predict pavement distress performance. The depth at which the calculations are performed varies according with the distress type:

For rutting, calculations are performed at mid-depth of each layer/sublayer, top of subgrade and 6 inches below the top of subgrade. For the fatigue cracking, calculations are performed at the surface (top-down cracking), 0.5 inches from the surface (top-down cracking) and at the bottom of the asphalt concrete layer (bottom-up cracking). In order to represent better the properties varying in the vertical direction, each pavement layer is subdivided into thinner sublayers.

The Finite Element Model (FEM) performs structural analysis of a multi-layer pavement section with material properties that differ both vertically and horizontally throughout the section. The FEM is implemented in MEPDG considering the following characteristics: linear elastic behavior for asphalt concrete, nonlinear elastic behavior with tension cut-off for unbound materials and fully bonded, full slip and intermediate interface conditions between layers.

The MLET, in the MEPDG, is implemented in a modified version of the JULEA algorithm (NCHRP, 2004). Single wheels can be combined spatially into multi-wheel axles to simulate a number of diverse axle configurations, using the principle of superposition. MLET requires a small set of input variables (layer thickness, modulus of elasticity and Poisson's ratio) for each layer, tire pressure and tire contact area which facilitates the use and implementation.

Mechanistic EICM is a one-dimensional coupled heat and moisture flow model to predict the changes in behavior and characteristics of pavement and unbound materials caused by environmental conditions that occur throughout the service life. With respect to flexible pavements, three major environmental effects are factored:

- Asphalt concrete temperature variations. The dynamic modulus of asphalt concrete mixtures is very sensitive to temperature. Temperature distributions are predicted and afterward used to define the stiffness of the mixture through the sublayers, also are employed as inputs for the thermal cracking prediction model.
- Subgrade and unbound materials moisture variation. Based on predicted moisture content, an adjustment factor is defined to rectify the resilient modulus.
- Freeze-thaw cycle for subgrade and unbound materials. For unbound materials located within the freezing zone, the resilient modulus is greater during freezing periods and lower during thawing periods. EICM defines the freezing zone and predicts the formation of ice lenses.

2.3.3 Design Criteria and Reliability Level

Pavement design inputs are subject to major uncertainties. The variability in design inputs is taking into account in the reliability methodology incorporated into MEPDG. For each pavement distress type, a reliability level is required. Performance indicators are compared individually against the design criteria default values or boundary limits.

Reliability is defined as the probability that the predicted pavement distresses will be less than the critical level of distress through the design period.

$$Reliability = Probability\ (Predicted\ Distress < Critical\ Distress) \qquad \text{Equation (1)}$$

The fundamental outcomes in MEPDG are the individual distresses, considered as the random variables of interest. The error for all pavement distresses is assumed to be normally distributed with a mean predicted value and an associated standard deviation. The desired reliability level for each distress type is given by the general formulation:

$$D_{Reliability} = D_{Mean} + S_d * Z_r \qquad \text{Equation (2)}$$

where:

$D_{Reliability}$= distress prediction at specified reliability level

D_{Mean}= distress prediction using mean inputs and 50 percent reliability

S_d= standard deviation for the distress prediction using mean inputs

Z_r= standardized normal deviate from the normal distribution at specified reliability level

2.4 Performance Prediction Models for Flexible Pavement

2.4.1 Bottom-Up Fatigue or Alligator Cracking Model

Bottom-Up Fatigue or Alligator Cracking is produced by repeated applications of tensile strain, resulting from wheel loading. Once developed, the cracks propagate upwards from the bottom of the HMA layer to the top. Bottom-Up fatigue cracking is usually a loading failure but other factors can contribute, such as, are inadequate structural support (loss of base, subbase or subgrade) and poor drainage or spring thaw resulting in a less rigid base. The tensile strain magnitude at the bottom of the asphalt concrete increases when soft layers are placed directly below the asphalt layer and consequently the probability of fatigue cracking increases. This distress is characterized by a series of interconnecting cracks in the asphalt layer. Tensile strains are higher at the bottom of the HMA layer, in thin pavement structures, from where cracks initiate and progress upwards in one or several longitudinal cracks. Agencies report this type of cracking based on severity, it is measured as a percentage of the total area and classified as low, medium or high level.

Alligator cracking is calculated by first predicting damage. Then, damage is converted into cracked area. The Asphalt Institute method was adopted to MEPDG and calibrated based on LTPP data. The number of axle load repetitions to failure for a certain load magnitude is calculated as follows:

$$N_{f-HMA} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{\beta_{f2}k_{f2}}(E_{HMA})^{\beta_{f3}k_{f3}} \quad \text{Equation (3)}$$

where:

N_{f-HMA} = number of allowable axle load repetitions for a flexible pavement

k_{f1}, k_{f2}, k_{f3} = regression parameters from global calibration

E_{HMA} = asphalt concrete stiffness (psi) / dynamic modulus of the HMA

ε_t = tensile strain at critical location within asphalt layer

$\beta_{f1}, \beta_{f2}, \beta_{f3}$ = field calibration coefficients

C = adjustment factor laboratory-field

C_H = thickness correction factor, dependent on type of cracking

The following values resulted from the global calibration of the model using the LTPP database: $k_{f1} = 0.007566, k_{f2} = -3.9492, k_{f3} = -1.281$. For this calibration, the field calibration coefficients $\beta_{f1}, \beta_{f2}, \beta_{f3}$ were assumed to be 1.

The thickness correction factor for bottom-up or alligator cracking is computed as follows:

$$C_H = \frac{1}{0.000398 + \frac{0.003602}{1+e^{(11.02-3.49*h_{AC})}}} \quad \text{Equation (4)}$$

where h_{AC} = total AC thickness. The adjustment factor laboratory-field is determined by:

$$C = 10^M \quad \text{Equation (5)}$$

$$M = 4.84\left(\frac{V_{beff}}{V_a + V_{beff}} - 0.69\right) \quad \text{Equation (6)}$$

where:

V_a = air voids in the HMA mixture (%)

V_{beff} = effective binder content by volume (%)

The incremental damage arising from a given load is then determined from the number of repetitions applying Miner's Law:

$$D = \sum_{i=1}^{T} \frac{n_i}{N_{f-HMA}}$$

Equation (7)

where:

D= damage (%)

T= total number of periods

n_i = actual number of axle load repetitions for period i

N_{f-HMA}= number of allowable axle load repetitions for a flexible pavement for period i

Finally, from the total damage (D), the following equation is used to predict the amount of alligator cracking on an area basis:

$$FC_{Bottom} = \left(\frac{C_4}{1+e^{(C_1C_1'+C_2C_2'*\log_{10}(D*100))}}\right) * \left(\frac{1}{60}\right) \qquad \text{Equation (8)}$$

$$C_2' = -2.40874 - 39.748(1 + h_{AC})^{-2.856} \qquad \text{Equation (9)}$$

$$C_1' = -2C_2' \qquad \text{Equation (10)}$$

where:

FC_{Bottom}= fatigue cracking "alligator" (% of total lane area)

D= damage (%)

C_1, C_2, C_4 = regression coefficients

h_{AC} = total AC thickness, mm

The following values for the regression constants resulted from the global calibration of the model using the LTPP database: $C_1 = 1, C_2 = 1$ and $C_4 = 6000$. Total area of the lane is deemed to be 12ft*500ft=6,000ft^2. The $\frac{1}{60}$ in the equation is used to convert square feet to percentage of alligator cracking.

2.4.2 Top-Down or Longitudinal Cracking Model

Top-down cracking develops at the pavement surface and propagates downward. In flexible pavements, longitudinal cracking development is conceptually identical to “alligator” fatigue cracking. Tensile strains at the top of the asphalt concrete surface layer caused by traffic loading generate the formation of cracks. Longitudinal cracking generally develops parallel to the pavement centerline and is usually produced by fatigue failure due to repeated traffic loading, however, other factors could contribute, for instance, poor construction paving joint, shrinkage of the asphalt, temperature variations and reflection from underlying layers. Agencies report this type of cracking based on severity. It is measured in meters per kilometer and it is classified in terms of low, moderate or high level. For top-down fatigue cracking, the damage is converted into longitudinal fatigue cracking using the following equation:

$$FC_{Top} = 10.56\left(\frac{C_4}{1+e^{(C_1 - C_2 * \log_{10}(D)}}\right) \quad \text{Equation (11)}$$

where:

FC_{Top}= longitudinal cracking (ft/mile) or (m/Km)

D= damage (%)

C_1, C_2, C_4 = calibration coefficients

The following values for the calibration coefficients resulted from the global calibration of the model using the LTPP database: $C_1 = 7.0, C_2 = 3.5$ and $C_4 = 1000$. The length of the LTPP sections is 500 feet. The maximum length of linear cracking that can result from two wheel paths of a 500 feet section is 1000 feet (2*500ft). A factor of 10.56 is used to convert the longitudinal cracking from feet per 500 feet into feet per mile.

2.4.3 Transverse Thermal Cracking Model

Thermal cracking is caused by cool/heat cycles that occurs in the asphalt concrete. The surface of the pavement cools down promptly and more intensively than the pavement core structure, which causes thermal cracking at the surface of pavement (low temperatures prevent friction at the bottom of the HMA surface). Thermal cracking generally manifests and extends in the transverse direction across the full width of the pavement. Cracks initiate at the surface of the pavement when the tensile stress at the bottom of the HMA layer exceed its tensile strength. Moisture in the pavement, daily temperature cycles and cold weather are other conditions that also contribute to the development of thermal cracking. Thermal cracking is reported based on severity, measured in meters per kilometer and is classified in terms of low, medium or high level.

The Paris law is used to compute the crack propagation for a given thermal cooling cycle that stimulate a crack to propagate, as follows:

$$\Delta C = A * \Delta K^{n} \qquad \text{Equation (12)}$$

where:

ΔC= change in crack depth for each thermal cycle

ΔK= change in stress intensity factor during thermal cycle

A, n= fracture parameters for the HMA mixture

$$n = 0.8\left(1 + \frac{1}{m}\right) \qquad \text{Equation (13)}$$

$$A = 10^{k_t \beta_t [4.389 - 2.52 * \log(E_{HMA} * \sigma_m * n)]} \qquad \text{Equation (14)}$$

where:

k_t= coefficient estimated through global calibration for each input level

(level 1=1.5, level 2=0.5 and level 3=1.5)

β_t= local calibration parameter

m= the m-value derived from the indirect tensile creep compliance curve measured in the laboratory

E_{HMA}= HMA indirect tensile modulus, psi

σ_m= HMA tensile strength, psi

The length of thermal cracking is predicted by relating the crack depth to the percentage of cracking in the pavement:

$$TC = \beta_1 * N\left(\frac{\log(C_d/h_{ac})}{\sigma}\right) \qquad \text{Equation (15)}$$

where:

TC= predicted thermal cracking, ft/mi

β_1= regression coefficient determined through global field calibration ($\beta_1 = 400$)

N= standard normal distribution evaluated at [z]

C_d= crack depth, in

h_{ac}= thickness of the asphalt concrete, in

σ= standard deviation of the log of the depth of cracks in the pavement (for global calibration $\sigma = 0.769$), in

2.4.4 Permanent Deformation (Rutting) Model

Permanent Deformation (Rutting) is defined as a depression in the wheel path (Figure 2-2). Rutting is a load-associated distress generated by cumulative load applications when the HMA has the lowest stiffness, i.e., at moderate and high temperatures. Rutting is commonly categorized into 3 stages. Primary deformation emerges early in the service life and is associated with mixture design. In the secondary stage, deformation increments are lower at a constant rate and the mixture is experiencing plastic shear deformations. Shear failure occurs in the tertiary stage and the rupture of the mixture takes place. Before this stage is achieved in pavements in operation, preventive maintenance and rehabilitation are required.

Empirical models are used to predict rutting in each layer throughout the analysis period, but only primary and secondary stages are outlined. The model for HMA materials is an improved version of Leahy's model (1989), modified by Ayres (1997) and Kaloush (2001). For unbound materials, the model is based on Tseng and Lytton's model (1989) modified by Ayres and then by El-Basyouny and Witczak (NCHRP, 2004).

This distress is not based on an incremental approach and instead measured in absolute terms. The empirical models included in MEPDG must be calibrated accounting for local conditions, given that temperature and moisture content are embedded in the computation of permanent deformation by their effect on dynamic modulus for asphalt concrete and resilient modulus for granular layers.

The model for computing total permanent deformation uses the plastic vertical strain under specific pavement conditions for the total number of repeated loads within that condition (AASHTO, 2008).

The total rutting is the summation of the rut depths from all layers, as follows:

$$\Delta p_{total} = \Delta p_{AC} + \Delta p_{soil}$$ Equation (16)

2.4.4.1 Asphalt Concrete Model

The AC layer is subdivided into sublayers and the total estimated rut depth for the layer is computed as follows:

$$\Delta p_{AC} = \sum_{i=1}^{N} \mathcal{E}_{P(HMA)} * h_{HMA} = \beta_{1r} k_z \varepsilon_{r(HMA)} 10^{k_{1r}} N^{k_{2r}\beta_{2r}} T^{k_{3r}\beta_{3r}}$$

Equation (17)

where:

Δp_{AC}= rut depth at the asphalt concrete layer

N= number of sublayers

$\mathcal{E}_{P(HMA)}$ = vertical plastic strain at mid-thickness of layer i

h_{HMA}= thickness of sublayer i

$\varepsilon_{r(HMA)}$ = computed vertical resilient or elastic strain at mid-thickness of sublayer i

T= mix or pavement temperature, °F

N= number of repetitions for a given load

k_z = depth correction factor

k_{1r}, k_{2r}, k_{3r} = global regression coefficients derived from laboratory testing

$\beta_{1r}, \beta_{2r}, \beta_{3r}$ = field calibration coefficients (all set to 1.0 for global calibration)

The depth correction factor, that refine the calculated plastic strain for confining pressure at varying depths, is a function of layer thickness and depth to mid-thickness of sublayer (computational point). This factor is given by:

$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth} \quad \text{Equation (18)}$$

$$C_1 = -0.1039 * (h_{AC})^2 + 2.4868 * h_{AC} - 17.342 \quad \text{Equation (19)}$$

$$C_2 = 0.0172 * (h_{AC})^2 - 1.7331 * h_{AC} + 27.428 \quad \text{Equation (20)}$$

where:

$depth$= depth to the point of strain calculation (below surface), in

h_{AC}= thickness of the asphalt layer, mm

The regression coefficients using the LTPP database are: $k_{1r} = -3.35412, k_{2r} = 0.4791, k_{3r} = 1.5606$.

2.4.4.2 Unbound Materials

All unbound granular materials are divided into sublayers in MEPDG. Therefore, the total rutting for each layer is the summation of the permanent deformation of all sublayers. The permanent deformation at any certain sublayer is calculated by the following equation:

$$\Delta p_{soil} = \beta_{s1} k_{s1} \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)^{\beta}} \varepsilon_v h_{soil} \quad \text{Equation (21)}$$

where:

Δp_{soil} = permanent deformation for sublayer i

β_{s1} = field calibration coefficient for unbound granular base and/or subgrade material

k_{s1} = global calibration regression coefficient, $k_{s1} = 1.673$ for granular materials and 1.35 for fine-grained materials

ε_0 = intercept determined repeated load permanent deformation tests (in laboratory)

ε_r = resilient strain imposed in laboratory test to obtain material properties

ε_v = computed vertical resilient or elastic strain at mid-thickness of sublayer i for a given load

N= number of repetitions for a given load

h_i= thickness of unbound sublayer i

For the global calibration procedure, β_{1GB} and β_{1SG} were set to 1.0. The model above is used to predict both granular base and subgrade permanent deformation. It is a modified model based on Tseng and Lytton's model (1989). The properties of the materials $\frac{\varepsilon_0}{\varepsilon_r}, \beta, \rho$ are derived from other properties under the following mathematical relationships:

$$log_\beta = -0.61119 - 0.017638 * W_c \qquad \text{Equation (22)}$$

$$\rho = 10^9 \left[\frac{C_0}{\left(1-(10^9)^\beta\right)}\right]^{\frac{1}{\beta}} \qquad \text{Equation (23)}$$

$$C_0 = ln\left[\frac{(a_1 M_r^{b_1})}{(a_9 M_r^{b_9})}\right] = 0.0075 \qquad \text{Equation (24)}$$

where:

W_c = water content (%)

M_r = resilient modulus of the unbounded layer/sublayer, *psi*

$a_{1,9}$ = regression coefficients; a_1=0.15 and a_9=20.0

$b_{1,9}$ = regression coefficients; b_1=0.0 and b_9=0.0

2.4.5 International Roughness Index (IRI) Model

Pavement Roughness is generally defined as a manifestation of irregularities in the pavement surface that negatively affects the ride quality. Roughness is recognized as the most representative distress of the overall serviceability of a roadway. Fatigue and thermal cracking and permanent deformation are acknowledged as the most prevailing distresses affecting roughness. Other influential factors are environmental conditions and supporting base type. International Roughness Index (IRI) is a roughness index obtained from measured longitudinal roadway profiles. LTTP data was used in the calibration process to develop three models for flexible pavements with distinct base layers: conventional granular base, cement-stabilized base and asphalt-treated base. All roughness models have similar form:

$$IRI = IRI_0 + 0.0150(SF) + 0.400(FC_{Total}) + 0.0080(TC) + 40(RD)$$

Equation (25)

where:

IRI_0 = initial IRI after construction, in/mi

SF = site factor

FC_{Total} = area of combined fatigue cracking (alligator, longitudinal and reflection cracking in the wheel path), in % of total lane area. All load related cracks are combined on an area basis (to convert length into area basis, length of cracks is multiplied by 1 foot)

TC = length of transverse cracking (including the reflection of transverse cracking in existing HMA pavements), ft/mi

RD = average rut depth, in

The site factor *(SF)* in the IRI model is computed as follows:

$$SF = Age[0.02003(PI + 1) + 0.00794(Precip + 1) + 0.000636(FI + 1)]$$

Equation (26)

where:

Age = age of the pavement in years

PI = plasticity index of the soil, %

$Precip$ = average annual precipitation or rainfall, in

FI = average annual freezing index in F^O

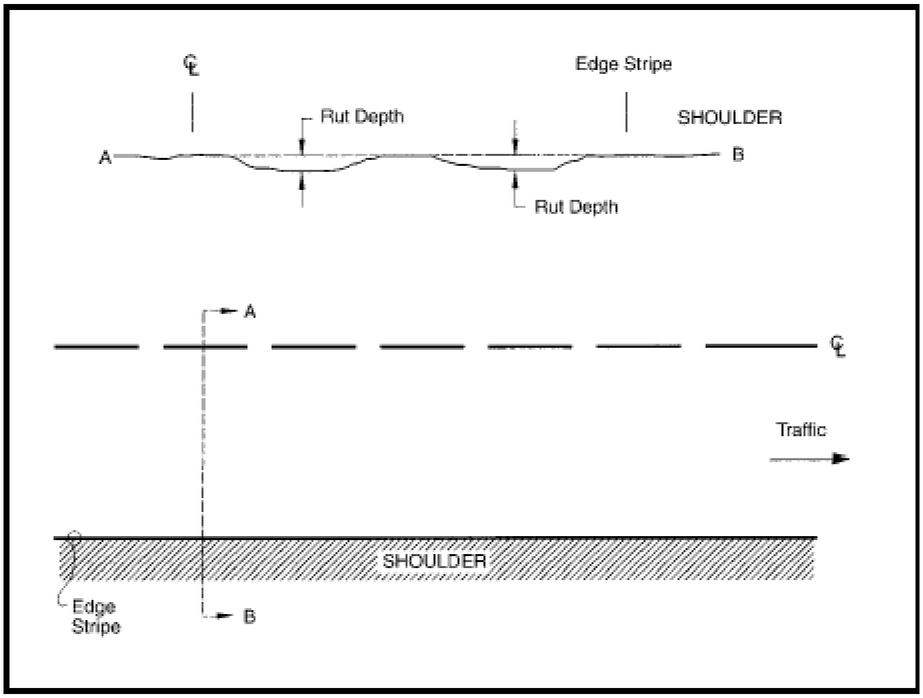

Figure 2-2 Rutting (FHWA, 2015)

3 Database Assembly

3.1 Local Calibration and Validation Plan

In any mechanistical-empirical procedure analysis, pavement distress prediction models are essential components. Calibration processes and subsequent validation with independent sets of data are required to improve the accuracy of the performance prediction models. The verification of an acceptable correlation between predicted and observed distresses increases the confidence level of a given transfer function.

The term calibration is attributed to the mathematical process through which the residual (generally termed as total error) or difference between predicted and observed values is reduced to a minimum. The term validation is provided to the process to legitimize that the calibrated model can return accurate and robust predictions for other cases beyond those used for the calibration model. Similar bias and precision statistics for the calibration model and validation model are required in order to assess the success of the validation procedure.

This chapter presents a plan for local calibration and validation of MEPDG for Ontario conditions. The performance models considered in this project were calibrated on a local level to observed field performance over a representative sample of pavement sections in Ontario. This plan is based on the directives outlined in the Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide (AASHTO, 2010).

3.2 Selection of Hierarchical Input Level

The first stage in the local calibration process is the selection of the hierarchical input level for each input parameter for pavement analysis and design. For the purpose of this research, the decision on each input parameter level was likely to be influenced by the availability of field and laboratory testing data and or results, consistency with current practices, material and construction specifications, traffic data availability and the recommendations offered by AASHTO on the MEPDG Manual of Practice.

The input level affects the final standard error for each distress prediction model and consequently material requirements, field investigation and construction costs. Table 3-1 to Table 3-5 provide the input levels for traffic, AC, granular, subgrade and climate parameters.

Table 3-1 - Input Level for Traffic Parameters

Parameter	Input Parameter	Input Level	Value	Data Source
AADTT	Two-way AADTT	Level 1	(a)	
	Number of lanes	Level 1	(d)	
	Percent of trucks in design direction	Level 3	(c)	
	Percent of trucks in design lane	Level 2	(b)	
	Operational speed	Level 1	(d)	
Axle Configuration	Average axle width (m)	Level 3	2.59	(c)
	Dual tire spacing (mm)	Level 3	305	(c)
	Tire pressure (kPa)	Level 3	827.4	(c)
	Tandem axle spacing (m)	Level 2	1.45	(b)
	Tridem axle spacing (m)	Level 2	1.68	(b)
	Quad axle spacing (m)	Level 2	1.32	(b)
Lateral Traffic Wander	Mean wheel location (mm)	Level 3	460	(c)
	Traffic wander standard deviation (mm)	Level 3	254	(c)
	Design lane width (m)	Level 1	3.75	(d)
Wheelbase	Short trucks - Average axle spacing (m)	Level 2	5.1	(b)
	Medium trucks - Average axle spacing (m)	Level 2	4.6	(b)
	Long trucks - Average axle spacing (m)	Level 2	4.7	(b)
	Percent short trucks	Level 3	33	(c)
	Percent medium trucks	Level 3	33	(c)
	Percent long trucks	Level 3	34	(c)
Traffic Volume Adjustment	Vehicle class distribution (Truck Traffic Classification - TTC)	Level 1	(a)	
	Traffic Growth Factor	Level 1	(a)	
	Monthly and Hourly adjustment	Level 3	(c)	
	Axles per truck	Level 1	(a)	
Axle Load Distribution	Axle distribution (Single, Tandem, Tridem, Quad)	Level 1	(a)	

Legend of data sources:

(a) *iCorridor* – Ministry of Transportation Ontario (MTO) web application.

(b) Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report.

(c) AASHTO.

(d) Project Specific Pavement Design Reports provided by MTO Offices - Provincial Highways Division/Geotechnical Engineering Section and Pavements and Foundations Section/Materials Engineering and Research.

(e) Computed using PI and gradation from Project Specific Pavement Design Reports.

Table 3-2 - Input Level for Asphalt Concrete Material Properties Parameters

Parameter	Input Parameter	Input Level	Value	Data Source
Asphalt Layer	Thickness	Level 1	(d)	
Mixture Volumetric	Unit Weight (Kg/m^3)	Level 2	(b)	
	Effective Binder Content by Volume (%)	Level 2	(b)	
	Air Voids (%)	Level 2	(b)	
Poisson's Ratio	Poisson's Ratio	Level 3	(b)	
Mechanical Properties	Dynamic Modulus	Level 3	(c)	
	Aggregate Gradation	Level 2	(b)	
	G* Predictive Model (Use Viscosity based model (Nationally calibrated)	Level 3	(c)	
	Reference Temperature (^{0}C)	Level 3	21.1	(c)
	Asphalt Binder[1]	Level 2	(b)	
	Indirect Tensile Strength at – 10^{0}C (MPa)	Level 3	Calculated	
	Creep Compliance (1/GPa)	Level 3	(c)	
Thermal	Thermal Conductivity (W/m-Kelvin)	Level 3	1.16	(c)
	Heat Capacity (J/Kg-Kelvin)	Level 3	963	(c)
	Thermal Contraction	Level 3	Calculated	

[1] For existing HMA assumed Pen Grade 85-100 in South Ontario, Pen Grade 200-300 in North Ontario.

Table 3-3 - Input Level for Climate Parameters

Parameter	Input Parameter	Input Level	Value	Data Source
Climate	Longitude, Latitude and Elevation (m)	Level 1	(d)	
	Depth of ground water table (m)	Level 1, 2	(d)[2]	

Table 3-4 - Input Level for Unbound Granular Material Properties Parameters

Parameter	Input Parameter	Input Level	Value	Data Source
Material	Material Type	Level 1	(d)	
Unbound	Thickness	Level 1	(d)	
	Poisson's Ratio	Level 3	(b)	
	Coefficient of lateral earth pressure (k_0)	Level 3	0.5	(b)
Modulus	Resilient Modulus	Level 2	(e)	
Sieve	Aggregate gradation	Level 1	(d)	
	Liquid Limit	Level 1	(d)	
	Plasticity Index	Level 1	(d)	
	Layer Compacted	Yes		
	Maximum Dry Unit Weight (Kg/m^3)	Level 1	Calculated	
	Saturated Hydraulic Conductivity (m/hr)	Level 1	Calculated	
	Specific Gravity of Solids	Level 1	Calculated	
	Optimum Gravimetric Water Content (T)	Level 1	Calculated	

[2] In some cases, depth of ground water table was not indicated in the Pavement Design Report and hence, the default value of 6.1m was used as recommended in Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report.

3.3 Performance Criteria Thresholds

AASHTOWare Pavement ME Design requires that performance criteria threshold values have to be defined in order to assess if a given pavement design passes or fails. These values are also needed for the computation of the sample size.

IRI is an appropriate indicator of pavement performance. In AASHTOWare Pavement ME Design, the initial IRI represents the baseline (value of the newly built pavement) and the terminal IRI represents the threshold value of IRI for a specific reliability. Design criteria represent the maximum accepted distresses in the pavement before proceeding with resurfacing or rehabilitation strategies and are normally established by the highway agency. Table 3-6 provides the input values defined in the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report for freeways and Ontario conditions:

Table 3-5 - Input Level for Subgrade Material Properties Parameters

Parameter	Input Parameter	Input Level	Value	Data Source
Material	Material Type	Level 1	(d)	
Unbound	Thickness	Semi-infinite		
	Poisson's Ratio	Level 2	(b)	
	Coefficient of lateral earth pressure (k_0)	Level 3	0.5	(b)
Modulus	Resilient Modulus	Level 3	(c)	
Sieve	Aggregate gradation	Level 1	(d)	
	Liquid Limit	Level 1	(d)	
	Plasticity Index	Level 1	(d)	
	Layer Compacted	Yes		
	Maximum Dry Unit Weight (Kg/m^3)	Level 1	Calculated	
	Saturated Hydraulic Conductivity (m/hr)	Level 1	Calculated	
	Specific Gravity of Solids	Level 1	Calculated	
	Optimum Gravimetric Water Content (T)	Level 1	Calculated	

Table 3-6 – Design Criteria Values

Performance Indicator		Threshold Values	
		SI Units	U.S. Units
IRI (Smoothness)	Initial	0.8 m/km	50 in/mi
	Terminal	1.9 m/km	120 in/mi
Rutting	Total	19 mm	0.75 in
	AC only	6 mm	0.24 in
Alligator Cracking (Bottom-up)		10 % lane area	
Longitudinal Cracking (Top-down)		380 m/km	2000 ft/mi
Transverse or Thermal Cracking		190 m/km	1000 ft/mi

3.4 Sample Size Computation

Both bias and precision are influenced by the sample size (number of pavement segments) used in the calibration process. The bias is defined as the average of residual errors, therefore, to correlate the sample size and the bias, the confidence interval on the mean can be used. Model error, confidence level and threshold value at a typical reliability level must be defined in order to determine the minimum number of pavement segments. The number of pavement segments required can be computed using the following expression:

$$N = \left(\frac{Z_{\alpha/2} * \sigma}{e_t}\right)^2 \quad \text{Equation (27)}$$

$$e_t = Z_{\alpha/2} * S_{ee} \quad \text{Equation (28)}$$

$$S_{ee} = \sqrt{\frac{\sum(Y - Y')^2}{n}} \quad \text{Equation (29)}$$

where:

N = number of segments required for each distress and IRI prediction model validation and local calibration

$Z_{\alpha/2}$ = 1.645 for a 95% confidence interval

σ = performance indicator threshold (varies with type of distress)

e_t = tolerate bias at 95% design reliability

S_{ee} = standard error of estimate

$(Y - Y')^2$ = squared error

n = number of observations

The Standard Error of Estimate (SEE) for IRI, rutting and alligator cracking models is based on recommendations from the MEPDG Local Calibration Guide. The acceptable bias is agency dependent. Then, the sample size computation based on the mean or bias is summarized in Table 3-7. Assumptions used in the estimations are also discerned in the table. The threshold values are based on the MEPDG Manual of Practice recommendations and standard error of estimate for each distress model and IRI are in line with MEPDG Local Calibration Guide. The confidence level selected was 95%. Regarding the design reliability level, Ontario recommends 95% for urban or rural freeways (Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report, 2012).

Calculations show that a minimum of 90 projects are required for IRI model calibration, which is a large value and therefore not feasible. Recognizing that IRI is a function of other distresses, one can assume that when rutting and cracking models are calibrated, the IRI model should deliver plausible predictions. Then, in this research, 16 segments are taken for calibration purposes.

Table 3-7 – Minimum Number of Segments Needed for Local Calibration and Validation

Performance Indicator		Threshold at 95% Reliability (σ)	Standard Error of Estimate (S_{ee})	***N***
Terminal IRI (m/km)		1.9	0.2	90
Rutting	Total (mm)	19	4.75	16
	AC only (mm)	6	1.5	16
Alligator Cracking (Bottom-up) (%)		10	3.5	8

3.5 Selection of Pavement Segments

The selection of pavement segments or sites for local calibration and validation of MEPDG should take into account primary parameters, such as, diversity of pavement structures, subgrade soil type and materials types. This selection process should also consider secondary parameters, such as climate and traffic.

The segments or projects should be selected to include a range of distress values that are of similar ages (historical distress data should represent nearly 10 year-periods) and comprehend segments exhibiting higher level of distress and segments showing good performance (low level of distress over long periods of operation). A minimum of 3 condition survey results are desirable for each project in order to assess the incremental increase in distress/IRI over time.

In accordance with AASHTO, segments that have a detailed history before and after overlay are of great value. Segments in which unconventional design or mixtures were used should be included in the calibration and validation process. In contrast, road segments in where complex technology or materials were experimented should not be used in the calibration and validation process.

Initially, the MTO has provided 26 pavement design reports of flexible pavement from 8 Provincial Highways, all of them located in the Central Region of Ontario. Each of the pavement design reports was divided by sections and consequently a total of 90 segments were tabulated. These reports were then analyzed in detail to confirm that pavement structure information, materials properties, traffic data and performance data was available.

Since this research does not consider rehabilitation and deals only with new pavements, sixteen (16) representative pavement sections were selected from widening and reconfiguration highway projects, combining a variety of pavement structures, subgrade soils, materials properties and traffic conditions. These sections, with pavement ages ranging from 3 to 17 years, also include a variety of distress levels.

Table 3-8 – List of Pavement Sections Selected

#	Project	Pavement Construction date	Performance Collection date	Design Period (Years)
1	Hwy 6 from Hwy 5 to Concession 6 W	05/02/99	10/01/14	15
2	Hwy 6 from Hwy 5 to Concession 6 W	05/02/99	10/01/14	15
7	401 at Newcastle EB (L3)	08/02/05	09/17/15	10
11	QEW from Brant St to Burloak Dr (L3)	08/16/11	09/25/15	4
18	QEW Winston Churchill Blvd to Trafalgar Rd (WBL4)	09/01/11	09/25/15	4
24	400 from Hwy 11 to Hwy 93 (Simcoe County)	08/02/09	07/06/15	6
27	403 Fiddler's Green Road Interchange	07/02/07	10/02/15	8
28	403 from Aberdeen Avenue to York Blvd (EB)	10/01/97	10/01/14	17
38	401 from Trafalgar Rd to Regional Rd 25 (EB)	09/15/97	10/01/14	17
39	401 from Trafalgar Rd to Regional Rd 25 (WB)	09/15/97	10/01/14	17
41	401 from Credit River to Trafalgar Rd (EB)	07/02/09	05/19/15	6
43	401 from Credit River to Trafalgar Rd (WB)	07/02/09	11/10/15	6
50	QEW from Third Line to Burloak Drive (EB)	09/01/09	09/25/15	6
60	Hwy 12 from County Rd 23 to City of Orillia	09/15/07	09/10/15	8
70	400 from King Road to South Canal Bridge (SB)	06/15/12	08/14/15	3
81	Hwy 12 Rama Road to Simcoe/Durham Boundary	09/30/10	09/10/15	5

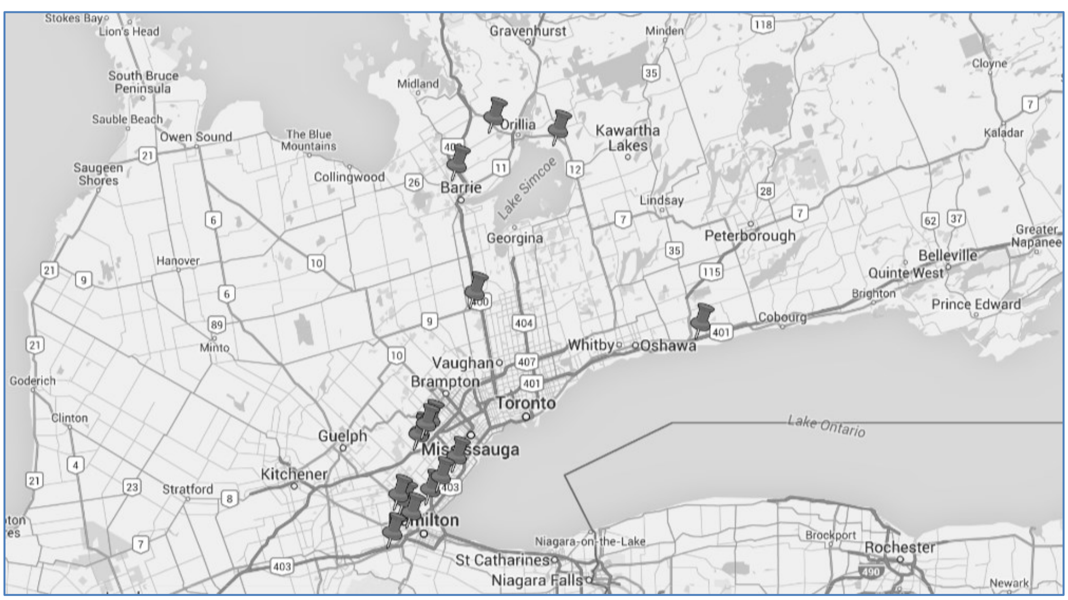

Figure 3-1. Map showing Pavement Sections Selected along Central Region of Ontario

3.6 Extract and Evaluate Distress and Project Data

3.6.1 Extraction, Assembly and Conversion of Measured Data

This step covers extracting and reviewing the distress data, comparing extracted data to the design criteria values, reviewing the data to identify outliers and extracting, reviewing and assembling all MEPDG inputs required for research distress/IRI predictions.

Firstly, it is recommended to review the data and check if distresses are measured in consistency with MEPDG predicted values. As shown in Table 3-9, MTO does not measure the rutting on the AC layer nor the total cracking (reflective + alligator). Since this research is focused only on rutting and alligator cracking, the lack of total cracking (reflective + alligator) is irrelevant.

Table 3-9 – Comparison of Units of Measured and Predicted Distresses

Performance Indicator		MEPDG Units	MTO Units
Terminal IRI		m/km	m/km
Rutting	Total	mm	mm
	AC only	mm	Not used
Alligator Cracking (Bottom-up)		%	m^2
Total Cracking (Reflective + Alligator)		%	Not used
Longitudinal Cracking (Top-down)		m/km	m/km
Transverse or Thermal Cracking		m/km	m/km

Other note from Table 3-9 is that alligator cracking is measured as an area in m^2. In order to convert those values to percentages, the total area of the section was computed by multiplying the total length of the segment by the width of one lane (3.75m; approx. 12 feet). Another detail is that MTO measures and records distresses, namely, alligator cracking in 3 different categories (slight, moderate and severe) and in 3 different places within the traffic lane (mid lane, edge of pavement and wheel path). The total area of alligator cracking was obtained by the summation of all six values without applying a weighting factor.

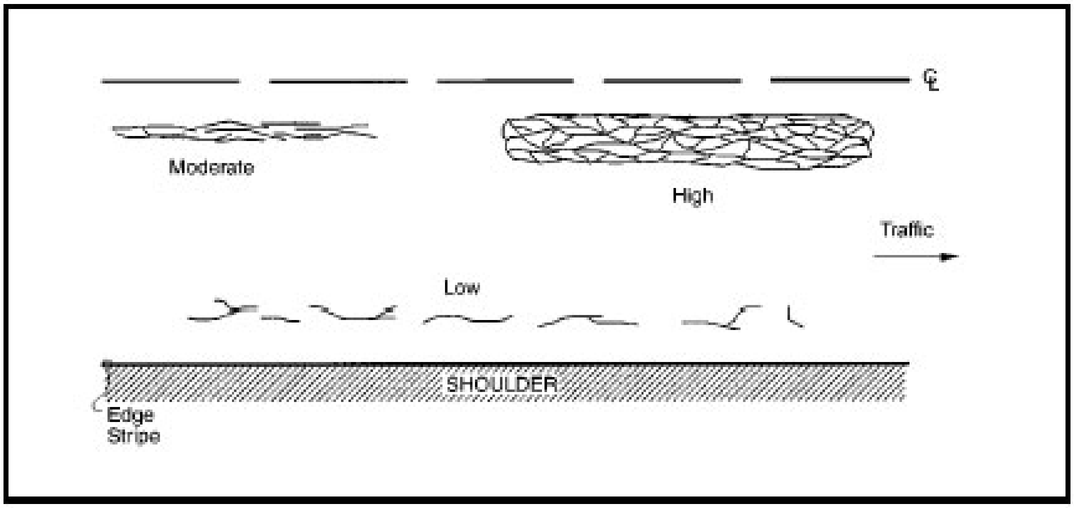

Figure 3-2 Bottom-Up or Alligator Cracking (FHWA, 2015)

3.6.2 Evaluation of Assembled Data

The next exercise is to review and evaluate the data to determine if information is deemed to be reasonable and accurate, to look for outliers and to identify any missing data. In addition, extracted data should be compared with design criteria or trigger values and assess if sampling shows values in the vicinity of the design criteria. The measured value of each distress should be taken and recorded and the average can be calculated. The following tables summarize the measured distress values and the pavement age for each performance distress.

Table 3-10 and Table 3-11 suggest that measured values are considered reasonable due to their magnitude and change in magnitude over time. Also, values are deemed to be plausible when compared with design criteria thresholds, although some of the distress magnitudes are more than two standard deviations below the design criteria. This observation can be associated with the fact that some projects were built very recently.

Table 3-10 – Extracted Measured Data and Pavement Age for Alligator Cracking

Project #	Pavement Age (years)	Measured Alligator Cracking (%)	Measured Total Rutting (mm)
1	15.42	9.0622	7.736
2	15.42	7.8471	7.056
7	10.08	0.8623	4.585
11	4.08	0.4129	3.250
18	4.00	0.5014	3.468
24	5.83	0.3191	2.972
27	8.25	6.7709	4.280
28	17.08	3.5210	10.997
38	17.08	1.3814	2.834
39	17.08	4.9978	5.394
41	5.83	3.7387	6.444
43	6.33	4.9319	4.364
50	6.00	0.3349	5.798
60	8.00	12.6882	5.057
70	3.17	0.8366	3.646
81	5.00	1.0257	3.446

Table 3-11 – Summary of Statistic Information for Alligator Cracking and Rutting

Distress or Performance Indicator	Mean	Maximum	Minimum	Standard Deviation	Design Criteria
Alligator Cracking, %	3.702	12.688	0.319	3.759	10
Rutting, mm	5.083	10.997	2.834	2.160	19

The next activity is to review the data to identify outliers and determine anomalous or erroneous data elements. In order to do this assessment, some strategies were used, including, computing basic statistics (mean, maximum, minimum and standard deviation) and developing scatter plots of key variables to evaluate reasonableness of magnitude and variability in magnitude over time. Alligator cracking or rutting observations with zero values must be removed from the database because that may imply that pavement has been rehabilitated or resurfaced. Any errors, outliers or anomalies that can be explained as an uncharacteristic condition should be flagged. Outliers can affect results and so measurement error or data entry should be corrected. Abnormal values associated with one-time event should be removed and analysis repeated. If any outliers are identified and removed, then it should be determined if the sample size is still appropriate.

Analyzing the scatter plots in Figure 3-3 and Figure 3-4, the data points are randomly dispersed and no points are identified to be widely separated from the main cluster of the data points, which indicates that no outliers exist.

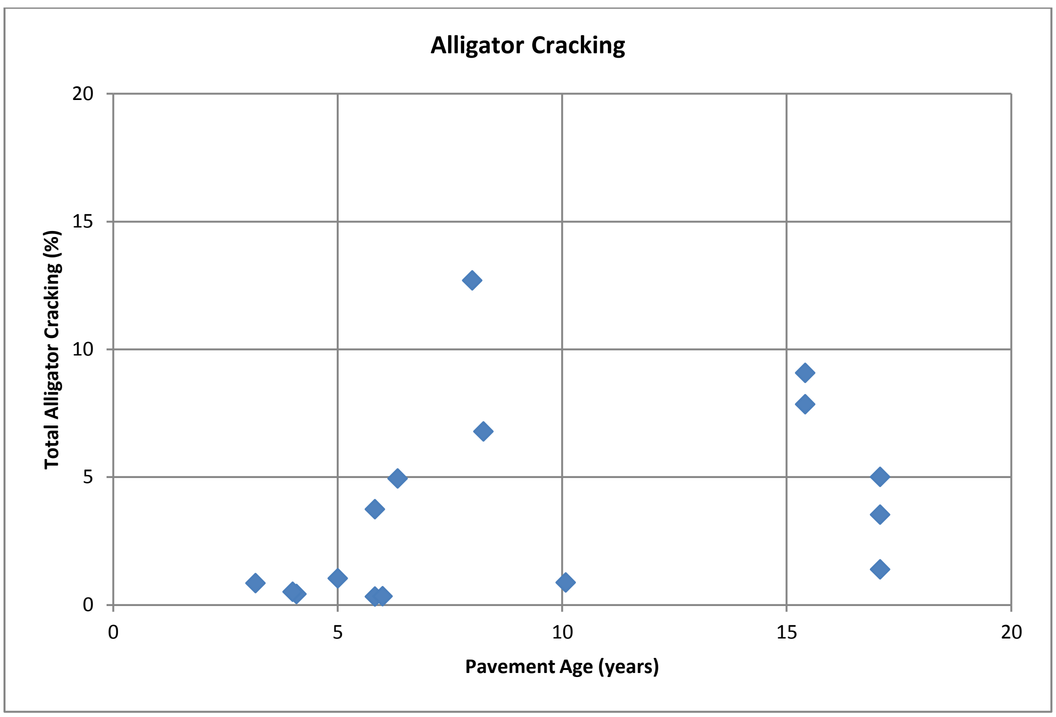

Figure 3-3. Plot showing Total Alligator Cracking (%) over time

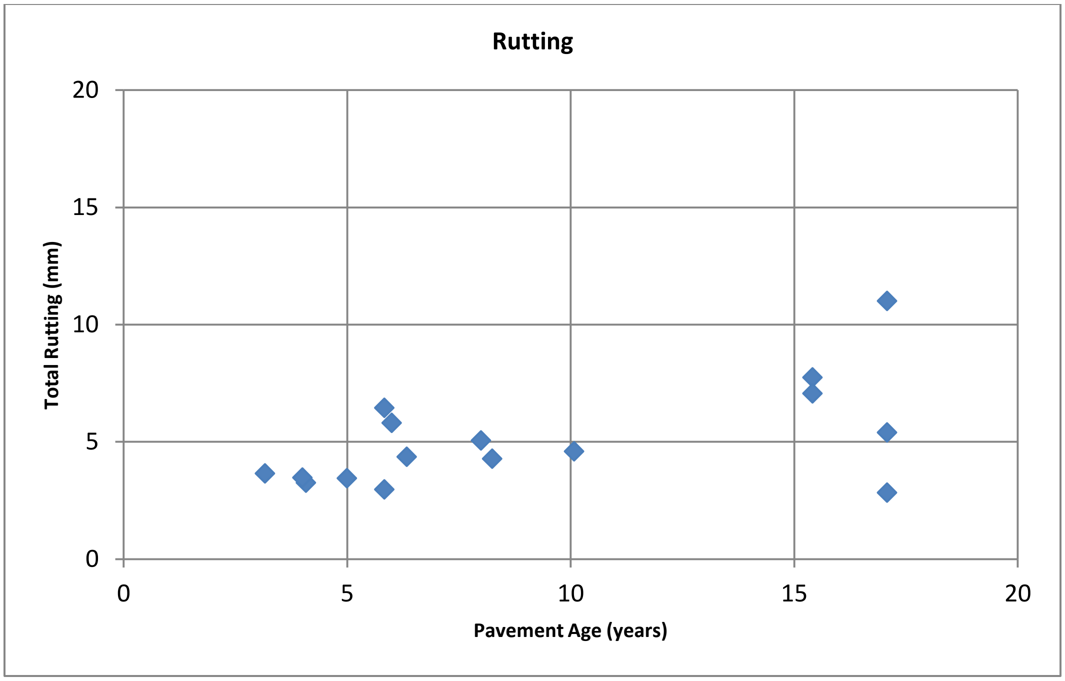

Figure 3-4. Plot showing Total Rutting (mm) over time

Lastly, for the roadway segments selected, all MEPDG inputs required for distress/IRI predictions should be extracted, reviewed and assembled. For this research, data was collected primarily from the following sources:

a) *iCorridor* – is a Ministry of Transportation Ontario (MTO) web application that provides traffic information;

b) MTO Pavements and Foundations Section/Materials Engineering and Research Office provided the pavement performance data;

c) MTO Provincial Highways Division/Geotechnical Engineering Section provided several pavement design reports from where construction history, cross sections, layer thicknesses and materials properties were extracted;

d) MTO - Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report provided default values;

e) Climate data – weather stations information embedded in AASHTOWare were used.

Evaluation of the assembled project information revealed some gaps in data required for AASHTOWare model calibration. A description of the relevant information missing and alternative sources is here described. Gradation analysis for the HMA mixtures was not available in the pavement design reports; the default values listed in the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report were used instead. Also, Dynamic modulus data was unavailable for HMA mixtures and Level 3 default values included in the AASHTOWare were assumed. The recommended value tabulated on the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report was assumed for Initial IRI.

3.7 Conduct Field and Forensic Investigations

Field and forensic investigations are needed to obtain additional information about the selected pavement sections. The fundamental information typically gathered from field and forensic investigations incorporates pavement visual condition surveys, subgrade strength and modulus, material properties and thickness. For this local calibration effort, information provided by MTO is used and no additional field and forensic investigations was conducted.

3.8 Data Inputs for Design

This section provides a description on the information extracted from sources listed in subchapter 3.6, for determining the inputs required to execute the MEPDG for each project.

3.8.1 Design Inputs

Construction dates were not very detailed in the pavement design reports; therefore, the construction date indicated in the reports was assumed as the pavement construction date. The base construction was assumed to be one month before the pavement and the traffic opening was assumed to be one (1) month after pavement construction. The dates of pavement performance surveys were provided by MTO.

3.8.2 Traffic Data

Traffic data was obtained primarily from the MTO *iCorridor* web application. For information not provided, default values tabulated on the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report and MEPDG were considered. Traffic information is tabulated in Appendix A. The following table tabulates the input level and the value inputted to run the AASHTOWare Pavement ME Design.

Average Annual Daily Truck Traffic (AADTT) – is defined as the two-way total volume of truck traffic on a highway segment during a reporting year, divided by the 365 days. AADTT is accessible in iCorridor, which is a map-based database that provides information for specific segments of Provincial Highways.

Percent of trucks in design direction – is defined as the percentage of trucks in the design direction compared to all trucks in the road segment in both directions. This value is not given in *iCorridor* and so 50% was used as recommended in MEPDG.

Percent of trucks in design lane – In MEPDG, this parameter is defined by the primary truck class for the roadway, since the primary truck class represents the truck class with the majority of applications (AASHTO, 2008). MTO recommends a percentage of trucks in design lane based on the number of lanes in one direction and the AADT (both directions), as given in Table 3-13.

Operational Speed – Truck speed affects the predicted E^* of HMA and therefore distresses. Higher incremental damage values computed by MEPDG (deeper rutting, more faulting and fatigue cracking) result from lower speeds. It is also defined as the speed at which drivers are observed operating during free-flow conditions. For this calibration effort, the posted speed was assumed as the operational speed.

Table 3-12 - Input Level for Traffic Parameters

Parameter	Input Parameter	Input Level	
		Value	**Description**
AADTT	Two-way AADTT	site specific	Downloaded from site specific traffic data systems. MTO files and store traffic data in a web application named *iCorridor*.
	Number of lanes in design direction		
	Percent of trucks in design direction	50%	MEPDG recommends a percent of trucks in design direction of 50% unless there is specific information available.
	Percent of trucks in design lane	site specific	Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report recommends a percentage of trucks in design lane based on the number of lanes in one direction and the AADT (both directions).
	Operational speed	site specific	Hierarchical levels are not appropriate for this variable. Defined as the speed at which drivers are observed operating during free-flow conditions. Posted speed was assumed as the operational speed.
Axle Configuration	Average axle width (m)	2.59	MEPDG default values were used for these variables.
	Dual tire spacing (mm)	305	
	Tire pressure (kPa)	827.4	
	Tandem axle spacing (m)	1.45	Used default values from Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report.
	Tridem axle spacing (m)	1.68	
	Quad axle spacing (m)	1.32	
Lateral Traffic Wander	Mean wheel location (mm)	460	MEPDG default values were used for these variables.
	Traffic wander standard deviation (mm)	254	
	Design lane width (m)	3.75	Lane width was obtained from site project reports.

Table 3-12 - *Continued*

Parameter	Input Parameter	Input Level	
		Value	Description
Wheelbase	Short trucks - Average axle spacing (m)	5.1	Used default values from Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report.
	Medium trucks - Average axle spacing (m)	4.6	
	Long trucks - Average axle spacing (m)	4.7	
	Percent short trucks	33	MEPDG default values were used for these variables.
	Percent medium trucks	33	
	Percent long trucks	34	
Traffic Volume Adjustment	Vehicle class distribution (Truck Traffic Classification - TTC)	site specific	Downloaded from site specific traffic data systems. MTO files and store traffic data in a web application named *iCorridor*.
	Traffic Growth Factor	site specific	
	Monthly and Hourly adjustment	site specific	MEPDG default values were used for these variables.
	Axles per truck	site specific	Downloaded from site specific traffic data systems. MTO files and store traffic data in a web application named *iCorridor*.
Axle Load Distribution	Axle distribution (Single, Tandem, Tridem, Quad)	site specific	

Axle-Load Configuration – For the standard truck classes, the spacing of the axles are relatively constant. The default values listed in Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report were used.

Dual Tire Spacing – AASHTOWare software presume that all standard truck axles contain dual tires. The default value of 305 mm (approx. 12 in) from AASHTOWare was considered based on the spacing of tires used by the majority of trucks.

Tire Pressure – Constant tire pressure is assumed in the AASHTOWare software for all loading conditions. The default value of 827.4 kPa (approx. 120 psi) from AASHTOWare is considered, which represents a median operating condition (hot inflation tire pressure).

Lateral Traffic Wander of Axle Loads – Constant wander is assumed in the AASHTOWare software for all trucks. The default value of 254 mm (approx. 10 in) was used for calibration procedure, regardless of the lane width.

Mean Wheel Location – is defined as the distance from the pavement marking to the outer edge of the wheel. The default value of 460 mm (approx. 18 in) is recommended by AASHTO.

Design Lane Width – is defined as the actual traffic lane width. The existing design lane width of 3.75 m (approx. 12 ft) was considered.

Wheelbase – the default values for average axle spacing and corresponding truck percentages for short, medium and long trucks are provided for Ontario's conditions.

Table 3-13 – Percentage of Trucks in Design Lane (Ontario 2012)

Number of lanes in one direction	AADT (both directions)	Percentage of Trucks in Design Lane
1	All	100
2	<15,000	90
	>15,000	80
3	<25,000	80
	25,000 to 40,000	70
	>40,000	60
4	<40,000	70
	>40,000	60
5	<50,000	60
	>50,000	60

Highway #: 401

LHRS Start:47575+0.9; LHRS End:47577+2.6

FHWA Veh. Class	Conf.	Truck Flow Vol.	Truck Flow %	FHWA Veh. Class	Conf.	Truck Flow Vol.	Truck Flow %
4		150	1.0 %	9		7,437	49.6%
5		1,734	11.6%	10		3,452	23.0 %
6		933	6.2%	11		59	0.4%
7		293	2.0%	12		30	0.2 %
8		207	1.4 %	13		705	4.7%
					Total	15,000	100%

Click on the links below to download the AASHTOWare Pavement ME traffic data files:

Traffic Data XML file (AADTT, FHWA Class Distribution and Number of Axles)

Axle Load Spectra ALF file

Summary Information Excel file (Traffic Data and Axle Load Spectra)

Figure 3-5: Example of Traffic Data Available in *iCorridor*

Normalized Vehicle (Truck) Class Distribution – represents the percentage of each truck class (class 4 through 13) within the AADTT for the selected year. The summation of all AADTT percentages should equal 1.

Traffic Growth Factor (TGF) – represents the annual rate of truck traffic growth over time. TGF are provided in iCorridor, as a compounded model, for each segment of Provincial Highways.

Monthly Adjustment Factors - represent the proportion of the annual truck traffic for a particular truck class that occurs in a given month. During calibration efforts, monthly distribution factors of 1.0 were used for all classes as defined in the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report.

Number of Axle Types per Truck Class – represents the average number of axles for each class 4 to 13 for each axle type (single, tandem, tridem and quad). These inputs are provided in iCorridor, based on the outcome of the CVS 2006 study for Ontario.

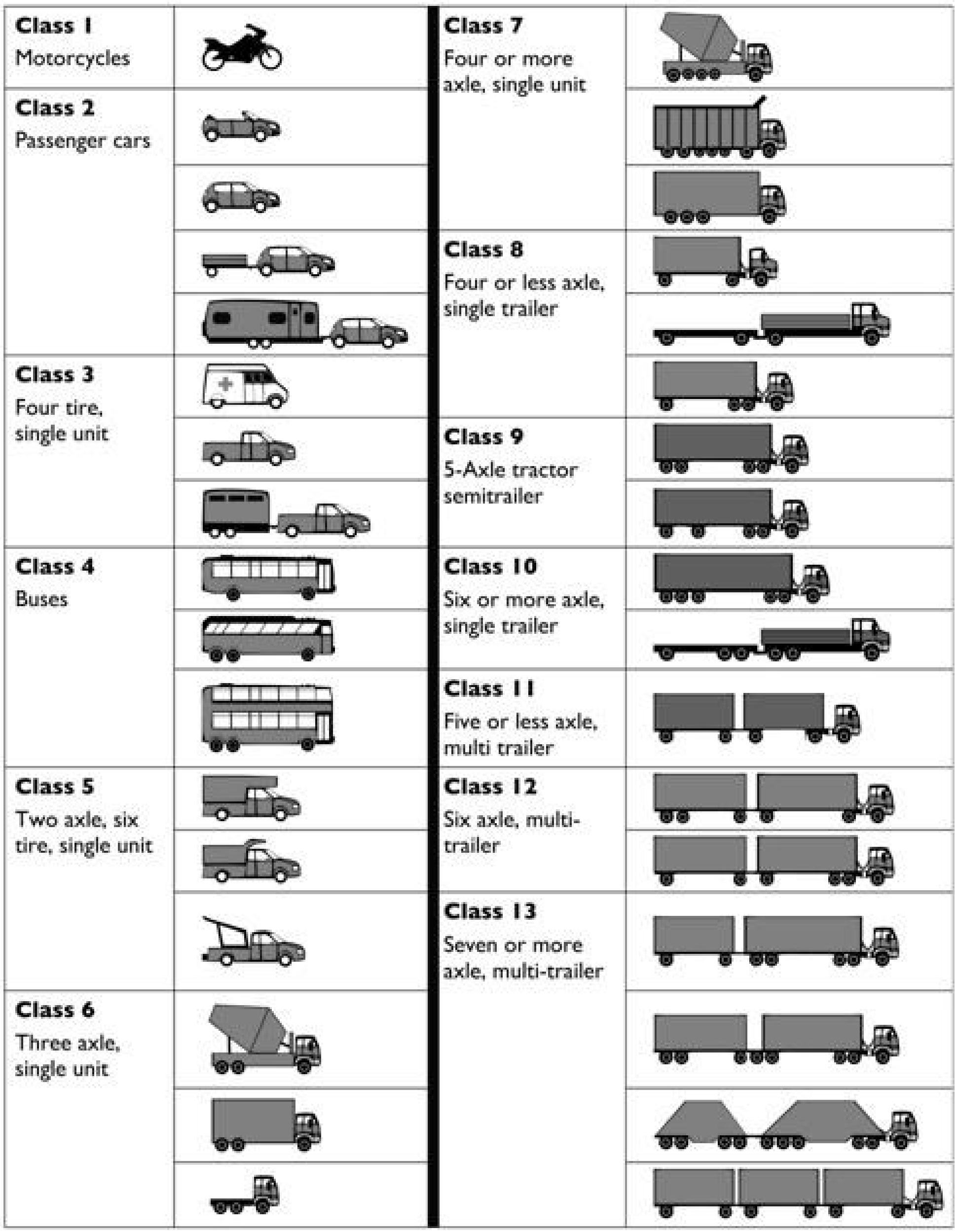

Figure 3-6: FHWA Vehicle Category Classification (FHWA, 2014)

Axle Load Distribution Factors/Spectra by Truck Class and Axle Type – represent the percentage of the total axle applications within each load interval for a specific axle type (single, tandem, tridem and quad) and vehicle class (classes 4 to 13). The normalized axle load distribution or spectra can only be determined from WIM data (NCHRP, 2004). MTO has developed axle load spectrum tables identifying vehicle classes and axle types based on the CVS 2006 study for Ontario. The Axle Load Spectra data was downloaded from iCorridor in ALF file format and then converted to Excel format.

3.8.3 Structural Layers and Materials Properties

Materials, layer thickness and properties for each pavement layer were obtained from the pavement reports provided by MTO. When data was not available, default values tabulated on the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report were considered. Structure information and materials properties are tabulated in Appendix B.

3.8.3.1 HMA

HMA Layer Thickness – the number of layers, type of material and thicknesses were obtained from each pavement design report, consulting tables, core and borehole information.

Volumetric Properties of Mixture – default values for unit weight, effective binder content, air voids and Poisson's ratio are tabulated on the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report and MEPDG.

Aggregate Gradation – Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report provides typical properties for SuperPave and Marshall mixtures.

Dynamic Modulus – in input Level 3, Witczak's model uses the aggregate gradation to compute the Dynamic Modulus (E^*) of HMA layers.

Asphalt Binder – Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report recommends specific penetration grades for a given mixture type.

Thermal - MEPDG default values are used for Thermal Conductivity and Heat Capacity. The Thermal Contraction is internally computed in MEPDG using the HMA volumetric properties, such as VMA and the thermal contraction coefficient for the aggregates. Thermal contraction coefficients are provided for different aggregate types.

Table 3-14 - Input Level for Asphalt Concrete material-related variables

Parameter	Input Parameter	Input Level: Value	Input Level: Description
General	HMA Layer Thickness	Site specific	Information was obtained from site project reports.
Structure	Surface Shortwave Absorptivity	0.85	MEPDG default value was used for this variable.
Volumetric Properties of Mixture (as built)	Unit Weight (Kg/m^3)	Site specific	Used default values from Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report.
	Effective Binder Content by Volume (%)	Site specific	
	Air Voids (%)	Site specific	
Poisson's Ratio	Poisson's Ratio	0.35	
Mechanical Properties	Dynamic Modulus	Site specific	Computed using PI and gradation from Project Specific Pavement Design Reports.
	Aggregate Gradation	Site specific	Used default values from Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report.
	G* Predictive Model[3]	Site specific	
	Reference Temperature (^{0}C)	21.1	
	Asphalt Binder	Site specific	
	Indirect Tensile Strength at – $10^{0}C$ (MPa)	Site specific	MEPDG calculates parameters based on binder type.
	Creep Compliance (1/GPa)	Site specific	
Thermal	Thermal Conductivity (W/m-Kelvin)	1.16	MEPDG default value.
	Heat Capacity (J/Kg-Kelvin)	963	
	Thermal Contraction		MEPDG calculates parameter based on HMA properties[4].

[3] Use Viscosity based model, Nationally calibrated.

[4] MEPDG computes internally using the HMA volumetric properties, such as, VMA and the thermal contraction coefficient for the aggregates. Thermal contraction coefficients are provided for different aggregate types.

3.8.3.2 Unbounded Material Properties

Granular Layer Thickness – the number of layers, type of material and thicknesses were obtained from each pavement design report, after checking the tables with borehole information.

Aggregate Gradation, Liquid Limit and Plastic Index – this data was extracted from borehole information from each pavement design report.

Resilient Modulus – input Level 2 based on gradation and Plastic Index (PI) was used to compute the Dynamic Modulus (E^*) of unbounded granular materials. Gradation and PI were obtained from borehole information from each pavement design report.

Additional information about unbound granular material parameters are given in Table 3-4.

3.8.3.3 Subgrade Soil Type and Properties

The soil type and aggregate gradation were extracted from borehole information from each pavement design report. The Liquid Limit and Plastic Index were available as well. The Resilient Modulus was included in the pavement design report. However, AASHTOWare Pavement ME Design was returning an error and thus Level 3 was selected.

Some project reports included the Depth of Ground Water Table from field investigation testing. For the remaining cases, the default value for Ontario conditions was used, which is 6.1m. Additional information about subgrade material parameters are given in Table 3-5.

3.8.4 Climate Data

AASHTOWare Pavement ME Design integrates processed climate information available from Environment Canada. There are, currently, 34 weather stations in Ontario. One may select a single weather station or a set of multiple weather stations in order to create a virtual weather station. Weather stations are sorted automatically by shortest distance to the project location. When creating virtual weather stations, more than one weather station must be selected in an effort to reduce error. By selecting a weather station (single or virtual), the software yields a climate summary for that location, which includes, mean annual air temperature, mean monthly temperatures (per month), mean annual precipitation, freezing index, average number of freeze/thaw cycles and number of wet days.

The latitude, longitude and elevation of each project were identified for the purpose of selecting the nearest weather stations or to create the virtual weather stations from existing Ontario weather stations.

Due to the limited functionality of the AASHTOWare Pavement ME Design weather station interpolation function, MTO recommends selecting the closest weather station to the project (Ontario, 2012).

3.8.5 Pavement Performance Data

The Initial IRI should be measured immediately after construction; however, this parameter was unavailable in the pavement design reports. The Initial IRI considered for this research was 0.8 m/km, which is the recommended value for new or reconstructed AC pavements for freeways tabulated in the Ontario’s Default Parameters for AASHTOWare Pavement ME Design – Interim Report.

In some cases, it was difficult to assess if the pavement performance data provided was exactly from the new constructed lane or section in reference. Therefore, assumptions were made considering that the existing sections were built with identical material properties. The pavement performance data is tabulated in Appendix C.

4 Development of Calibration Models

This chapter presents and describes the development of the procedure to calibrate the MEPDG performance models (alligator cracking and permanent deformation) for local conditions. The criteria to proceed with local calibration was based on (a) whether distresses were predicted without significant bias and (b) whether a given global model demonstrates a reasonable goodness of fit (between predicted and measured data points). The absence or presence of bias was assessed based on the hypothesis test described in the following subchapters, while the reasonable goodness of fit was assessed based on the Standard Error of Estimate (SEE). The criteria to assess the adequacy of models was defined as shown in Table 4-1.

Table 4-1 – Criteria for Determining Global Models Adequacy

Criterion of Reference	**Statistic Test**	**Thresholds**		**Rating**
Bias	Hypothesis testing of intercept of the linear predicted vs measured distress	p-value for the intercept		Reject if p-value is <0.05
Goodness of fit	SEE, Alligator Cracking model	<5 percent		Good
		5 to 10 percent		Fair
		>10 percent		Poor
	SEE, Total Rutting model	<0.1 in	<2.54 mm	Good
		0.1 to 0.2 in	2.54 to 5.08 mm	Fair
		>0.2 in	> 5.08 mm	Poor

4.1 Assess Local Bias from Global Calibration Parameters

The AASHTOWare Pavement ME software was run using the global calibration parameters, for each selected project to predict pavement distresses and IRI over the life of the project at a 50 percent reliability level.

A comparison was made between the measured and predicted distresses to determine bias, standard error of estimate and residuals. A linear regression model was used to determine the correlation between predicted distress (dependent variable, Y) and the measured distress (independent variable, X). The local bias is assessed by studying the null hypothesis that the bias (average residual error) is zero at a 95 percent confidence level.

$$H_0 = \sum_{i=1}^{n} \left(Y_{measured} - X_{predicted}\right) = 0$$

Equation (30)

Also, scatter charts of the measured versus the predicted distresses were plotted to investigate the dispersion of the points in relation to the line of equality.

Table 4-2 – Summary of Statistic Parameters – Global Calibration

Performance Indicator	Regression Coefficients	Mean Error (e_r)	p-value	Hypothesis; $H_0: \sum(Y_m - X_p) = 0$
Rutting	$\beta_{1r} = 1$ $\beta_{s1(GB)} = 1$ $\beta_{s1\,(SG)} = 1$	-7.98	0.462	null hypothesis is not rejected; $p>0.05$
Alligator Cracking	$C_1 = 1$ $C_2 = 1$	3.68	0.083	null hypothesis is not rejected; $p>0.05$

4.1.1 Assess Local Bias for Alligator Cracking Model

The p-value approach was used to conduct this hypothesis test. The output from regression gives a p-value of 0.083 which is greater than 0.05. Therefore, the null hypothesis cannot be rejected. Thus, it cannot be said that the transfer functions produce biased predictions.

The plot of measured versus predicted alligator cracking is given in Figure 4-1. The figure shows a poor linear relationship between measured and predicted distress. The measured cracks are of much larger magnitude than predicted values, which reflects that the MEPDG model under predicted alligator cracking. Therefore, the local calibration procedure must be conducted in order to improve this transfer function.

As shown in Figure 4-1, the intercept (b_0) is significantly different from zero and the slope (m) estimator is significantly different from 1. Thus, the transfer function must be calibrated to local conditions.

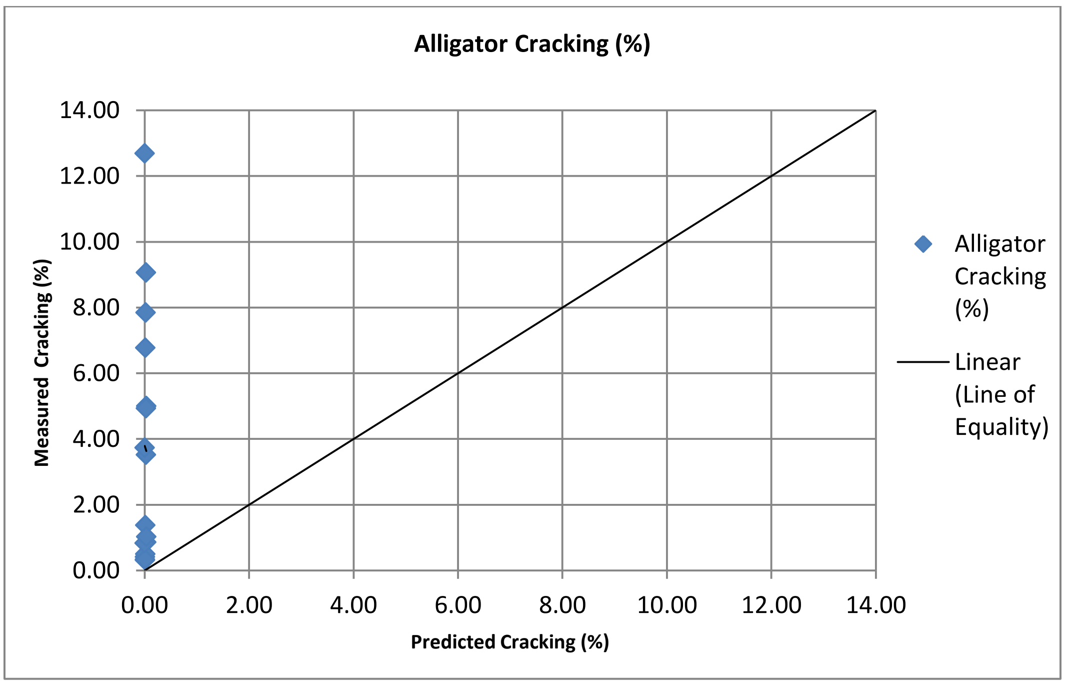

Figure 4-1: Measured vs Predicted Alligator Cracking Using Global Calibration Parameters

4.1.2 Assess Local Bias for Rutting Model

The output from regression analysis gives a p-value of 0.462 which is greater than $\alpha = 0.05$; therefore, the null hypothesis cannot be rejected. Thus, it cannot be stated that the transfer functions produce biased predictions.

The plot of measured versus predicted total rutting was performed. Figure 4-2 shows a wide dispersion between the measured and predicted rut depths, which suggests a poor correlation between measured and predicted rutting. Rutting is over predicted using the global calibration parameters; therefore, local calibration must be conducted for this transfer function.

Also, the intercept (b_0) is significantly different from zero and the slope (m) estimator is significantly different from 1. Thus, the transfer function will produce biased predictions and accordingly should be calibrated to local conditions.

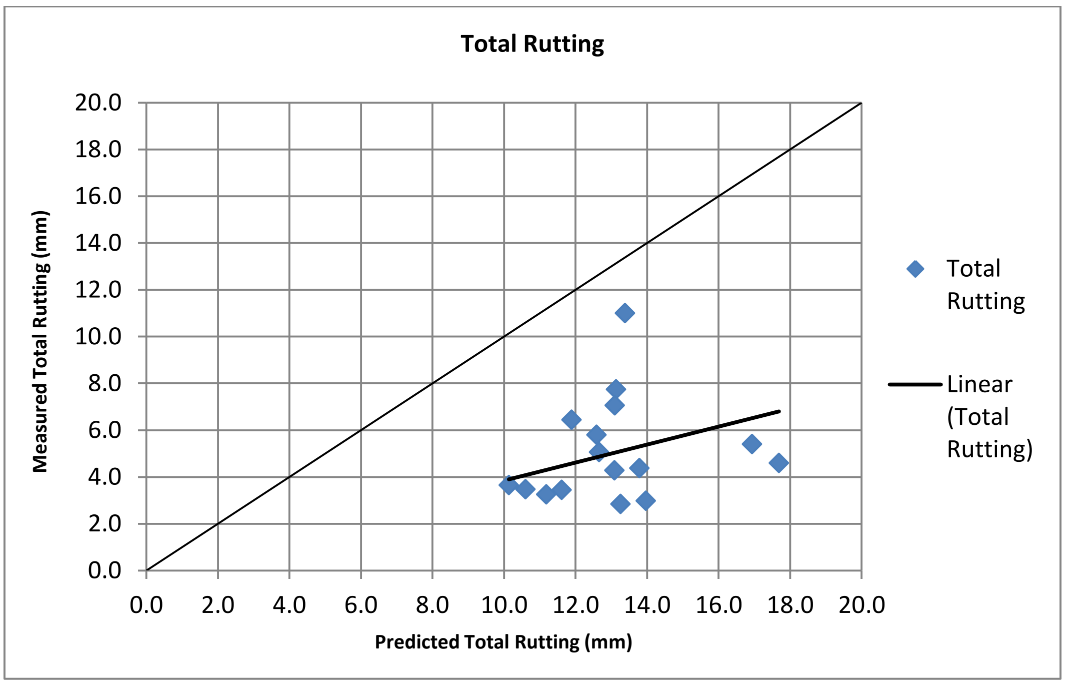

Figure 4-2: Measured vs Predicted Total Rutting Using Global Calibration Parameters

4.2 Eliminate Local Bias of Distress Prediction Models

The procedure to eliminate the bias begins with the identification of the cause for bias. This is done by adjusting the local calibration coefficients for the biased models. The MEPDG local calibration coefficients can be amended to reduce bias and/or standard error, as presented in Table 4-3:

Table 4-3 – Potential Cause of Bias and Corrective Action (CDOT, 2013)

Potential Cause of Bias	Corrective Action
Check and compare predicted against measured distress/IRI plots for each individual project in order to identify projects wherein predicted and measured values vary significantly.	In case that these projects are a small percentage, verify data for mistaken inputs and assumptions and correct them. Then, repeat process to check if bias was eliminated and use global coefficients. Otherwise, iterate global/local coefficients as required until significant bias is eliminated.
Examine key input variables that influence each distress type and develop a residuals plot – predicted/measured distress versus a particular variable. Identify trends in the plot. Upward or downward trends are indicators of correlation.	Identify and track the global/local coefficients influencing more the key inputs that relate to bias. Iterate global/local coefficients as required until significant bias is eliminated.
Establish that bias is random with not assigned cause.	Iterate global/local coefficients as required until significant bias is eliminated

Microsoft Excel Solver and Regression Analysis are used to estimate the calibration factors of the performance models. The procedure to eliminate the local bias was performed according to the following steps:

a) The distresses were predicted for the selected projects using the AASHTOWare Pavement ME Design with the global calibration coefficients;

b) The computed distresses corresponding to the same month and year of performance data were extracted from ME Design outputs of each project and tabulated along with measured distresses;

c) The Error, Squared Error, Sum of Squared Errors (SSE) and Standard Error of Estimate (SEE) were calculated, along with other basic statistics (Mean, Maximum, Minimum and Standard Deviation);

d) Regression Analysis was employed to determine the calibration factors of the rutting models while Microsoft Excel Solver was used to minimize SSE by adjusting the C1 and C2 factors of the alligator model.

4.2.1 Eliminate Local Bias for Alligator Cracking Model

The bias of bottom-up fatigue cracking can be improved through four coefficients: $\beta_{f1}, \beta_{f2}, \beta_{f3}$ and C_2. The C_2 term is related with the percentage of area of fatigue cracking from the damage index, while the *beta* terms are associated to estimating the allowable number of load applications for a specific condition and layer. The coefficients β_{f1} and C_2 are usually used to eliminate the model bias and the standard error of the estimate (SEE).

A Microsoft Solver numerical optimization routine was used to find the regression coefficients C_1 and C_2. The optimization was set by first predicting, thru AASHTOWare Pavement ME Design software, the damage for each section, assuming an initial value for C_1=1 and C_2=1. Then, by using Equation (8), alligator cracking was computed from the predicted damage percentage. Then, the predicted alligator cracking was subtracted from the observed alligator cracking to obtain the error. Then, the errors were squared and summed to obtain the total sum of squared error (SSE).

At this point, the Microsoft Solver was set to minimize the sum of the squared errors (SSE) by changing the coefficients C_1 and C_2 for the first iteration. The first iteration returned C2=0. When C2 is zero, the computed damage has no effect on the alligator cracking percentage. Then, Solver was run again and C2 returned negative on the second iteration. When C2 is negative, more damage leads to less cracking, which is not reasonable. Thence, these runs were discarded.

At this point, C1 was fixed to 1.0, the original value, and the calibration was continued only for C2. The local calibration coefficients obtained are: C1 = 1 and C2 = 0.968267 (Table 4-4).

Table 4-4 – Computation of the Calibration Coefficients C1 and C2

	Predicted (from M-E Design)				Predicted (by formula)					Measured			
	Month	Pavement Age (years)	Max Damage (%)	Max Cracking (%)	h_{AC} (mm)	h_{AC} (in)	C1'	C2'	Predicted Cracking (%)	Total Alligator Cracking (%)	Error	|Error|	Squared Error
Project 1	10/2014	15.42	0.0468	0.0296	285	11.2205	4.8799	-2.4400	3.5693	9.0622	-5.4930	5.4930	30.1725
Project 2	10/2014	15.42	0.0333	0.0208	290	11.4173	4.8772	-2.4386	2.5492	7.8471	-5.2979	5.2979	28.0675
Project 7	9/2015	10.08	0.0505	0.0330	320	12.5984	4.8635	-2.4318	3.8883	0.8623	3.0259	3.0259	9.1563
Project 11	9/2015	4.08	0.0202	0.0131	440	17.3228	4.8371	-2.4186	1.5953	0.4129	1.1823	1.1823	1.3979
Project 18	9/2015	4.00	0.0254	0.0166	410	16.1417	4.8412	-2.4206	1.9989	0.5014	1.4975	1.4975	2.2424
Project 24	6/2015	5.83	0.0157	0.0096	320	12.5984	4.8635	-2.4318	1.2101	0.3191	0.8910	0.8910	0.7939
Project 27	10/2015	8.25	0.0276	0.0179	375	14.7638	4.8477	-2.4238	2.1607	6.7709	-4.6102	4.6102	21.2540
Project 28	10/2014	17.08	0.0485	0.0323	360	14.1732	4.8511	-2.4256	3.7666	3.5210	0.2455	0.2455	0.0603
Project 38	10/2014	17.08	0.0301	0.0199	435	17.1260	4.8377	-2.4189	2.3733	1.3814	0.9919	0.9919	0.9839
Project 39	10/2014	17.08	0.0515	0.0350	420	16.5354	4.8397	-2.4199	4.0236	4.9978	-0.9742	0.9742	0.9491
Project 41	5/2015	5.83	0.0107	0.0068	450	17.7165	4.8360	-2.4180	0.8433	3.7387	-2.8954	2.8954	8.3836
Project 43	11/2015	6.33	0.0471	0.0313	360	14.1732	4.8511	-2.4256	3.6598	4.9319	-1.2721	1.2721	1.6183
Project 50	9/2015	6.00	0.0186	0.0119	395	15.5512	4.8437	-2.4219	1.4604	0.3349	1.1255	1.1255	1.2667
Project 60	9/2015	8.00	0.0055	0.0029	255	10.0394	4.9010	-2.4505	0.4024	12.6882	-12.2858	12.2858	150.9410
Project 70	8/2015	3.17	0.0077	0.0047	390	15.3543	4.8447	-2.4223	0.6018	0.8366	-0.2348	0.2348	0.0551
Project 81	9/2015	5.00	0.0728	0.0340	160	6.2992	5.0896	-2.5448	4.9019	1.0257	3.8762	3.8762	15.0250

SSE
272.3675

C1 = 1

C2 = 0.968267437

Subsequently, using the local calibration factors, the bottom-up cracking was calculated. Then, the error for the full set of data was calculated from the difference between predicted alligator cracking and measured alligator cracking. The errors were squared and then summed to obtain the total sum of squared error (SSE).

The plot of measured versus predicted alligator cracking, using the local calibration coefficients, is given in Figure 4-3. This figure shows that the points are randomly scattered along the equality line and that the predicted values closer to the measured values than in Figure 4-1, suggesting an improvement in the prediction of alligator cracking.

Figure 4-4 shows that the residual errors are randomly distributed with the exception of one project. Also, the errors do not increase significantly as the percentage of cracking increases.

The presence of bias between predicted and measured alligator cracking was investigated through a linear regression analysis between the measured alligator cracking as the dependent variable and predicted alligator cracking as the independent variable. The p-value of the intercept was used to test the hypothesis that there is no bias in the prediction. The intercept in the linear regression would be the bias, the consistent or systematic difference between the measured and predicted values with the calibrated model. The output from regression (Table 4-5) gives a p-value of 0.07 which is greater than $\alpha = 0.05$; therefore, the null hypothesis cannot be rejected. Thus, for the specified level of confidence, it cannot be stated that the calibrated transfer function will produce biased predictions.

As shown, the mean error has improved in comparison to the use of the global calibration values, suggesting that the local calibration procedure has improved the alligator cracking model.

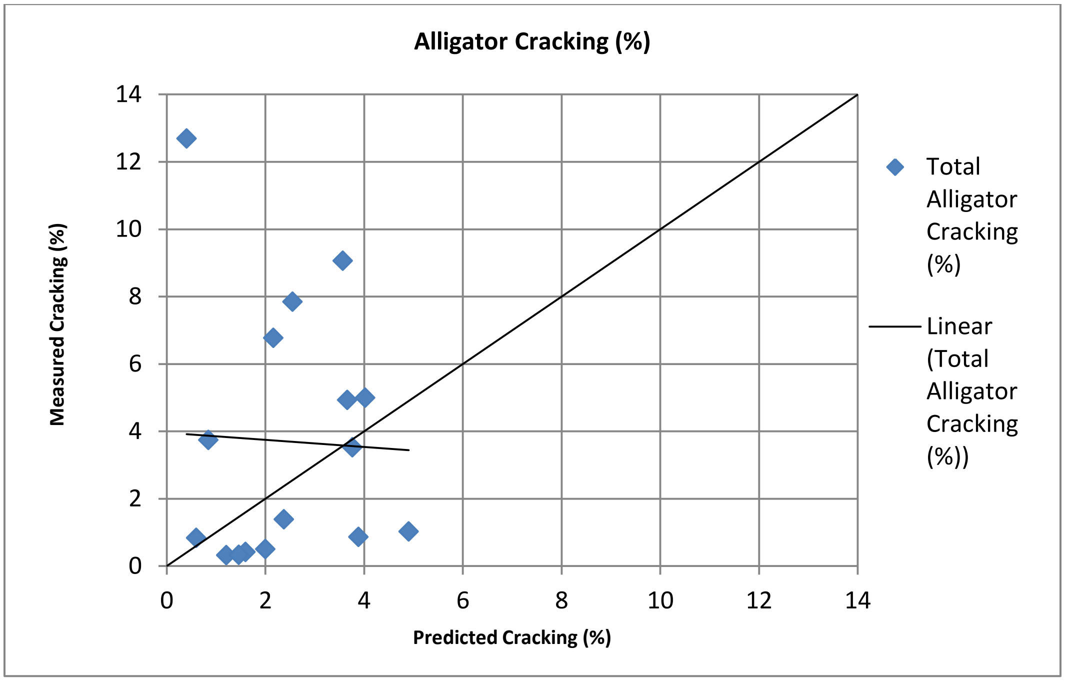

Figure 4-3: Measured vs Predicted Alligator Cracking Using Local Calibration Parameters

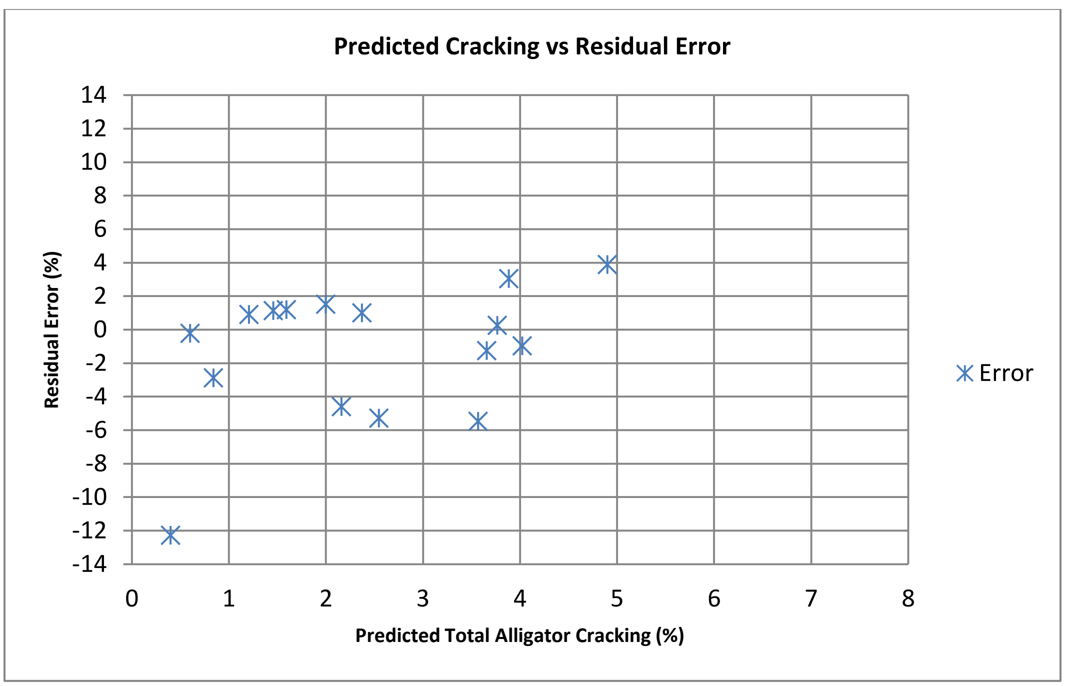

Figure 4-4: Residual Error versus Predicted Alligator Cracking

Table 4-5 – Summary of Statistic Parameters – Local Calibration

Performance Indicator	**Regression Coefficients**	**Mean Error (e_r)**	**p-value**	**Hypothesis; $H_0: \sum(Y_m - X_p) = 0$**
Alligator Cracking	$C_1 = 1$ $C_2 = 0.968267$	-1.26 %	0.07	null hypothesis cannot be rejected; p>0.05

4.2.2 Eliminate Local Bias for Rutting Models

There are a total of five calibration factors in the permanent deformation model, three for the asphalt layers, one for the unbound granular base and another for the unbound subgrade. These calibration factors are correction constants applied to the initial models to improve the final calibrated permanent deformation model. The following paragraphs describe the approach used in the calibration procedure. Note that no trench studies were conducted; therefore, the actual layer rut depths are not provided. Other mathematical approaches were used instead in this exercise.

A Microsoft Excel Regression Analysis was used to find the permanent deformation calibration factors β_{1r}, β_{s1GB} and β_{s1SG}. The optimization was set by first predicting, through the AASHTOWare Pavement ME Design software, the rutting for each section after assuming an initial value for $\beta_{1r} = 1, \beta_{s1GB} = 1$ and $\beta_{s1SG} = 1$. The optimization was done using the approach of minimizing the sum of squared error (SSE) for the rutting in each of the three layers separately. Therefore, three calibration factors (β_{1r}, β_{s1GB} and β_{s1SG}) were obtained separately. The remaining calibration coefficients were kept fixed: $\beta_{2r} = 1$ (temperature) and $\beta_{3r} = 1$ (number of load repetitions).

The M-E Design predicts rutting values for the AC, base, subgrade layers and the total rutting. However, from performance reports, only total rutting is given. In order to address this, the ratios of rutting in each layer relative to the total rutting was calculated for the rutting values predicted with the globally calibrated models. Then, the measured total rutting was multiplied by those ratios to compute the measured rutting values for the AC, base and subgrade layers.

After tabulating the ratios for predicted and measured rutting values for the AC, base and subgrade layers, the local calibration procedure was executed by performing a simple linear regression, by setting the measured total rutting as the dependent variable (Y) and the predicted total rutting as the independent variable (X). The intercept was fixed as zero to fit the regression line through the origin.

The same procedure was carried out for each layer in order to obtain the three calibration factors, $\beta_{1r}, \beta_{s1(GB)}$ and $\beta_{s1(SG)}$. The obtained regression coefficients are given in Table 4-6 and are the local calibration coefficients.

Table 4-6 –Local Calibration Coefficients

Layer	**Model**	**Regression Coefficients**
HMA	$\Delta p_{AC} = \beta_{1r} k_z \varepsilon_{r(HMA)} 10^{k_{1r}} N^{k_{2r}\beta_{2r}} T^{k_{3r}\beta_{3r}}$	$\beta_{1r} = 0.385452$
Base	$\Delta p_{soil} = \beta_{s1} k_{s1} \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)^{\beta}} \varepsilon_v h_{soil}$	$\beta_{s1GB} = 0.413484$
Subgrade	$\Delta p_{soil} = \beta_{s1} k_{s1} \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)^{\beta}} \varepsilon_v h_{soil}$	$\beta_{s1SG} = 0.372472$

After that, using the local calibration factors, the total rutting was calculated using Equations (16) and (21). Then, the residual error for the full set of data was calculated as the difference between predicted total rutting and measured total rutting. The SSE was then obtained from the squared errors.

The plot of measured versus predicted rutting in each layer of the pavement, using the local calibration coefficients, was performed as shown in Figure 4-5 to Figure 4-11. The measured total rutting versus the predicted total rutting was also plotted.

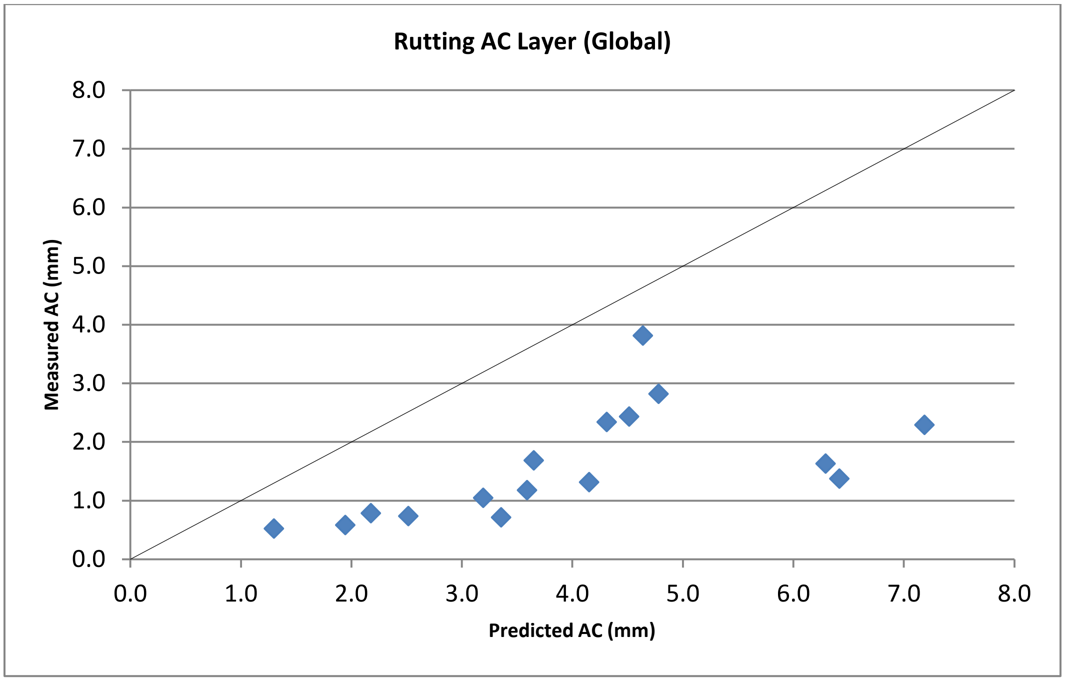

Figure 4-5: Measured vs Predicted AC Rutting Using Global Calibration Parameters

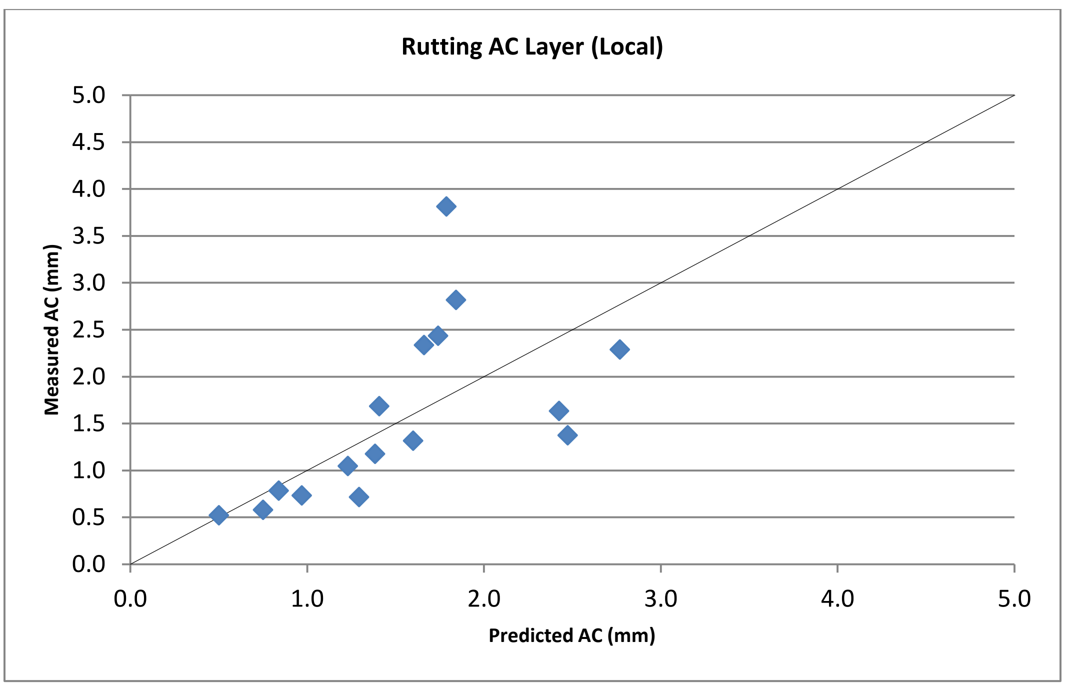

Figure 4-6: Measured vs Predicted AC Rutting Using Local Calibration Parameters

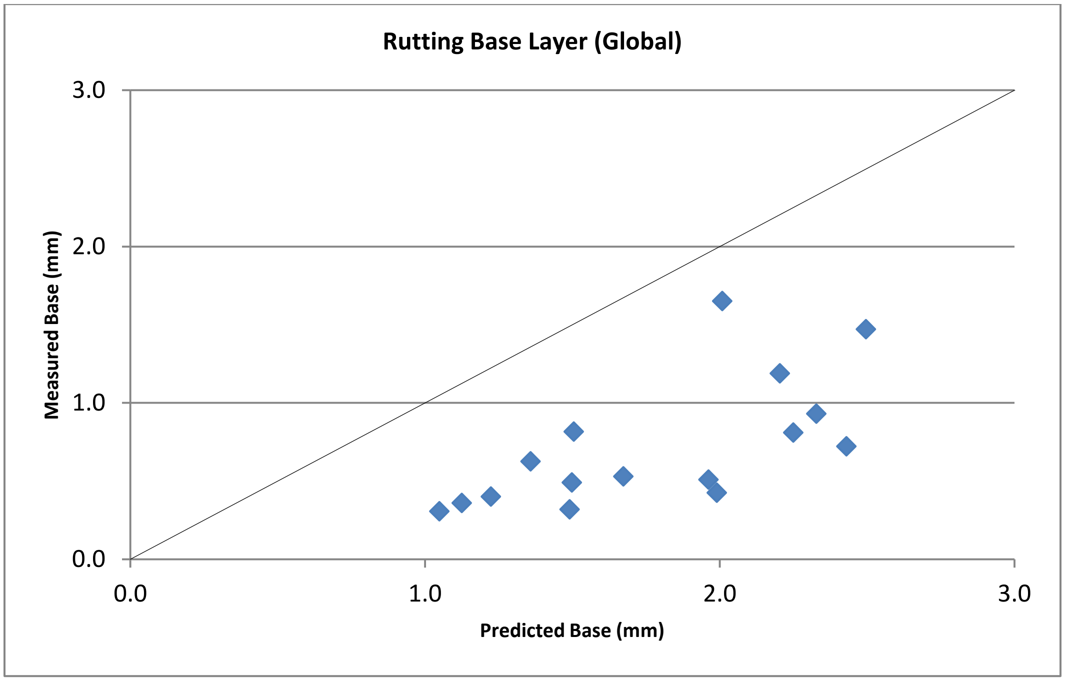

Figure 4-7: Measured vs Predicted Base Rutting Using Global Calibration Parameters

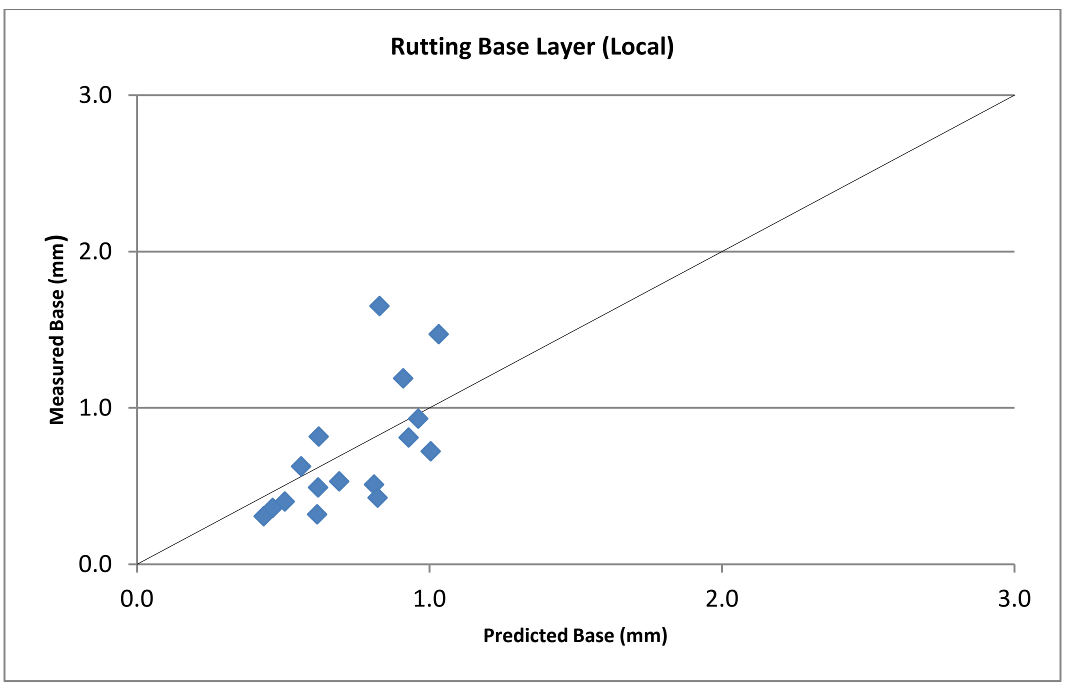

Figure 4-8: Measured vs Predicted Base Rutting Using Local Calibration Parameters

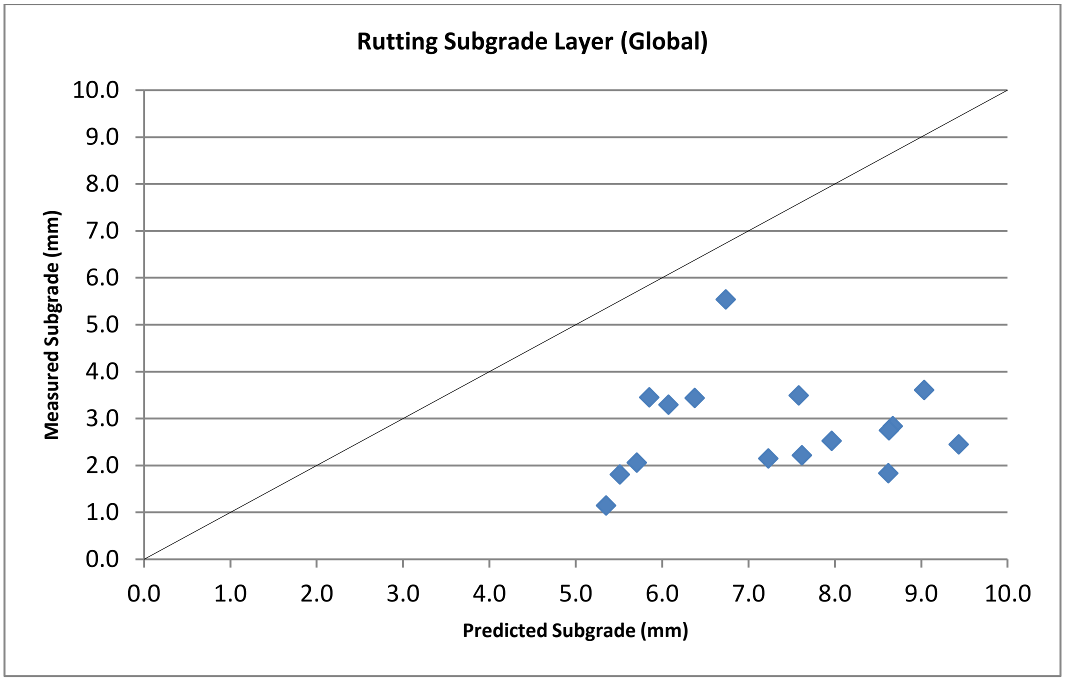

Figure 4-9: Measured vs Predicted Subgrade Rutting Using Global Calibration Parameters

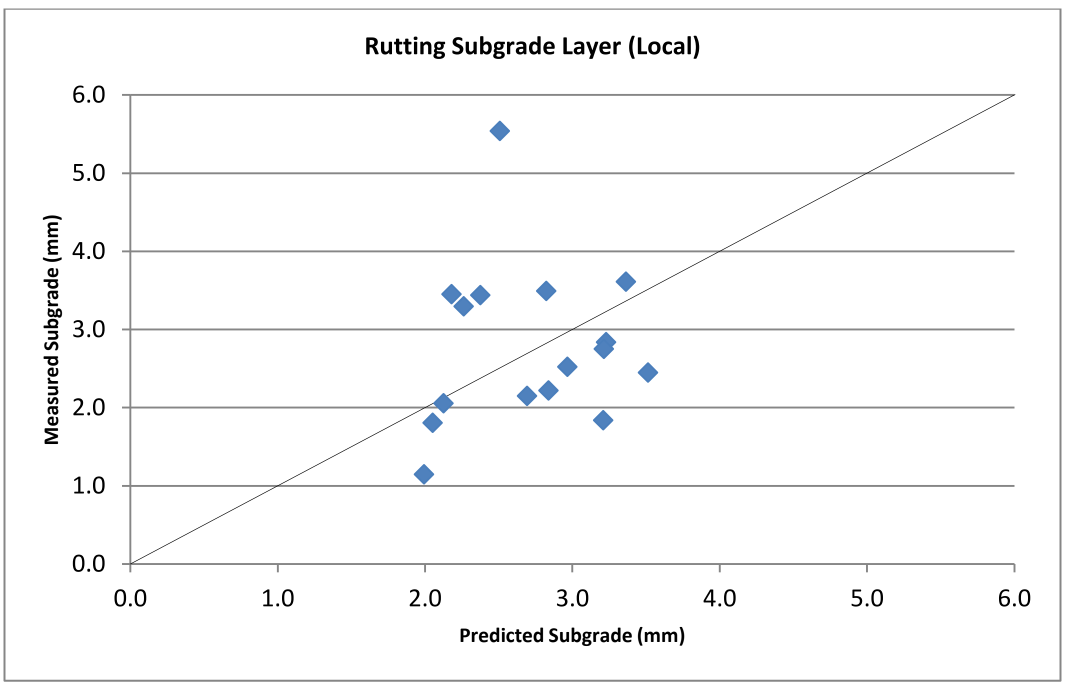

Figure 4-10: Measured vs Predicted Subgrade Rutting Using Local Calibration Parameters

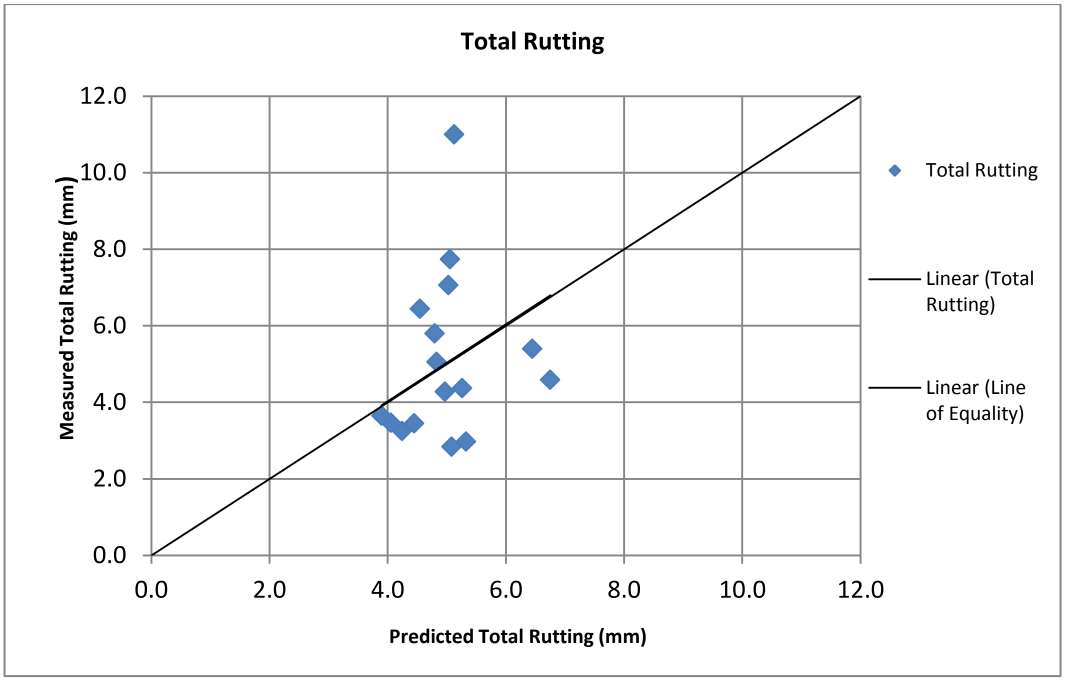

Figure 4-11: Measured vs Predicted Total Rutting Using Local Calibration Parameters

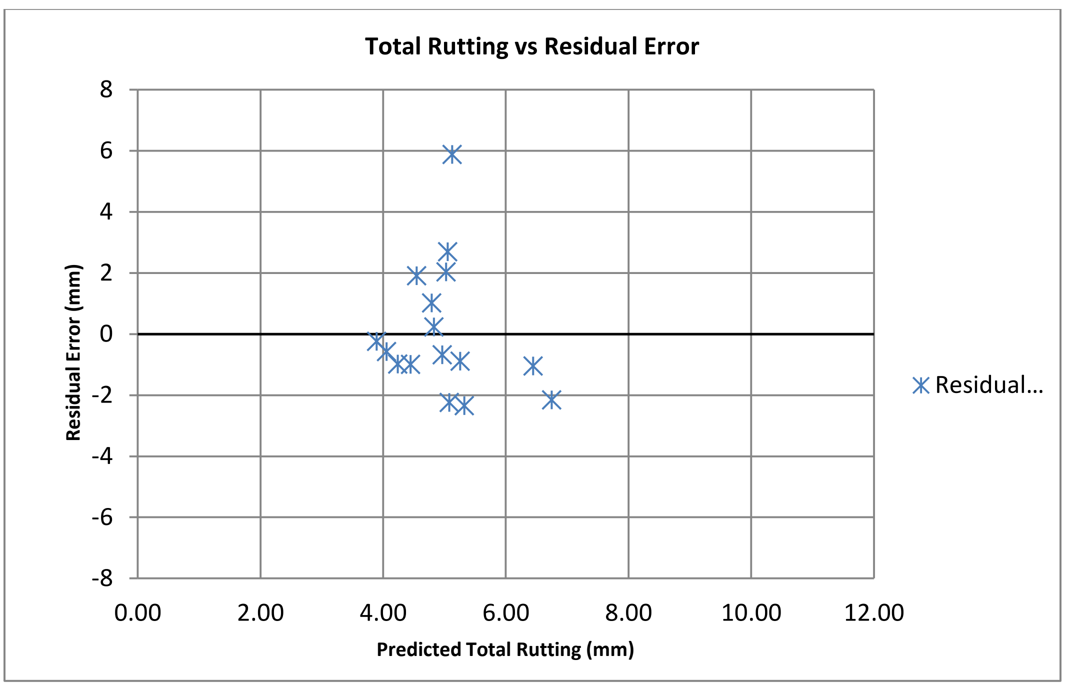

Figure 4-12: Predicted Total Rutting versus Residual Error

Figure 4-5 to Figure 4-11 show that the local calibration coefficients provide a better fit between predicted and measured rutting, in all layers separately and for the total rutting, which means that the prediction of permanent deformation has improved. Lastly, Figure 4-12 shows that the residual errors are randomly distributed and not correlated to the predicted rutting.

The presence of bias between the predicted and measured total rutting was investigated through a linear regression analysis between the measured total rutting as the dependent variable and predicted total rutting as the independent variable. The p-value of the intercept was used to test the hypothesis that there is no bias in the prediction. The intercept in the linear regression would be the bias, the consistent or systematic difference between the measured and predicted total rutting with the calibrated model. The output from regression (Table 4-7) shows a p-value for the intercept of 0.49, which is much greater than α = 0.05; therefore, the null hypothesis cannot be rejected. Thus, for the specified level of confidence, it cannot be stated that the transfer function will produce biased predictions.

The average error or bias was reduced and the accuracy of the transfer function has increased in comparison to the use of the global calibration values, suggesting that the local calibration procedure has improved the rutting model.

Table 4-7 – Summary of Statistic Parameters – Local Calibration

Performance Indicator	**Regression Coefficients**	**Mean Error (e_r)**	**p-value**	**Hypothesis;** $\boldsymbol{H_0: \sum(Y_m - X_p) = 0}$
Rutting	$\beta_{1r} = 0.385452$ $\beta_{s1GB} = 0.413484$ $\beta_{s1SG} = 0.372472$	0.09 mm	0.49	null hypothesis cannot be rejected; p>0.05

4.3 Assess the Goodness of Fit

In order to assess the goodness of fit, the standard error of the estimate (SEE) was obtained from the analysis performed in subchapter 4.2 (Microsoft Excel Solver and Regression Analysis) following the determination of the SSE. Then, these parameters were compared with the parameters from global calibration models, as shown in Table 4-8.

Table 4-8 – MEPDG Statistic Parameters – Global and Local Calibration

Performance Model	**Mean Error (e_r)**		**Standard Error of Estimate (SEE)**	
	Global	**Local**	**Global**	**Local**
Total Rutting	-7.98 mm	0.09 mm	8.68 mm	2.17 mm
Alligator Cracking	3.68 %	-1.26%	5.35 %	4.26 %

The standard error of estimate is the main indicator to measure if the model improved or not. Table 4-8 shows that using local calibration coefficients, the standard error of estimate (SEE) for alligator cracking improved slightly from 5.35 to 4.26 percent of lane area. With regard to rutting model, the SEE improved significantly from 8.68 to 2.17 mm. It can be concluded that local calibration procedure has improved the prediction models. In addition, SEE parameters are considered good when compared with thresholds defined in Table 4-1.

In the case of alligator cracking, in order to improve accuracy and precision of the models, more data is required, in particular, more pavement sections exhibiting higher level of distresses.

5 Summary, Conclusions and Recommendations

5.1 Summary

The primary objective of this project was to calibrate the performance models used in the AASHTOWare Pavement ME for the design of new flexible pavement structures to local conditions of Ontario, Canada, considering local traffic conditions, materials properties and structure information from pavement design reports and local climate information. MTO pavement management database was a major source for traffic, field-testing structural information and performance data. For climatic data, local weather stations embedded in AASHTOWare software were considered.

The calibration procedure followed the Guide for Mechanistic-Empirical Design (AASHTO, 2008) and guidelines in the Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report (Ontario's Interim Report, 2012). The AASHTOWare Pavement ME Design^{TM} software was used to predict pavement distresses.

This research deals only with new flexible pavement structures. Sixteen (16) representative pavement sections were selected from widening and reconfiguration highway projects, combining a variety of pavement structures, subgrade soils, materials properties and traffic conditions. These sections, with pavement ages ranging from 3 to 17 years, include also a variety of distress levels.

5.2 Conclusions

The pavement distress models evaluated and calibrated in this research are (i) total rutting and (ii) AC bottom-up fatigue cracking. The calibration procedure includes the comparison between measured (from performance data) and predicted values (from M-E Design software). The statistical analysis showed that, using the global calibration coefficients, the AASHTOWare models under predicted alligator cracking and over predicted total rutting.

An optimization process was undertaken to obtain the calibration coefficients for the bottom-up fatigue cracking model, C_1 and C_2. The final coefficients obtained for local calibration were: $C_1 = 1$ and $C_2 = 0.968267$. The local calibration factors have improved the mean prediction error (from 3.68 to -1.26) and standard error of the estimate (from 5.35 to 4.26).

The plot of measured versus predicted alligator cracking, showed that the predicted values moved closer to the measured values due to the local calibration. In addition, the statistical analysis could not state that the transfer function will produce biased predictions, at the specified level of confidence.

However, one possible justification for the slight improvement in SEE could be the measurement procedure used by the agency. The agency measures and records alligator cracking in 3 different categories (slight, moderate and severe) and in 3 different places within the traffic lane (mid lane, edge of pavement and wheel path). The total area of alligator cracking is obtained by the summation of all six values without applying any weighting factor. Finally, it is important to recall that this research deals with Provincial Highways (fairly compared to Interstate Highways in the United States). Therefore, high percentages of alligator cracking are not expected due to the high design standards and the strict rehabilitation program.

Regression analysis was performed to find the following coefficients for the local calibration of the rutting model: $\beta_{1r} = 0.385452$, $\beta_{s1GB} = 0.413484$ and $\beta_{s1SG} = 0.372472$. The local calibration factors have improved bias (from -7.98 to 0.09), and reduced the standard error of the estimate (from 8.68 to 2.17). Using the local calibration coefficients, the average error or bias was reduced and the accuracy of transfer function has increased in comparison to the use of the global calibration values, suggesting that the local calibration procedure has improved the rutting model.

5.3 Recommendations

This research led to the following recommendations for future research:

(i) The database used in this research was selected from widening and reconfiguration highway projects, combining a variety of pavement structures, subgrade soils, materials properties, traffic conditions and a variety of distress levels. In order to enhance the models, improve accuracy and precision, more data is required, in particular, for new pavement sections that are closer to failure.

(ii) Trench studies would be recommended in order to determine the contribution of each layer to total rutting.

(iii) Finally, transverse cracking data should be collected in order to allow the calibration of the thermal cracking models to local conditions.

Appendix A

Traffic Data

Table A-1 - Traffic Inputs

Project #	AADTT	Number of Lanes in Design Direction	% of Trucks in Design Lane	Operational Speed (Km/h)	AADTT Growth Rate (%)
1	3,200	2	80	70	2.15
2	3,000	2	80	70	2.15
7	11,200	3	60	100	2.78
11	17,400	3	60	100	2.05
18	22,000	3	60	100	2.08
24	4,520	2	80	100	2.51
27	11,560	2	80	100	2.08
28	12,000	3	60	100	2.09
38	20,546	3	60	100	2.43
39	20,546	3	60	100	2.43
41	23,400	3	60	100	2.42
43	20,600	3	60	100	2.43
50	18,000	3	60	100	2.06
60	414	1	100	70	2.94
70	9,040	3	60	100	2.35
81	528	1	100	70	2.57

Table A-2 - Distribution by Vehicle Class (%)

Project #	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
1	1.50	14.12	8.21	4.36	1.65	44.88	22.63	0.00	0.00	2.65
2	1.50	14.12	8.21	4.36	1.65	44.93	22.58	0.00	0.00	2.65
7	1.00	8.22	5.37	0.56	1.27	55.69	23.29	0.40	0.20	4.00
11	2.00	17.92	7.95	2.38	1.74	43.59	21.61	0.26	0.19	2.36
18	2.00	18.45	9.36	3.97	1.98	40.29	20.78	0.25	0.17	2.75
24	1.50	13.43	5.29	1.76	1.29	42.81	24.76	0.00	0.20	8.96
27	1.50	13.77	5.84	0.65	1.12	50.98	23.08	0.19	0.13	2.74
28	1.50	16.32	6.77	2.10	1.46	44.87	23.22	0.13	0.13	3.50
38	1.50	10.14	7.60	2.35	1.72	53.60	20.50	0.17	0.17	2.25
39	1.50	10.14	7.60	2.35	1.72	53.60	20.50	0.17	0.17	2.25
41	1.50	10.20	7.57	2.16	1.71	54.10	20.12	0.17	0.17	2.30
43	1.50	10.13	7.58	2.27	1.70	53.93	20.26	0.17	0.17	2.29
50	2.00	18.45	8.98	3.66	1.87	41.64	20.65	0.24	0.18	2.33
60	0.75	15.61	25.36	0.00	9.76	21.69	22.93	0.00	0.00	3.90
70	1.50	18.94	10.03	4.05	1.57	34.58	22.71	0.13	0.18	6.31
81	0.75	23.19	10.14	0.00	0.00	22.44	33.33	0.00	2.90	7.25

Table A-3 - Number of Axles per Truck

Project #	Axles	Vehicle Class									
		4	5	6	7	8	9	10	11	12	13
1	Single	1.62	2.00	1.01	1.41	2.00	1.12	1.62	0.00	0.00	1.15
	Tandem	0.39	0.00	0.99	0.99	0.97	1.94	1.41	0.00	0.00	2.08
	Tridem	0.00	0.00	0.00	0.02	0.00	0.00	0.56	0.00	0.00	0.74
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.04
2	Single	1.62	2.00	1.01	1.41	2.00	1.12	1.62	0.00	0.00	1.15
	Tandem	0.39	0.00	0.99	0.99	0.97	1.94	1.42	0.00	0.00	2.08
	Tridem	0.00	0.00	0.00	0.02	0.00	0.00	0.56	0.00	0.00	0.74
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.04
7	Single	1.62	2.00	1.01	1.09	2.14	1.04	1.26	4.28	2.80	1.11
	Tandem	0.39	0.00	0.99	0.88	0.88	1.98	1.13	0.25	1.60	2.08
	Tridem	0.00	0.00	0.00	0.14	0.00	0.00	0.85	0.00	0.00	0.86
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.02
11	Single	1.62	2.00	1.01	1.36	2.09	1.05	1.44	4.49	2.85	1.14
	Tandem	0.39	0.00	0.99	0.99	0.91	1.97	1.26	0.14	1.50	2.04
	Tridem	0.00	0.00	0.00	0.02	0.00	0.00	0.68	0.00	0.00	0.88
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.00	0.00	0.03
18	Single	1.62	2.00	1.02	1.30	2.12	1.04	1.42	4.46	2.85	1.28
	Tandem	0.39	0.00	0.99	0.99	0.89	1.97	1.25	0.22	1.50	2.06
	Tridem	0.00	0.00	0.00	0.02	0.00	0.00	0.70	0.00	0.00	0.79
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.00	0.00	0.02
24	Single	1.62	2.00	1.01	1.39	2.68	1.05	1.44	0.00	3.00	1.05
	Tandem	0.39	0.00	1.00	1.00	0.58	1.95	1.30	0.00	1.00	2.04
	Tridem	0.00	0.00	0.00	0.00	0.00	0.02	0.72	0.00	0.00	0.94
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.00	0.00	0.01
27	Single	1.62	2.00	1.01	1.27	2.06	1.10	1.55	4.44	3.00	1.58
	Tandem	0.39	0.00	0.99	0.97	0.98	1.95	1.31	0.33	1.67	1.92
	Tridem	0.00	0.00	0.00	0.03	0.00	0.00	0.54	0.00	0.00	0.89
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.19	0.00	0.00	0.02
28	Single	1.62	2.00	1.00	1.47	2.04	1.08	1.47	4.63	2.63	1.26
	Tandem	0.39	0.00	1.00	0.94	0.96	1.96	1.25	0.00	1.63	1.99
	Tridem	0.00	0.00	0.00	0.05	0.00	0.00	0.64	0.00	0.00	0.89
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.13	0.00	0.00	0.03

Table A-3 - Number of Axles per Truck (Continued)

Project #	Axles	Vehicle Class									
		4	5	6	7	8	9	10	11	12	13
38	Single	1.62	2.00	1.01	1.32	2.12	1.03	1.41	4.77	2.94	1.32
	Tandem	0.39	0.00	0.99	0.99	0.87	1.98	1.37	0.14	1.47	2.13
	Tridem	0.00	0.00	0.00	0.03	0.00	0.00	0.59	0.00	0.00	0.72
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.00	0.00	0.03
39	Single	1.62	2.00	1.01	1.32	2.12	1.03	1.41	4.77	2.94	1.32
	Tandem	0.39	0.00	0.99	0.99	0.87	1.98	1.37	0.14	1.47	2.13
	Tridem	0.00	0.00	0.00	0.03	0.00	0.00	0.59	0.00	0.00	0.72
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.00	0.00	0.03
41	Single	1.62	2.00	1.01	1.32	2.13	1.03	1.41	4.77	2.94	1.32
	Tandem	0.39	0.00	0.99	0.99	0.87	1.98	1.37	0.15	1.47	2.13
	Tridem	0.00	0.00	0.00	0.03	0.00	0.00	0.59	0.00	0.00	0.71
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.00	0.00	0.03
43	Single	1.62	2.00	1.01	1.32	2.13	1.03	1.41	4.77	2.94	1.32
	Tandem	0.39	0.00	0.99	0.99	0.87	1.98	1.37	0.14	1.47	2.14
	Tridem	0.00	0.00	0.00	0.04	0.00	0.00	0.59	0.00	0.00	0.71
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.00	0.00	0.03
50	Single	1.62	2.00	1.03	1.30	2.10	1.04	1.42	4.49	2.85	1.13
	Tandem	0.39	0.00	0.98	0.99	0.89	1.97	1.25	0.14	1.50	2.05
	Tridem	0.00	0.00	0.00	0.02	0.00	0.00	0.70	0.00	0.00	0.87
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.00	0.00	0.03
60	Single	1.62	2.00	1.00	0.00	3.80	1.41	1.51	0.00	0.00	1.38
	Tandem	0.39	0.00	1.00	0.00	0.00	1.80	1.45	0.00	0.00	1.75
	Tridem	0.00	0.00	0.00	0.00	0.00	0.00	0.60	0.00	0.00	1.13
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
70	Single	1.62	2.00	1.00	1.21	2.24	1.06	1.46	5.00	2.55	1.08
	Tandem	0.39	0.00	1.00	0.98	0.76	1.95	1.30	0.13	1.73	2.02
	Tridem	0.00	0.00	0.00	0.02	0.00	0.02	0.67	0.00	0.00	0.93
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.00	0.00	0.01
81	Single	1.62	2.00	1.00	0.00	0.00	1.13	1.48	0.00	1.00	1.40
	Tandem	0.39	0.00	1.00	0.00	0.00	2.00	1.30	0.00	1.50	1.20
	Tridem	0.00	0.00	0.00	0.00	0.00	0.00	0.74	0.00	0.00	1.00
	Quad	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Appendix B

Pavement Structure Information

Table B-1 - Pavement Structure Information

Project #	Project Description	Layer Number	Layer Type	Representative Thickness (mm)	% Passing 150 mm	26.5 mm	19 mm	13.2 mm	9.5 mm	4.75 mm (#4)	2 mm (#10)	1.18 mm	0.425 mm (#40)	0.300 mm	0.075 mm (#200)	0.05 mm	0.02 mm	Unit Weight (Kg/m^3)	Va (%)	Vbe (%)	Asphalt Binder	Liquid Limit (LL)	Plastic Limit (PL)	Plastic Index (PI)	Resilient Modulus (Mpa)	Groundwater Table Depth (m)
1	Hwy 6 from Hwy 5 to Concession 6 W	1	DFC	45			100		82.5	52.5					2.5			2520	3.5	12.4	85-100					
		2	HDBC	105			97		63	43.5					3			2460	4	10.9	85-100					
		3	HL8	135			97		63	42.5					3			2460	4	10.9	85-100					
		4	GrA	125		100	100	92	78	66		45		27	14							6	6	0	194	
		5	GrB	790	100	100				75		37		21	13							11	11	0	194	
		6	CL	Semi-infinite						94	91		83		70	33	24					27	15	12	35	6.1
2	Hwy 6 from Hwy 5 to Concession 6 W	1	DFC	50			100		82.5	52.5					2.5			2520	3.5	12.4	85-100					
		2	HDBC	120			97		63	43.5					3			2460	4	10.9	85-100					
		3	HL8	120			97		63	42.5					3			2460	4	10.9	85-100					
		4	GrA	190		100	100	100	98	92		69		32	13							6	6	0	215	
		5	GrB	590	100	75				60		55		33.5	4							11	11	0	215	
		6	CL	Semi-infinite						96	94		86		67	30	24					29	17	12	35	6.1
7	401 at Newcastle EB (L3)	1	SP 12.5	40			100		83.2	54					4			2460	4	11.8	PG 64-28					
		2	SP 19	100		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		3	SP 25	180		100	89.1		63.3	49.3					3.8			2469	4	10.4	PG 58-28					
		4	GrA	300		100	100	78	67	53		36		18	8							6	6	0	250	
		5	GrB	380	100	100				68		52		30	6							11	11	0	250	
		6	ML	Semi-infinite						97	94		87		59	18	14					12	10	2	35	6.1
11	QEW from Brant St to Burloak Dr (L3)	1	SP 12.5	100			100		83.2	54					4			2460	4	11.8	PG 64-28					
		2	SP 19	160		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		3	SP 25	180		100	89.1		63.3	49.3					3.8			2469	4	10.4	PG 58-28					
		4	GrB	380		96	92	89	84	77		66		56	49							11	11	0	128	
		5	CI	Semi-infinite						90	84		77		71	38	29					41	20	21	30	6.1
18	QEW from Winston Churchill Blvd to Trafalgar Rd (WBL4)	1	SP 12.5	50			100		83.2	54					4			2460	4	11.8	PG 64-28					
		2	SP 19	160		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		3	SP 25	200		100	89.1		63.3	49.3					3.8			2469	4	10.4	PG 58-28					
		4	GrB	490		96	92	89	84	77		66		56	49							11	11	0	128	
		5	CH	Semi-infinite						82					59	32	24					39	19	20	50	6.1
24	400 from Hwy 11 to Hwy 93 (Simcoe County)	1	SP 12.5	50			100		83.2	54					4			2460	4	11.8	PG 64-28					
		2	SP 19	120		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		3	SP 19	150		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		4	GrA	520		100	88	80	69	59		46		23	9							6	6	0	216	
		5	GrB	200	100	89				77		63		33	12							11	11	0		
		6	SM	Semi-infinite						99	98		87		42	16	12					18	14	4	30	1.3

Table B-1 - Pavement Structure Information (Continued)

Project #	Project Description	Layer Number	Layer Type	Representative Thickness (mm)	% Passing 150 mm	26.5 mm	19 mm	13.2 mm	9.5 mm	4.75 mm (#4)	2 mm (#10)	1.18 mm	0.425 mm (#40)	0.300 mm	0.075 mm (#200)	0.05 mm	0.02 mm	Unit Weight (Kg/m^3)	Va (%)	Vbe (%)	Asphalt Binder	Liquid Limit (LL)	Plastic Limit (PL)	Plastic Index (PI)	Resilient Modulus (Mpa)	Groundwater Table Depth (m)
27	403 Fiddler's Green Road Interchange	1	DFC	55			100		82.5	52.5					2.5			2520	3.5	12.4	85-100					
		2	HDBC	90			97		63	43.5					3			2460	4	10.9	85-100					
		3	HL8	230			97		63	42.5					3			2460	4	10.9	85-100					
		4	GrA	460		100	99			59		42		21	7							6	6	0	215	
		5	CL	Semi-infinite						100	99		92		83	19	15					26	19	7	35	6.1
28	403 from Aberdeen Avenue to York Blvd (EB)	1	DFC	50			100		82.5	52.5					2.5			2520	3.5	12.4	85-100					
		2	HDBC	150			97		63	43.5					3			2460	4	10.9	85-100					
		3	HL8	160			97		63	42.5					3			2460	4	10.9	85-100					
		4	GrA	690		100	100	81	66	51		30		19	13							6	6	0	234	
		5	CL-ML	Semi-infinite						76	64		57		54	24	17					21	14	7	25	6.1
38	401 from Trafalgar Rd to Regional Rd 25 (EB)	1	DFC	40			100		82.5	52.5					2.5			2520	3.5	12.4	85-100					
		2	HDBC	185			97		63	43.5					3			2460	4	10.9	85-100					
		3	HL8	210		100	97		63	42.5					3			2460	4	10.9	85-100					
		4	GrA	330		100	92	85	78	59		39		24	16							6	6	0	215	
		5	GrB	300	100	75				60		55		33.5	4							11	10	1	215	
		6	CI	Semi-infinite						99	96		89		79	47	34					39	20	19	30	6.1
39	401 from Trafalgar Rd to Regional Rd 25 (WB)	1	DFC	40			100		82.5	52.5					2.5			2520	3.5	12.4	85-100					
		2	HDBC	180			97		63	43.5					3			2460	4	10.9	85-100					
		3	HL8	200			97		63	42.5					3			2460	4	10.9	85-100					
		4	GrA	350		100	100	88	77	61		41		25	16							6	6	0	250	
		5	GrB	300	100	75				60		55		33.5	4							11	10	1	250	
		6	CL-CI	Semi-infinite						93	91		84		75	41	33					36	21	15	30	6.1
41	401 from Credit River to Trafalgar Rd (EB)	1	SP 12.5	60			100		83.2	54					4			2460	4	11.8	PG 64-28					
		2	SP 19	200		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		3	SP 19	190		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		4	GrA	400		100	95	86	78	58		38		24	15							6	6	0	216	
		5	GrB	250	100	75				60		55		33.5	4							14	12	2	215	
		6	CI	Semi-infinite						94	92		87		76	42	30					38	17	21	30	6.1
43	401 from Credit River to Trafalgar Rd (WB)	1	SP 12.5	60			100		83.2	54					4			2460	4	11.8	PG 64-28					
		2	SP 19	160		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		3	SP 25	140		100	89.1		63.3	49.3					3.8			2469	4	10.4	PG 58-28					
		4	GrA	240		100	100	87	76	55		34		23	14							6	6	0	221	
		5	GrB	280	100	100				71		45		30	18							11	11	0	221	
		6	CI	Semi-infinite						97	94		90		78	44	33					40	16	24	30	1.3

Table B-1 - Pavement Structure Information (Continued)

Project #	Project Description	Layer Number	Layer Type	Representative Thickness (mm)	% Passing 150 mm	26.5 mm	19 mm	13.2 mm	9.5 mm	4.75 mm (#4)	2 mm (#10)	1.18 mm	0.425 mm (#40)	0.300 mm	0.075 mm (#200)	0.05 mm	0.02 mm	Unit Weight (Kg/m^3)	Va (%)	Vbe (%)	Asphalt Binder	Liquid Limit (LL)	Plastic Limit (PL)	Plastic Index (PI)	Resilient Modulus (Mpa)	Groundwater Table Depth (m)
50	QEW from Third Line to Burloak Drive (EB)	1	SP 12.5	45			100		83.2	54					4			2460	4	11.8	PG 64-28					
		2	SP 19	120		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		3	SP 19	230		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		4	GrA	495		100	100	93	88	72		44		26	16							6	6	0	186	
		5	CL	Semi-infinite						100					88	35						33	18	15	30	6.1
60	Hwy 12 from County Rd 23 to City of Orillia	1	HL1	40			100		82.5	55					2.5			2520	4	12.4	85-100					
		2	HL2	110			100		100	92.5					5.5			2410	5	14.2	85-100					
		3	HL4	105			100		72	53.5					3			2480	4	12.2	85-100					
		4	GrA	255		100	93	78	62	45		28		14	5							6	6	0	245	
		5	SW-SM	405			100	94	84	75		59		30	11							18	14	4	65	
		6	SM	Semi-infinite		100	97	95	93	90		82		65	41	29	20					18	14	4	25	6.1
70	400 from King Road to South Canal Bridge (SB)	1	SP 12.5	40			100		83.2	54					4			2460	4	11.8	PG 64-28					
		2	SP 19	185		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		3	SP 25	165		100	89.1		63.3	49.3					3.8			2469	4	10.4	PG 58-28					
		4	GrA	320		100	97	93	85	67		51		32	18							6	6	0	185	
		5	GrB	590	100	100				98		97		80	8							11	11	0	185	
		6	CL	Semi-infinite						100	99		96		73	35	23					26	14	12	30	6.1
81	Hwy 12 from Rama Rd to Simcoe/Durham Boundary	1	SP 12.5	30			100		83.2	54					4			2460	4	11.8	PG 64-28					
		2	SP 19	70		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		3	SP 19	60		100	96.9		72.5	52.8					3.9			2460	4	11.2	PG 58-28					
		4	GrB	500	100	100				60		46		24	10							11	11	0	213	
		5	CL	Semi-infinite						96					58	28	21					27	15	12	35	6.1

Appendix C

Pavement Performance Data

Table C-1 - Pavement Performance Data

Project #	Total Section Length (m)	Highway Designation	Average Total Rutting (mm)	Average Total Rutting (in)	Total Area of Alligator Cracking in the section (m^2)	Total Area of section (m^2)	Total Alligator Cracking (%)
1	6,025	6	7.736	0.30	2,047	22,592	9.062%
2	6,100	6	7.056	0.28	1,795	22,875	7.847%
7	11,634	401	4.585	0.18	376	43,629	0.862%
11	2,926	1	3.250	0.13	45	10,972	0.413%
18	3,004	1	3.468	0.14	56	11,267	0.501%
24	9,738	400	2.972	0.12	117	36,518	0.319%
27	7,473	403	4.280	0.17	1,897	28,023	6.771%
28	2,950	403	10.997	0.43	390	11,063	3.521%
38	9,450	401	2.834	0.11	490	35,438	1.381%
39	9,268	401	5.394	0.21	1,737	34,755	4.998%
41	2,404	401	6.444	0.25	337	9,014	3.739%
43	2,193	401	4.364	0.17	406	8,224	4.932%
50	5,032	1	5.798	0.23	63	18,871	0.335%
60	5,542	12	5.057	0.20	2,637	20,783	12.688%
70	12,308	400	3.646	0.14	386	46,156	0.837%
81	15,954	12	3.446	0.14	614	59,829	1.026%

Appendix D

Predicted Distresses from AASHTOWare and Calculations

Table D-1 - Global Calibration - Predicted Alligator Cracking

Project #	Predicted (from M-E Design)				Predicted (by formula)				
	Month	Pavement Age (years)	Max Damage (%)	Max Cracking (%)	h_{AC} (mm)	h_{AC} (in)	C1'	C2'	Predicted Cracking (%)
1	10/2014	15.42	0.0468	0.0296	285	11.2205	4.8799	-2.4400	0.0296
2	10/2014	15.42	0.0333	0.0208	290	11.4173	4.8772	-2.4386	0.0207
7	9/2015	10.08	0.0505	0.0330	320	12.5984	4.8635	-2.4318	0.0330
11	9/2015	4.08	0.0202	0.0131	440	17.3228	4.8371	-2.4186	0.0132
18	9/2015	4.00	0.0254	0.0166	410	16.1417	4.8412	-2.4206	0.0166
24	6/2015	5.83	0.0157	0.0096	320	12.5984	4.8635	-2.4318	0.0096
27	10/2015	8.25	0.0276	0.0179	375	14.7638	4.8477	-2.4238	0.0179
28	10/2014	17.08	0.0485	0.0323	360	14.1732	4.8511	-2.4256	0.0323
38	10/2014	17.08	0.0301	0.0199	435	17.1260	4.8377	-2.4189	0.0200
39	10/2014	17.08	0.0515	0.0350	420	16.5354	4.8397	-2.4199	0.0350
41	5/2015	5.83	0.0107	0.0068	450	17.7165	4.8360	-2.4180	0.0068
43	11/2015	6.33	0.0471	0.0313	360	14.1732	4.8511	-2.4256	0.0313
50	9/2015	6.00	0.0186	0.0119	395	15.5512	4.8437	-2.4219	0.0119
60	9/2015	8.00	0.0055	0.0029	255	10.0394	4.9010	-2.4505	0.0029
70	8/2015	3.17	0.0077	0.0047	390	15.3543	4.8447	-2.4223	0.0047
81	9/2015	5.00	0.0728	0.0340	160	6.2992	5.0896	-2.5448	0.0340

C1 = 1

C2 = 1

Table D-2 - Local Calibration - Predicted Alligator Cracking

Project #	Predicted (from M-E Design)				Predicted (by formula)				
	Month	Pavement Age (years)	Max Damage (%)	Max Cracking (%)	h_{AC} (mm)	h_{AC} (in)	C1'	C2'	Predicted Cracking (%)
1	10/2014	15.42	0.0468	0.0296	285	11.2205	4.8799	-2.4400	3.5693
2	10/2014	15.42	0.0333	0.0208	290	11.4173	4.8772	-2.4386	2.5492
7	9/2015	10.08	0.0505	0.0330	320	12.5984	4.8635	-2.4318	3.8883
11	9/2015	4.08	0.0202	0.0131	440	17.3228	4.8371	-2.4186	1.5953
18	9/2015	4.00	0.0254	0.0166	410	16.1417	4.8412	-2.4206	1.9989
24	6/2015	5.83	0.0157	0.0096	320	12.5984	4.8635	-2.4318	1.2101
27	10/2015	8.25	0.0276	0.0179	375	14.7638	4.8477	-2.4238	2.1607
28	10/2014	17.08	0.0485	0.0323	360	14.1732	4.8511	-2.4256	3.7666
38	10/2014	17.08	0.0301	0.0199	435	17.1260	4.8377	-2.4189	2.3733
39	10/2014	17.08	0.0515	0.0350	420	16.5354	4.8397	-2.4199	4.0236
41	5/2015	5.83	0.0107	0.0068	450	17.7165	4.8360	-2.4180	0.8433
43	11/2015	6.33	0.0471	0.0313	360	14.1732	4.8511	-2.4256	3.6598
50	9/2015	6.00	0.0186	0.0119	395	15.5512	4.8437	-2.4219	1.4604
60	9/2015	8.00	0.0055	0.0029	255	10.0394	4.9010	-2.4505	0.4024
70	8/2015	3.17	0.0077	0.0047	390	15.3543	4.8447	-2.4223	0.6018
81	9/2015	5.00	0.0728	0.0340	160	6.2992	5.0896	-2.5448	4.9019

C1 = 1

C2 = 0.968267437

Table D-3 - Global Calibration - Predicted Total Rutting

			Predicted (from M-E Design)							Measured				Predicted (Global Calibration Coeffiecients)			
Project #	Month	Pavement Age (years)	X_{AC} Predicted AC	X_B Predicted Base	X_{SG} Predicted Subgrade	X_T Predicted Total	AC ratio	Base ratio	Subgrade ratio	Y_{AC} Measured AC	Y_B Measured Base	Y_{SG} Measured Subgrade	Y_T Measured Total (Avg)	$\tilde{Y}_{AC}$ AC	$\tilde{Y}_B$ Base	$\tilde{Y}_{SG}$ Subgrade	$\tilde{Y}_T$ Total
1	10/2014	15.42	4.783	2.497	5.855	13.135	0.364	0.190	0.446	2.817	1.471	3.449	7.736	4.783	2.497	5.855	13.135
2	10/2014	15.42	4.516	2.205	6.380	13.101	0.345	0.168	0.487	2.432	1.188	3.436	7.056	4.516	2.205	6.38	13.101
7	9/2015	10.08	6.294	1.963	9.436	17.693	0.356	0.111	0.533	1.631	0.509	2.445	4.585	6.294	1.963	9.436	17.693
11	9/2015	4.08	2.517	1.049	7.623	11.189	0.225	0.094	0.681	0.731	0.305	2.214	3.250	2.517	1.049	7.623	11.189
18	9/2015	4.00	3.592	1.499	5.512	10.603	0.339	0.141	0.520	1.175	0.490	1.803	3.468	3.592	1.499	5.512	10.603
24	6/2015	5.83	3.358	1.991	8.623	13.972	0.240	0.142	0.617	0.714	0.423	1.834	2.972	3.358	1.991	8.623	13.972
27	10/2015	8.25	3.195	1.224	8.674	13.093	0.244	0.093	0.662	1.044	0.400	2.835	4.280	3.195	1.224	8.674	13.093
28	10/2014	17.08	4.641	2.009	6.739	13.389	0.347	0.150	0.503	3.812	1.650	5.535	10.997	4.641	2.009	6.739	13.389
38	10/2014	17.08	6.419	1.491	5.352	13.262	0.484	0.112	0.404	1.372	0.319	1.144	2.834	6.419	1.491	5.352	13.262
39	10/2014	17.08	7.188	1.125	8.631	16.944	0.424	0.066	0.509	2.288	0.358	2.748	5.394	7.188	1.125	8.631	16.944
41	5/2015	5.83	4.313	1.506	6.078	11.897	0.363	0.127	0.511	2.336	0.816	3.292	6.444	4.313	1.506	6.078	11.897
43	11/2015	6.33	4.153	1.674	7.965	13.792	0.301	0.121	0.578	1.314	0.530	2.520	4.364	4.153	1.674	7.965	13.792
50	9/2015	6.00	3.653	1.359	7.584	12.596	0.290	0.108	0.602	1.681	0.626	3.491	5.798	3.653	1.359	7.584	12.596
60	9/2015	8.00	1.300	2.329	9.035	12.664	0.103	0.184	0.713	0.519	0.930	3.608	5.057	1.3	2.329	9.035	12.664
70	8/2015	3.17	2.179	2.250	5.710	10.139	0.215	0.222	0.563	0.784	0.809	2.054	3.646	2.179	2.25	5.71	10.139
81	9/2015	5.00	1.948	2.431	7.234	11.613	0.168	0.209	0.623	0.578	0.721	2.146	3.446	1.948	2.431	7.234	11.613

Br1	1
Bgb	1
Bsg	1

Table D-4 - Local Calibration - Predicted Total Rutting

			Predicted (from M-E Design)							Measured				Predicted (Local Calibration Coeffiecients)			
			X_{AC}	X_B	X_{SG}	X_T				Y_{AC}	Y_B	Y_{SG}	Y_T	$\bar{Y}_{AC}$	$\bar{Y}_B$	$\bar{Y}_{SG}$	$\bar{Y}_T$
Project #	Month	Pavement Age (years)	Predicted AC	Predicted Base	Predicted Subgrade	Predicted Total	AC ratio	Base ratio	Subgrade ratio	Measured AC	Measured Base	Measured Subgrade	Measured Total (Avg) (mm)	AC	Base	Subgrade	Total
1	10/2014	15.42	4.783	2.497	5.855	13.135	0.364	0.190	0.446	2.817	1.471	3.449	7.736	1.844	1.032	2.181	5.057
2	10/2014	15.42	4.516	2.205	6.380	13.101	0.345	0.168	0.487	2.432	1.188	3.436	7.056	1.741	0.912	2.376	5.029
7	9/2015	10.08	6.294	1.963	9.436	17.693	0.356	0.111	0.533	1.631	0.509	2.445	4.585	2.426	0.812	3.515	6.752
11	9/2015	4.08	2.517	1.049	7.623	11.189	0.225	0.094	0.681	0.731	0.305	2.214	3.250	0.970	0.434	2.839	4.243
18	9/2015	4.00	3.592	1.499	5.512	10.603	0.339	0.141	0.520	1.175	0.490	1.803	3.468	1.385	0.620	2.053	4.057
24	6/2015	5.83	3.358	1.991	8.623	13.972	0.240	0.142	0.617	0.714	0.423	1.834	2.972	1.294	0.823	3.212	5.329
27	10/2015	8.25	3.195	1.224	8.674	13.093	0.244	0.093	0.662	1.044	0.400	2.835	4.280	1.232	0.506	3.231	4.968
28	10/2014	17.08	4.641	2.009	6.739	13.389	0.347	0.150	0.503	3.812	1.650	5.535	10.997	1.789	0.831	2.510	5.130
38	10/2014	17.08	6.419	1.491	5.352	13.262	0.484	0.112	0.404	1.372	0.319	1.144	2.834	2.474	0.617	1.993	5.084
39	10/2014	17.08	7.188	1.125	8.631	16.944	0.424	0.066	0.509	2.288	0.358	2.748	5.394	2.771	0.465	3.215	6.451
41	5/2015	5.83	4.313	1.506	6.078	11.897	0.363	0.127	0.511	2.336	0.816	3.292	6.444	1.662	0.623	2.264	4.549
43	11/2015	6.33	4.153	1.674	7.965	13.792	0.301	0.121	0.578	1.314	0.530	2.520	4.364	1.601	0.692	2.967	5.260
50	9/2015	6.00	3.653	1.359	7.584	12.596	0.290	0.108	0.602	1.681	0.626	3.491	5.798	1.408	0.562	2.825	4.795
60	9/2015	8.00	1.300	2.329	9.035	12.664	0.103	0.184	0.713	0.519	0.930	3.608	5.057	0.501	0.963	3.365	4.829
70	8/2015	3.17	2.179	2.250	5.710	10.139	0.215	0.222	0.563	0.784	0.809	2.054	3.646	0.840	0.930	2.127	3.897
81	9/2015	5.00	1.948	2.431	7.234	11.613	0.168	0.209	0.623	0.578	0.721	2.146	3.446	0.751	1.005	2.694	4.450

Br1	0.385452
Bgb	0.413484
Bsg	0.372472

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Biographical Information

Nelson Fernando Cunha Coelho is Portuguese citizen, born in Oporto. He received his technological course in 1998 from the Technology Professional Center of Industry and Construction in Oporto. Nelson worked for one of the biggest construction companies in Portugal for 4 years and then in 2003 joined one of the world's largest infrastructure and engineering company, participating in several major projects in Portugal. In 2008, he received his Bachelor degree in Occupational Safety Engineering from the University Institute of Maia (Oporto). In 2011, he migrated to the United States of America to participate in some of the world's largest highway projects at Dallas-Fort Worth. In 2012, Nelson completed the requirements of Master of Engineering and Construction Management and received his Master degree from the University of Azores (Ponta Delgada - S. Miguel). In 2013, he started his Masters of Science in Civil Engineering at University of Texas Arlington, in the area of Transportation and Pavement Engineering. He was taught, guided and supervised by Dr. Stefan Romanoschi and graduated in July 2016. Nelson will pursue a doctoral degree in civil engineering and seeks to continue to work on major highway projects across the world.