Comparison of Ontario Pavement Designs Using the AASHTO 1993 Empirical Method and the Mechanistic-Empirical Pavement Design Guide Method

by

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AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners. I understand that my thesis may be made electronically available to the public.

Abstract

The AASHTO 1993 Guide for Design of Pavement Structures is the most widely used pavement design method in both Canada and the United States, and is currently used by the Ministry of Transportation of Ontario (MTO) for both flexible and rigid pavement design. Despite its widespread use, the AASHTO 1993 pavement design method has significant limitations stemming primarily from the limited range of conditions observed at the AASHTO Road Test from which its empirical relationships were derived. The Mechanistic-Empirical Pavement Design Guide (MEPDG) was developed to address the perceived limitations of the AASHTO 1993 Guide. Although the MEPDG provides a rational pavement design procedure with a solid foundation in engineering mechanics, a considerable amount of work is required to adapt and validate the MEPDG to Ontario conditions.

The purpose of this research was to conduct a comparative analysis of Ontario structural pavement designs using the AASHTO 1993 Guide for Design of Pavement Structures and the Mechanistic-Empirical Pavement Design Guide. Historical flexible, rigid, and asphalt overlay pavement designs completed using the AASHTO 1993 pavement design method for the MTO were evaluated using a two-stage procedure. First, the nationally-calibrated MEPDG pavement distress models were used to predict the performance of the pavements designed using the AASHTO 1993 method. The purpose of this stage of the analysis was to determine whether the two methods predicted pavement performance in a consistent manner across a range of design conditions typical of Ontario. Finally, the AASHTO 1993 and MEPDG methods were compared based on the thickness of the asphalt concrete or Portland cement concrete layers required to satisfy their respective design criteria.

The results of the comparative analysis demonstrate that the AASHTO 1993 method generally over-predicted pavement performance relative to the MEPDG for new flexible pavements and asphalt overlays of flexible pavements. The MEPDG predicted that most of the new flexible pavements and asphalt overlays of flexible pavements designed using the AASHTO 1993 method would fail primarily due to permanent deformation and / or roughness. The asphalt layer thicknesses obtained using the MEPDG exceeded the asphalt layer thicknesses obtained using the AASHTO 1993 method, and a poor correlation was observed between the asphalt layer thicknesses obtained using the two methods. Many of the new flexible pavements and asphalt overlays of existing flexible pavements could not be re-designed to meet the MEPDG performance criteria by increasing the asphalt layer thickness.

The results of the comparative analysis showed that the AASHTO 1993 method generally under-predicted rigid pavement performance relative to the MEPDG, although the results varied widely between alternative rigid pavement designs. The AASHTO 1993 rigid pavement designs that the MEPDG predicted would not meet the rigid pavement performance criteria generally failed due to pavement roughness. A very poor correlation was observed between the Portland cement concrete layer thicknesses obtained using the MEPDG and AASHTO 1993 design methods. The MEPDG predicted thinner Portland cement concrete layer thicknesses than the AASHTO 1993 design method for most of the rigid pavement designs.

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Chapter 1
Introduction

1.1 Background

The AASHTO 1993 Guide for Design of Pavement Structures is the most widely used pavement design method in both Canada and the United States. In a 2007 survey of all fifty US State Departments of Transportation (DOTs) conducted by the United States Federal Highway Administration (FHWA), 63% reported using the AASHTO 1993 Guide, 12% reported using the earlier AASHTO 1972 Guide, and 8% reported using a combination of the AASHTO 1993 Guide and a local state design procedure for flexible pavement design. For rigid pavement design, 55% of State DOTs reported using the AASHTO 1993 Guide (either with or without the 1998 supplement), 10% reported using a combination of AASHTO 1993 and a local state agency procedure, and 6% reported using either the 1972 or 1986 AASHTO Guide (Crawford 2009). Similarly, a recent survey of Canadian Federal and Provincial Departments of Transportation found that 75% used the AASHTO 1993 Guide for the design of flexible pavement structures, and 100% used the AASHTO 1993 method (either with or without the 1998 supplement) for rigid pavement design (TAC 2011). The AASHTO 1993 Guide is also currently used by the Ministry of Transportation of Ontario (MTO) for both flexible and rigid pavement design.

Despite its widespread use, the AASHTO 1993 pavement design method has some significant limitations. The AASHTO 1993 method is based primarily on the empirical models developed from field performance data from the AASHTO Road Test. The AASHTO Road Test was a series of experiments designed to determine how traffic loading contributed to pavement deterioration and loss of serviceability. The last major experiment, which became the basis for the various versions of the AASHTO Guide for Design of Pavement Structures, was conducted from 1958 – 1960 in the vicinity of Ottawa, Illinois. The primary limitations of the AASHTO 1993 pavement design method stem from its empirical nature. The AASHTO Road Test was conducted under a limited range of conditions, including: relatively low traffic loading; a single climate; a single subgrade material; and, construction materials and methods characteristic of the 1950s. The relationships between traffic loading and pavement functional performance developed at the AASHTO Road Test are directly applicable only to the specific set of conditions observed at the AASHTO Road Test. The AASHTO 1993 method extrapolates these relationships for conditions that were not observed at the AASHTO Road Test; however, this requires assumptions that may compromise the accuracy of the method. In addition, the AASHTO 1993 Guide employs only a single pavement performance indicator, the present serviceability index, which is based on road users' subjective evaluation of the pavement functional performance. As a result, the AASHTO 1993 method does not have the capability to assess how a pavement structural design will perform with respect to the common pavement distresses that typically dictate pavement maintenance and rehabilitation schedules, and are thus of greatest concern to modern transportation engineers and highway agencies.

The Mechanistic-Empirical Pavement Design Guide (MEPDG) was developed to address the perceived limitations of the AASHTO 1993 pavement design method. In March 1996, the AASHTO Joint Task Force on Pavements, the National Cooperative Highway Research Program (NHCRP), and the Federal Highway Administration (FHWA) proposed a research program to develop a pavement design guide based on mechanistic-empirical principles. The development of the new pavement design guide began in 1998 under the National Cooperative Highway Research Program Project 1-37A (TAC, 2011). The final report was completed and published in 2004, followed by the publication of the AASHTO Mechanistic-Empirical Pavement Design Guide in 2008 (ARA 2004, AASHTO 2008). The most recent version of the MEPDG design software is the AASHTOWare Pavement ME Design Software, which automates the pavement design procedure outlined in the MEPDG. It is anticipated that the MEPDG will replace the AASHTO 1993 Guide as the standard pavement design method for the foreseeable future.

Although the MEPDG provides a rational pavement design procedure with a solid foundation in engineering mechanics, Ontario transportation agencies must exercise due diligence and care during its implementation. A significant amount of work will be required to ensure that pavement design inputs accurately reflect Ontario conditions and produce consistent and realistic pavement performance predictions. In particular, the empirical models in the MEPDG that relate mechanistic pavement structural responses to predicted pavement distresses were nationally calibrated using pavement sections primarily from the Long-Term Pavement Performance (LTPP) database. Since these pavement sections were located primarily throughout the United States, the nationally-calibrated empirical pavement distress models may not be representative of pavement performance based on the typical traffic loading, climatic conditions, and construction materials and methods in Ontario.

MTO is currently working toward the adoption and validation of the MEPDG and associated AASHTOWare Pavement ME design software for Ontario conditions. MTO currently has well-established pavement design practices based on the AASHTO 1993 method that have been adapted and verified to reflect Ontario conditions (ERES, 2008). A prudent step in the MEPDG implementation process is to compare typical Ontario pavement designs obtained using the MEPDG and the AASHTO 1993 method using the MEPDG nationally-calibrated pavement distress models.

1.2 Purpose

The purpose of this research was to conduct a comparative analysis of Ontario structural pavement designs using the AASHTO 1993 Guide for Design of Pavement Structures and the Mechanistic-Empirical Pavement Design Guide.

1.3 Objectives

The three objectives of this research were:

1. Provide a comprehensive literature review summarizing research that has: compared pavement structural designs produced using the AASHTO 1993 and MEPDG pavement design methods; examined the sensitivity of the MEPDG to various pavement design inputs; and, verified the accuracy of the nationally-calibrated MEPDG pavement distress models.
2. Determine whether the MEPDG and AASHTO 1993 pavement design methods predict pavement performance in a consistent manner across a range of pavement types and design conditions typical of Ontario.
3. Determine how the MEPDG and AASHTO 1993 pavement design methods compare in terms of the pavement structural designs produced for a range of pavement types and design conditions typical of Ontario.

1.4 Research Tasks

The following tasks were undertaken as part of this research to accomplish the research objectives (see Figure 1-1):

1. Provide a comprehensive literature review of research that has: compared pavement structural designs obtained using the AASHTO 1993 and MEPDG pavement design methods; examined the sensitivity of the MEPDG to various design inputs; and, verified

the accuracy of the MEPDG nationally-calibrated pavement distress models using pavement performance data.

2. Select a set of historical MTO pavement structural designs obtained using the AASHTO 1993 method that are representative of the range of pavement types, traffic loading, climatic conditions, and construction materials / methods typical of Ontario.

3. Determine appropriate MEPDG pavement design inputs to reflect Ontario conditions.

4. Examine the predicted pavement performance of the historical MTO flexible, rigid, and asphalt overlay pavement sections designed using the AASHTO 1993 pavement design method using the nationally-calibrated MEPDG pavement distress prediction models to determine if the two pavement design methodologies predict pavement performance in a consistent manner across a range of design conditions.

5. Compare typical Ontario flexible, rigid, and asphalt overlay pavement designs completed using the AASHTO 1993 and MEPDG pavement design methods on the basis of asphalt concrete or Portland Cement Concrete thickness.

6. Summarize study conclusions and provide recommendations for future study.

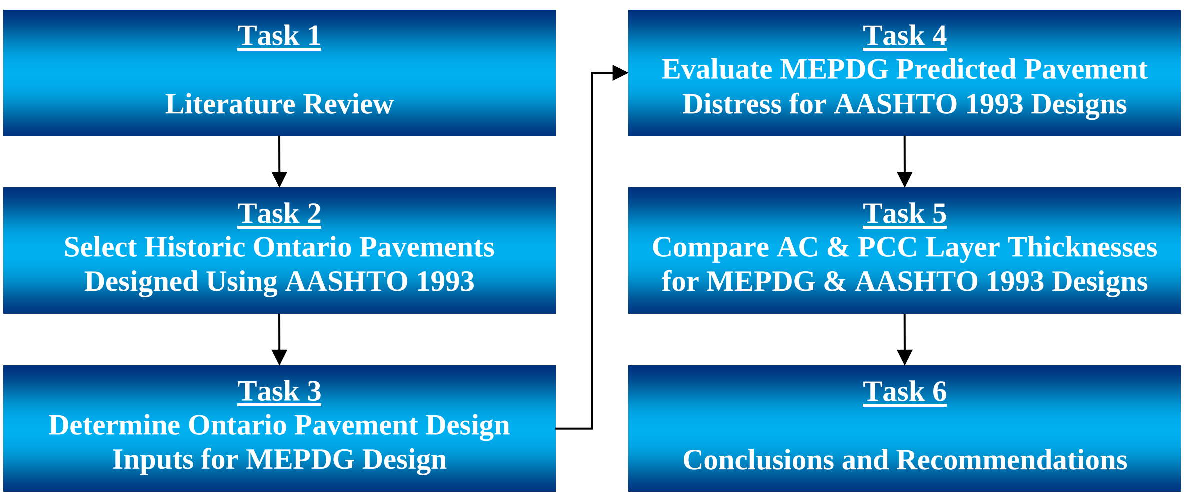


Figure 1-1: Research Tasks

1.5 Organization

The thesis is organized into the following five chapters:

1. Chapter One introduces the project and outlines the purpose, scope, objectives, research tasks, and organization of the thesis.

2. Chapter Two provides a literature review that examines: (a) the AASHTO 1993 and MEPDG pavement design methods; (b) studies comparing pavement structural designs obtained using the AASHTO 1993 and MEPDG pavement design methods; (c) studies examining the sensitivity of the MEPDG pavement distress models to various traffic, climatic, and material design inputs; and, (d) studies verifying the accuracy of the nationally-calibrated MEPDG performance models to predict local pavement performance.

3. Chapter Three describes the historical pavement design data and pavement design inputs used to complete this study.

4. Chapter Four compares the AASHTO 1993 and MEPDG pavement design methods based on MEPDG-predicted pavement performance and the thickness of the asphalt concrete / Portland cement concrete layers.

5. Chapter Five provides the research conclusions and recommendations for future research.

Chapter 2

Literature Review

2.1 Introduction

Chapter 2 presents the results of the comprehensive literature review conducted as part of this research and is organized as follows:

- Section 2.2 provides an overview of typical approaches to pavement design including the AASHTO 1993 and MEPDG pavement design methods.
- Section 2.3 summarizes studies that have compared pavement structural designs completed using the AASHTO 1993 and MEPDG pavement design methods.
- Section 2.4 summarizes the results of studies that have examined the sensitivity of the MEPDG pavement distress models to various traffic, climate, materials, and pavement structure design inputs.
- Section 2.5 summarizes studies that have verified the accuracy of the nationally-calibrated MEPDG pavement distress models using local pavement performance data.

2.2 Pavement Design Methods

2.2.1 Pavement Design Method Approaches

There are a number of alternative approaches to the design of pavement structures. Pavement design methods generally fall into the following four categories (TAC 2011):

1. Standard Sections
2. Empirical Pavement Design Methods
3. Mechanistic Pavement Design Methods
4. Mechanistic-Empirical Pavement Design Methods

Standard section pavement design methods select an appropriate pavement design for given set of design conditions based on experience of past performance. The primary limitation of these methods is that they are only applicable to the specific set of conditions under which they were developed (TAC 2011). This method is suitable for low-volume highways where the likelihood of traffic characteristics changing over the design life of the pavement is low and the risk of premature pavement failure is low.

Empirical pavement design methods are based solely on the results of experiments or experience. Observations of pavement responses to known traffic loading and subgrade conditions are used to establish correlations between pavement design inputs and pavement performance. The primary advantage of empirical methods is that they avoid the issue of defining theoretically the complex cause-effect relationship between pavement design and observed pavement distresses. The primary disadvantage of empirical pavement design methods is that the validity of the relationships is limited to the conditions under which they were observed. Extrapolating these relationships to other conditions requires assumptions that may undermine the accuracy of the method. The most commonly used empirical method for designing new and rehabilitated pavements in Canada and the United States is the AASHTO 1993 Guide for Design of Pavement Structures (AASHTO 1993, TAC 2011).

Mechanistic pavement design methods use the theories of engineering mechanics to relate traffic loading and environmental conditions to pavement structural behaviour and performance. Mechanistic methods determine stresses, strains, and deflections at critical points in the pavement structure based on specified traffic loading and environmental conditions. The pavement structure is modelled as a multi-layered linear elastic system to capture the dynamic responses of the various pavement materials. One disadvantage of mechanistic pavement design models is that they are strictly theoretical and do not incorporate observed pavement performance in the field. In addition, the assumption of linear-elastic material behaviour is generally incompatible with the prediction of non-linear inelastic pavement distresses (Carvalho & Schwartz 2006). Since pavement performance is defined in terms of pavement distresses and not pavement structural responses, this is a significant limitation of purely mechanistic pavement design methods. For these reasons, attempts to develop fully mechanistic pavement design approach have generally been unsuccessful to-date (Carvalho et al. 2006).

Mechanistic-empirical pavement design methods afford the advantages of mechanistic pavement design while addressing its primary limitations. The mechanic component of the model calculates pavement structural responses (i.e. stresses, strains, deflections) resulting from traffic loading, environmental conditions, and material properties. These pavement responses are then related to pavement performance through the use of empirical pavement distress prediction models. The empirical distress prediction models are developed and calibrated using observed pavement performance in the field. The most comprehensive mechanistic-empirical pavement design method is the Mechanistic-Empirical Pavement Design Guide (MEPDG), which was recently developed under NCHRP Project 1-37A (ARA, 2004).

2.2.2 AASHTO 1993 Empirical Method

The AASHTO 1993 pavement design method is based primarily on the empirical models developed from field performance data observed at the AASHTO Road Test. The AASHTO Road Test was a series of experiments designed to determine how traffic loading contributed to pavement deterioration and loss of serviceability. The last major experiment, conducted from 1958 – 1960 in the vicinity of Ottawa, Illinois, became the basis for the AASHTO Interim Guide for Design of Pavement Structures published in 1972 (AASHTO 1972). A subsequent version entitled AASHTO Guide for Design of Pavement Structures was published in 1986 and contained a number of notable additions to the method, including improved materials characterization. The AASHTO 1993 Guide further enhanced the method by including a section on pavement rehabilitation design, and new methods to account for the impact of drainage and environmental conditions in pavement design. The AASHTO 1993 Guide includes separate design procedures for flexible and rigid pavements.

2.2.2.1 AASHTO 1993 Flexible Pavement Design

The AASHTO 1993 flexible pavement design method is based fundamentally on the relationship between traffic loading, subgrade strength, and the functional performance of the pavement. The AASHTO 1993 method uses Equation 2-1 for the design of flexible pavements (AASHTO 1993).

$$\log_{10} W_{18} = Z_R S_0 + 9.36\log(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{\left[0.4+\frac{1094}{(SN+1)^{5.19}}\right]} + 2.32\log_{10} M_{R-8.07} \quad \text{Equation 2-1}$$

Where:

W_{18} = predicted number of 18-kip (80kN) equivalent single axle load applications

Z_R = standard normal deviate

S_0 = combined standard error of the traffic prediction and performance prediction

ΔPSI = initial serviceability index (p_o) minus terminal serviceability index (p_t)

SN = Structural Number

M_R = subgrade resilient modulus

To complete a flexible pavement design using the AASHTO 1993 method, the pavement designer must first determine the representative resilient modulus of the underlying subgrade materials (M_R). This can be determined either directly through laboratory testing of representative samples of subgrade material, or assumed based on soil classification and anticipated drainage conditions. The designer must also determine the cumulative traffic loading experienced over the performance period of the pavement (W_{18}). The AASHTO 1993 method characterized traffic loading in terms of number of Equivalent Single Axle Loads (ESALs). An ESAL represents the damage experienced by a pavement structure as a result of loading from an 18,000 lb. single axle. All traffic loading from a mixed stream of traffic of different axle loads and axle configurations predicted over the design life of the pavement is converted into an equivalent number of ESALs for design. The designer must also select a suitable value for design reliability. Reliability represents the probability that the pavement design will meet or exceed its design life, and is typically based on the highway functional classification and the risk associated with premature failure of the pavement. Finally, the designer must select the deterioration rate in terms of loss of serviceability (ΔPSI). The AASHTO 1993 design method characterized pavement performance solely in terms of functional performance as measured using the Pavement Serviceability Index (PSI). Equation 2-2 is used to calculate PSI (TAC 2011):

$$PSI = 5.03 - 1.91\log(1 + SV) - 0.01(C + P)^{0.5} - 1.38RD^2 \qquad \text{Equation 2-2}$$

Where:

SV = longitudinal cracking in the wheel path

C = cracked area

P = patched area

RD = average rut depth for both wheel paths

As shown in Equation 2-2, PSI is a composite performance measure that is influenced primarily by pavement roughness. The selection of suitable initial and terminal serviceability values is typically dependent on highway functional class and local agency policy. The output of the AASHTO 1993 flexible pavement design method is a Structural Number (SN) required for the pavement to function adequately over the design period at the specified level of reliability. The pavement SN is related to pavement layer thicknesses and drainage conditions using Equation 2-3 (AASHTO 1993):

$$\mathrm{SN} = \mathrm{a}_1\mathrm{D}_1 + \mathrm{m}_2\mathrm{a}_2\mathrm{D}_2 + \mathrm{m}_3\mathrm{a}_3\mathrm{D}_3$$ Equation 2-3

Where:

a_i = structure layer coefficient (e.g. 0.42 asphalt, 0.14 granular base, etc.)

m_i = drainage layer coefficient (e.g. 1.0 good drainage, 0.9 fair drainage, etc.)

D_i = layer thickness

The designer must select the individual pavement layer thicknesses to satisfy the required SN with consideration to producing a cost-effective design.

2.2.2.2 AASHTO 1993 Rigid Pavement Design

The basic procedure of the AASHTO 1993 rigid pavement design is very similar to the flexible pavement design method, albeit with a number of different design input parameters required. The AASHTO 1993 method uses Equation 2-4 for the design of rigid pavements (AASHTO 1993).

$$\log_{10} W_{18} = Z_R\, S_O + 7.35 \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1 + \frac{1.624\times 10^7}{(D+1)^{8.46}}}$$

$$+(4.22 - 0.32 p_t)\log_{10}\left[\frac{S'_c C_d\left(D^{0.75} - 1.132\right)}{215.63 J\left(D^{0.75} - \frac{18.42}{(E_c \div k)^{0.25}}\right)}\right]$$ Equation 2-4

Where:

W_{18} = predicted number of 18-kip (80kN) equivalent single axle load applications

Z_R = standard normal deviate

S_0 = combined standard error of the traffic prediction and performance prediction

ΔPSI = initial serviceability index (p_o) minus terminal serviceability index (p_t)

D = thickness of concrete pavement slab

S'_c = modulus of rupture for Portland cement concrete (psi)

J = load transfer coefficient

C_D = drainage coefficient

E_C = modulus of elasticity for Portland cement concrete (psi)

K = modulus of subgrade reaction (pci)

Many of the design inputs used in AASHTO 1993 rigid pavement design are the same as those used in the AASHTO 1993 flexible pavement design method. Unlike the AASHTO 1993 flexible pavement design method, subgrade support in AASHTO 1993 rigid pavement design is characterized in terms of modulus of subgrade reaction (k); the modulus of subgrade reaction can be determined either directly through laboratory tests or through correlation with soil classification and anticipated drainage characteristics. The drainage coefficient (C_D) is selected by the designer to account for the difference in the quality of drainage relative to the drainage conditions at the AASHTO Road Test. The load transfer coefficient (J) is used to account for the overall quality of load transfer between slabs, and has recommended values based on type of reinforcement, load transfer devices, and shoulders used in the pavement design. The remaining inputs are characteristics of the concrete mix which can be determined directly using laboratory tests or estimated using correlations with other concrete properties. The output of the AASHTO 1993 rigid pavement design method is the thickness of the concrete slab (D).

2.2.2.3 AASHTO 1993 Limitations

The fact that the AASHTO 1993 design method has functioned adequately for over 60 years is a testament to the robustness of the method. However, the method does have significant limitations due primarily to the empirical nature of the relationships employed and the limited range of conditions present at the AASHTO Road Test. The primary limitations of the AASHTO 1993 design method cited in the literature are (Ahammed, Kass, Hilderman, & Tang 2011, Bayomy, El-Badawy, & Awed 2012, Carvalho et al. 2006, Hall & Beam 2005, and Schwartz 2007):

- The AASHTO Road Test did not consider pavement rehabilitation procedures. These are of primary importance to the modern pavement engineer due to the high proportion of pavement rehabilitation work relative to new construction, widening and reconstruction.
- The AASHTO Road Test duration was only 2 years, and so did not account for the effects of material aging on pavement performance.
- The AASHTO Road Test experienced a maximum traffic loading of approximately 2 million ESALs. The relationships derived from this very limited range of traffic loading

have been extrapolated to design pavements with traffic loading in excess of 200 million ESALs.

- The AASHTO Road Test employed vehicles with characteristics that differ widely from modern vehicles in terms of vehicle suspension, axle configurations, tire pressures, tire types, etc.
- The AASHTO Road Test used vehicles with identical axle loads and configurations, which is not representative of the mixed traffic conditions on highways.
- The AASHTO Road Test was conducted at a single geographic location. The method must make assumptions to account for the impact of different climatic conditions on predicted pavement performance.
- The AASHTO Road Test considered only limited asphalt concrete properties and binders (i.e. no Superpave, Stone Mastic Asphalt, etc.).
- The AASHTO Road Test main pavement sections all had unstabilized, dense granular bases. The method must make assumptions to account for the impact of different base layers (i.e. stabilized base layers) on pavement performance.
- The AASHTO Road Test contained only one type of subgrade soil (AASHTO Soil Classification A-6 clay). The method must make assumptions to account for the impact of different subgrade soils on pavement performance.
- The AASHTO Road Test used pavement designs, materials, and construction methods characteristic of the 1950s (i.e. no subdrains), which are not representative of those employed today.
- The AASHTO 1993 design equations do not account for variation in material stiffness due to changes in applied load or stress.
- The AASHTO 1993 method relates structural integrity exclusively to pavement thickness. However, pavement thickness is not the primary variable influencing all pavement distresses.

- The AASHTO Road Test and AASHTO 1993 design method consider only one subjective performance measure (loss of serviceability), which frequently does not dictate when pavement maintenance and rehabilitation is required.

In addition, it is reported that the AASHTO 1993 Guide produces overly thick pavement designs for highways with high traffic loading, especially for rigid pavement design (Li, Uhlmeyer, Mahoney, & Muench 2011, TAC 2011). It has also been reported that the AASHTO 1993 method produces pavement designs that vary widely in observed performance (Mallela, Glover, Darter, Von Quintus, Gotlif, Stanley & Sadasivam 2009). The perceived limitations of the AASHTO 1993 pavement design method were the driving force behind the development of a more comprehensive mechanistic-empirical pavement design method under NCHRP Project 1-37A.

2.2.3 Mechanistic-Empirical Pavement Design Guide Method

The development of the MEPDG began in 1998 under the National Cooperative Highway Research Program Project 1-37A (TAC, 2011). The final report was completed and published in 2004, followed by the publication of the AASHTO Mechanistic-Empirical Pavement Design Guide in 2008 (ARA 2004, AASHTO 2008). The objective of the MEPDG was "to provide the highway community with a state-of-the-practice analysis tool for the design and analysis of new and rehabilitated pavement structures based on mechanistic-empirical principles" (AASHTO 2008). A detailed explanation of the MEPDG pavement design method can be found in these sources; this section provides an overview of the MEPDG design approach.

The MEPDG represents a fundamental change in the philosophy and methodology of pavement design. Figure 2-1 illustrates the MEPDG pavement design method (Schwartz 2007).

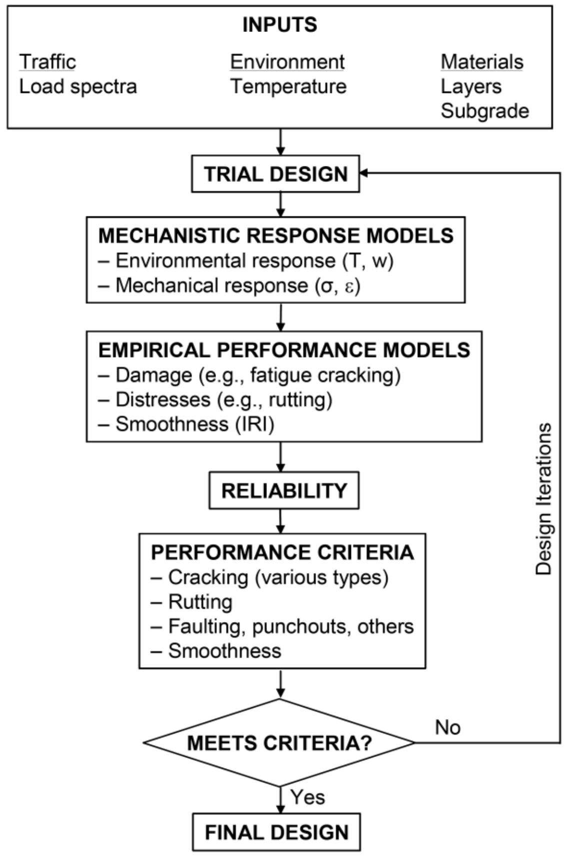


Figure 2-1: Mechanistic-Empirical Pavement Design Method (Schwartz 2007)

As shown in Figure 2-1, the pavement designer is required to provide inputs related to the traffic loading, climate, and material properties. The designer must then input a trial pavement design to be evaluated by the MEPDG pavement performance models. Next, pavement structural responses (i.e. stress, strain, and deflection) are calculated at critical locations within the pavement structure using mechanistic models (multi-layer elastic theory or finite element analysis) based on the principles of engineering mechanics. Environmental responses, including the distribution of heat and moisture throughout the pavement structure, are also determined mechanistically through the use of the Enhanced Integrated Climate Model (EICM). The output from these mechanistic models is then provided as input to the empirical performance models that relate these responses to observed pavement distresses such as permanent deformation, fatigue cracking, thermal cracking, joint faulting, and pavement roughness. The empirical models included in the MEPDG were calibrated using hundreds of pavement test sections across the United States, primarily from the LTPP database. The MEPDG output consists of the predicted pavement distresses for the trial section based on the specified level of reliability. The reliability level represents the probability that the predicted performance indicator of the trail design will not exceed the design criteria within the design-analysis period (AASHTO 2008). The MEPDG predicted pavement distresses are then compared to the performance criteria established by the highway agency for each of the pavement distresses to determine whether the trial design meets the design criteria. If the trial design meets the specified performance criteria, it is an acceptable design from a structural and functional perspective. If the trial design does not meet the specified criteria, the process must be repeated with a revised trial pavement design to address the specific deficiencies observed in the original trial design (ARA 2004, AASHTO 2008, Schwartz 2007).

One significant difference between the MEPDG and AASHTO 1993 design methods is the characterization of design inputs. The MEPDG employs a hierarchical approach to pavement design inputs; this approach is designed to reflect the practical consideration that the level of effort and overall cost expended in completing a pavement design should be proportional to the importance of the project and risk of premature pavement failure. Level 1 inputs provide the highest degree of accuracy and reliability, but typically require detailed project-specific laboratory testing and field measurements to characterize traffic, materials, and climate. Level 2 inputs represent an intermediate level of accuracy, and are typically obtained from limited laboratory and field testing or estimated using correlations or transportation agency experience. Level 3 inputs provide the lowest level of accuracy and highest level of uncertainty, and are typically based on default values derived from the transportation agency's experience at a regional level (ARA 2004, AASHTO 2008, Schwartz 2007). Transportation agencies are able to utilize different input levels for different inputs within the same pavement design.

The MEPDG also requires significantly more design inputs at a much higher level of detail than the AASHTO 1993 method, even when using the Level 3 input level. A primary challenge for transportation agencies in implementation of the MEPDG is to develop strategies to modify existing data collection and storage procedures to reflect the data required for MEPDG pavement design. This includes prioritizing which data is critical and cost-effective to collect, and which inputs can be specified at lower input levels without compromising pavement performance.

Another significant challenge for transportation agencies seeking to implement the MEPDG is the need to verify, calibrate, and validate the empirical pavement distress models to reflect local conditions. As previously mentioned, the empirical pavement distress models that relate pavement structural and environmental responses in the MEPDG were calibrated using hundreds of pavement test sections across the United States primarily from the Long Term Pavement Performance (LTPP) database. As such, they may not be representative of the traffic loading patterns, climate, materials, and construction methods for each local transportation agency. Local transportation agencies must first determine the suitability of the nationally-calibrated MEPDG models to reflect local conditions, and if necessary, recalibrate and validate the models based on local conditions. Many State DOTs have completed studies examining the MEPDG and adapting it to suit local conditions. These studies typically include one or more of the following activities:

- Compare pavement designs obtained using the current state pavement design method (typically AASHTO 1993) and the nationally-calibrated MEPDG
- Determine the sensitivity of the MEPDG to the various traffic, climate, material, and pavement structure inputs required for pavement design
- Verify the accuracy of the nationally-calibrated MEPDG pavement distress models based on local pavement designs and performance data, typically from the local Pavement Management System (PMS), and re-calibrate the models as necessary.

The following sections summarize the most relevant findings from these studies.

2.3 Comparisons of AASHTO 1993 and MEPDG Pavement Designs

Since to the completion of NCHRP Project 1-37A in 2004, a number of studies have directly compared pavement structures obtained using the AASHTO 1993 and MEPDG pavement design methods. This section provides an overview of their key findings.

Timm (2006) conducted a direct comparison of rigid pavement slab thicknesses obtained using the NCHRP 1-37A (M-E Design Guide Release 3) and AASHTO 1993 methods. The study examined 125 design scenarios encompassing a range of design inputs. Rigid pavement designs were produced for traffic loading scenarios of 5 million and 10 million ESALs, and reliability levels of 50% and 90%. Direct conversions were made between input variables for traffic, material properties, load transfer coefficient, and pavement performance where possible; otherwise, default values were assumed for the NCHRP 1-37A method. Timm found that the slab thicknesses designed using the NCHRP 1-37A method were 9% thinner on average than those designed using AASHTO 1993 for reasonable serviceability loss ($\Delta PSI = 1.2$) (see Figure 2-2). NCHRP 1-37A slab thicknesses were 5%

thinner on average when all serviceability loss scenarios were considered (ΔPSI = 0.2, 0.7, and 1.2). As shown in Figure 2-2, a strong correlation was observed between slab thicknesses obtained using the two pavement design methods.

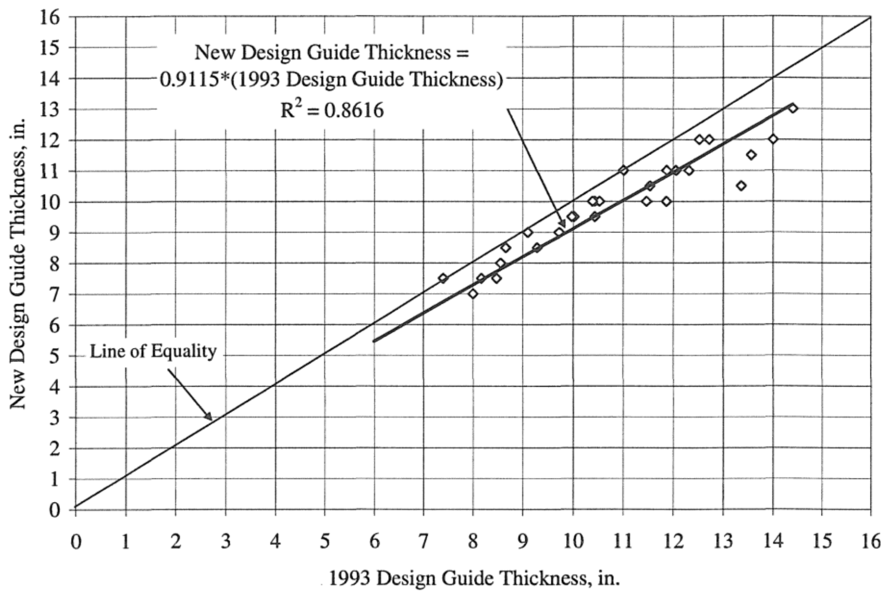


Figure 2-2: AASHTO 1993 vs. MEPDG PCC Slab Thickness ΔPSI = 1.2 (Timm 2006)

Carvalho et al. (2006) compared flexible pavement designs and performance between the NCHRP 1-37A method (M-E version 0.700) and AASHTO 1993. Flexible pavement designs were completed for five locations selected to be representative of the range of climates, subgrades, material properties, and local design preferences in the United States. All of the flexible pavement designs consisted of asphalt layers over granular base layers. Three traffic loading scenarios were examined: low (3.8 million ESALs), medium (15 million ESALs), and high (55 million ESALs). A design reliability of 95% was used for both the AASHTO 1993 pavement designs and MEPDG pavement performance predictions. Carvalho et al. correctly noted the difficulty of a direct comparison of the two methods due to the disparity in the number and detail of design inputs required between the two methods, the dependence of design thickness on specified design criteria, and the ability for multiple designs to satisfy the same performance criteria. Instead of design thickness, this study examined the M-E predicted performance of flexible pavements designed using the AASHTO 1993 method based on the same loss of serviceability. It was assumed that the flexible pavements designed for the same loss of serviceability with the AASHTO 1993 method should exhibit similar predicted performance in the M-E models, and any discrepancies would be indicative of one design method being more conservative than the other. It was also assumed that the M-E design method was the more accurate of the two based on the extent of its national calibration. Based on the above, the following conclusions were reached:

- The AASHTO 1993 method underestimated rutting and bottom-up fatigue cracking (i.e. overestimated performance) for flexible pavements in warm climates
- The AASHTO 1993 method underestimated rutting and bottom-up fatigue cracking (i.e. overestimated performance) for flexible pavements with high traffic loading (i.e. 55 million ESALs) (see Figure 2-3)
- The AASHTO 1993 designs were less reliable at higher traffic levels as demonstrated through increased variability in predicted pavement distresses
- The AASHTO 1993 designs had low variability in predicted pavement distresses for pavements with low traffic loading and low to moderate temperatures

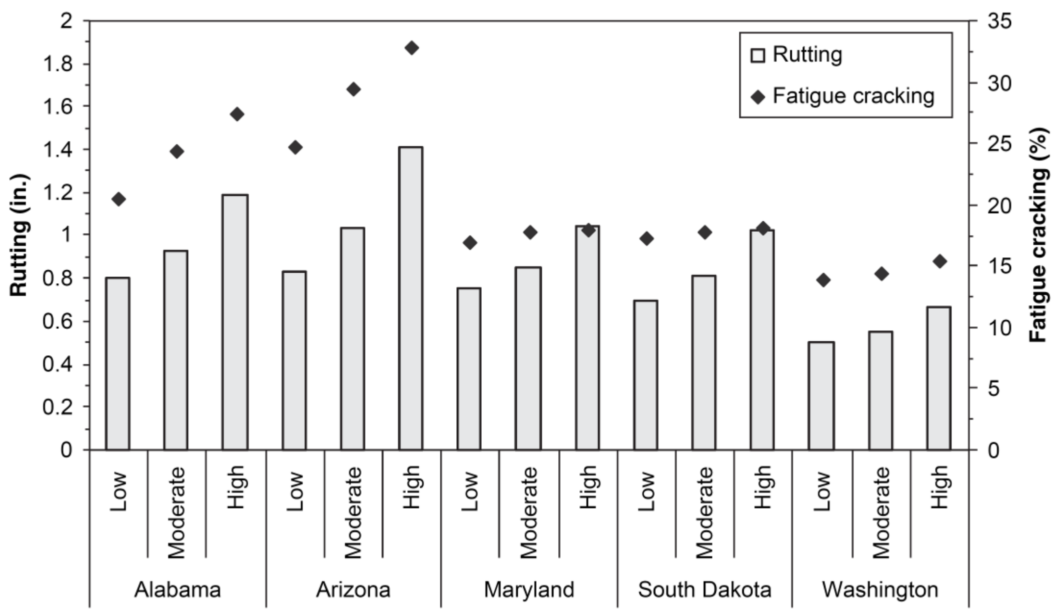


Figure 2-3: Effect of Traffic Loading on MEPDG-Predicted Pavement Distresses in Flexible Pavements (Carvalho et al. 2006)

Ahammed et al. (2011) examined the predicted performance of a typical Manitoba flexible pavement designed using the AASHTO 1993 method using the nationally-calibrated MEPDG pavement distress models. The flexible pavement consisted of asphalt concrete over Granular A base over Granular C subbase. Four traffic loading scenarios were analyzed: 4.3 million ESALs, 8.6

million ESALs, 17.3 million ESALs, and 28.8 million ESALs. A separate AASHTO 1993 flexible pavement design was prepared for each traffic scenario. MEPDG traffic inputs consisted of Manitoba-specific axle load spectra, and MEPDG climate data was obtained from the Winnipeg climate station. A 90% reliability level was used for both the AASHTO 1993 and MEPDG pavement design methods. All of the AASHTO 1993 designs were unable to satisfy the MEPDG performance criteria, indicating that the AASHTO 1993 method overestimated flexible pavement performance relative to the nationally-calibrated MEPDG. Terminal IRI and total permanent deformation were found to govern the flexible pavement designs, with total permanent deformation governing the predicted design life. It was also found that the MEPDG predicted design life decreased significantly as the traffic loading increased, especially with respect to total permanent deformation.

Mulandi, Khanum, Hossani and Schieber (2006) redesigned 5 in-service JPCP rigid pavements in Kansas as equivalent rigid and flexible pavement structures using both the MEPDG and AASHTO 1993 methods. All five projects were constructed on Lime Treated Subgrade at 90% reliability. Initial AADTT ranged from 968 to 3690 vehicles per day with 10 and 20-year design lives for flexible and rigid pavements, respectively. The designs were compared on the basis of PCC and AC thickness obtained using both the default MEPDG distress thresholds and revised thresholds typical of Kansas. Traffic distribution data was obtained from Weigh-In-Motion data, and virtual weather stations were created using the EICM for each project location. The MEPDG produced designs with AC layers that were 34.6% thinner on average than the AASHTO 1993 method using the default MEPDG performance criteria, and 29.4% thinner using the revised Kansas performance criteria. The MEPDG rigid pavement designs had 9.84% thinner PCC layers on average than the AASHTO 1993 method for default MEPDG performance criteria. Flexible pavements were observed to fail primarily in longitudinal cracking, while rigid pavements failed mostly in transverse cracking. Gedafa, Mulandi, Hossain and Schieber (2011) completed a similar analysis for the same five pavement sections using MEPDG software versions 1.0 and 1.1 and obtained similar results.

El-Badawy, Bayomy, Santi and Clawson (2011) compared flexible pavement designs obtained using the MEPDG, AASHTO 1993, and Idaho Transportation Department (IDT) empirical pavement design method. The flexible pavements consisted of asphalt over granular base / subbase over a variety of subgrades. The analysis was conducted using MEPDG software version 1.1 at 50% reliability. Design traffic loading ranged from 0.8 to 7.9 million ESALs. AASHTO 1993 ESALs were entered directly into the MEPDG as 100% 18-kip axles, and the EICM was deactivated for all

MEPDG simulations. The study found reasonable agreement between AASHTO 1993 and MEPDG in terms of total pavement structure (see Figure 2-4, left chart). No significant difference was observed in predicted total rutting and IRI between the AASHTO 1993 and MEPDG pavement designs, and the AASHTO 1993 designs generally conformed to the MEPDG default performance criteria. In general, MEPDG pavement structural design was found to be governed by total permanent deformation.

El-Badawy (2011) reported additional results from the above study in a presentation made to the 51st Idaho Asphalt Conference. As shown in Figure 2-4 (right chart), the MEPDG was found to yield thicker asphalt layers than AASHTO 1993 at a higher (85%) reliability level, especially for pavements on weak subgrades.

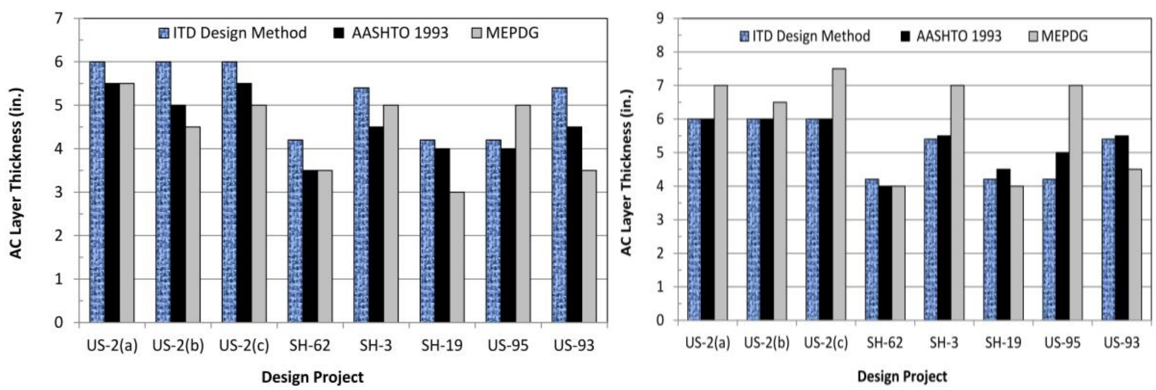


Figure 2-4: AASHTO 1993 and MEPDG Asphalt Layer Thicknesses at 50% Reliability (Left) and 85% Reliability (Right) (El-Badawy 2011)

Li, Uhlmeyer, Mahoney and Muench (2011) described the preparation of a revised pavement thickness catalogue for flexible and rigid pavements for the Washington State Department of Transportation (WSDOT). The MEPDG software version 1.0 was used with all default MEPDG performance criteria. The MEPDG pavement distress models were successfully re-calibrated for Washington State conditions prior to the analysis; thus, unlike the previous studies cited in this section, the nationally calibrated MEPDG models were not used. Traffic distribution data was obtained from Weigh-In-Motion scales with AADTT and traffic growth from historical records. Six traffic loading scenarios were examined for a 50-year design life: 5 million, 10 million, 25 million, 50 million, 100 million, and 200 million ESALs. The former two traffic loading scenarios were designed at 85% reliability while the latter four scenarios were designed at 95% reliability. The EICM was activated for all runs using default weather stations within Washington State. It was observed that pavements designed using the MEPDG averaged 35 mm less asphalt layer thickness and 70 mm less PCC thickness than AASHTO 1993 pavement designs. It was observed that the MEPDG predicted very high rutting in the asphalt layers for pavements with heavy traffic loading. The AASHTO 1993 method was found to produce flexible pavement designs comparable to the MEPDG when the asphalt structure layer coefficient was increased from 0.44 to 0.50.

In summary, the studies comparing pavement structure designs obtained using the MEPDG and AASHTO 1993 have had different results for flexible and rigid pavements. The studies indicate that the AASHTO 1993 overdesigns PCC thickness in rigid pavement structure by approximately 10% relative to the nationally-calibrated MEPDG. However, these results are based on studies with relatively low traffic volumes (< 10 million ESALs). This correlates with the general consensus that AASHTO 1993 overdesigns rigid pavement structures (TAC 2011). The results for flexible pavement structures are more varied. When compared to flexible pavement designs obtained using the nationally-calibrated MEPDG, the AASHTO 1993 designs vary from significantly under-designed, to comparable, to significantly overdesigned for low traffic loading (< 10 million ESALs). At high traffic loadings and/or high design reliabilities, the nationally-calibrated MEPDG generally resulted in significantly thicker asphalt layers for flexible pavements relative to the AASHTO 1993 method.

2.4 MEPDG Sensitivity Analysis

Since the completion of NCHRP Project 1-37A, a myriad of studies have been conducted by State DOTs in the United States to assess the sensitivity of the pavement distress models to various input parameters. This is often one of the initial steps taken by State Departments of Transportation in the MEPDG implementation process to both provide a sense of confidence in the MEPDG method, and identify the most important design input variables that will need to be collected accurately prior to MEPDG implementation. This section provides a summary of the results of a selection of these sensitivity studies for the MEPDG flexible and rigid pavement distress models.

2.4.1 Flexible Pavement and AC Overlay Permanent Deformation (Rutting)

The MEPDG permanent deformation models have been demonstrated to be sensitive to a wide range of inputs. The sensitivity of the MEPDG permanent deformation models is of particular interest since they have been found to govern MEPDG flexible pavement designs in many cases (Ahammed et al. 2011, El-Badawy 2011, Li et al 2011, Schwartz 2007). This section summarizes the sensitivity of the

MEPDG permanent deformation models to various traffic, climate, pavement structure, and material inputs.

2.4.1.1 Permanent Deformation Sensitivity to Traffic

Total permanent deformation in flexible pavement structures has been shown to be highly sensitive to Average Annual Daily Truck Traffic (AADTT) in numerous studies (Ali 2005, Bayomy et al. 2012, Ceylan, Coree, & Gopalakrishnan, 2009, Ceylan, Kim, Heitzman, & Gopalakrishnan, 2006, Graves, & Mahboub 2006, Hoerner, Zimmerman, Smith & Cooley 2007, Li, Jiang, Zhu & Nantung 2007). Some studies have identified AADTT as the most significant variable influencing MEPDG predicted total permanent deformation in flexible pavements (Ceylan et al. 2006, Graves et al. 2006). It has also been observed that MEPDG predicted total permanent deformation in flexible pavement structures designed using the AASHTO 1993 method increases with increasing traffic level (Ahammed et al 2011, Schwartz 2007). MEPDG predicted permanent deformation in the asphalt layers has also been found to be highly sensitive to AADTT (Ali 2005, Bayoymy et al. 2012, Graves et al. 2006, Hoerner et al, 2007, Schwartz 2012, Schwartz, Li, Kim, Ceylan & Gopalakrishnan 2011). One study found AADTT to be the most significant variable influencing asphalt layer rutting (Graves et al. 2006). The impact of AADTT on permanent deformation in the unbound layers is less conclusive. Bayoymy et al. (2012) found that AADTT significantly influenced permanent deformation in the base and subbase layers, however, other studies concluded that it was not significant (Ali 2005, Ceylan et al 2009). Similarly, Ceylan et al. (2009) found that AADTT significantly influenced permanent deformation in the subgrade, while Ali (2005) determined AADTT was not significant. Figure 2-5 illustrates the typical effect of AADTT on MEPDG predicted permanent deformation (Ali 2005).

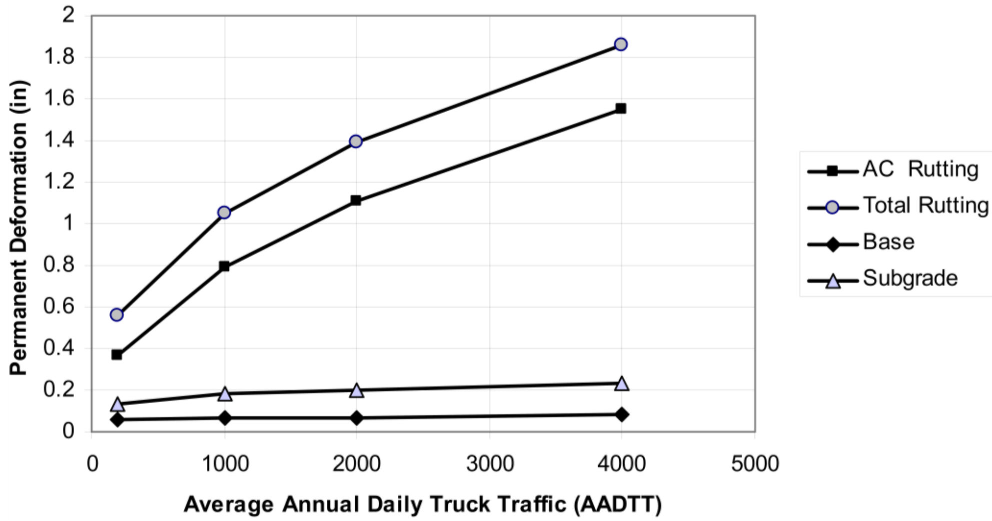


Figure 2-5: Typical Effect of AADTT on MEPDG Permanent Deformation

MEPDG predicted permanent deformation has also been found to be highly sensitive to axle load spectra. Schwartz (2007) reported that a full axle load spectrum leads to higher predicted permanent deformation in the MEPDG relative to an axle load spectrum consisting exclusively of 18-kip single axles, even when the total number of ESALs was the same. Ahammed et al. (2011) found that the use of the MEPDG default axle load spectra resulted in a 2 year decrease in flexible pavement service life in terms of predicted total rutting relative to an axle load distribution specific to the Province of Manitoba. Al-Yagout, Mahoney, Pierce, & Hallenbeck (2005) found that MEPDG predicted rutting was particularly sensitive to significantly overestimated and underestimated axle load spectra, and moderately sensitive to slightly underestimated axle load spectra. Li et al. (2007) found the MEPDG rutting model to be highly sensitive to axle load distribution, and Timm, Newcomb & Galambos (2000) observed that variability in rutting performance is overwhelmingly affected by variability in axle weight. Bayomy et al. (2012) also found that an increase from light to heavy axle load spectra resulted in an increase in permanent deformation in all pavement layers.

MEPDG predicted permanent deformation has been found to be highly sensitive to vehicle class distribution. A recent study conducted in the Province of Manitoba found that flexible pavement service life decreased by 5 years based on total permanent deformation for a truck distribution consisting of a high proportion of multi-trailer trucks compared to a high proportion of single-trailer trucks (Ahammed et al. 2011). Schwartz (2007) found that a vehicle class distribution for a principal arterial highway predicted significantly more permanent deformation than a vehicle class distribution for a minor collector roadway. Tran & Hall (2007) also found that the use of a statewide vehicle class distribution in Arkansas lead to a significant difference in predicted rutting compared to the MEPDG default. However, at least one study in the literature reported a low sensitivity to vehicle class distribution (Li et al. 2007).

MEPDG predicted permanent deformation has been found to be insensitive to hourly traffic distribution (Ahammed et al. 2011, Li et al. 2007), but highly sensitive to monthly traffic distribution (NJDOT 2006). The latter finding appears to be consistent with the nature of the permanent deformation models and their ability to account for seasonal variations in material properties to predict permanent deformation. Operational speed is generally reported to affect MEPDG predicted rutting via an inverse relationship with sensitivity ranging from marginal (Bayomy et al. 2012) to very sensitive (Schwartz 2012, Schwartz et al. 2011). Flexible pavement rutting has also been shown to be sensitive to the use of Level 1 versus Level 3 traffic data inputs (Zanghoul et al. 2006b).

2.4.1.2 Permanent Deformation Sensitivity to Climate

The MEPDG flexible pavement permanent deformation model has been shown to be significantly sensitive to variations in MEPDG climate inputs (Ali 2005, Bayomy et al. 2012, Graves et al. 2006, Li Pierce & Uhlmeyer 2009b). The MEPDG rutting model appears to be capable of capturing the effect of temperature on stiffness and rutting in the asphalt layers, with an increase in temperature resulting in an increase in asphalt rutting (Ali 2005, Bayomy et al. 2012, Schwartz 2007). Li, Schwartz & Forman (2013) found that, of the climatic inputs examined, the total and asphalt layer permanent deformation models were most sensitive to average annual temperature and average annual temperature parameters. These models were also found to be sensitive to percent sunshine and wind speed, which also influence the temperature in the asphalt layers. Schwartz (2007) found that interstate climatic variations in the State of Maryland had a non-negligible influence on MEPDG predicted rutting, with increased temperatures and precipitation resulting in increased predicted rutting. Tighe, Mills, Andrey, Smith & Huen (2009) also observed a moderate increase in MEPDG predicted total pavement and asphalt layer rutting under various climate-change scenarios involving increased temperatures and precipitation for low-volume flexible pavements in Southern Canada.

2.4.1.3 Permanent Deformation Sensitivity to Pavement Structure

MEPDG predicted total permanent deformation has been found to be highly sensitive to the thickness of the asphalt layer in numerous studies (Ali 2005, Bayomy et al. 2012, Graves et al. 2006, Hoerner et al. 2007, Mallela et al. 2009, Schwartz 2012, Schwartz et al. 2011), although at least one study did not observe a significant effect (Ceylan et al. 2009). Asphalt layer thickness has been found to reduce predicted rutting in the asphalt layer for both new flexible pavements and asphalt overlays of existing flexible pavements (Hoerner et al. 2007). Asphalt layer thickness has also been found to significantly reduce permanent deformation in the underlying pavement layers (Bayomy et al. 2009, Schwartz 2007). Variability in rutting performance has also been found to be strongly influenced by asphalt layer thickness (Timm et al. 2000). Ceylan et al. (2006) noted in one study that rutting in the asphalt layer dominated total rutting, but noted that it may have been the result of the thick asphalt layers in the pavements examined in the study.

The sensitivity of flexible pavement rutting to unbound material thickness is somewhat mixed in the literature. Ali (2005) found that an increase in base thickness resulted in a minor decrease in subgrade rutting and no change in asphalt layer rutting. Bayomy et al. (2012) confirmed that increasing base thickness had no impact on asphalt layer rutting, but also found a significant reduction in subgrade and total rutting accompanied by a minor increase in base layer rutting. Other studies have confirmed the impact of base thickness on predicted total rutting although degree of sensitivity observed varied (Graves et al. 2006, Timm et al. 2000). Conversely, Schwartz (2007) found granular base layer thickness had limited influence on predicted rutting, and postulated that the MEPDG may underestimate the contribution of base layers as a direct consequence of the multilayer linear elastic theory employed by the mechanistic models to predict stresses and strains in the pavement structure.

MEPDG predicted rutting in asphalt overlays of flexible pavements has been shown to be very sensitive to existing pavement condition rating and existing asphalt thickness (Harsini, Brink, Haider, Chatti, Buch, Baladi, & Kutay 2013). Rutting in the asphalt overly was also found to be sensitive to rutting in the existing asphalt pavement (Hoerner et al. 2007).

2.4.1.4 Permanent Deformation Sensitivity to Material Properties

MEPDG predicted permanent deformation has been shown to be sensitive to a variety of asphalt material properties. MEPDG predicted permanent deformation in both the total pavement structure and asphalt layers of new flexible pavements has been shown to be sensitive to the asphalt binder grade (Ali 2005, Graves et al. 2006, Hoerner et al. 2007, Schwartz 2007). MEPDG predicted rutting in the asphalt layer for asphalt overlays of existing flexible pavement has also been shown to be sensitive to asphalt binder grade (Hoerner et al. 2007). In addition, MEPDG predicted rutting in the asphalt layers has been shown to be influenced by asphalt effective binder content, with sensitivity ranging from marginal to very high (Ali 2005, Bayomy et al. 2012, Mallela et al. 2009). Asphalt effective binder content was not found to influence rutting in the underlying base, subbase, and subgrade layers. Several studies have verified that asphalt air voids do significantly affect rutting in flexible pavements, although the reported influence of air voids on subgrade rutting was not conclusive (Ali 2005, Bayomy et al. 2012, Mallela et al. 2009). In contrast, Schwartz (2007) reported no variation in predicted rutting as a result of high asphalt air void content. Increasing the stiffness of the asphalt mix has been shown to decrease observed predicted asphalt and total permanent deformation, however, the impact ranged from marginal (Mallela et al. 2009) to significant (Bayomy et al. 2012). Schwartz et al. (2011) conducted a comprehensive sensitivity analysis of over 25 design inputs and found that the α and δ parameters used in the MEPDG Level 1 equation to calculate asphalt dynamic modulus (E*) were by far the most sensitive parameters influencing predicted permanent deformation. MEPDG predicted total and asphalt layer rutting were also shown to be very sensitive to HMA shortwave absorptivity and HMA Poisson's ratio, and moderately sensitive to HMA unit weight, HMA heat capacity, and HMA thermal conductivity (Schwartz et al. 2011, Schwartz 2012). Schwartz (2007) also found in an earlier study that the MEPDG was not able to account for the reduced rutting observed using Stone Mastic Asphalt (SMA) mixtures.

The observed impact of base modulus on predicted permanent deformation is somewhat varied in the literature. A number of studies cite a significant influence of base resilient modulus on MEPDG predicted rutting (Hoerner et al. 2007, Li et al. 2009b, Mallela et al. 2009, Schwartz 2012, Schwartz et al. 2011), however, other studies found a negligible influence (Ali 2005, Li et al 2012, Schwartz 2007). Similar observations were reported with respect to the sensitivity of MEPDG predicted rutting to subgrade resilient modulus. Most studies reported that MEPDG predicted rutting was only moderately sensitive to subgrade resilient modulus; increasing subgrade resilient modulus resulted in only a minor decrease in predicted rutting (Ali 2005, Mallela et al 2009, Schwartz 2007), although at least one study did report a more significant relationship (Bayomy et al 2012). Li et al (2012) and Schwartz (2007) both noted that unbound material resilient moduli had an insignificant impact on MEPDG predicted total and asphalt layer permanent deformation, and speculated that the MEPDG underestimates the structural contribution of unbound pavement and subgrade layers. Ali (2005) also noted that the MEPDG reflected an insufficient sensitivity to unbound material properties, and noted that changes made to the properties of one unbound layer did not appear to affect the other unbound layers. A recent advisory issued by AASHTO has acknowledged that the current MEPDG model for unbound pavement materials underestimate the structural impact of high quality aggregate base layers, and several efforts were currently underway to address the issue (AASHTO 2013).

2.4.2 Flexible Pavement and AC Overlay Bottom-Up Fatigue (Alligator) Cracking

The MEPDG bottom-up fatigue cracking model has been demonstrated to be sensitive to a number of key inputs. The sensitivity of the MEPDG bottom-up fatigue cracking model is of particular concern for pavement structures with thinner asphalt layers, as MEPDG predicted bottom-up fatigue cracking has generally not been found to be a critical pavement distress for flexible pavement structures with relatively thick asphalt layers (Ceylan et al. 2006). This section summarizes the sensitivity of the MEPDG bottom-up fatigue cracking model to various traffic, climate, material and pavement structure inputs.

2.4.2.1 Bottom-Up Fatigue Cracking Sensitivity to Traffic

Numerous studies have demonstrated that MEPDG predicted bottom-up fatigue cracking in flexible pavement structures is highly sensitive to AADTT (Ali 2005, Bayomy et al. 2012, Ceylan et al. 2009, Graves et al. 2006, Hoerner et al. 2007, Li et al. 2007, Schwartz 2012, Schwartz et al. 2011), with one study citing AADTT as the most significant factor influencing predicted bottom-up fatigue cracking (Graves et al. 2006). Studies have also shown that MEPDG predicted bottom-up fatigue cracking is moderately to highly sensitive to axle load spectra (Li, Pierce, Hallenbeck, & Uhlmeyer, 2009a, Li et al. 2007), and is particularly sensitive to significantly overestimated and underestimated axle load spectra (Al-Yagout et al. 2005). Variability in predicted bottom-up fatigue cracking has also been shown to be overwhelming affected by variability in axle weight variability (Timm et al. 2000). MEPDG predicted bottom-up fatigue cracking has also generally been shown to be sensitive to vehicle class distribution (Li et al. 2007, Schwartz 2007, Tran et al. 2007), although one study did not observe a significant relationship (Ceylan et al. 2009). Mallela et al. (2009) noted that bottom-up fatigue cracking was moderately influenced by vehicle class distribution primarily in terms of the percentage of trucks of FHWA Class 9 or greater. NJDOT (2006) found that MEPDG predicted alligator cracking was sensitive to both hourly and monthly traffic distribution, and several studies observed a relationship between operational speed and predicted alligator cracking ranging from insignificant to moderate (Ali 2005, Bayomy et. al 2012, Li et al. 2007, Schwartz 2012, Schwartz et al. 2011).

2.4.2.2 Bottom-Up Fatigue Cracking Sensitivity to Climate

In general, studies appear to show that MEPDG predicted bottom-up fatigue cracking is only moderately affected by climate (Mallela et al. 2009). Ali (2005) found that MEPDG predicted alligator cracking was higher for warm climates relative to colder climates, and Bayomy et al. (2012) observed that an increase in Mean Average Total Temperature resulted in a marginal increased in predicted alligator cracking. Schwartz (2007) observed that interstate climatic variations in Maryland had a non-negligible effect on MEPDG predicted fatigue cracking, with higher temperature and precipitation linked to increase fatigue cracking. Tighe et al. (2009) examined MEPDG predicted pavement performance under climate change scenarios and found moderate increase in MEPDG predicted bottom-up fatigue cracking for increasing temperature and precipitation.

2.4.2.3 Bottom-Up Fatigue Cracking Sensitivity to Pavement Structure

The thickness of the asphalt layer is the most significant variable influencing MEPDG predicted alligator cracking (Graves et al. 2006, Hoerner et al. 2007, Mallela et al. 2009, Schwartz 2012, Schwartz et al. 2011). Ali (2005) reported a significant relationship between asphalt layer thickness and predicted alligator cracking, which is reproduced in Figure 2-6.

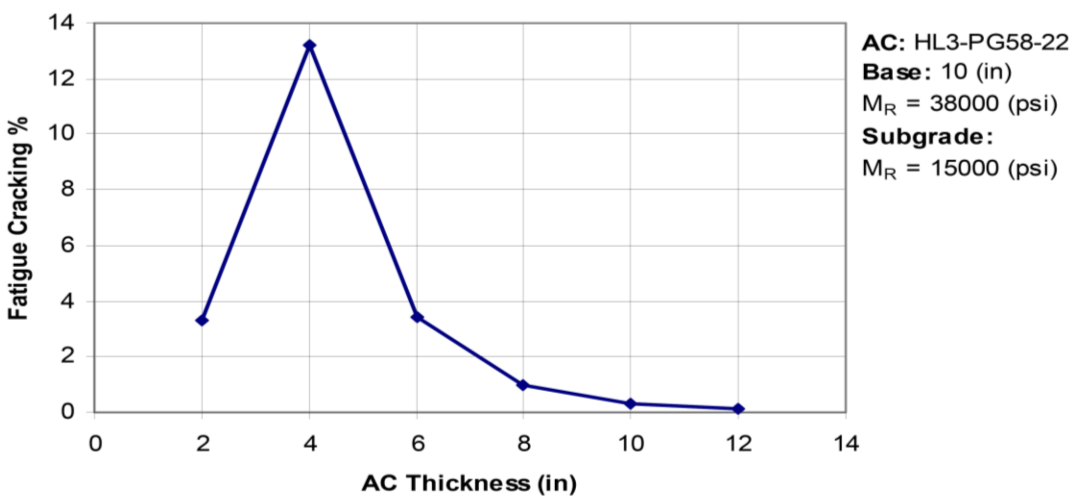


Figure 2-6: Typical Effect of AC Thickness on MEPDG Bottom-Up Fatigue Cracking

As shown in Figure 2-6, Ali (2005) observed that MEPDG predicted bottom-up fatigue cracking increased when asphalt layer thickness was increased from 50mm to 100mm, and then rapidly declined with increasing asphalt layer thickness. The highest alligator cracking was observed in flexible pavements with asphalt layer thicknesses between 50mm and 150mm, with negligible alligator cracking observed in pavements with asphalt layer thicknesses exceeding 200mm. Bayomy et al. (2012) confirmed this observation and reported that predicted alligator cracking: had a significant impact on asphalt layer thickness; was highest for asphalt layer thicknesses between 50mm and 125mm; and, was negligible for asphalt layer thicknesses exceeding 175mm. Ceylan et al. (2009) also reported that alligator cracking was insensitive to changes in asphalt layer thickness for flexible pavements with thick asphalt layers. Timm et al. (2000) observed that variability in asphalt layer thickness was the most significant variable influencing variability in predicted bottom-up fatigue cracking.

A number of studies have found that MEPDG predicted bottom-up fatigue cracking is very sensitive to the thickness of the underlying base layers (Bayomy et al. 2012, Ceylan et al. 2006, Mallela et al. 2009, Schwartz 2012, Schwartz et al. 2011). Ceylan et al. 2006 reported that base thickness, along with base modulus, was the most significant factor influencing MEPDG predicted alligator cracking for flexible pavement structures in Iowa.

MEPDG predicted bottom-up fatigue cracking for asphalt overlays of existing flexible pavements has been shown to be highly sensitive to existing pavement condition rating, existing asphalt layer thickness, and the existing asphalt binder grade (Harsini et al. 2013, Hoerner et al. 2007).

2.4.2.4 Bottom-Up Fatigue Cracking Sensitivity to Material Properties

MEPDG predicted bottom-up fatigue cracking has been shown to be very sensitive to asphalt binder grade (Ali 2005, Graves et al. 2006, Hoerner et al. 2007, Schwartz 2007), effective asphalt binder content (Ali 2005, Bayomy et al. 2012, Mallela et al. 2009, Schwartz 2012, Schwartz et al. 2011), and asphalt air voids (Ali 2005, Bayomy et al. 2012, Mallella et al. 2009, Schwartz 2007, Schwartz 2012, Schwartz et al. 2011). MEPDG predicted bottom-up fatigue cracking has also been shown to be sensitive to asphalt mix stiffness (Ali 2005, Li et al. 2009b, Schwartz 2007). Bayomy et al. (2012) observed an insignificant relationship between asphalt mix stiffness and predicted alligator cracking for flexible pavements with thin asphalt layers, but noted that the relationship is dependent on asphalt thickness and may not be applicable to thicker asphalt pavement structures. MEPDG predicted alligator cracking has also been shown to be very sensitive to HMA shortwave absorptivity and HMA Poisson’s ratio, and moderately sensitive to HMA unit weight, HMA heat capacity, HMA thermal conductivity, and HMA binder grade (Schwartz 2012, Schwartz et al. 2011).

With respect to the base layer, MEPDG predicted bottom-up fatigue cracking has been shown to be very sensitive to base modulus (Hoerner et al. 2007, Mallela et al. 2009, Schwartz 2012, Schwartz et al. 2011) and moderately sensitive to base Poisson ratio (Schwartz 2012, Schwartz et al. 2011). The effect of subgrade resilient modulus appears to be less conclusive. Some studies have reported the effect of subgrade resilient modulus on MEPDG predicted alligator cracking to be insignificant (Bayomy et al. 2012, Li et al. 2012) or moderate (Mallela et al. 2009). However, Schwartz (2007) found that unbound resilient modulus values reasonably influenced predicted bottom-up fatigue cracking, especially for low values associated with soft subgrade soils. MEPDG predicted alligator cracking was also found to be very sensitive to subgrade resilient modulus in a comprehensive sensitivity analysis (Schwartz 2012, Schwartz et al. 2011). Other subgrade properties with moderate influence on MEPDG predicted bottom-up fatigue cracking are subgrade gradation, subgrade liquid limit, subgrade Poisson's ratio, and subgrade plasticity index (Schwartz 2012, Schwartz et al. 2011).

2.4.3 Flexible Pavement and AC Overlay Top-Down Fatigue (Longitudinal) Cracking

The MEPDG top-down fatigue cracking model has been demonstrated to be sensitive to a number of key inputs. Similar to the bottom-up fatigue cracking model, the sensitivity of the top-down fatigue cracking model is of particular concern for pavement structures with thinner asphalt layers. This section summarizes the sensitivity of the MEPDG longitudinal cracking models to various traffic, climate, pavement structure and material inputs.

2.4.3.1 Top-Down Fatigue Cracking Sensitivity to Traffic

MEPDG predicted longitudinal cracking has been shown to be very sensitive to AADTT for both new flexible pavements and asphalt overlays in a number of studies (Bayomy et al. 2012, Ceylan et al. 2009, Ceylan et al. 2006, Graves et al. 2006, Hoerner et al. 2007, Schwartz 2012, Schwartz et al. 2011). MEPDG predicted longitudinal cracking has also been found to be highly sensitive to axle load spectra (Li et al. 2007). Bayomy et al. (2012) found that changing from a light to heavy axle load spectra significantly increase predicted longitudinal cracking. MEPDG predicted longitudinal cracking has also been shown to be highly sensitive to vehicle class distribution (Ceylan et al. 2009, Li et al. 2007). For example, Ahammed et al. (2011) found that flexible pavement service life in Manitoba decreased by over 8 years in terms of longitudinal cracking when using a vehicle class distribution with a high proportion of multi-trailer trucks compared to a vehicle class distribution with a high proportion of single-trailer trucks. MEPDG predicted longitudinal cracking has been found to be insensitive to hourly traffic distribution (Ahammed et al 2011) and only moderately sensitive to monthly traffic distribution (Li et al. 2007). MEPDG predicted longitudinal cracking has been found to be only moderately sensitive to operational speed (Bayomy et al. 2012, Li et al. 2007, Schwartz 2012, and Schwartz et al. 2011). Zanghloul et al. (2006b) found that the impact of traffic variables on longitudinal cracking was not sensitive to the use of Level 1 versus Level 3 traffic inputs.

2.4.3.2 Top-Down Fatigue Cracking Sensitivity to Climate

The MEPDG longitudinal cracking model has been found to be sensitive to various climatic parameters. For example, Bayomy et al. (2012) found that an increase in Mean Average Daily Temperature significantly increased predicted longitudinal cracking, while an increase in the Groundwater Table resulted in only a moderate increase in longitudinal cracking. Li et al. (2013) examined the sensitivity of the MEPDG longitudinal cracking model to a variety of climatic inputs and determined that average annual temperature and average annual temperature range had the most significant impact on predicted longitudinal cracking, followed by percent sunshine and wind speed. The above studies show that, similar to alligator cracking, higher pavement temperatures generally increase MEPDG longitudinal cracking predictions.

2.4.3.3 Top-Down Fatigue Cracking Sensitivity to Pavement Structure

The MEPDG longitudinal cracking model has been found to be very sensitive to the thickness of the asphalt layer in numerous studies (Bayomy et al. 2012, Graves et al. 2006, Hoerner et al. 2007, Li et al. 2009b, Mallela et al. 2009, Schwartz 2012, and Schwartz et al. 2011). Bayomy et al. (2012) observed a relationship between asphalt layer thickness and MEPDG predicted longitudinal cracking similar to that observed for alligator cracking. As shown in Figure 2-7, longitudinal cracking was found to be highest in flexible pavements with asphalt layer thicknesses between 75mm and 125mm, with negligible longitudinal cracking observed for asphalt layer thicknesses greater than 175mm. Mallela et al. (2009) also found that, for flexible pavements in Ohio, asphalt layer thickness had a significant impact on longitudinal cracking for flexible pavements with asphalt layers less than 200mm; for asphalt layers exceeding 200mm, asphalt layer thickness had no effect on predicted longitudinal cracking.

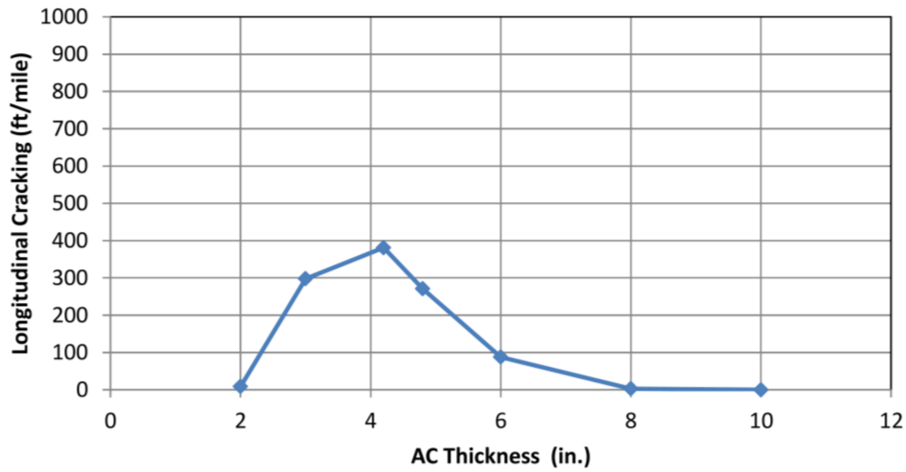


Figure 2-7: Effect of Asphalt Layer Thickness on MEPDG Predicted Longitudinal Cracking

The MEPDG longitudinal cracking model has also been found to be very sensitive to base layer thickness for both new flexible pavements and asphalt overlays of existing flexible pavements (Bayomy et al. 2012, Hoerner et al. 2007, Li et al. 2012, Schwartz 2012, Schwartz et al. 2011). Bayomy et al. (2012) observed this effect for base thicknesses up to 550mm, at which point increasing base thickness actually resulted in increased longitudinal cracking, possibly due to the base acting as a stiff foundation.

MEPDG predicted cracking in asphalt overlays of flexible pavements has been found to be highly sensitive to existing pavement condition and existing HMA thickness (Harsini et al. 2013, Hoerner et al. 2007).

2.4.3.4 Top-Down Fatigue Cracking Sensitivity to Material Properties

The MEPDG longitudinal cracking model has been found to be highly sensitive to asphalt binder grade for both new flexible pavements and asphalt overlays of existing flexible pavements (Ceylan et al. 2006, Hoerner et al. 2007, Li et al. 2009b, Schwartz 2012, and Schwartz et al. 2011). MEPDG predicted longitudinal cracking has also been found to be very sensitive to asphalt mix effective binder content and asphalt percent air voids (Bayomy et al. 2012, Schwartz 2012, and Schwartz et al. 2011). Bayomy et al. (2012) found that, for thin asphalt layers, an increase in asphalt stiffness actually caused an increase in longitudinal cracking; the relationship is reversed for thicker asphalt layers. The MEPDG longitudinal cracking model was also found to be very sensitive to surface shortwave absorptivity and HMA Poisson's ratio, and moderately sensitive to HMA unit weight, HMA heat capacity, and HMA thermal conductivity (Schwartz 2012, and Schwartz et al. 2011).

The MEPDG longitudinal cracking model has been found to be sensitive to base resilient modulus for both new flexible pavements and asphalt overlays of existing flexible pavements (Hoerner et al. 2007, Li et al. 2012). MEPDG predicted longitudinal cracking has also been found to be moderately sensitive to base Poisson's ratio (Li et al. 2012, Schwartz 2012, and Schwartz et al. 2011).

The MEPDG longitudinal cracking model has also been found to be very sensitive to subgrade resilient modulus and subgrade gradation (Bayomy et al. 2012, Ceylan et al. 2006, Graves et al. 2006, Schwartz 2012, and Schwartz et al. 2011) (see Figure 2-8), and moderately sensitive to subgrade liquid limit, subgrade plasticity index, and subgrade Poisson's ratio (Li et al. 2012, Schwartz 2012, and Schwartz et al. 2011).

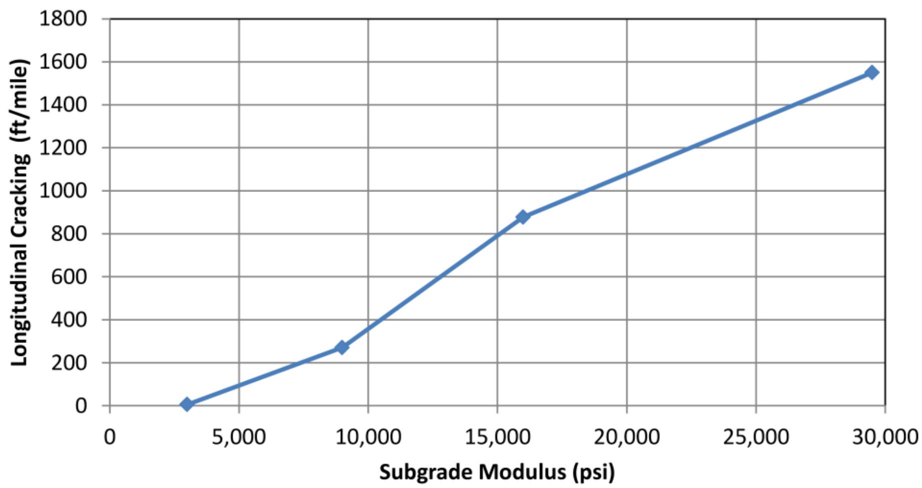


Figure 2-8: Typical Effect of Subgrade Resilient Modulus on MEPDG Predicted Longitudinal Cracking (Bayomy et al. 2012)

2.4.4 Flexible Pavement and AC Overlay Thermal (Transverse) Cracking

The MEPDG thermal cracking model has been demonstrated to be sensitive to only a few pavement design inputs. MEPDG predicted thermal cracking has typically been observed to be minimal to none provided an appropriate asphalt binder grade is selected (Bayomy et al. 2012, Schwartz 2007). This section summarizes the sensitivity of the MEPDG thermal cracking model to various traffic, climate, material, and pavement structure inputs.

2.4.4.1 Thermal Cracking Sensitivity to Traffic

The studies reviewed did not examine the sensitivity of the MEPDG thermal cracking model to traffic loading.

2.4.4.2 Thermal Cracking Sensitivity to Climate

The MEPDG thermal cracking model is very sensitive to climate (Ceylan et al. 2006, Li et al. 2009b, and Mallella et al. 2009). Mallella et al. (2009) found that the lowest temperature achieved was the most significant climatic variable influencing MEPDG predicted thermal cracking in Ohio.

2.4.4.3 Thermal Cracking Sensitivity to Pavement Structure

The MEPDG thermal cracking model has been found to be insensitive to asphalt layer thickness (Ceylan et al. 2009). Most studies examined did not consider the influence of asphalt or base layer thickness on thermal cracking.

2.4.4.4 Thermal Cracking Sensitivity to Material Properties

The MEPDG thermal cracking model is very sensitive to asphalt material properties including HMA binder type, HMA air voids, HMA modulus of elasticity, HMA creep compliance, and HMA tensile strength (Ceylan et al 2006, Schwartz 2012, and Schwartz et al. 2011). The MEPDG thermal cracking model is also moderately sensitive to surface shortwave absorptivity, HMA Poisson's ratio, HMA effective binder volume, HMA unit weight, HMA heat capacity, and HMA thermal conductivity (Schwartz 2012, Schwartz et al. 2011).

2.4.5 Flexible Pavement and AC Overlay International Roughness Index (IRI)

The MEPDG flexible pavement IRI model calculates roughness as a composite measure of the other pavement distresses predicted by the MEPDG (AASHTO 2008). As a composite measure, flexible pavement IRI should be most affected by the variables that have the greatest affect on the other pavement distresses. This section summarizes the sensitivity of the MEPDG flexible pavement IRI model to various traffic, climate, pavement structure and material inputs.

2.4.5.1 Flexible pavement IRI Sensitivity to Traffic

The MEPDG flexible pavement IRI model has been found to be very sensitive to AADTT (Bayomy et al. 2012, Li et al. 2007, Schwartz 2012, Schwartz et al. 2011 and Zhou, Huang, Shu, & Dong 2013). Conversely, MEPDG predicted flexible pavement IRI is reported to be insensitive to axle load spectra (Al-Yagout et al. 2005, and Li et al. 2009a), vehicle class distribution (Ceylan et al. 2009 and Mallela et al. 2009), and operational speed (Schwartz 2012 and Schwartz et al. 2011).

2.4.5.2 Flexible pavement IRI Sensitivity to Climate

The MEPDG flexible pavement IRI model has been found to be relatively insensitive to climate (Mallela et al. 2009).

2.4.5.3 Flexible pavement IRI Sensitivity to Pavement Structure

The reported effect of asphalt layer thickness on MEPDG predicted flexible pavement IRI is varied in the literature. Ceylan et al. (2009) reported that the MEPDG flexible pavement IRI model was not sensitive to asphalt layer thickness, while other sensitivity studies found that the model to be very sensitive to asphalt layer thickness (Mallela et al. 2009, Schwartz 2012 and Schwartz et al. 2011).

The MEPDG flexible pavement IRI model is not sensitive to the thickness of granular base and subbase layers (Bayomy et al. 2012). Ahammed et al. (2011) observed that an increase in granular subbase thickness of 200mm resulted in a 0.25 year increase in pavement service life based on MEPDG predicted flexible pavement IRI; the same increase in subbase thickness resulted in a doubling of pavement service life using the AASHTO 1993 method.

MEPDG predicted flexible pavement IRI for asphalt overlays of existing flexible pavement has been found to be highly sensitive to both existing condition rating and existing asphalt thickness of the flexible pavement (Harisni et al. 2013).

2.4.5.4 Flexible pavement IRI Sensitivity to Material Properties

The MEPDG flexible pavement IRI model has been found to be only moderately sensitive to select asphalt material properties, including surface shortwave absorptivity, HMA air voids, HMA Poisson's ratio, HMA effective binder volume, and HMA heat capacity (Mallela et al. 2009, Schwartz 2012 and Schwartz et al. 2011). The MEPDG flexible pavement IRI model has also been found to be only moderately sensitive to unbound material properties such as base resilient modulus, subgrade resilient modulus, subgrade gradation, and subgrade stabilization (Schwartz 2012, and Schwartz et al. 2011).

2.4.6 Rigid Pavement Mean Joint Faulting

The MEPDG mean joint faulting model has been demonstrated to be sensitive to a number of variables. This section summarizes the sensitivity of the MEPDG mean joint faulting model to traffic, climate, pavement structure and material inputs.

2.4.6.1 Mean Joint Faulting Sensitivity to Traffic

The MEPDG mean joint faulting model has been shown to be sensitive to AADTT (Schwartz et al. 2011), and only moderately sensitive to vehicle class distribution (Mallela et al. 2009).

2.4.6.2 Mean Joint Faulting Sensitivity to Climate

Mallela et al. (2009) found the MEPDG mean joint faulting model to be only marginally influenced by climatic input variables.

2.4.6.3 Mean Joint Faulting Sensitivity to Pavement Structure

The MEPDG mean joint faulting model has been shown to be sensitive to PCC layer thickness, although this is not one of the primary variables influencing its performance predictions (Hall et al. 2005, Mallela et al. 2009, and Schwartz et al. 2011). Where PCC thickness was found to have a very significant influence on predicted mean joint faulting, this appeared to be primarily attributable to its correlation with dowel diameter (Mallela et. al 2009). The MEPDG predicted mean joint faulting has also been found to be moderately sensitive to base layer thickness (Schwartz et al. 2011).

Variables related to the actual design of the rigid pavement have been shown to be the significant parameters influencing MEPDG mean joint faulting prediction. For example, the MEPDG mean joint faulting model has been shown to be highly sensitive to slab width, dowel diameter, PCC joint spacing, with edge support exhibiting more moderate influence (Guclu, Ceylan, Gopalakrishnan & Kim 2009, Hall et al 2005, Mallela et al. 2009 and Schwartz et al. 2011).

2.4.6.4 Mean Joint Faulting Sensitivity to Material Properties

PCC material properties have a very strong influence on the MEPDG mean joint faulting model. PCC curl / warp effective temperature difference , PCC coefficient of thermal expansion (CTE), and PCC unit weight are consistently reported as having a very strong influence on MEPDG predict mean joint faulting (Guclu et al. 2009, Hall et al. 2005, Mallela et al. 2009, Schwartz et. al 2011, and Tanesi et al. 2007). Tanesi, Kutay, Abbas & Meininger (2007) also reported that higher laboratory prediction error for the PCC CTE parameter significantly increased the variability of MEPDG predicted mean joint faulting; a mean prediction error of only 0.3×10^{-6} in/in/^{o}F for a PCC with a mean CTE of 6.5×10^{-6} in/in/^{o}F resulted in a difference in 0.5mm for MEPDG predicted mean joint faulting. PCC material properties with a moderate influence on MEPDG predicted mean joint faulting include surface shortwave absorptivity, Poisson’s ratio, thermal conductivity, cement content, water / cement ratio, flexural strength / modulus, and mix coarse aggregate type (Guclu et al. 2009, Mallela et al. 2009, and Schwartz et al 2011).

The MEPDG mean joint faulting model was found to be only moderately sensitive to unbound material properties including base resilient modulus, subgrade resilient modulus, and Erodibility index (Mallela et al 2009 and Schwartz et al. 2011).

2.4.7 Rigid Pavement Transverse Cracking

The MEPDG transverse cracking model has been demonstrated to be sensitive to a number of pavement design inputs. This section summarizes the sensitivity of the MEPDG transverse cracking model to traffic, climate, material, and pavement structure inputs.

2.4.7.1 Transverse Cracking Sensitivity to Traffic

Among traffic variables, the MEPDG transverse cracking model has been shown to be very sensitive to vehicle class distribution, primarily the percentage of FHWA Class 5 to 8 trucks, and more moderately sensitive to AADTT (Mallela et al. 2009, Schwartz et al. 2011).

2.4.7.2 Transverse Cracking Sensitivity to Climate

The MEPDG rigid pavement transverse cracking model has been found to be highly sensitive to climatic inputs including average daily temperature range and percent sunshine (Li et al. 2013 and Mallela et al. 2009). Johanneck and Khazanovich (2010) examined the impact of climate on MEPDG rigid pavement transverse cracking prediction, and observed that the effect of climate was more significant for new rigid pavements compared to asphalt overlays of existing rigid pavements; it was postulated that this may be due to the insulating effect of the asphalt layer on the underlying concrete. MEPDG predicted transverse cracking was also found to be higher in warmer climates and inland regions. MEPDG predicted transverse cracking was found to vary widely between climate stations in very close proximity due to poor climate data at some stations; this finding emphasized the very high influence of climate on predicted transverse cracking.

2.4.7.3 Transverse Cracking Sensitivity to Pavement Structure

The MEPDG rigid pavement transverse cracking model has been found to be highly sensitive to PCC layer thickness in several studies (Guclu et al. 2009, Hall et al. 2005, Mallela et al. 2009, and Schwartz et al. 2011). MEPDG predicted transverse cracking has also been found to be moderately sensitive to base layer thickness (Schwartz et al. 2011). The MEPDG rigid pavement transverse cracking model has also been shown to be highly sensitive to joint spacing, slab width, and pavement edge support (Guclu et al. 2009, Hall et al. 2005, Li, Muench, Mahoney, Sivaneswaran & Pierce 2006, Mallela et al. 2009 and Schwartz et al. 2011). MEPDG predicted transverse cracking has been shown to be moderately sensitive to dowel diameter (Schwartz et al. 2011).

2.4.7.4 Transverse Cracking Sensitivity to Material Properties

The MEPDG rigid pavement transverse cracking model has been found to be highly sensitive to a number of PCC material properties including: curl / warp effective temperature difference, CTE, thermal conductivity, thermal expansion, 28-day modulus of rupture, 28-day compressive strength, unit weight, and water /cement ratio (Guclu et al. 2009, Hall et al. 2005 and Schwartz et al. 2011). Tanesi et al. (2007) reported that higher laboratory prediction error for the PCC CTE parameter significantly increased the variability of MEPDG predicted transverse cracking; a mean prediction error of only 0.3 x 10^{-6} in/in/^{o}F for a PCC with a mean CTE of 6.5 x 10^{-6} in/in/^{o}F resulted in a difference in 10% for MEPDG predicted transverse cracking. PCC material properties with moderate influence on MEPDG predicted transverse cracking included Poisson's ratio, surface shortwave absorptivity, 28-day elastic modulus, and cement content (Guclu et al. 2009 and Schwartz et al. 2011). The MEPDG rigid pavement transverse cracking model has been found to be moderately sensitive to unbound material properties including base resilient modulus, subgrade resilient modulus, Erodibility index, and subgrade stabilization (Mallela et al. 2009 and Schwartz et al. 2011).

2.4.8 Rigid Pavement International Roughness Index (IRI)

The MEPDG rigid pavement IRI model calculates roughness as a composite measure of the other rigid pavement distresses predicted by the MEPDG. As a composite measure, rigid pavement IRI should be affected by the same variables that affect the other rigid pavement distresses. Input variables with a high influence on both of the other rigid pavement distress models should have the greatest impact on rigid pavement IRI. This section summarizes the sensitivity of the MEPDG flexible pavement IRI model to various traffic, climate, material, and pavement structure inputs.

2.4.8.1 Rigid Pavement IRI Sensitivity to Traffic

Among traffic variables, the MEPDG rigid pavement IRI model has been shown to be sensitive to AADTT (Li et al. 2006, Schwartz et. al 2011), and marginally sensitive to vehicle class distribution (Mallela et al. 2009).

2.4.8.2 Rigid Pavement IRI Sensitivity to Climate

The reported influence of climate on the MEPDG rigid pavement IRI model is mixed. A study conducted by Washington State DOT reported a significant influence (Li et. al 2006), while a study from OHIO DOT reported only a moderate influence (Mallela et al. 2009).

2.4.8.3 Rigid Pavement IRI Sensitivity to Pavement Structure

The MEPDG rigid pavement IRI model is reported to be at least moderately sensitive to PCC layer thickness, with one study reported a high sensitivity to this input (Guclu et al.2009, Hall et al. 2005, Mallela et al. 2009 and Schwartz et al. 2011). The MEPDG rigid pavement IRI model was also found to be highly sensitive to slab width, joint spacing, and dowel bar use (Li et al. 2006, Mallela et al. 2009 and Schwartz et al. 2011). The model was also found to be moderately sensitive to dowel diameter and pavement edge support (Guclu et al.2009, Hall et al. 2005, Mallela et al. 2009 and Schwartz et al. 2011).

2.4.8.4 Rigid Pavement IRI Sensitivity to Material Properties

The MEPDG rigid pavement IRI model has been found to be very sensitive to PCC curl / warp effective temperature difference, CTE, and unit weight (Guclu et al.2009, Hall et al. 2005, Mallela et al. 2009 and Schwartz et al. 2011). Tanesi et al. (2007) reported that higher laboratory prediction error for the PCC CTE parameter significantly increased the variability of MEPDG predicted IRI; a mean prediction error of only 0.3 x 10^{-6} in/in/^{o}F for a PCC with a mean CTE of 6.5 x 10^{-6} in/in/^{o}F resulted in a difference in 0.284 m / km for MEPDG predicted IRI. PCC material properties with moderate influence on MEPDG predicted transverse cracking included: 28 day modulus of rupture, 28 day elastic modulus, surface shortwave absorptivity, water / cement ratio, thermal conductivity, coefficient of thermal expansion, Poisson’s ratio, cement content, and mix type / coarse aggregate (Guclu et al.2009, Hall et al. 2005, Mallela et al. 2009 and Schwartz et al. 2011). Base resilient modulus, subgrade resilient modulus, and Erodibility index were also found to exert a moderate influence on MEPDG predict rigid pavement IRI (Li et al. 2006, Mallela et al. 2009 and Schwartz et al. 2011).

2.4.9 MEPDG Sensitivity Summary

In summary, the input variables that have consistently been demonstrated to exert a very significant influence on the various MEPDG flexible pavement distress models in the studies examined were as follows:

- Permanent Deformation: AADTT, Axle Load Spectra, Vehicle Class Distribution, Monthly Traffic Distribution, AC Layer Thickness, Existing Pavement Condition Rating, Existing AC Thickness
- Bottom-Up Fatigue Cracking: AADTT, AC Thickness, Existing Pavement Condition Rating, Existing AC Thickness, Existing AC Binder Grade, AC Binder Grade, AC Air Voids, AC Effective Binder Content, Base Resilient Modulus
- Top-Down Fatigue Cracking: AADTT, Axle Load Spectra, Vehicle Class Distribution, Climate, AC Thickness, Base Thickness, Existing Pavement Condition, Existing AC Thickness, AC Binder Grade, AC Air Voids, AC Effective Binder Content, AC Surface Shortwave Absorptivity, AC Poisson's Ratio, Subgrade Resilient Modulus, Subgrade Gradation
- Thermal Cracking: Climate, AC Binder Grade, AC Air Voids, AC Effective Binder Content
- International Roughness Index: AADTT, Existing Pavement Condition, Existing AC Thickness

The input variables that have consistently been demonstrated to exert a very significant influence on the various MEPDG rigid pavement performance models in the studies examined were as follows:

- Mean Joint Faulting: Slab Width, Dowel Use, Dowel Diameter, Joint Spacing, PCC Curl / Warp Effective Temperature Difference, PCC Coefficient of Thermal Expansion, PCC Unit Weight
- Transverse Cracking Vehicle Class Distribution, Climate, PCC Thickness,

Joint Spacing, Slab Width, PCC Curl / Warp Effective Temperature Difference, PCC Coefficient of Thermal Expansion, PCC Thermal Conductivity, PCC Thermal Expansion, PCC 28-Day Modulus of Rupture, PCC 28-day Compressive Strength, PCC Unit Weight, and PCC Water /Cement Ratio

- International Roughness Index: Slab Width, Joint Spacing, Dowel Bar Use, PCC Curl / Warp Effective Temperature Difference, PCC Coefficient of Thermal Expansion

2.5 MEPDG Verification Studies

Since the completion of NCHRP Project 1-37A a number of studies have been conducted by State DOTs in United States to determine the verify the accuracy of the nationally calibrated MEPDG performance prediction models using local pavement performance data. Since the MEPDG distress models were nationally calibrated using hundreds of pavement sections located throughout the United States, it is sometimes necessary for State DOTs to recalibrate the performance models to reflect local traffic, climate, materials, and construction practices. Since the recalibration of the MEPDG pavement performance models can be a relatively long and costly process, these verification studies allow State DOTs to: (1) determine whether the default MEPDG pavement performance models need to be recalibrated using local pavement performance data; and, (2) prioritize the recalibration of the performance models. They also provide insight into the ability of the nationally calibrated models to accurately model actual pavement performance measured in the field. Field measured pavement performance is typically obtained either from the State Pavement Management System (PMS), LTPP pavement sections, or other test track pavement sections.

2.5.1 Verification of the Flexible Pavement Permanent Deformation Models

A significant amount of research has been conducted by states and provinces to examine the accuracy of the nationally-calibrated MEPDG permanent deformation models. The vast majority of studies reviewed found that the nationally calibrated MEPDG permanent deformation models significantly over-predicted rutting in new flexible pavements. This conclusion was reached in studies conducted in Michigan (Buch, Chatti, Haider, & Manik 2008, Goh & You 2009), Utah (Darter, Titus-Glover, Von Quintus 2009), Arkansas (Hall, Xiao, & Wang 2011), Alberta (He, Juhasz, Crockett &

Lakkavalli 2011), Minnesota (Hoegh, Khazanovich, & Jensen, 2010), Iowa (Kim, Ceylan, Gopalakrishnan, & Smadi 2010), Ohio (Mallela et al. 2009), Montana (Von Quintus & Moulthrop 2007), and Louisiana (Wu & Yang 2012). Only one study, conducted for the Washington State DOT, found that the MEPDG permanent deformation models under-predicted new flexible pavement rutting (Li et al. 2009).

A number of studies have examined the accuracy of the nationally calibrated MEPG permanent deformation models as a function of pavement structure with somewhat mixed results. For example, He et al (2011) compared MEPDG predicted rutting with data from the Alberta PMS and found that the MEPDG consistently over predicted total pavement rutting for new flexible pavements with granular base courses by 10mm or more over a 20 year design life (see Figure 2-9). In contrast, the MEPDG was found to accurately predict the total rutting for flexible pavements rehabilitated with straight asphalt overlays, and actually under predicted total rutting in flexible pavements rehabilitated with mill and overlay techniques by 1 – 5 mm over a 20 year design life. Kim et al. (2010) found that the MEPDG over predicted total rutting for new flexible pavements and asphalt overlays of existing flexible pavements in Iowa, but slightly under predicted rutting for asphalt overlays of existing rigid pavement. Yang & Wu (2012) examined the accuracy of the MEPDG permanent deformation models in Louisiana and found that the nationally calibrated models over-predicted rutting for asphalt over rubblized concrete base, asphalt over crushed stone, and asphalt over cement stabilized base pavement structures; in contrast, the models were found to be unbiased for asphalt overlays of existing flexible pavements. Darter et al. (2009) found that the MEPDG significantly over-predicted rutting in both new flexible pavements and asphalt overlays of existing flexible pavement in Utah. Zhou et al. (2013) found that the MEPDG significantly over predicted both total and asphalt layer rutting in asphalt overlays of both existing flexible pavement and existing rigid pavement in Tennessee.

In summary, the studies examined suggest that the nationally calibrated MEPG permanent deformation models consistently and significantly over-predict rutting in new flexible pavement structures, but the results are less consistent for asphalt overlays of existing flexible and rigid pavements.

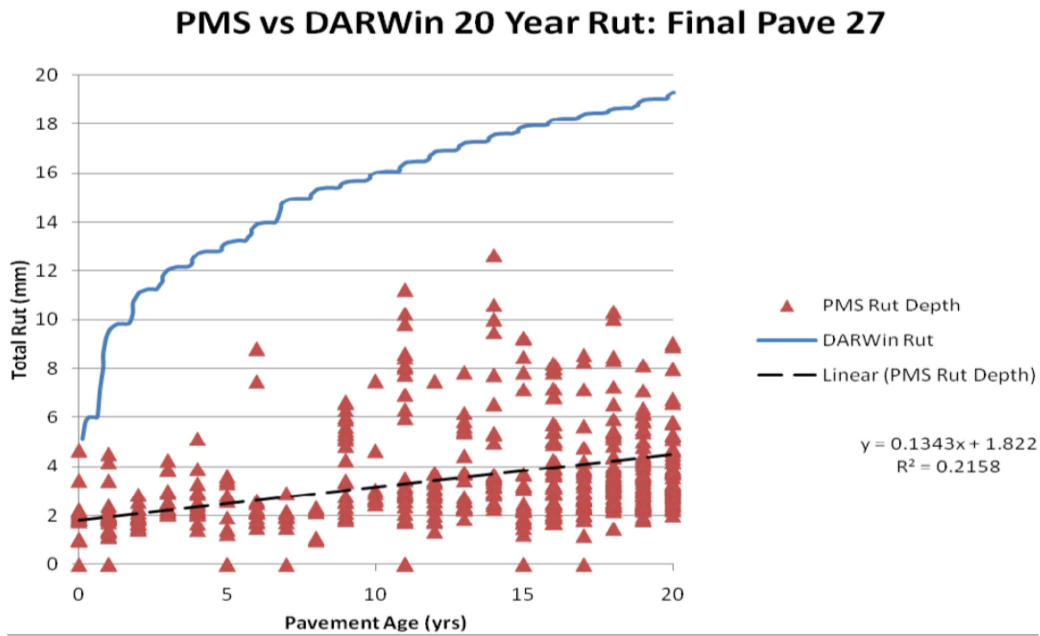


Figure 2-9: MEPDG Predicted versus PMS Measured Total Rutting for New Flexible Pavements in Alberta

The over prediction of total rutting in the MEPDG permanent deformation models has been attributed partly to an over prediction of rutting in the unbound layers, especially early in the pavement design life. As part of NCHRP Project 1-40B, Von Quintus, Darter & Mallella (2005b) determined that the MEPDG model used to predict plastic deformations in unbound layers over predicted rutting in the unbound pavement layers and subgrade soils. This result was verified in several validation studies undertaken by State DOTs. Hall et al. (2011) found that the nationally-calibrated MEPDG permanent deformation models significantly over predicted subgrade rutting while under predicting asphalt layer rutting, leading to an over prediction in total rutting for flexible pavements in Arkansas. He et al. (2011) found that much of the MEPDG predicted rutting for new flexible pavements in Alberta occurred early in the performance period in the base and subgrade layers, which resulted in a significant prediction in overall rutting over the pavement design life. Hoegh et al. (2012) found that the MEPDG predicted asphalt layer rutting matched well with the total permanent deformation observed for some pavement test sections at the MnRoad test facility in Minnesota. It was also observed that the MEPDG base and subgrade rutting predictions for the first month of pavement life were consistently and significantly over predicted. Hoegh et al. recommended that this rutting in the unbound layers during the first month be excluded from the analysis. Similar results were observed in studies conducted in Montana (Von Quintus et al. 2007), Louisiana (Wu et al. 2012), and Tennessee (Zhou et al. 2013).

2.5.2 Verification of the Flexible Pavement Bottom-Up Fatigue Cracking Model

A number of State DOTs have examined the nationally calibrated MEPDG bottom-up fatigue cracking model using data from the local PMS or LTPP test sections with mixed results. For example, the nationally calibrated MEPDG models were found to be reasonably accurate for studies conducted in Michigan (Buch et al 2008) and Utah (Darter et al 2009), although the latter study noted that all of the pavement sections examined had only low to moderate measured alligator cracking. However, the nationally calibrated MEPDG bottom-up fatigue cracking model was found to significantly under-predict measured alligator cracking in studies conducted in Arkansas (Hall et al. 2011), Wisconsin (Kang & Adams 2008), Washington (Li et al. 2009b), and Arizona (Souliman, Mamlouk, El-Basyouy, Zapata 2010). Von Quintus et al. (2007) found that the nationally calibrated MEPDG model under predicted alligator cracking for asphalt overlays of existing flexible pavement, but over predicted alligator cracking for new flexible pavements and in-place pulverization flexible pavements. It was also noted that the MEPDG model over predicted alligator cracking in pavements where pavement preservation techniques had been employed.

2.5.3 Verification of the Flexible Pavement Top-Down Fatigue Cracking Model

The nationally calibrated MEPDG top-down fatigue cracking model has been generally regarded as having very poor predictive power and low reliability (i.e. Schwartz 2007, Velasquez, Hoegh, Yut, Funk, Cochran, Marasteanu and Khazanovich 2009). It has also been noted that the initial calibration of the model had a very high standard of error (ARA 2004). Based on the work completed as part of NCHRP Projects 9-30 and 1-40B, it was recommended that the MEPDG longitudinal cracking model not be used or calibrated (Von Quintus, Andrei & Schwartz 2005a, Von Quintus et al. 2005b). State agencies that have attempted to validate and calibrate this model have generally experienced significant difficulty. For example, Hall et al. (2011) found that the MEPDG top-down fatigue cracking model significantly over predicted longitudinal cracking in Arkansas, and were unable to successfully recalibrate the model. Kang et al. (2008) found that the nationally calibrated MEPDG top-down fatigue cracking model significantly under predicted longitudinal cracking in Wisconsin, and the recalibrated model was also found to have very poor prediction power. Kim et al. (2010) did not attempt to validate or calibrate the MEPDG top-down fatigue cracking model for Iowa based on the recommendation of NCHRP Report 1-40B. Li et al. (2009b) and Souliman et al. (2010) found that the nationally calibrated MEPDG top-down fatigue cracking model over and under predicted longitudinal cracking for Washington and Arizona, respectively. Of the studies examined, only Michigan found the default MEPDG model to reasonably predict longitudinal cracking (Buch et al. 2008).

2.5.4 Verification of the Flexible Pavement Thermal Cracking Model

The nationally calibrated MEPDG transverse cracking model has been examined by a number of State DOTs using data from the local PMS or LTPP test sections with mixed results. The MEPDG model was found to accurately predict transverse cracking in both new flexible pavements and asphalt overlays in Utah and Washington (Darter et al. 2009 and Li et al. 2009b). However, Darter et al. (2009) noted that while the nationally calibrated MEPDG transverse cracking model was adequate for new flexible pavements with Superpave asphalt binders, it was inadequate for older flexible pavements that used conventional asphalt binders. Von Quintus et al. (2007) noted that the nationally calibrated MEPDG transverse cracking model over predicted transverse cracking in Montana and under predicted transverse cracking in LTPP sections in Idaho, North Dakota, South Dakota, Wyoming, Alberta, and Saskatchewan. Buch et al. (2008) found that the MEPDG transverse cracking model over predicted cracking compared to the Michigan PMS.

Von Quintus et al. (2007) commented that the nationally-calibrated MEPDG transverse cracking model has been found to be generally acceptable for flexible pavements in northern climates. However, Mallela et al (2009) noted that the MEPDG transverse cracking model generally overestimates asphalt creep compliance of HMA mixes and consequently underestimates thermal cracking, a discrepancy that is more prevalent in colder climates.

2.5.5 Verification of the Flexible Pavement International Roughness Index Model

The nationally calibrated MEPDG IRI model has generally been found to be the most accurate MEPDG flexible pavement distress models (Von Quintus et al. 2007). The original national calibration of the MEPDG IRI model was found to have reasonable standard error terms despite the hundreds of test sections with diverse pavement types and site conditions included from the LTPP database (ARA, 2004). The nationally calibrated MEPDG IRI model was found to be reasonably accurate in verification studies conducted in Michigan (Buch et al. 2008), Utah (Darter et al. 2009), Iowa (Kim et al. 2010), and Montana (Von Quintus 2007). Kim et al. (2010) found the MEPDG IRI model had good agreement for new flexible pavements, asphalt overlays of existing flexible pavements, and asphalt overlays of existing concrete pavement. In contrast, Mallela et al. (2009) found an extremely poor correlation ($R^2 < 0.008$) between predicted and measured IRI in Ohio; the MEPDG was found to over predict IRI for low values of measured IRI and under predict IRI for high values of measured IRI.

2.5.6 Verification of the Rigid Pavement Mean Joint Faulting Model

The reported accuracy of the globally-calibrated MEPDG mean joint faulting model is varied. The model was found to have acceptable correlation with measured values in Utah and Ohio and was not recalibrated in either state (Darter et al. 2009 and Mallela et al. 2009). In contrast, the nationally calibrated MEPDG mean joint faulting model was found to under predict measured faulting in Iowa (Kim et al. 2010), and predict faulting trends that were significantly different than those observed in Washington (Li et al. 2006).

2.5.7 Verification of the Rigid Pavement Transverse Cracking Model

The reported accuracy of the nationally calibrated MEPDG transverse cracking model is varied. The model was found to have very good correlation with measured values for JPCP pavements in Utah and Ohio and was not recalibrated (Darter et al. 2009, Mallela et al. 2009). However, the nationally calibrated MEPDG transverse cracking model was found to over predict observed cracking in Washington State (Li et al. 2006). The nationally calibrated MEPDG transverse cracking model could not be validated in Iowa since the definition of transverse cracking differed between the MEPDG and Iowa PMS (Kim et al. 2010).

2.5.8 Verification of the Rigid Pavement International Roughness Index Model

The reported accuracy of the nationally calibrated MEPDG rigid pavement IRI model is varied. Darter et al. (2009) found the model had a very good correlation for JPCP pavements in Utah and did not need to be recalibrated. Mallela et al. (2009) found that the globally-calibrated MEPDG rigid pavement IRI model had an excellent correlation ($R^2 = 0.98$) but significant bias between predicted and measured rigid pavement IRI. In contrast, Li et al. (2006) found that the MEPDG rigid pavement IRI model under predicted roughness in Washington State, while Guclu et al. (2009) found predicted rigid pavement IRI values were twice as high as actual measured values in the Iowa PMS.

2.5.9 Summary of MEPDG Verification Studies

In summary, the results of State DOT verification studies assessing the ability of the MEPDG pavement distress prediction models to accurately predict local pavement performance have been mixed. Since the MEPDG pavement distress prediction models were calibrated based on a national data set comprised of pavement test sections located across the United States, it is expected that their ability to accurately predict pavement distresses in each local highway agency will vary. This result highlights the importance of local verification, calibration, and validation of the MEPDG pavement distress models. However, the review of local verification studies did identify the following concerns that were consistently noted in the studies examined:

- The nationally calibrated MEPDG total permanent deformation model consistently over-predicted total pavement rutting for new flexible pavements. This has been attributed primarily to an over-prediction of rutting in the unbound layers early in the design life of the flexible pavement.
- The nationally calibrated MEPDG top-down fatigue cracking model has generally been found to have poor predictive power and low reliability. It has been recommended that this model not be used or calibrated, and attempts to calibrate the model have frequently been unsuccessful.

Chapter 3
Research Method and Data

3.1 Introduction

Chapter 3 presents the research method and data used to complete this research. Chapter 3 is organized as follows:

- Section 3.2 describes the research method used to compare the AASHTO 1993 and MEPDG pavement design methods.
- Section 3.3 gives an overview of the historical MTO pavement designs analyzed in this research.
- Section 3.4 describes the typical pavement design inputs used in the historical MTO pavement designs using the AASHTO 1993 method.
- Section 3.5 describes the pavement design inputs used for the MEPDG pavement designs completed as part of this research.
- Section 3.6 describes the pavement design software used to complete the MEPDG analysis.

3.2 Research Method

The purpose of this research was to conduct a comparative analysis of Ontario structural pavement designs using the AASHTO 1993 Guide for Design of Pavement Structures and the Mechanistic-Empirical Pavement Design Guide. The research method consisted of Tasks 2 to 5 shown in Figure 1-1.

In Task 2, a group of pavement structural designs were selected for inclusion in the analysis that represented the range of traffic loading, climatic conditions, pavement types, and construction materials characteristic of pavement design in Ontario. The first step was to obtain historical Pavement Design Reports (PDRs) completed for the MTO over the past 15 years for flexible pavements, rigid pavements, and asphalt overlays of existing flexible and rigid pavements. A group of PDRs representative of the different pavement types, traffic loading, climatic conditions, and construction materials was selected for analysis. Since the AASHTO 1993 method has been used as the primary pavement design method for the past 20 years, most of the historical PDRs reviewed used the AASHTO 1993 method for pavement structural design. The AASHTO 1993 pavement structural designs included in the PDRs were used as the AASHTO 1993 pavement design in the analysis. Section 3.3 provides an overview of the historical MTO AASHTO 1993 pavement designs selected for inclusion in this research.

Task 3 consisted of specifying the pavement design inputs to be used in the MEPDG analysis. Ideally, the pavement design inputs used in the MEPDG analysis would be exactly equivalent to the inputs used to complete the AASHTO 1993 pavement designs in the PDRs. However, specifying exactly equivalent pavement design inputs is not possible due to the very different manner in which inputs are characterized in the two design methods. As discussed in Section 2.2.3, the MEPDG uses a hierarchical approach to characterizing pavement design inputs based on level of detail. Even at the least detailed design input level (Level 3), the pavement design inputs required by the MEPDG are vastly more numerous and detailed than required for the AASHTO 1993 method. To address this challenge, project-specific information contained in the PDRs was used to determine MEPDG design inputs for variables common to both methods (i.e. AADTT). This information was supplemented by recommended MEPDG pavement design inputs developed by the MTO Materials Engineering and Research Office (MERO) Pavements and Foundations section and published in a November 2012 interim report entitled "Ontario's Default Parameters for AASHTOWare Pavement ME Design Interim Report" (MTO 2012). The philosophy behind this approach was to complete the MEPDG pavement designs and analysis based on how it would be completed with the quality of pavement design information available to MTO at this time of this research. The results of the comparative analysis will show how the MEPDG method compares to the AASHTO 1993 method if no further work were completed adapt, calibrate, and validate the MEPDG for Ontario conditions. Section 3.5 documents the MEPDG pavement design inputs used in the analysis.

The comparative analysis of the Ontario structural pavement designs using the AASHTO 1993 and MEPDG method was completed in two stages.

Task 4 was the first stage of the comparative analysis, and consisted of examining the MEPDG predicted pavement performance of the historical MTO flexible, rigid, and asphalt overlay pavement sections designed using the AASHTO 1993 pavement design method. The MEPDG analysis was completed using the nationally-calibrated pavement distress prediction models, since these models have not yet been calibrated and validated for Ontario conditions. The goal of this analysis was to determine if the two pavement design methodologies predict pavement performance in a consistent manner across a range of design conditions typical of Ontario. Since the historical AASHTO 1993 pavement structural designs within each highway functional classification were designed based the same levels of overall pavement performance (ΔPSI), it would be reasonable to expect that the MEPDG would predicted equivalent pavement performance for pavements within the same highway functional classification if the two pavement methodologies accounted for pavement performance in the same manner. Any discrepancies observed in the MEPDG predicted pavement performance trends would indicate that the pavement design methodologies did not predict pavement performance in the same manner. This analysis also examined the key factors that contributed to trends in MEPDG predicted pavement performance to determine which pavement designs parameters accounted for the difference in pavement performance predictions between the two methodologies. The results of this analysis are presented in Chapter 4.

Task 5 comprised the second stage of the comparative analysis, and consisted of comparing the MEPDG and AASHTO 1993 pavement design methods based on either the asphalt concrete or Portland cement concrete layer thickness of the pavement structure produced. This stage of the analysis examined the practical significance of the differences in predicted pavement performance observed between the two methodologies in Task 4. As previously noted, the AASHTO 1993 pavement structure design was available from the historical MTO PDRs. The MEPDG pavement structure design was determined using the following procedure:

1. Input the AASHTO 1993 pavement structure design into the MEPDG software.
2. Run the MEPDG software to determine the predicted pavement distresses in the AASHTO 1993 pavement structure design during the design period.
3. Determine whether the MEPDG predicted pavement distress values exceed the required pavement performance criteria:
    a. If the pavement performance criteria are exceeded, increase the thickness of the asphalt concrete or Portland cement concrete layer
    b. If the pavement performance criteria are not exceeded, decrease the thickness of the asphalt concrete or Portland cement concrete layer

4. Repeat steps 2 – 3 to determine the thinnest asphalt concrete or Portland cement concrete layer that satisfies the MEPDG pavement performance criteria. This was selected as the MEPDG pavement structure design.

The results of this analysis are documents in Chapter 4.

3.3 Selection of Historic Ontario Pavement Designs

The Ontario pavement designs and pavement design input information used in this research were obtained from historical MTO PDRs filed with the MTO MERO Pavements and Foundations Section. In total, 209 PDRs were obtained from the MTO MERO Pavements and Foundations section extending back to the year 1999. Of the 209 PDRs obtained, 123 were found to be suitable for use in this research. The remaining PDRs were unsuitable either because they were not designed using the AASHTO 1993 method, or more commonly, because the copy on file did not include all of the documentation (i.e. Appendices) required to determine the design inputs used in the AASHTO 1993 pavement structure design.

The historic pavement designs included in this research were selected to represent the range of pavement types, traffic loading, climatic conditions, and construction materials and methods typical in Ontario. A total of 140 pavement designs from 29 historical MTO PDRs were selected for inclusion in the analysis.

Figure 3-1 shows the distribution of the MTO historical pavement designs by pavement type and highway functional classification.

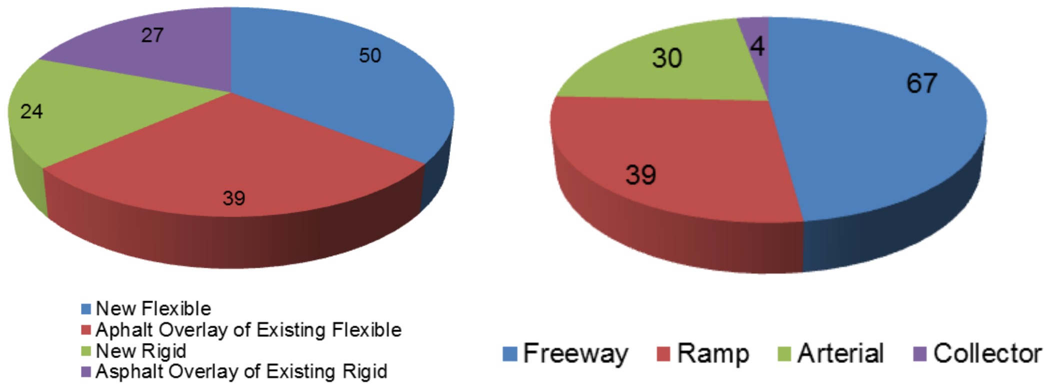


Figure 3-1: MTO Historical AASHTO 1993 Pavement Designs by Pavement Type and Highway Functional Classification

The four pavement design types included in the analysis were those most frequently employed by the MTO. As shown in Figure 3-1, most of the MTO pavement designs examined consisted of pavements on freeways or freeway ramps. This reflects the fact that the MTOs provincial highway network consists primarily of high-speed, controlled access highways designed to serve a high proportion of inter-regional trips. The very few collector roads included in the analysis were not actually under the jurisdiction of the MTO, but were roads rehabilitated or reconstructed as a result of MTO projects that intersected these roads. Thus, the results of this analysis will be primarily applicable to freeways and arterials, not collector and local roads.

Figure 3-2 shows the distribution of traffic loading, expressed as AASHTO 1993 ESALs, for the MTO pavement designs included in the analysis.

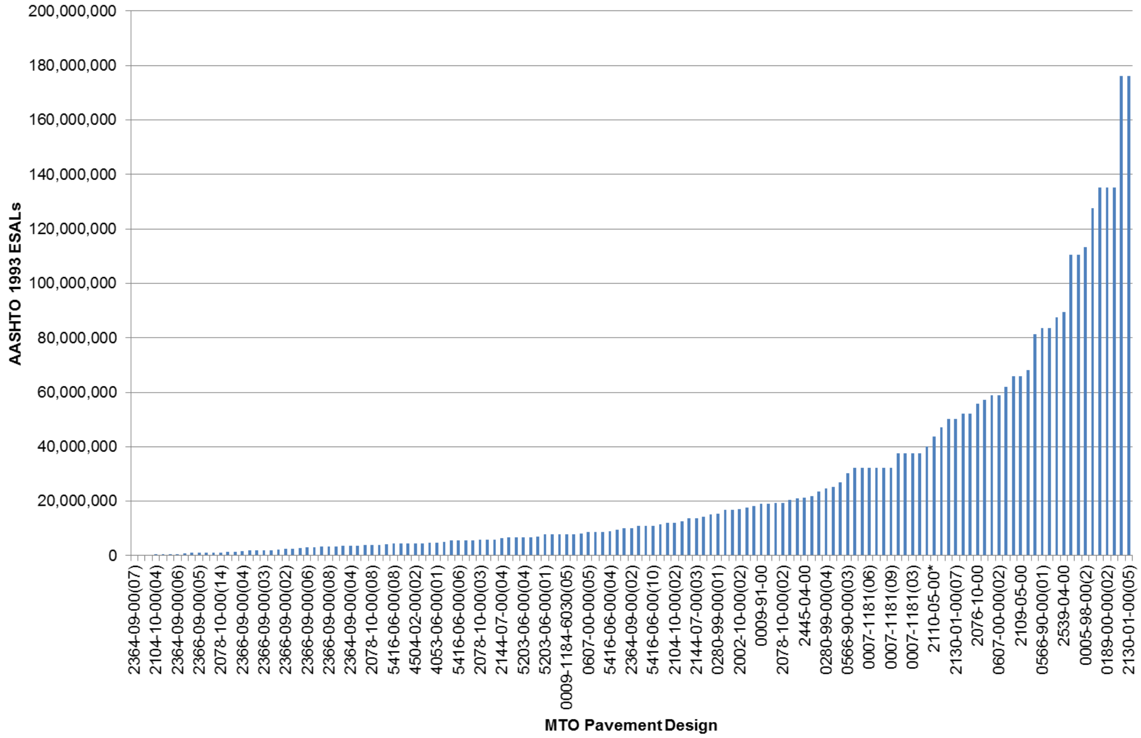


Figure 3-2: AASHTO 1993 ESALs over Design Period by Historical MTO Pavement Design

As shown in Figure 3-2, the MTO pavement designs included in the analysis were selected to represent the range of traffic loading experienced on MTO highways. Traffic loading ranged from approximately 500,000 to 180 million ESALs as determined in the AASHTO 1993 analysis. Figure 3-3 shows the distribution of historical MTO AASHTO 1993 pavement designs by traffic loading category.

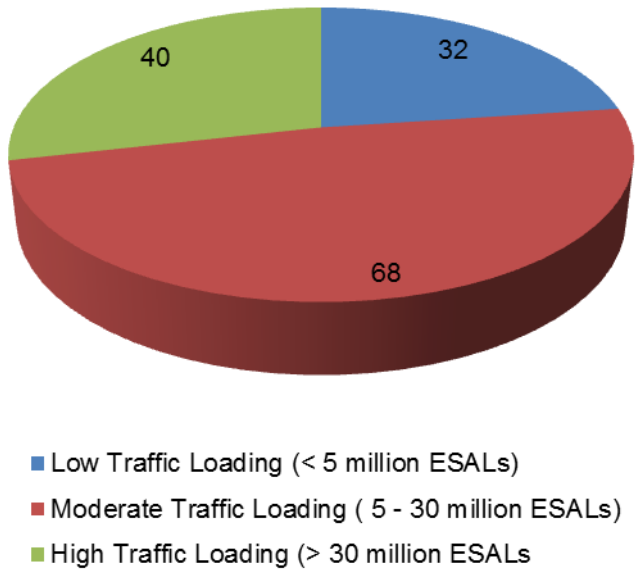


Figure 3-3: Historical MTO AASHTO 1993 Pavement Design by Traffic Loading Category

Figure 3-4 shows the distribution of historical MTO AASHTO 1993 pavement designs by the nearest climate station.

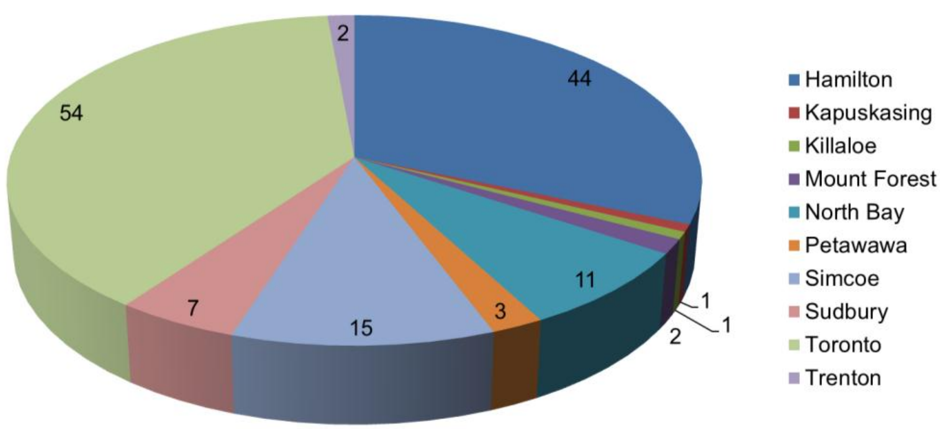


Figure 3-4: Historical MTO AASHTO 1993 Pavement Designs by MEPDG Climate Station

As shown in Figure 3-4, a high proportion of the pavement designs examined in this analysis were located in closest proximity to either the Toronto or Hamilton climate stations. This resulted from a number of different factors. First, almost all of the rigid pavement and asphalt overlay of rigid pavement designs included in the study were located in MTOs Central Region, which encompasses the regions of Niagara, Hamilton, Peel, Halton, York, Simcoe, Toronto, and Durham. This was because most of the highways with sufficiently high traffic volumes to warrant the use of rigid pavement were located in MTO Central Region. Second, a disproportionate number of the available MTO PDRs that were suitable for inclusion in the study were from MTOs Central Region. For example, many of the PDRs from the northern regions of Ontario either did not use the AASHTO 1993 method or contained insufficient documentation (i.e. Appendices) to determine the AASHTO 1993 design inputs. Third, the range of traffic volumes observed in the PDRs examined outside Central Region was relatively small compared to highways within Central Region. For example, very few of the pavement designs examined located outside of the Golden Horseshoe region of Ontario experienced traffic loading in excess of 15 million ESALs. Therefore, to select a range of pavement designs representative of the range of traffic loading in Ontario, it was necessary to select a higher proportion of pavement designs within MTOs Central Region. Finally, the distribution of climate stations in southern Ontario with sufficient data for MEPDG analysis was relatively sparse. As a result, the Toronto and Hamilton climate stations were used to represent most of the pavement sections in Central Region.

Figure 3-5 shows the distribution of historical MTO AASHTO 1993 pavement designs by the predominant subgrade material.

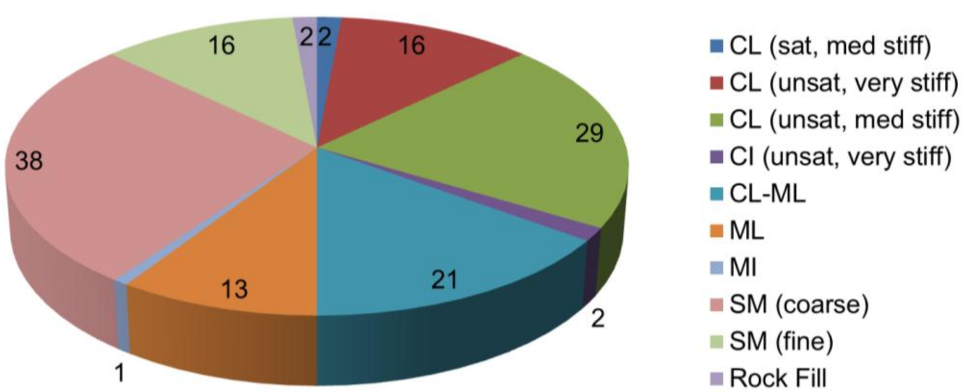


Figure 3-5: Historical MTO AASHTO 1993 Pavements Designs by Design Subgrade MTO Soil Classification

The distribution of design subgrades shown in Figure 3-5 is generally representative of the typical subgrade material types in Ontario, which are usually comprised predominantly of silty-sands, silts, and clays.

3.4 AASHTO 1993 Pavement Design Inputs

The AASHTO Guide for Design of Pavement Structures in its various versions has been used by the MTO to design highway pavement structures since 1990; the AASHTO 1993 Guide has been used for this purpose since its inception in 1993 (ERES 2008). In order to ensure that the AASHTO 1993 pavement design input parameters accurately reflected Ontario conditions and produced consistent and realistic performance trends, the MTO retained ERES Consultants, a division of Applied Research Associates, to complete a comprehensive study to adapt the AASHTO 1993 Design Guide to Ontario conditions and provide detailed guidelines for the selection of the various input parameters. The report, entitled "Adaptation and Verification of AASHTO Pavement Design Guide for Ontario Conditions", went through several revisions throughout the years with the Final Report published in March 19, 2008 (ERES 2008). The recommendations included in this report provided the basis for both consultant and MTO in-house pavement designers to select appropriate pavement design input parameters when completing pavement designs using the AASHTO 1993 method in Ontario. The purpose of this section is to briefly summarize the AASHTO 1993 pavement design input parameters recommended for use in Ontario conditions based on the results of the ERES report. These recommended pavement design input values were generally used in the historical MTO AASHTO 1993 pavement designs included in this study.

3.4.1 AASHTO 1993 General Design Parameters

3.4.1.1 Pavement Performance Period

The pavement performance period used in Ontario pavement designs varies by pavement design type. MTO typically uses a pavement performance period of 19 – 21 years for new flexible pavements, 28 years for new rigid pavements, and 10 – 13 years for asphalt overlays (Rangaraju, Amirkhanian, & Guven 2008). The range of pavement performance periods observed in the pavement designs included in the analysis were 15 – 40 years for new flexible pavements, 28 – 40 years for new rigid pavements, and 11 – 18 years for asphalt overlays.

3.4.1.2 Pavement Performance Measures and Reliability

Table 3-1 shows the pavement initial serviceability and terminal serviceability values recommended for use on MTO projects by highway functional classification (ERES 2008).

Table 3-1: Recommended AASHTO 1993 Pavement Performance Values for MTO Projects by Highway Functional Classification

Highway Functional Classification	Initial Serviceability (p_0)		Terminal Serviceability (p_t)
	New Construction	Rehabilitation	
Freeway	4.5	4.1 – 4.5	2.6
Arterial	4.5	4.1 – 4.5	2.5
Collector	4.4	4.0 – 4.4	2.2
Local	4.2	3.9 – 4.3	2.0

Table 3-2 shows the overall standard deviation and reliability values recommended for use on MTO projects by highway functional classification and pavement type (ERES 2008).

Table 3-2: Recommended AASHTO 1993 Values for Overall Standard Deviation and Reliability for MTO Projects

Highway Functional Classification	Overall Standard Deviation		Design Reliability
	Flexible	Rigid	
Freeways	0.44 – 0.49	0.34 – 0.39	90% - 95%
Arterial			85% - 95%
Collector			85% - 90%
Local			80% - 90%

3.4.2 AASHTO 1993 Traffic Inputs

The Average Annual Daily Traffic (AADT), percentage of commercial vehicles, and average annual traffic growth input values were typically assigned based on existing MTO data, site-specific survey data, or the most recent Commercial Vehicle Survey (CVS) in the PDRs examined.

Table 3-3 shows the typical truck values recommended for use on MTO projects by major truck class (ERES 2008). In general, the typical truck factors shown were used in most of the PDRs examined.

Table 3-3: Recommended AASHTO 1993 Typical Truck Factors

Major Truck Class	Typical Truck Factor	Range of Typical Truck Factors
2- and 3- axle trucks	0.5	0.05 – 1.00
4-axle trucks	2.3	0.20 – 4.00
5-axle trucks	1.6	0.30 – 3.50
6-axle trucks	5.5	2.00 – 7.00

In general, the historical MTO AASHTO 1993 pavement designs used a directional distribution factor of 0.5. Table 3-4 shows the lane distribution factor values recommended for use on MTO projects by number of lanes per direction and AADT (ERES 2008).

Table 3-4: Recommended AASHTO 1993 Lane Distribution Factors

Number of Lanes in One Direction	Average Annual Daily Traffic Volume (Both Directions)	Lane Distribution Factor
1	All	1.0
2	< 15,000	0.9
	> 15,000	0.8
3	< 25,000	0.8
	25,000 – 40,000	0.7
	> 40,000	0.6
4	< 40,000	0.7
	> 40,000	0.6
5	< 50,000	0.6
	> 50,000	0.6

3.4.3 AASHTO 1993 Environmental Inputs

The historical MTO AASHTO 1993 pavement designs examined did not utilize the AASHTO 1993 procedure for incorporating the effects of frost-heave or expansive subgrades into the pavement design. Similarly, the seasonally adjusted subgrade resilient modulus was not used in any of the historical MTO AASHTO 1993 pavement designs examined; instead, a single representative subgrade resilient modulus was selected.

Table 3-5 shows the drainage layer coefficient values recommended for use on MTO projects by observed drainage condition (ERES 2008).

Table 3-5: Recommended AASHTO 1993 Drainage Layer Coefficients for Unbound Granular Materials

Unbound Granular Material	Drainage Conditions	Recommended Drainage Coefficients
All granular materials recognized by MTO specifications, including Granular A with up to 50% RAP	**Standard** All typical situations involving new designs and standard drainage features.	1.0
	Questionable One or two minor deviations from accepted drainage standards.	0.9
	Inadequate Few minor deviations or one or more major deviations from accepted drainage standards.	0.5 – 0.8

In general, the historical MTO AASHTO 1993 pavement designs included in the analysis assigned a drainage coefficient value of 1.0 to new granular base and subbase layers, and a drainage coefficient value of 0.9 to existing granular base and subbase layers for rehabilitation projects.

Table 3-6 shows the rigid pavement drainage coefficient values recommended for use on MTO projects by overall drainage quality and climate (ERES 2008).

Table 3-6: Recommended AASHTO 1993 Rigid Pavement Drainage Coefficient Values

Overall Drainage Quality	Climate	
	Dry[1]	Wet[2]
Excellent	1.15 – 1.20	1.10
Good	1.10 – 1.15	1.00
Fair	1.00 – 1.10	0.90
Poor	0.90 – 1.00	0.80
Very Poor	0.80 – 0.90	0.70

[1] Precipitation less than or equal to 508 mm per year

[2] Precipitation greater than 508 mm per year

In general, most of the historical MTO AASHTO 1993 rigid pavement designs examined selected a value of 1.0 for the rigid pavement drainage coefficient regardless of overall drainage quality and climate.

3.4.4 AASHTO 1993 Pavement Structure Inputs

3.4.4.1 Subgrade Support Characterization

The AASHTO 1993 flexible pavement design procedure requires the pavement designer to input an effective subgrade soil resilient modulus to characterize the combined effect of all seasonal resilient modulus values. Recommended values of M_R soil resilient modulus based on MTO Soil Classification and typical subgrade condition have been developed for use on MTO projects; these recommended values are shown in Table 3-7 (ERES 2008).

Similarly, the AASHTO 1993 rigid pavement design procedure requires the pavement designer to input an effective modulus of subgrade reaction (k-value) to account for seasonal variations in the support provided to the concrete slab. Recommended values of effective modulus of subgrade reaction, based on the MTO Soil Classification and typical subgrade condition, have been developed for use on MTO projects; these values are shown in Table 3-8 (ERES 2008).

Table 3-7: Recommended AASHTO 1993 Subgrade Resilient Modulus (M_R) Values for Flexible Pavement Design

Brief Description	Category No.	MTO Classification	Drainage Characteristics	Susceptibility to Frost Action	Resilient Modulus (M_R) for Typical Subgrade Conditions (MPa)		
					Good	Fair	Poor
Rock, rock fill, shattered rock, boulders / cobbles	1	Boulders / cobbles	Excellent	None	90	80	70
Well graded gravels and sands suitable as granular borrow	2	GW, SW	Excellent	Negligible	80	70	50
Poorly graded gravels and sands	3	GP, SP	Excellent to fair	Negligible to slight	70	50	35
Silty gravels and sands	4	GM, SM	Fair to semi-impervious	Slight to moderate	50	35	30
Clayey gravels and sands	5	GC, SC	Practically impervious	Negligible to slight	40	30	25
Silts and sandy silts	6	ML, MI	Typically poor	Severe	30	25	18
Low plasticity clays and compressible silts	7	CL, MH	Practically impervious	Slight to severe	35	20	15
Medium to high plasticity clays	8	CI, CH	Semi-impervious to impervious	Negligible to severe	30	20	15

Note: M_R values should be reduced by 20% for locations in Northern Ontario

Table 3-8: Recommended AASHTO 1993 Effective Modulus of Subgrade Reaction (k) Values for Rigid Pavement Design

Brief Description	Category No.	MTO Classification	Drainage Characteristics	Susceptibility to Frost Action	Effective Modulus of Subgrade Reaction Values for Typical Subgrade Conditions (MPa)		
					Good	Fair	Poor
Rock, rock fill, shattered rock, boulders / cobbles	1	Boulders / cobbles	Excellent	None	140	110	100
Well graded gravels and sands suitable as granular borrow	2	GW, SW	Excellent	Negligible	120	100	80
Poorly graded gravels and sands	3	GP, SP	Excellent to fair	Negligible to slight	110	90	70
Silty gravels and sands	4	GM, SM	Fair to semi-impervious	Slight to moderate	110	70	60
Clayey gravels and sands	5	GC, SC	Practically impervious	Negligible to slight	90	60	40
Silts and sandy silts	6	ML, MI	Typically poor	Severe	80	40	20
Low plasticity clays and compressible silts	7	CL, MH	Practically impervious	Slight to severe	60	30	15
Medium to high plasticity clays	8	CI, CH	Semi-impervious to impervious	Negligible to severe	60	30	10

Note: k-values should be reduced by 20% for locations in Northern Ontario

For flexible pavements, most of the historical MTO AASHTO 1993 pavement designs reviewed in this study had subgrade resilient modulus values of 25 – 30 MPa; the values for effective modulus of subgrade reaction in rigid pavements varied between 30 and 103.

3.4.4.2 Unbound and Stabilized Base Materials Characterization

The AASHTO 1993 pavement design method assigns each pavement layer a structural layer coefficient (a_i) to characterize the relative ability of the material to function as a structural component of the pavement.

Table 3-9 shows the structural layer coefficient values for unbound and stabilized base materials recommended for use on MTO projects (ERES 2008).

Table 3-9: Recommended AASHTO 1993 Structural Layer Coefficient Values for Unbound and Stabilized Base Materials

Layer Material Type	AASHTO 1993 Structural Layer Coefficient (a_i)
Granular A	0.14
Granular A with up to 50% RAP	0.14
Pulverize bituminous surface mixed with existing granular material (up to 50% RAP)	0.10 – 0.14
Existing Granular A (with or without RAP)	0.10 – 0.14
Granular B, Type I	0.09
Existing Granular B, Type I	0.05 – 0.09
Granular B, Type II	0.09 – 0.14
Existing Granular B, Type II	0.06 – 0.14
Granular O	0.14
Rubblized PCC Slab	0.14 – 0.30
Open Graded Drainage Layer (Untreated, Asphalt Stabilized, or PCC Stabilized)	0.14
Portland Cement Treated Base	0.28 – 0.34

The structural layer coefficients assigned to the existing granular and stabilized base layers was not consistent across the historical MTO AASHTO 1993 pavement designs examined; however, the structural layer coefficient values assigned generally varied within the recommended ranges shown in Table 3-9.

3.4.4.3 Asphalt Material Characterization

Table 3-10 shows the structural layer coefficient values for bituminous materials recommended for use on MTO projects (ERES 2008).

Table 3-10: Recommended AASHTO 1993 Structural Layer Coefficient Values for Bituminous Materials

Layer Material Type	AASHTO 1993 Structural Layer Coefficient (a_i)
New or recycled hot mix asphalt	0.42
Existing hot mix asphalt	0.14 – 0.28
Cold recycling of RAP off-site or in-place	0.28 – 0.38
Cold In-Place Recycling with Expanded Asphalt Cement	0.20 – 0.25
Existing Cold Mix Asphalt	0.11 – 0.24
Bituminous treated Granular A (3% AC content)	0.31

As with the granular and stabilize base materials, the structural layer coefficients assigned to the existing bituminous layers was not consistent across the historical MTO AASHTO 1993 pavement designs examined; however, the structural layer coefficient values assigned generally varied within the recommended ranges shown in Table 3-10.

3.4.4.4 Concrete Material Characterization

The PCC modulus of rupture (S_c`) and elastic modulus of concrete (E_{PCC}) values recommended for use on MTPO projects was 5.0 MPa and 30 GPa, respectively (ERES 2008). These values were used in the historical MTO AASHTO 1993 rigid pavement designs included in the analysis.

3.5 MEPDG Pavement Design Inputs

The purpose of this section is document the pavement design inputs used in the MEPDG analysis completed using the AASHTOWare Pavement ME Design software. As previously mentioned, the MTO MERO Pavements and Foundations sections released a document in November 2012 entitled "Ontario's Default Parameters for AASHTOWare Pavement ME Design Interim Report" (MTO

2012). This interim document introduced customized default MEPDG traffic, climatic, and materials input parameters for pavement design in Ontario conditions (MTO 2012). The recommended input values included in this document served as the basis for the input values used in the MEPDG analysis.

3.5.1 MEPDG General Project Information Inputs

3.5.1.1 Design Analysis Life

The design life of the pavements was obtained directly from the PDRs to match the design life of the AASHTO 1993 pavement design.

3.5.1.2 Construction and Traffic Opening Dates

Since the Pavement Design Reports were prepared during the design phase of the contracts, they did not specify when the contract was constructed. Therefore, the construction and traffic opening dates selected were not based on historical construction data; instead, the following dates were selected for all the pavement designs considered in this study:

- Base Construction: May 2014
- Pavement Construction: July 2014
- Traffic Opening: September 2014

For asphalt overlays of existing pavements, the original construction date for the existing pavement construction was either obtained directly from the PDR, or estimated based the construction and rehabilitation history included in the PDR.

3.5.1.3 Performance Criteria and Reliability

The MEPDG requires the pavement designer to specify performance criteria thresholds for each of the pavement distress models included in the MEPDG. The pavement performance thresholds are the maximum amount of distress in the pavement before rehabilitation is performed. The pavement performance thresholds need to be satisfied at the specified reliability level over the entire design life of the pavement in order for the pavement design to pass the MEPDG evaluation.

The six performance criteria for new flexible pavement structures were: Terminal International Roughness Index (IRI) (m/km); Top-Down Fatigue Cracking (m/km); Bottom-Up Fatigue Cracking (% Total Area); Thermal Cracking (m / km); Permanent Deformation – Total Pavement (mm); and

Permanent Deformation – AC (mm). Asphalt overlays also included a Total Cracking (Reflective + Alligator) performance criterion. The three performance criteria for new rigid pavement structures were: Terminal IRI (m/km); Transverse Cracking (% Total Slabs); Mean Joint Faulting (mm).

In addition to specifying a Terminal IRI value, the pavement designer also specifies an initial IRI value representing the smoothness of the pavement surface immediately subsequent to construction or rehabilitation. MTO has developed recommended initial IRI values for various highway functional classifications and pavement rehabilitation treatments based on historical IRI data contained in the MTO Pavement Management System 2 (PMS-2). Table 3-11 shows the MTO recommended initial IRI values for the pavement design projects included in this study (MTO 2012).

Table 3-11: MEPDG Initial IRI Inputs (m/km)

Treatments	**Highway Functional Class**		
	Freeway	**Arterial**	**Collector**
New or Reconstruction to AC	0.8	1.0	1.0
New or Reconstruction to PCC	1.3	1.5	-
HMA Overlay 1 Lift	1.0	1.0	1.0
HMA Overlay 2 Lifts	0.9	0.9	0.9
Mill and HMA Overlay	1.0	1.0	1.0
CIR and HMA Overlay 1 Lift	-	1.0	1.0
CIR and HMA Overlay 2 Lifts	-	0.9	1.0
FDR and HMA Overlay	0.9	0.9	0.9
FDR with EAS and HMA Overlay	-	0.9	0.9
Mill to Concrete and HMA Overlay 2 Lifts	0.9	1.0	1.1

Table 3-12 shows the MTO recommended values for terminal IRI based on highway function class and pavement type employed in this study (MTO 2012).

Table 3-12: MEPDG Terminal IRI Inputs

Highway Functional Class	Terminal IRI (m/km)	
	Asphalt	Concrete
Freeway	1.9	2.4
Arterial	2.3	2.7
Collector	2.7	2.7

MTO has not yet developed performance criteria thresholds for the remainder of the pavement distresses predicted by the MEPDG; therefore, MTO currently recommends using of the default performance criteria thresholds recommended for use with the MEPDG and AASHTOWare Pavement ME Design software. These default performance criteria, which were used in the MEPDG analysis completed as part of this research, are provided in Table 3-13 (MTO 2012).

Table 3-13: MEPDG Pavement Performance Criteria

Performance Criteria	Target Values
Flexible Pavements	
AC Top-Down Fatigue Cracking (m/km)	380
AC Bottom-Up Fatigue Cracking (%)	Freeway: 10 Arterial: 20 Collector: 35
AC Thermal Cracking (m/km)	190
Permanent Deformation – Total Pavement (mm)	19
Permanent Deformation – AC Only (mm)	6
Total Cracking (Reflective + Alligator) (%)	100
Rigid Pavements	
Transverse Cracking (% slabs)	Freeway: 10 Arterial: 15 Collector: 20
Mean Joint Faulting (mm)	3

In addition to the performance criteria thresholds, the MEPDG requires the pavement designer to specify a reliability level for each pavement distress model. The reliability level represents the probability that the predicted distress will be less than the specified threshold over the design life of the pavement structure (AASHTO 2008). MTO has not developed recommended reliability levels specific for use in Ontario pavement design, and recommends the use of the default reliability levels included in the MEPDG. Table 3-14 provides the default MEPDG reliability levels used for the pavement designs included in this research.

Table 3-14: MEPDG Design Reliability Inputs

Highway Functional Class	Reliability	
	Urban	Rural
Freeway	95%	95%
Arterial	90%	85%
Collector	80%	75%

3.5.2 MEPDG Traffic Inputs

The MEPDG requires much more extensive and comprehensive traffic design inputs than the AASHTO 1993 pavement design method. In addition to standard traffic inputs such as Average Annual Daily Truck Traffic (AADTT) and annual truck traffic growth rate, the MEPDG requires the pavement designer to provide detailed inputs regarding traffic distribution including number of axles per truck, axle load distribution, monthly traffic distribution, and hourly traffic distribution.

3.5.2.1 MTO iCorridor

The MTO Systems Analysis and Forecasting Office (SAFO) has developed a web-based mapping program called iCorridor that includes the capability to download detailed traffic and other data for highway segments under MTO's jurisdiction (MTO 2013). One of the modules included in iCorridor allows the user to download site specific traffic data for use in MEPDG pavement design using the AASHTOWare Pavement ME software (see Figure 3-6). This site specific traffic data corresponds to Level 1 MEPDG traffic inputs, and includes AADTT, vehicle class distribution, number of axles per truck, and axle load distributions for single, tandem, tridem, and quad axles. The Level 1 traffic data obtained from iCorridor formed the foundation of the MEPDG traffic inputs used for the pavement designs in this research, and were modified as appropriate to reflect the historical traffic condition of the pavement sections included in the study.

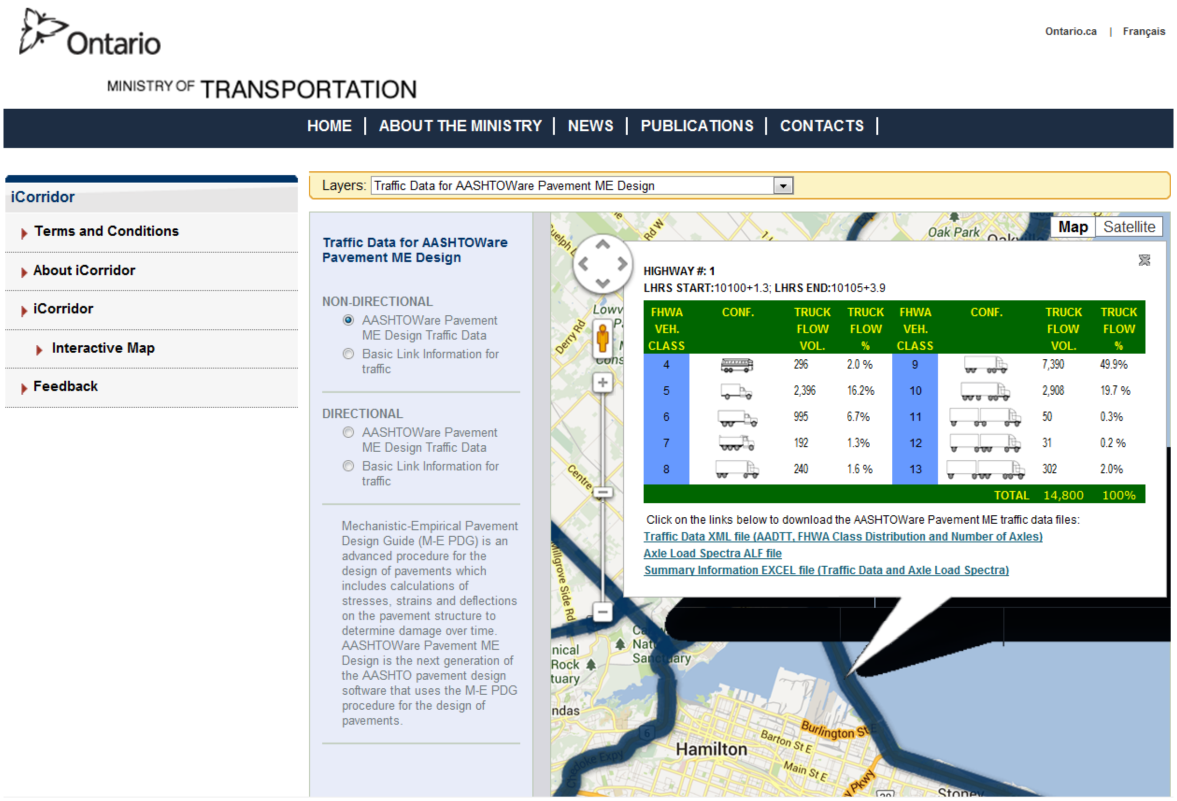


Figure 3-6: MTO iCorridor Web-Based Application

3.5.2.2 Traffic Volume

The MEPDG characterizes traffic volume over the design life of the pavement using the following input variables: base year Average Annual Daily Truck Traffic (AADTT); percentage of trucks in design direction; percentage of trucks in design lane; and, average annual growth in truck traffic. Since the traffic data provided by iCorridor represents the most recent traffic data available for each highway segment, the data was not necessarily representative of the traffic volumes at the time the historical pavement designs included in this study were designed and constructed. Therefore, the traffic volume inputs used in the MEPDG analysis were obtained directly from the Pavement Design Reports (PDRs).

It should be noted that the PDRs frequently assumed that the average annual growth rate in truck volume would vary over the design life of the pavement; however, the MEPDG only permits the pavement designer to input a single annual traffic growth rate over the performance period. In these cases, a weighted average of the average annual growth rate was calculated over the design life of the pavement and used in the MEPDG analysis.

3.5.2.3 Traffic Capacity

The AASHTOWare Pavement ME program permits the pavement designer to specify whether to enforce a cap on traffic volume based on the actual traffic capacity of the highway corridor. Based on the recommendation of MTO, and in the absence of any traffic capacity data for the pavement sections designed in this report, the “not enforced” option was selected for all MEPDG simulations. This assumption corresponds to the AASHTO 1993 design approach which does not use a traffic volume cap concept.

3.5.2.4 Operational Speed

The MEPDG requires the pavement designer to input the operational speed of the highway. It was assumed that the operational speed of the highways included in the study was equal to the posted speed limit. This is consistent with the assumption used in the national calibration of the MEPDG pavement distress models (AASHTO 2008). For freeway ramps, an operational speed of 60 km / hour was assumed.

3.5.2.5 Monthly and Hourly Traffic Distribution

The MEPDG requires the pavement designer to input monthly traffic distribution to account for seasonal variations in traffic over the calendar year. The MEPDG default for monthly distribution assumes that traffic volume is evenly distributed throughout all of the months of the year. Since MTO also recommends assuming a uniform distribution of traffic throughout the months of the year, this assumption was used for the MEPDG analysis completed in this study (MTO 2012).

The MEPDG also requires the pavement designer to input an hourly traffic distribution for rigid pavement design. Figure 3-7 shows the MEPDG default hourly traffic distribution that was used in the MEPDG analysis.

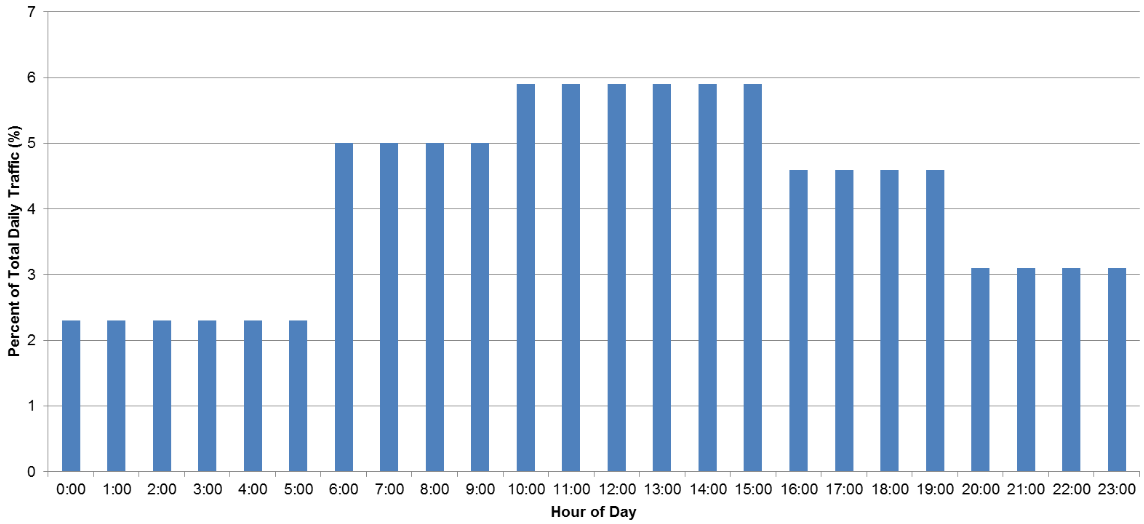


Figure 3-7: Default MEPDG Hourly Traffic Distribution

3.5.2.6 Axle Configuration

The AASHTOWare Pavement ME software also requires traffic inputs for axle configuration and spacing. Based on the MTO 2002 Commercial Vehicle Survey (CVS), the default axle configuration inputs included in the MEPDG and AASHTOWare Pavement ME software are applicable for Ontario pavement design (MTO 2012). These values, which were used in the MEPDG simulations, are provided in Table 3-15 (MTO 2012).

Table 3-15: MEPDG Axle Configuration Inputs

Axle Configuration	Values
Average Axle Width (m)	2.59
Dual Tire Pressure (mm)	305.00
Tire Pressure (kPa)	827.40

The MTO 2002 CVS determined that the average spacing between axles within each axle group in Ontario differed from the default values provided in the MEPDG. The Ontario values for average axle spacing, which were used in the MEPDG analysis, are provided in Table 3-16 (MTO 2012).

Table 3-16: MEPDG Axle Spacing Inputs

Axle Type	Average Axle Spacing Within Axle Group (m)
Tandem	1.45
Tridem	1.68
Quad	1.32

3.5.2.7 Vehicle Lateral Wander

The MTO 2002 CVS determined that the default MEPDG and AASHTOWare Pavement ME design values for traffic lateral wander were applicable to Ontario conditions (MTO 2012). These values, which were used in the MEPDG analysis, are provided in Table 3-17 (MTO 2012).

Table 3-17: MEPDG Lateral Traffic Wander Inputs

Factors	Default Values
Mean Wheel Location (mm)	460
Traffic Wander Standard Deviation (mm)	254
Design Lane Width (m)	3.75

3.5.2.8 Average Axle Spacing

Table 3-18 shows the MTO recommended values for average axle spacing between axle groups for the various truck types (MTO 2012); these values were used in the MEPDG analysis included in this research.

Table 3-18: MEPDG Typical Spacing between Major Axle Groups Inputs

Truck Type	Average Axle Spacing Between Axle Groups (m)	Percent of Trucks
Short	5.1	33%
Medium	4.6	33%
Long	4.7	34%

3.5.2.9 Axles per Truck

The MEPDG uses the Federal Highway Administration (FHWA) truck classification system to categorize truck traffic. Level 1 inputs for the number and type of axles for each FHWA truck class were obtained from the MTO *iCorridor* web-based application for existing highways under MTO jurisdiction. For highways that did not have traffic data information available from *iCorridor*, the axles per truck inputs provided in Table 3-19 and Table 3-20 were used in Southern Ontario and Northern Ontario, respectively, as recommended by MTO. These values were based on the results of the 2006 CVS conducted by MTO (MTO 2012).

Table 3-19: Southern Ontario Typical Axle per Trucks Table

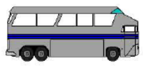
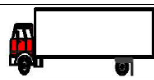
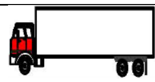
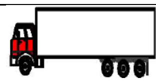
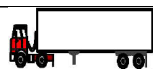
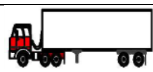
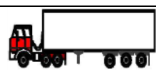
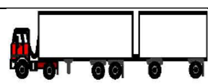
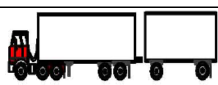
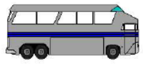
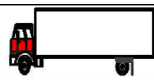
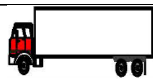
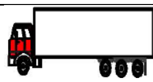
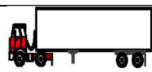
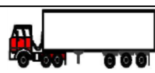
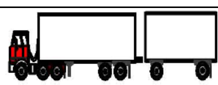
FHWA Class		Singles	Tandems	Tridems	Quads	Total
4		1.620	0.390	0.000	0.000	2.400
5		2.000	0.000	0.000	0.000	2.000
6		1.010	0.993	0.000	0.000	2.996
7		1.314	0.989	0.030	0.000	3.382
8		2.163	0.845	0.000	0.000	3.853
9		1.055	1.968	0.003	0.000	5.000
10		1.446	1.234	0.700	0.088	6.366
11		4.546	0.168	0.000	0.000	4.882
12		2.857	1.526	0.000	0.000	5.909
13		1.201	2.058	0.848	0.024	7.957

Table 3-20: Northern Ontario Typical Axle per Trucks Table

FHWA Class	Singles	Tandems	Tridems	Quads	Total
4 	1.620	0.390	0.000	0.000	2.400
5 	2.000	0.000	0.000	0.000	2.000
6 	1.014	0.993	0.000	0.000	3.000
7 	1.244	0.962	0.043	0.000	3.297
8 	2.414	0.674	0.000	0.000	3.762
9 	1.048	1.955	0.014	0.000	5.000
10 	1.358	1.165	0.840	0.044	6.384
11 	3.849	0.538	0.000	0.000	4.925
12 	2.910	1.514	0.021	0.000	6.001
13 	1.100	2.012	0.945	0.011	8.003

3.5.2.10 Vehicle Class Distribution

For existing highways under MTO jurisdiction, the vehicle class distribution was obtained directly from the MTO *iCorridor* web-based application. For extensions of existing highways (i.e. Highway 404 extension), the vehicle class distribution on the highway segment immediately adjacent to the highway extension was taken to be representative of the vehicle class distribution on the highway extension. The vehicle class distribution on freeway ramps was assumed to be the same as the vehicle class distribution on the corresponding mainline of the freeway. For highways not under MTO jurisdiction, default MEPDG Truck Traffic Classification (TTC) groups that best represented the assumed truck traffic mix were selected. For example, for collector roads, TTCs with a higher proportion of single unit trucks were selected.

3.5.2.11 Axle Load Distribution

For existing highways under MTO jurisdiction, the axle load distributions were obtained directly from the MTO iCorridor web-based application. For extensions of existing highways (i.e. Highway 404 extension), the axle load distributions on the highway segment immediately adjacent to the highway extension was taken to be representative of the axle load distributions on the highway extension. The axle load distributions on freeway ramps were assumed to be the same as the axle load distributions on the corresponding mainline of the freeway.

MTO has developed axle load spectrum data for Ontario conditions using data from the 2006 CVS data. Separate axle load spectrum tables have been developed for Northern Ontario and Southern Ontario, and can be found in Appendix A. These axle load spectra were used for highways where no axle load spectrum data was available or assumed using the above method.

3.5.3 MEPDG Environmental Inputs

The MEPDG method requires detailed climate data for use in the mechanistic Enhanced Integrated Climate Model (EICM). This climate data is typically obtained from existing climate data stations. At the time of this report, thirty-four Environment Canada weather stations in Ontario were available for use in the AASHTOWare Pavement ME application (see Figure 3-8).

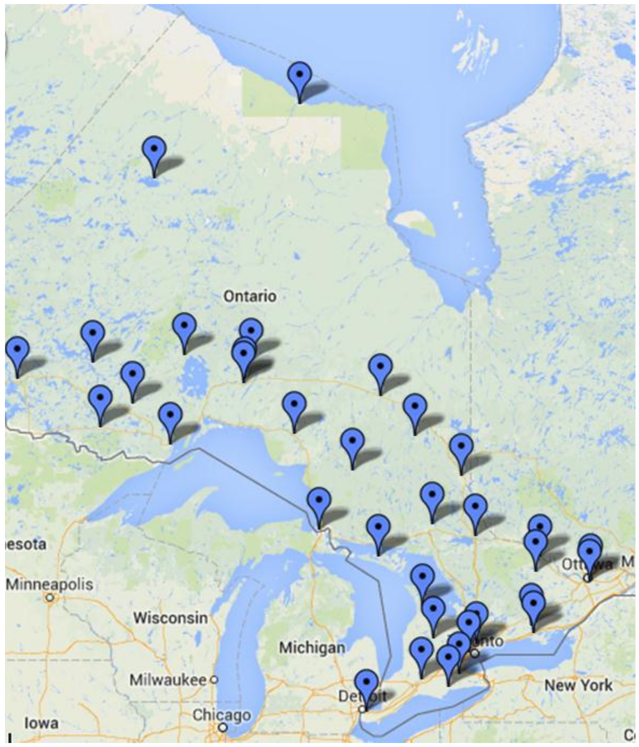


Figure 3-8: Ontario Weather Stations

The AASHTOWare Pavement ME application permits the user to input climate data in two ways. The pavement designer may select a single weather station to represent the climate data for the project. Alternatively, the pavement designer may create a virtual weather station for the project location using data interpolated from up to six weather stations located in proximity to the project location. The climate data from each weather station is assigned a weighting based on its proximity to the project location, and the various climate inputs are averaged based on their assigned weights to create the virtual weather station data.

For the MEPDG analysis completed as part of this research, a single weather station was used to represent the climate at every project location. This is consistent with MTO's recommendation for the use of climate data in MEPDG analysis. Most of the weather stations included in this study contained significant data errors and omissions. For example, the Lester B Pearson International Airport weather station had several years of data missing within the dataset included in the AASHTOWare Pavement ME software. Other weather stations had clearly erroneous data inputs, such as ambient air temperature changes from 30°C to -21°C within a single hour. When using a single weather station, the AASHTOWare Pavement ME program flags missing and suspicious climate data to the pavement designer so it can be corrected or adjusted as appropriate prior to running the performance simulation. When using a virtual weather station, the climate data from the weather stations is used without any correction or adjustment, leading to the incorporation of any erroneous climate data into the virtual weather station. This can lead to significant variation in the MEPDG pavement performance predictions, especially for pavement distresses that are highly affected by climate data (see Johanneck and Khazanovich 2010). For this reason, only a single weather station was used for the MEPDG analysis completed as part of this research. The climate data was cleaned and corrected prior to running an MEPDG analysis.

3.5.4 MEPDG Pavement Structure Layer Inputs

MTO has developed default MEPDG inputs for common pavement construction materials and designs used in Ontario. These inputs are equivalent to Level 3 MEPDG inputs and are recommended for use when no project-specific information is available. Given that many of the inputs required for a Level 1 MEPDG analysis were not required for AASHTO 1993 pavement design, the MTO PDRs examined did not include the level of detail required for Level 1 inputs of pavement material information. Therefore, the MTO recommended Level 3 inputs included in the following sections were used for the MEPDG analysis conducted for this research.

3.5.4.1 Hot Mix Asphalt Material Properties

MTO has developed Level 3 asphalt concrete material properties for Superpave and Marshall asphalt mixes commonly used in Ontario based on previous contract mix design information. Table 3-21 shows the recommended Level 3 MEPDG asphalt concrete material properties inputs for use in MTO pavement design projects (MTO 2012).

Table 3-21: MEPDG Asphalt Concrete Properties Inputs

Asphalt Layers		SP 12.5	SP 19.0	SP 25.0	SMA 12.5	Existing Asphalt Northern Ontario	Existing Asphalt Southern Ontario
Unit Weight (kg / m^3)		2460[1]	2460	2469	2520	2480	2460[1]
Effective Binder Content by Volume (%)		11.8	11.2	10.4	14.6	12.2	10.9
Air Voids (%)		4.0%					
Poisson's Ratio		0.35					
Dynamic Modulus		"Input Level 3"					
Aggregate Gradation	% Passing 19mm Sieve	100.0%	96.9%	89.1%	100.0%	100.0%	97.0%
	% Passing 9.5mm Sieve	83.2%	72.5%	63.3%	73.1%	72.0%	63.0%
	% Passing 4.75mm Sieve	54.0%	52.8%	49.3%	29.7%	53.5%	42.5%
	% Passing 75µm Sieve	4.0%	3.9%	3.8%	9.3%	3.0%	3.0%
G Star Predictive Model		"Use Viscosity Based Model (Nationally Calibrated)"					
Reference Temperature		21.1^{o}C					
Asphalt Binder[2]		PG 64-28	PG 58-28	PG 58-28	PG 70-28	Penetration Grade	
Indirect Tensile Strength – 10^{o}C (MPa)		Calculated					
Creep Compliance (1 / GPa)		"Input Level 3"					
Thermal Conductivity (W/m-Kelvin)		1.16					
Heat Capacity (J/kg-Kelvin)		963					
Thermal Contraction		Calculated					

[1] For SP 12.5FC1, FC2, and SMA 12.5: Central and North Regions 2,520 kg/m^3, East Region 2,390kg/m^3, West Region 2,530 kg/m^3

[2] New HMA mixtures asphalt binder obtained from Pavement Design Report or default value used. For existing HMA assumed Pen Grade 85-100 in S. Ontario, Pen Grade 200-300 in N. Ontario

The asphalt concrete material properties for existing asphalt layers was not available for the historical MTO pavement designs examine in this study. Consequently, the asphalt concrete material properties had to be assumed for all existing pavement layers. The assumed asphalt concrete material properties were based on an HL-4 Marshall Mix for Northern Ontario and an HL-8 Marshall Mix for Southern Ontario. These asphalt mixes were commonly used as asphalt binder courses in the respective regions prior to the implementation of Superpave mixes on all MTO projects. It should be noted that the asphalt concrete material properties used for existing pavements were developed to represent asphalt layers at time of initial construction, and may not necessarily represent the material properties of aged asphalt concrete layers. This is a limitation of the assumption made with respect to the asphalt material properties of existing asphalt concrete layers.

Table 3-22 shows the MTO recommended values for MEPDG asphalt stabilized material properties design inputs (MTO 2012). The asphalt stabilized materials used by the MTO include Cold-In Place Recycled (CIR) asphalt, Cold-In Place Recycled Asphalt with Expanded Asphalt Mix (CIREAM), and Expanded Asphalt Stabilization (EAS).

Table 3-22: MEPDG Stabilized Material Properties Design Inputs

Asphalt Layers		CIR	CIREAM	EAS
Unit Weight (kg/m^3)		2240	2110	2170
Effective Binder Content – by Volume (%)		12.5	13.5	11.7
Air Voids (%)		9.0	13.5	10.0
Poisson's Ratio		0.35	0.35	0.35
Dynamic Modulus		"Input Level 3"		
Aggregate Gradation	% Passing 19mm Sieve	100%	100%	97%
	% Passing 9.5mm Sieve	83%	83%	73%
	% Passing 4.75mm Sieve	63%	63%	58%
	% Passing 75μm Sieve	6%	6%	7%
G Star Predictive Model		"Use Viscosity Based Model (Nationally Calibrated)"		
Reference Temperature		21.1^{o}C		
Asphalt Binder[2]		Note 1	PG 58-28	PG 58-28
Indirect Tensile Strength – 10^{o}C (MPa)		Calculated		
Creep Compliance (1 / GPa)		"Input Level 3"		
Thermal Conductivity (W/m-Kelvin)		1.16		
Heat Capacity (J/kg-Kelvin)		963		
Thermal Contraction		Calculated		

Note 1: PGAC should be the same as binder grade of asphalt materials

As noted in Table 3-22, MTO recommends that the PGAC grade of CIR be equal to the PGAC grade of the existing asphalt materials. However, the PGAC grade of the existing asphalt materials was not known for the pavement designs examined in this study. Therefore, a PGAC grade of PG 58-28 was assumed for CIR in Southern Ontario, and a PGAC grade of PG 58-34 was assumed for CIR in Northern Ontario.

The amount of pre-overlay rutting in the existing flexible pavements was not known for the pavements examined in this study. As recommended by MTO, it was assumed that pre-overlay rutting in existing flexible pavements was equal to 7 mm. This value was the average pre-overlay rutting recorded in the PMS for MTO pavements at time of overlay (MTO 2012).

3.5.4.2 Concrete Material Properties

MTO has developed Level 3 concrete material properties for concrete pavement mixes commonly used in Ontario. Table 3-23 shows the recommended Level 3 MEPDG concrete material properties inputs for use in MTO pavement design projects (MTO 2012).

Table 3-23: MEPDG PCC Properties Inputs

PCC	
Unit Weight (kg/m^3)	2320
Poisson's Ratio	0.20
PCC Thermal Properties	
PCC Coefficient of Thermal Expansion (mm/mm C^o x 10^{-6})	7.80
PCC Thermal Conductivity (watt/meter-Kelvin)	2.16
PCC Heat Capacity (joule/kg-Kelvin)	1172
PCC Cement Mix	
Cement Type	GU (Type 1)
Cementitious Material Content (kg / m^3)	335.0
Water / Cement Ratio	0.45
Aggregate Type	Limestone
PCC Set Temperature	Calculated
Ultimate Shrinkage (Microstrain)	Calculated
Reversible Shrinkage (% Ultimate Shrinkage)	50%
Time to Develop 50% Ultimate Shrinkage (Days)	35
Curing Method	Curing Compound
PCC Strength	
PCC Strength and Modulus	"Level 3"
28 Day Modulus of Rupture (MPa)	5.6
Elastic Modulus (GPa)	29.6

3.5.4.3 Jointed Plain Concrete Pavement Design Properties

Jointed Plain Concrete Pavement (JPCP) with doweled joints and widened slab is the rigid pavement design most widely employed in Ontario and by the MTO (MTO 2008). Table 3-24 shows the MEPDG JPCP pavement design properties recommended for use by the MTO (MTO 2008).

Table 3-24: MEPDG JPCP Design Properties

JPCP Design	
PCC Surface Shortwave Absorptivity	0.85
PCC Joint Spacing (m)	3.5, 4, 4.3, 4.5 (random)
Sealant Type	Other
Doweled Joints	Spacing (300mm), Diameter (32mm)
Widened Slab	Widened (4.25m)
Tied Shoulders	Tied with long-term load transfer efficiency of 70
Erodibility Index	Very Erodible
PCC-Base Contact Friction	Full friction with friction loss at 240 months
Permanent Curl/Warp Effective Temp Difference (^{o}C)	5.6

3.5.4.4 Granular Material Properties

MTO has developed default Level 3 inputs for granular materials typically used for base and subbase construction in Ontario. Table 3-25 shows the MEPDG inputs for granular material properties recommended for use by the MTO (MTO 2008).

Table 3-25: MEPDG Granular Material Properties Inputs

Unbound		Granular A	Granular B-I	Granular B-II
Poisson’s Ratio		0.35		
Coefficient of Lateral Pressure (k^0)		0.5		
Resilient Modulus (MPa)		250	150	200
Aggregate Gradation (% passing)	75µm	5.0	4.0	5.0
	380µm	13.5	33.5	13.5
	1.18mm	27.5	55.0	25.0
	4.75mm	45.0	60.0	37.5
	9.50mm	61.5	-	-
	13.20mm	77.5	-	-
	19.00mm	92.5	-	-
	25.00mm	100.0	75.0	75.0
Liquid Limit		6	11	11
Plasticity Index		0		
Layer Compacted?		Yes		
Maximum Dry Unit Weight (kg/m^3)		Calculated		
Saturated Hydraulic Conductivity (m/hr)		Calculated		
Specific Gravity of Solids		Calculated		
Optimum Gravimetric Water Content (T)		Calculated		

3.5.4.5 Stabilized Base Material Properties

MTO has developed default Level 3 inputs for Open Graded Drainage Layer (OGDL) base material, which is the chemically stabilized base material properties typically used in rigid pavement construction in Ontario. Table 3-26 shows the MEPDG inputs for OGDL base material properties recommended for use by the MTO (MTO 2008).

Table 3-26: MEPDG Chemically Stabilized Base Material Properties

OGDL Properties	OGDL
Unit Weight (kg/m^3)	1700.00
Poisson's Ratio	0.40
Elastic / Resilient Modulus (MPa)	690.00
Thermal Conductivity (watt/meter-Kelvin)	2.16
Heat Capacity (joule/kg-Kelvin)	1172.30

3.5.4.6 Subgrade Material Properties

MTO has developed default Level 3 inputs for subgrade materials typically encountered in pavement construction in Ontario. Table 3-27 shows the MEPDG inputs for subgrade material properties recommended for use by the MTO (MTO 2008).

Table 3-27: MEPDG Subgrade Material Properties

		Subgrade Type								
		CL	**CI**	**CH**	**CL-ML**	**ML**	**MI**	**MH**	**SM**	**SC**
Layer Thickness (mm)		Semi-Infinite								
Poisson's Ratio		0.45 (saturated) 0.20 (unsaturated)			0.325				0.30 (dense) 0.15 (coarse) 0.25 (fine)	
Coefficient of Lateral Earth Pressure		0.65 (very stiff) 0.72 (medium stiff)			0.730				0.68	
Resilient Modulus		35	25-30		30	30	30	35	50	40
Aggregate Gradation (% Passing)	0.002mm	30	37	60	16	11	25	40	8	13
	0.075mm	80	88	92	84	74	82	84	29	32
	0.180mm	84	92	94	89	86	91	91	58	48
	0.425mm	91	95	96	92	91	95	95	72	56
	2.000mm	95	98	98	96	95	98	98	84	86
	4.750mm	97	99	99	98	96	100	100	90	93
	9.500mm	99	100	100	99	100	100	100	94	100
	12.50mm	100	100	100	100	100	100	100	97	100
	19.00mm	100	100	100	100	100	100	100	98	100
	25.00mm	100	100	100	100	100	100	100	100	100
Liquid Limit		26	41	67	22	26	42	53	18	22
Plasticity Index		12	21	43	6	3	15	21	4	10
Layer Compacted?		Yes								
Maximum Dry Unit Density (kg/m^3)		Calculated								
Saturated Hydraulic Conductivity (m/hr)		Calculated								
Specific Gravity of Solids		Calculated								
Optimum Gravimetric Water Content (T)		Calculated								

3.6 MEPDG Software

The MEPDG pavement design method is so computationally-intensive that it must be completed using a computer software program. The MEPDG analysis included in this research was completed using the AASHTOWare Pavement ME Design software, version 1.3.28, released on February 13, 2013.

Chapter 4
Results and Discussion

4.1 Introduction

This chapter presents the results of the comparison of pavement structural designs obtained using the AASHTO 1993 Guide for Design of Pavement Structures and the Mechanistic-Empirical Pavement Design Guide for Ontario conditions. As stated in Section 3.2, the analysis was completed using a two-stage procedure for each pavement design type. First, the nationally-calibrated MEPDG pavement distress models were used to predict the performance of the pavements designed using the AASHTO 1993 method. The purpose of this stage of the analysis was to determine whether the two methods predicted pavement performance in a consistent manner across a range of design conditions typical of Ontario. Second, the AASHTO 1993 and MEPDG methods were compared based on the thickness of the asphalt concrete or Portland cement concrete layers required to satisfy their respective design criteria. Chapter 4 is organized based on the type of pavement design as follows:

- Section 4.2 New Flexible Pavements
- Section 4.3 Asphalt Overlays of Existing Flexible Pavements
- Section 4.4 New Rigid Pavements
- Section 4.5 Asphalt Overlays of Existing Rigid Pavements

4.2 New Flexible Pavements

4.2.1 MEPDG Predicted Performance of AASHTO 1993 New Flexible Pavements

The purpose of this section is to analyze the MEPDG predicted performance of new flexible pavement structural designs completed using the AASHTO 1993 pavement design methodology.

4.2.1.1 MEPDG Predicted Permanent Deformation for AASHTO 1993 New Flexible Pavements

4.2.1.1.1 Results

Figure 4-1 shows the MEPDG predicted permanent deformation for the new flexible pavements designed using the AASHTO 1993 method. The new flexible pavement sections are arranged from left to right in ascending order of traffic loading expressed in terms of MEPDG estimated ESALs.

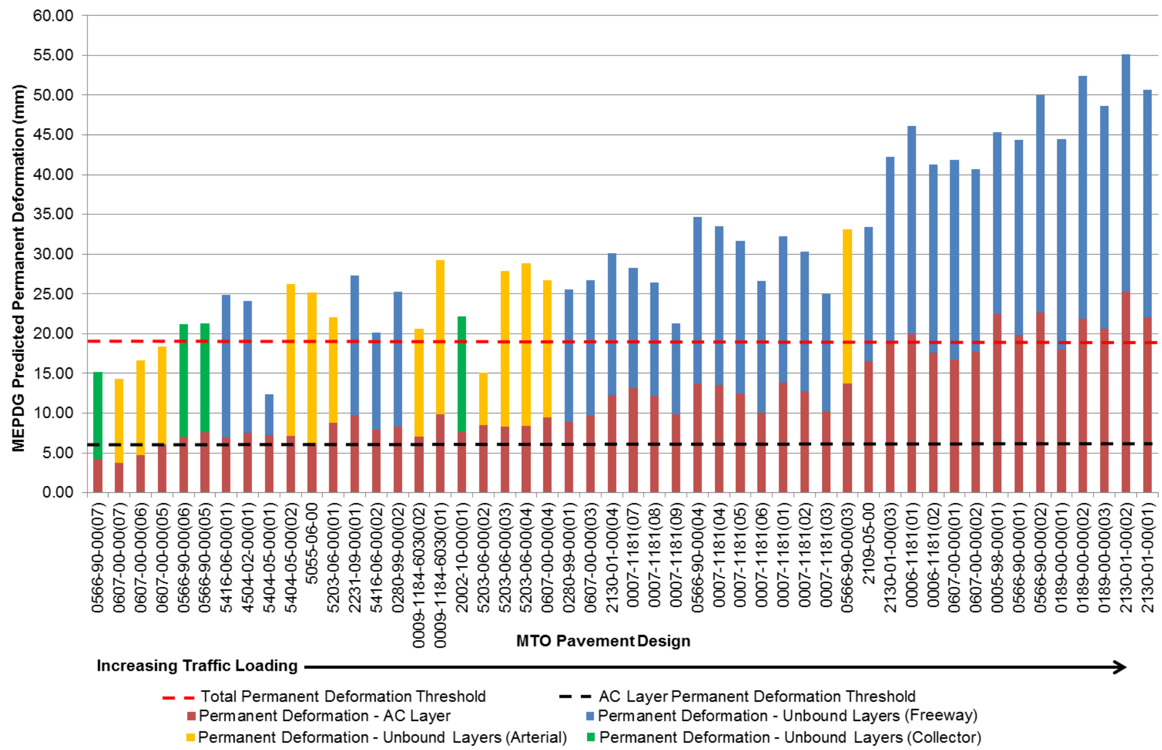


Figure 4-1: MEPDG Predicted Permanent Deformation for AASHTO 1993 New Flexible Pavement Designs (All Highway Functional Classes)

As shown in Figure 4-1, forty-seven of the fifty new flexible pavements designed using the AASHTO 1993 method failed to meet the MEPDG performance criteria for permanent deformation. Of the fifty new flexible pavement designs examined, only three met the MEPDG performance criteria for permanent deformation in the asphalt layers, while eight met the criteria for permanent deformation in the total pavement structure. The three pavement designs that met the MEPDG performance criteria for both total pavement and asphalt layer permanent deformation had the lowest traffic loading of the new flexible pavements (approx. 1 - 2 million MEPDG estimated ESALs). Figure 4-1 also shows a general trend of increasing permanent deformation, both in the asphalt layers and total pavement structure, with increased traffic loading within each highway functional classification.

Figure 4-2 and Figure 4-3 show the relationship between traffic loading, expressed as MEPDG estimated ESALs, and MEPDG predicted permanent deformation in the total pavement structure and asphalt layers, respectively.

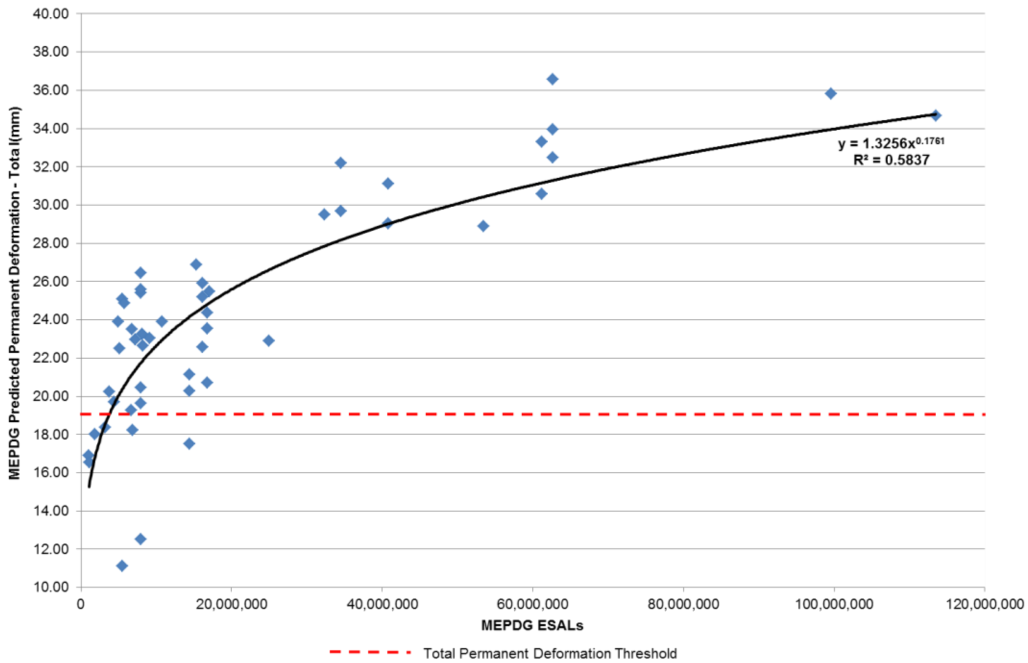


Figure 4-2: MEPDG Predicted Total Permanent Deformation versus MEPDG Estimated ESALs for AASHTO 1993 New Flexible Pavements

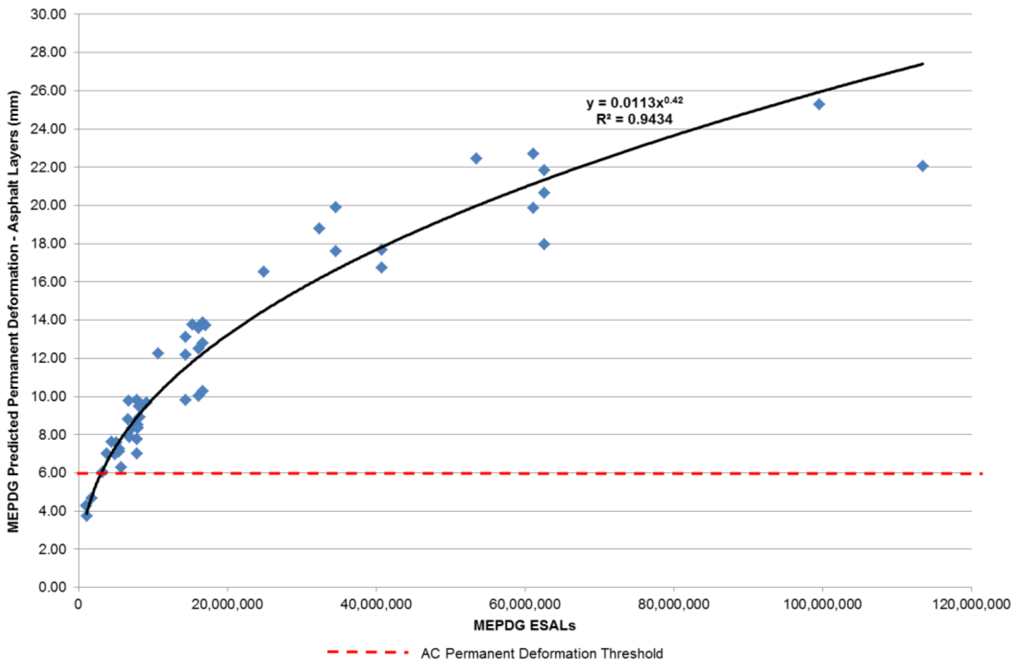


Figure 4-3: MEPDG Predicted Asphalt Layer Permanent Deformation versus MEPDG Estimated ESALs for AASHTO 1993 New Flexible Pavements

As shown in Figure 4-2 and Figure 4-3, a very strong relationship was observed between traffic loading and MEPDG predicted permanent deformation in both the total pavement structure and the asphalt layers. The very high R^2 values indicate that variations in traffic loading accounted for a substantial amount of the variation observed in MEPDG predicted permanent deformation; this was particularly true for permanent deformation in the asphalt layers. Based on the relationships observed in Figure 4-2, any new flexible pavement with traffic loading in excess of 15 million ESALs would fail to meet the MEPDG total pavement permanent deformation performance criterion. Similarly, the relationship observed in Figure 4-3 suggests that any new flexible pavement with traffic loading in excess of 3 million ESALs would fail to meet the MEPDG asphalt layer permanent deformation performance criterion. As shown in the above Figures, these levels of traffic loading were far less than those observed in the new flexible pavements included in this study and typically used by the MTO for new flexible pavement design.

Figure 4-4 and Figure 4-5 show the MEPDG predicted permanent deformation in the new flexible pavement designs by highway functional classification for the total pavement structure and asphalt layers, respectively.

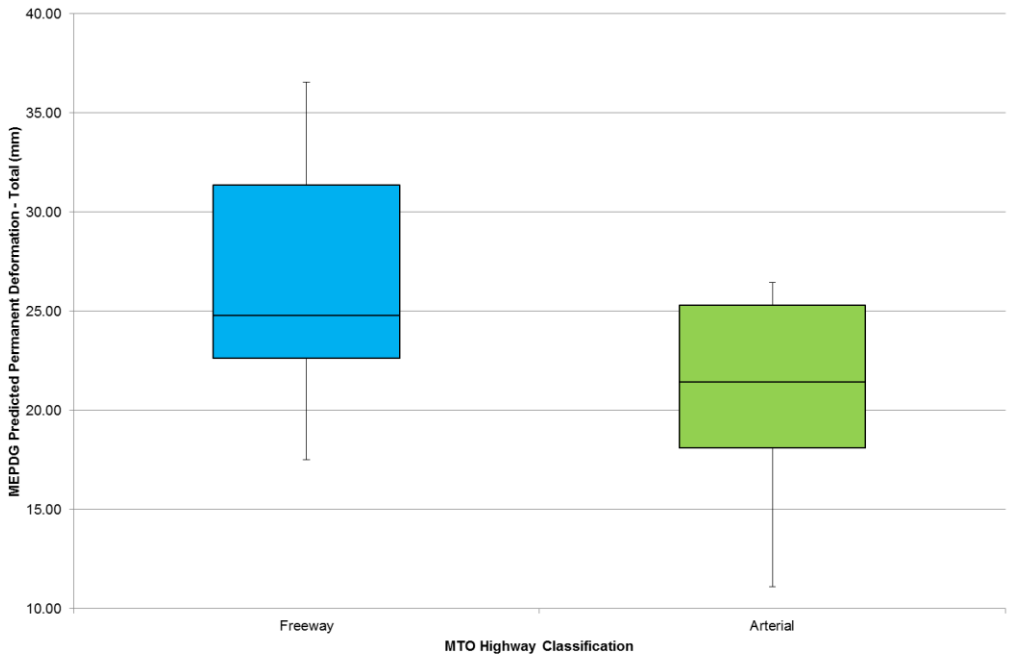


Figure 4-4: MEPDG Predicted Total Permanent Deformation for AASHTO 1993 New Flexible Pavements by Highway Functional Classification

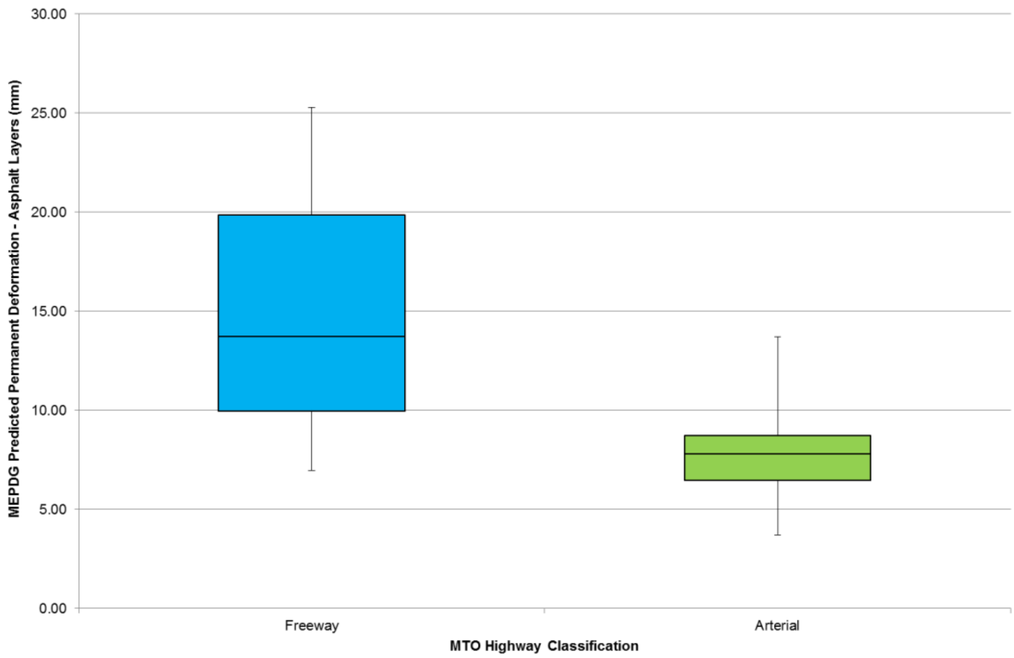


Figure 4-5: MEPDG Predicted Asphalt Layer Permanent Deformation for AASHTO 1993 New Flexible Pavements by Highway Functional Classification

As shown in Figure 4-4 and Figure 4-5, the MEPDG predicted permanent deformation in the freeway category was much higher than the arterial category. In addition, the observed variability in MEPDG predicted permanent deformation in the freeway category was also higher than the arterial category.

Figure 4-6 shows the average MEPDG predicted permanent deformation for the new flexible pavements designed using the AASHTO 1993 method plotted against time for all of the freeways included in the analysis.

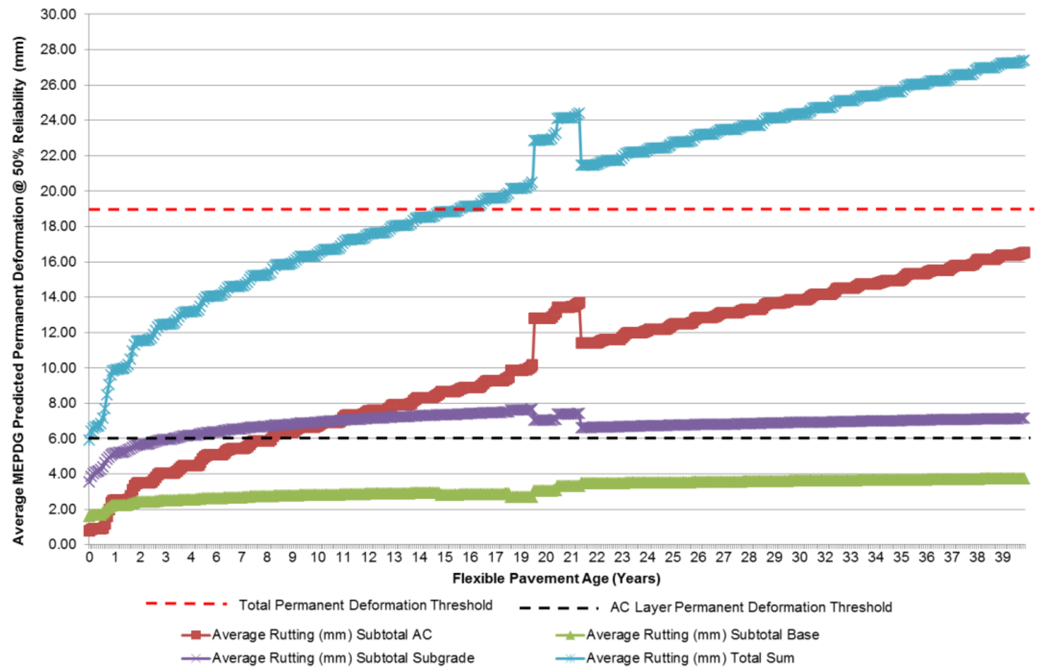


Figure 4-6: Average MEPDG Predicted Total Permanent Deformation @ 50% Reliability versus Pavement Age for New Flexible Pavements (Freeways)

As shown in Figure 4-6, the average permanent deformation observed in the new flexible pavement freeways exceeded the specified performance criteria threshold after approximately 15 years for the total pavement structure and approximately 8 years for the asphalt layer. These results were observed at 50% reliability, far less than the 80% - 95% reliability used in actual MEPDG pavement designs. Figure 4-6 also shows that the new flexible pavements accumulated a significant amount of permanent deformation within the first year of the pavement design life, particularly in the unbound layers. For example, the new flexible pavements examined accumulated 6 mm of predicted total permanent deformation in the first month alone, with 5.5 mm of the predicted permanent deformation in the unbound layers. By the end of the first year, the MEPDG predicted total permanent deformation was 10 mm, over half of the MEPDG performance threshold, with approximately 7 mm of the permanent deformation predicted in the unbound layers.

To examine the impact of high quality granular base thickness on new flexible pavement performance, a sensitivity analysis was conducted using alternative new flexible pavement designs developed for the same pavement sections. Table 4-1 shows the design inputs for the new flexible pavement design scenarios examined for each MTO project.

Table 4-1: New Flexible Pavement Design Scenarios – Design Inputs

MTO Pavement Design ID	Design Life (Years)	Initial IRI (m/km)	Design Reliability (%)	AADTT Base Year	MEPDG Est. Total ESALs	Climate Station	Subgrade Soil Classification
Project #1							
0007-1181(07)	19	0.80	95.00%	4,230	14,360,000	Toronto	SM (fine)
0007-1181(08)	19	0.80	95.00%	4,230	14,360,000	Toronto	SM (fine)
0007-1181(09)	19	0.80	95.00%	4,230	14,360,000	Toronto	SM (fine)
Project #2							
0007-1181(04)	19	0.80	95.00%	4,088	16,160,000	Toronto	CL-ML
0007-1181(05)	19	0.80	95.00%	4,088	16,160,000	Toronto	CL-ML
0007-1181(06)	19	0.80	95.00%	4,088	16,160,000	Toronto	CL-ML
Project #3							
0007-1181(01)	19	0.80	95.00%	3,880	16,750,000	Toronto	CL (unsat, med stiff)
0007-1181(02)	19	0.80	95.00%	3,880	16,750,000	Toronto	CL (unsat, med stiff)
0007-1181(03)	19	0.80	95.00%	3,880	16,750,000	Toronto	CL (unsat, med stiff)
Project #4							
0006-1181(01)	19	0.80	95.00%	15,236	34,550,000	Toronto	CL (unsat, very stiff)
0006-1181(02)	19	0.80	95.00%	15,236	34,550,000	Toronto	CL (unsat, very stiff)
Project #5							
0189-00-00(02)	21	0.80	95.00%	24,056	62,620,000	Toronto	CL (unsat, very stiff)
0189-00-00(01)	21	0.80	95.00%	24,056	62,620,000	Toronto	CL (unsat, very stiff)

All of the new flexible pavement design scenarios examined were multi-lane freeways. Figure 4-7 shows the pavement structure designs obtained using the AASHTO 1993 method.

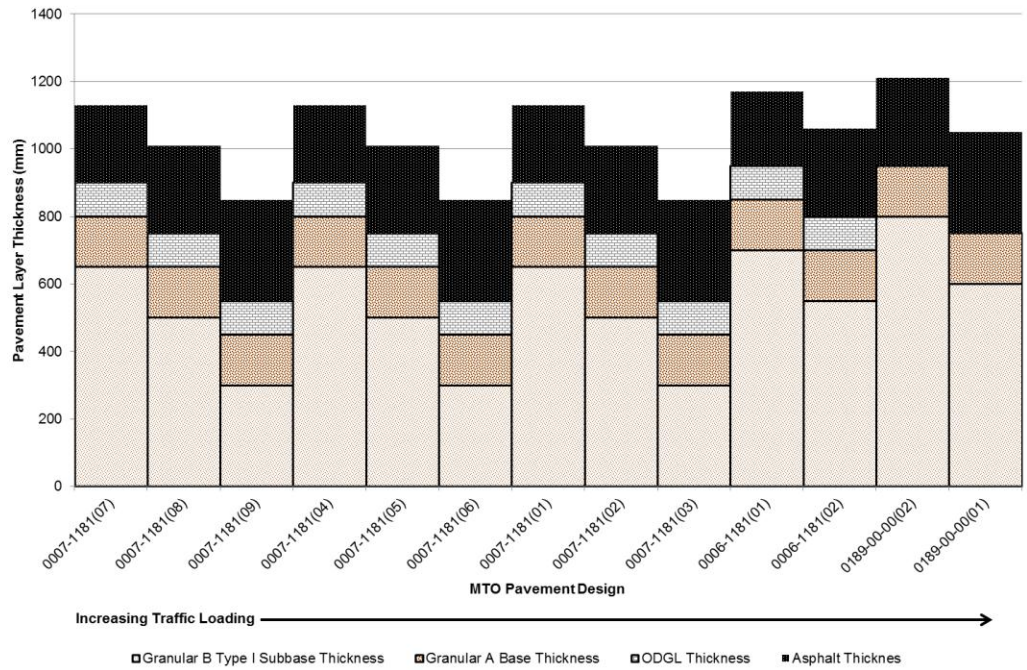


Figure 4-7: New Flexible Pavement Design Scenarios - AASHTO 1993 Pavement Thickness

As shown in Figure 4-7, the alternative new flexible pavement structure designs varied only in terms of the relative thickness of the asphalt and granular base / subbase layers; all other design inputs remained constant. The alternative flexible pavement designs within each project also had equivalent Structural Numbers as determined using the AASHTO 1993 method. The alternative pavement designs within each MTO project are sorted in ascending order of asphalt layer thickness from left to right. Figure 4-8 shows the MEPDG predicted permanent deformation for the alternative new flexible pavement design scenarios.

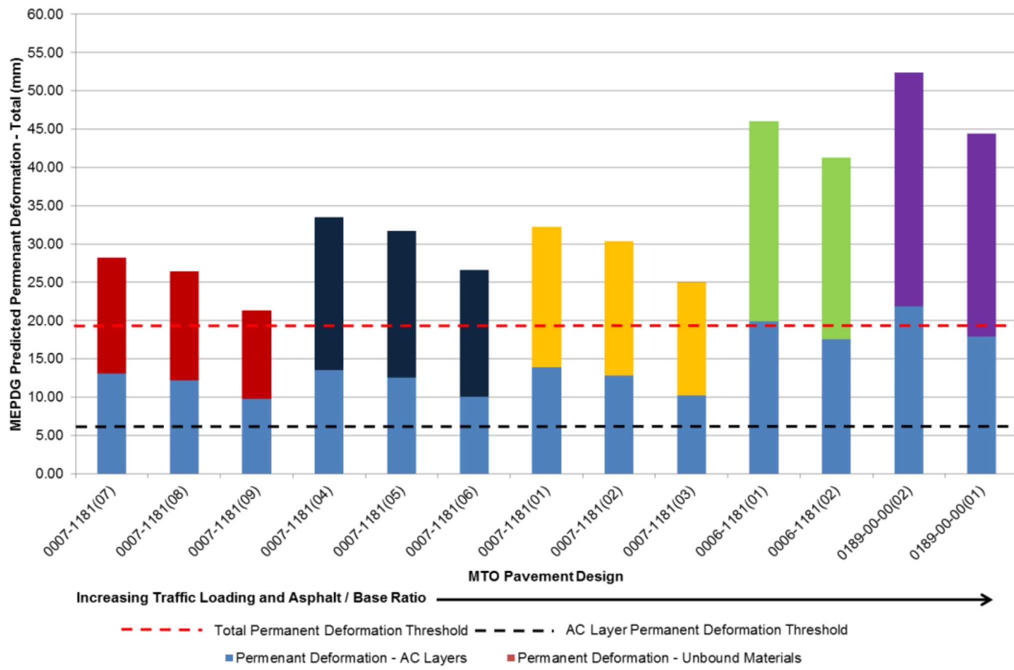


Figure 4-8: New Flexible Pavement Design Scenarios - MEPDG Predicted Permanent Deformation

As shown in Figure 4-8, the MEPDG predicted permanent deformation in both the asphalt layers and unbound pavement layers decreased significantly as the ratio of asphalt layer thickness to unbound layer thickness increased.

4.2.1.1.2 Discussion

Permanent deformation was the most significant MEPDG predicted pavement distress contributing to the failure of the new flexible pavement designs completed using the AASHTO 1993 method. The MEPDG predicted that 94% of the new flexible pavements designed using the AASHTO 1993 method would fail based on permanent deformation in the total pavement structure and / or asphalt layers. The disparity in MEPDG predicted permanent deformation within each highway functional classification suggests that the MEPDG over-predicts permanent deformation relative to the AASHTO 1993 method.

The high values of MEPDG predicted permanent deformation values were found to result primarily due to the following three factors: very strong relationship between permanent deformation and traffic loading; unreasonably high predicted permanent deformation in the unbound layers in the first year of pavement design life; and, weak structural contribution assigned by the MEPDG to unbound granular layers.

The MEPDG predicted permanent deformation in both the total pavement structure and asphalt layers was found to be very sensitive to traffic loading. As noted in Section 2.4.1.1., MEPDG sensitivity studies have consistently found that MEPDG predicted permanent deformation in both the total pavement structure and the asphalt layers is very sensitive to traffic inputs, particularly AADTT, axle load spectra, and vehicle class distribution. A number of studies have found that traffic loading variables, particularly AADTT, were the most significant variables influencing MEPDG estimated permanent deformation in both the total pavement structure and the asphalt layers. While MEPDG sensitivity studies have observed this strong relationship when all other pavement design inputs were held constant, the strong relationship observed this study was present despite varying climates, pavement structures, and materials in the new flexible pavements examined. The relationships observed in this study suggested that most of the observed variation in MEPDG predicted permanent deformation was the resulted of variation in traffic loading; this was particularly true for permanent deformation in the asphalt layers. In fact, none of the new flexible pavement structures with traffic loading in excess of 3 million MEPDG estimated ESALs met the performance criteria for permanent deformation. Given that most MTO highways in Ontario are designed for traffic loading much higher than 3 million ESALs, it is unlikely that the vast majority of MTO new flexible pavement designs would be able to meet the MEPDG performance criteria for permanent deformation using the nationally-calibrated MEPDG permanent deformation models.

The MEPDG predicted permanent deformation in the freeways was also found to be much higher and more variable compared to the arterial highways. This effect was most likely due to the strong relationship between MEPDG predicted permanent deformation and traffic loading, and the wider range of traffic loading observed in freeways relative to arterial highways.

The MEPDG predicted permanent deformation in the unbound layers was found to be unreasonably high within the first year of the pavement design life. As noted in Section 2.5.1, numerous studies have documented the significant over-prediction of permanent deformation in the unbound layers during the initial period of the pavement design life using the nationally-calibrated MEPDG permanent deformation models. One study has recommended that this early permanent deformation be subtracted from the total predicted permanent deformation to achieve a more accurate result. This recommendation could be the subject of future study in Ontario if the MEPDG permanent deformation models are not modified to resolve this issue.

The results of the sensitivity analysis demonstrated that high quality granular materials are assumed to provide less overall structural contribution to the pavement structure using the MEPDG method relative to the AASHTO 1993 method. As noted in Section 2.4.1.3 and Section 2.4.1.4, a number of studies have noted that the MEPDG predicted permanent deformation is insensitive to base layer thickness or unbound material resilient modulus. This has led several researchers to speculate that the MEPDG underestimates the structural contribution of unbound pavement and subgrade layers due to flaws inherent in the MEPDG models. AASHTO has recognized that the MEPDG and the associated AASHTOWare Pavement ME design software currently do not adequately account for the structural contribution of unbound pavement layers. At the time of this research, AASHTO has released an advisory entitled "Model for Unbound Pavement Materials in AASHTOWare Pavement ME Design" that states the following:

> "AASHTO has recently determined that the current model for unbound pavement materials underestimates the structural impact of high quality aggregate base. Because the Pavement ME Design software implements the model as it is presented in the Mechanistic Empirical Pavement Design Guide (MEPDG), this issue also impacts the guide. Users of the guide and the software are also advised that several efforts are already underway to address the unbound materials model issue."

Since relatively thick layers of high quality granular base and subbase materials are typically used in Ontario flexible pavement structures, the inability of the MEPDG to adequately account for the structural contribution of these layers is of particular concern for Ontario flexible pavement designs.

As noted in Section 2.5.1, almost all of the State DOT verification studies examined in the literature review found that the nationally-calibrated MEPDG permanent deformation models significantly over-predicted permanent deformation in new flexible pavements compared to actual pavement performance data recorded in the LTPP database or local PMSs (see Figure 2-9). In addition, based on the rutting data found in MTOs PMS, the average pre-overlay rutting observed in MTO pavements was 7 mm (MTO 2012), far less than the MEPDG predicted total permanent deformation values observed in this study at the end of the design life for new flexible pavements. Based on these considerations, it is seems likely that the MEPDG over-predicted permanent deformation in AASHTO 1993 new flexible pavements observed in this analysis. This could be confirmed in future studies using pavement performance data from the MTO PMS or LTPP database.

4.2.1.2 MEPDG Predicted Fatigue Cracking for AASHTO 1993 New Flexible Pavements

4.2.1.2.1 Results

Figure 4-9 shows the MEPDG predicted bottom-up and top-down fatigue cracking observed in the new flexible pavements designed using the AASHTO 1993 method. The MTO pavement designs are arranged from left to right in ascending order of asphalt layer thickness.

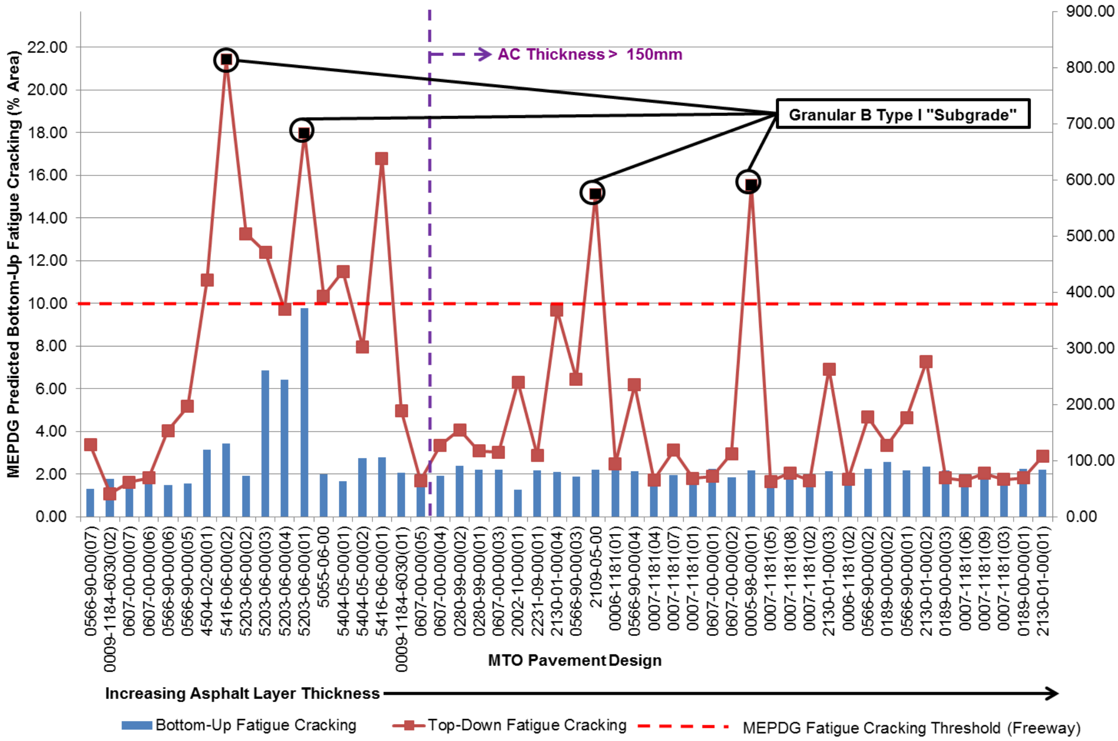


Figure 4-9: MEPDG Predicted Top-Down and Bottom-Up Fatigue Cracking for AASHTO 1993 New Flexible Pavement Designs (All Highway Functional Classes)

As shown in Figure 4-9, ten of the fifty new flexible pavements designed using the AASHTO 1993 method failed to meet the MEPDG performance criteria for top-down fatigue cracking, while all of the new flexible pavements met the MEPDG performance criteria for bottom-up fatigue cracking. Bottom-up fatigue cracking was generally low and did not vary significantly across the new flexible pavement designs. No significant bottom-up fatigue cracking was observed in new flexible pavements where the asphalt layer was more than 150 mm thick. Only three new flexible pavement structures had MEPDG predicted bottom-up fatigue cracking in excess of 4% of total pavement area; all of these pavements had 140 mm thick asphalt layers and either stiff, coarse subgrades, or very thick granular bases and / or subbases. However, other pavement designs with similar pavement structures, the same climate data, and comparable traffic loading experienced minimal bottom-up fatigue cracking, so it could not be determined conclusively exactly what combinations of factors lead to higher predicted bottom-up fatigue cracking in these three pavement sections.

Figure 4-9 shows that most of the new flexible pavement sections that failed the MEPDG performance criteria for top-down fatigue cracking had total asphalt layer thicknesses of less than 150 mm. Figure 4-10 shows a box and whisker plot of MEPDG predicted longitudinal cracking in the new flexible pavements designed using the AASHTO 1993 method.

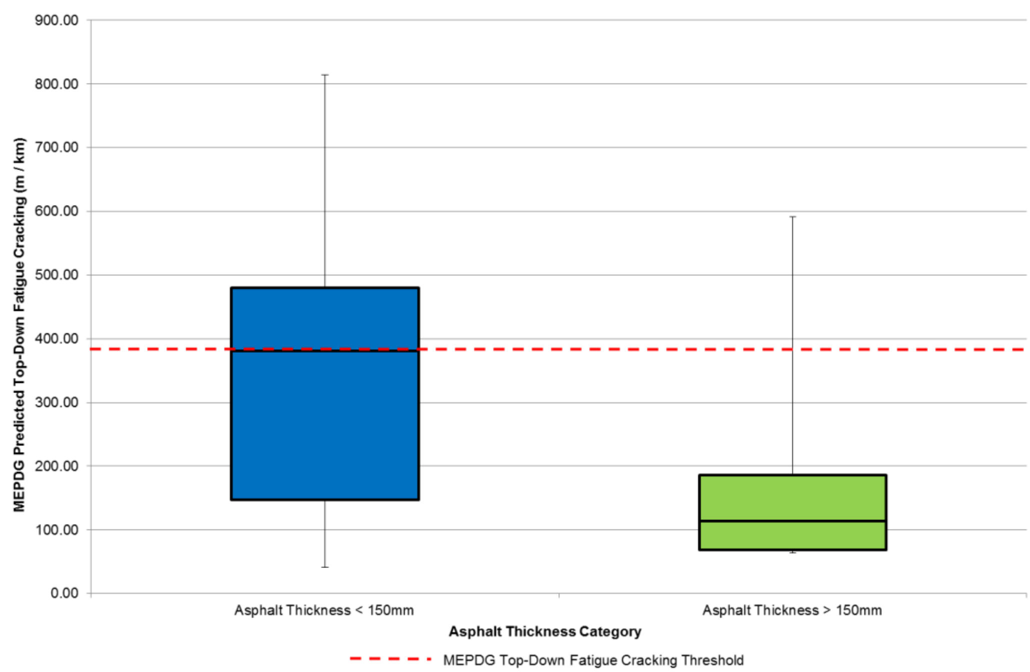


Figure 4-10: MEPDG Top-Down Fatigue Cracking for AASHTO 1993 New Flexible Pavements by Asphalt Layer Thickness Category (All Highway Functional Classifications)

As shown in Figure 4-10, new flexible pavements with asphalt thicknesses greater than 150 mm had less overall MEPDG predicted top-down fatigue cracking and less variability in MEPDG predicted top-down fatigue cracking. Figure 4-9 shows that only two new flexible pavements with total asphalt layers greater than 150 mm failed the MEPDG top-down fatigue cracking performance criteria. In contrast, approximately fifty percent of the new flexible pavements with total asphalt thicknesses less than 150 mm failed the MEPDG top-down fatigue cracking performance criteria.

In addition to asphalt layer thickness, subgrade resilient modulus and gradation was also found to exert a significant influence on MEPDG predicted top-down fatigue cracking. Figure 4-11 shows a box and whisker plot of the MEPDG predicted longitudinal cracking for new flexible pavements designed using the AASHTO 1993 method grouped by subgrade. The subgrade materials are arranged from left to right in ascending order of subgrade resilient modulus and coarseness.

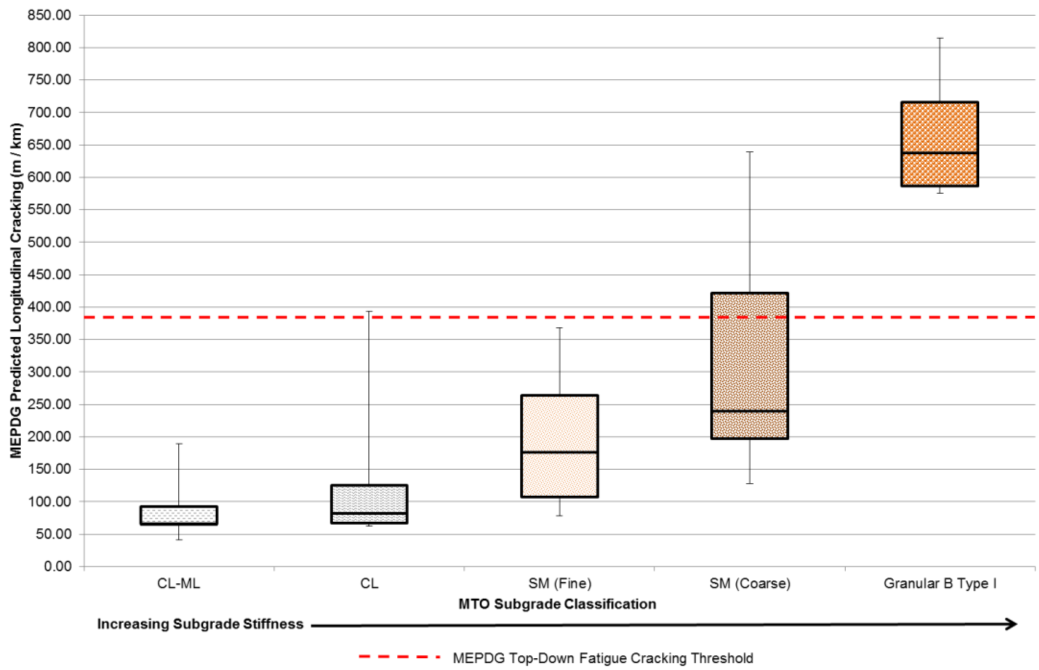


Figure 4-11: MEPDG Predicted Top-Down Fatigue Cracking for AASHTO 1993 New Flexible Pavements by MTO Subgrade Classification (All Highway Functional Classifications)

As shown in Figure 4-11, MEPDG predicted top-down fatigue cracking in the new flexible pavements increased significantly in subgrades with higher resilient modulus and coarseness. In addition, the observed range of MEPDG predicted top-down fatigue cracking also increased significantly with increasing subgrade resilient modulus and coarseness.

4.2.1.2.2 Discussion

None of the new flexible pavements designed using the AASHTO 1993 method failed the MEPDG performance criteria for bottom-up fatigue cracking. MEPDG predicted bottom-up fatigue cracking was generally very low among the new flexible pavements designed using the AASHTO 1993 method. The only new flexible pavement sections with any significant MEPDG predicted bottom-up fatigue cracking had asphalt layer thicknesses less than 150 mm. This coincides with the findings of many studies in the literature as cited in Section 2.4.2.3 (see Figure 2-6). However, since MEPDG predicted bottom-up fatigue cracking generally did not vary across the new flexible pavements examined, no other significant relationships were observed with other pavement design inputs.

As cited in Section 2.5.2, most State DOT verification studies have found that the nationally-calibrated MEPDG bottom-up fatigue cracking model significantly under-predicts bottom-up fatigue cracking when compared to local pavement performance data in the LTPP database or local PMS data. Although two studies did find the nationally-calibrated MEPDG model to be reasonably accurate, one of the studies noted that all of the pavement sections examined had only low to moderate measured alligator cracking. The generally low values of MEPDG predicted bottom-up fatigue cracking observed in this analysis combined with the findings from previous verification studies suggest that the MEPDG model may significantly under-predict bottom-up fatigue cracking; this could be verified for Ontario conditions in future research by using MTO PMS and / or pavement performance data.

MEPDG predicted top-down fatigue cracking was a significant pavement distress contributing to the predicted failure of the new flexible pavements designed using the AASHTO 1993 method; approximately 20% of the new flexible pavements failed to meet the MEPDG performance criteria for top-down fatigue cracking. MEPDG predicted top-down fatigue cracking was much higher in new flexible pavements with a total asphalt thickness of less than 150 mm, and was generally low in new flexible pavements with total asphalt thicknesses exceeding 150mm. These results correspond with the findings of a number of previous studies as cited in Section 2.4.3.3 (see Figure 2-7).

MEPDG predicted top-down fatigue cracking was also found to be sensitive to the resilient modulus and gradation of the subgrade material. This result corresponds with the findings of MEPDG sensitivity studies cited in Section 2.4.3.4 (see Figure 2-8). The presence of “Granular B Type I” subgrades in the analysis requires some additional discussion. The AASHTOWare Pavement ME software program, which automates the MEPDG pavement design method, analyzes the pavement structure by breaking each pavement layer into a number of distinct sub layers. When the pavement structure is very thick, typically due to the thickness of the granular base and / or subbase layers, the AASHTOWare Pavement ME program is required to break the pavement structure down into more than 19 sub layers. This results in an error when the program attempts to produce the pavement analysis output files. The solution recommended by the AASHTOWare Pavement ME program is to reduce the thickness of the granular base / subbase layer immediately above the subgrade, and change the subgrade material to match the granular base / subbase material immediately above the subgrade. This was done for four of the fifty new flexible pavement sections included in this analysis in order to successfully model these pavements using the AASHTOWare Pavement ME program. As shown in Figure 4-9, all of these four new flexible pavements exceeded the MEPDG top-down fatigue cracking performance criteria threshold significantly. In addition, two of these pavements were the only new flexible pavements with asphalt thicknesses greater than 150 mm to fail the MEPDG top-down fatigue cracking performance criteria. The MEPDG predicted top-down fatigue cracking in these two pavements exceeded that predicted in any other new flexible pavement with an asphalt layer thickness greater than 150 mm by a wide margin. Although based on a limited sample size, these observations suggest that new flexible pavement structures with very thick granular base / subbase layers that cannot be successfully modeled by the AASHTOWare Pavement ME program will be susceptible to very high MEPDG top-down fatigue cracking predictions.

Although MEPDG predicted top-down fatigue cracking was found to have a strong relationship with the design input variables discussed above, it was noted that the model did predict widely different values for pavement sections with very similar design inputs. It was not possible to explain the cause of these anomalies in the predicted results on the basis of the design inputs.

As noted in Section 2.5.3, the nationally calibrated MEPDG top-down fatigue cracking model has been generally regarded as having very poor predictive power and low reliability. In fact, based on the work completed as part of NCHRP Projects 9-30 and 1-40B, it was recommended that the MEPDG longitudinal cracking model not be used or calibrated. Many State DOTs that have attempted to calibrate and validate the model based on local PMS performance data have not been successful. Further research is needed to determine whether the MEPDG top-down fatigue cracking model can be successfully calibrated and validated based on Ontario conditions.

4.2.1.3 MEPDG Predicted Thermal Cracking for AASHTO 1993 New Flexible Pavements

4.2.1.3.1 Results

Figure 4-12 shows the MEPDG predicted transverse cracking in the new flexible pavements designed using the AASHTO 1993 method.

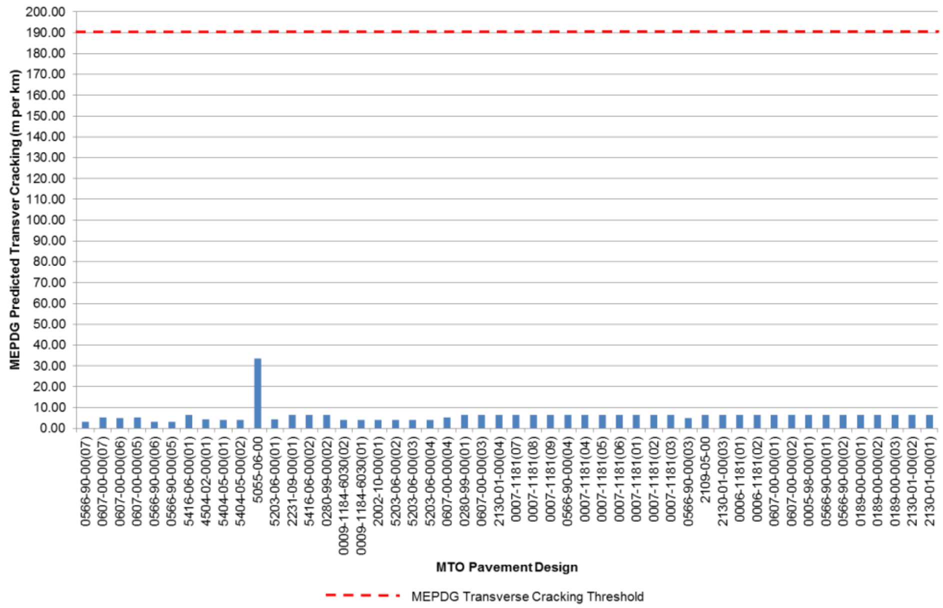


Figure 4-12: MEPDG Predicted Transverse Cracking in AASHTO 1993 New Flexible Pavements (All Highway Functional Classifications)

As shown in Figure 4-12, the MEPDG predicted transverse cracking was negligible and did not vary significantly between the new flexible pavement designs. MTO Pavement Design 5055-06-00, which was the only new flexible pavement design to use climate data from the Kapuskasing climate station, was the only new flexible pavement design to have MEPDG predicted longitudinal cracking in excess of 7 m / km. This pavement design also had nearly five times more MEPDG predicted transverse cracking than any of the remaining pavement designs. Upon closer examination of the climate data, it was determined that the Kapuskasing climate station was the only climate station included in the study that routinely experienced minimum temperatures in the range of -40^{o}C to -45^{o}C (see Figure 4-13).

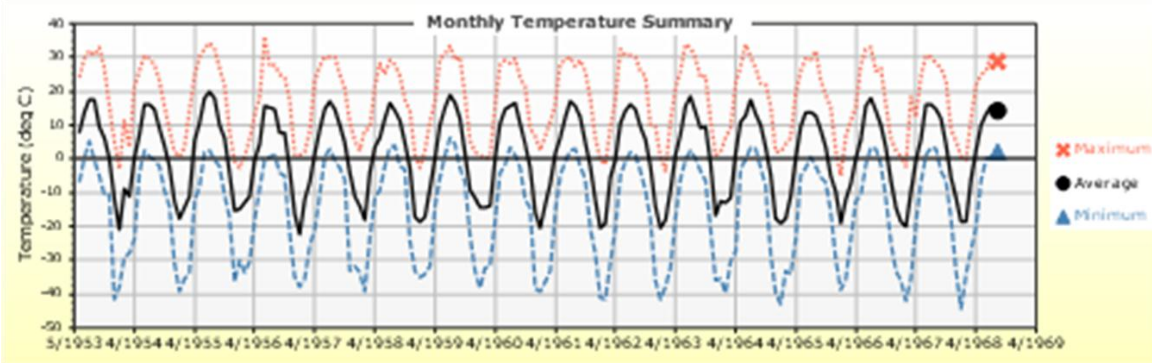


Figure 4-13: Kapuskasing Climate Station Temperature Data

Given that PGAC grade 58-34 was used in the asphalt concrete for MTO Pavement Design 5055-06-00, transverse cracking would be expected in this pavement given the extremely cold temperatures experienced over the design life of the pavement.

Given the very low values for MEPDG predicted transverse cracking in the new flexible pavements, a sensitivity analysis was conducted to assess the response of the nationally-calibrated MEPDG transverse cracking model to a change in asphalt PGAC grade. As per the recommendations in the MTO PDRs, asphalt mix designs in Southern Ontario used PGAC grade 58-28 while PGAC grade 58-34 was used in Northern Ontario. To assess the sensitivity of the MEPDG transverse cracking model, the PGAC grade of the new flexible pavements in Northern Ontario was changed to PGAC 58-28; the results are shown in Table 4-2.

Table 4-2: Effect of PGAC Grade on MEPDG Predicted Transverse Cracking in AASHTO 1993 New Flexible Pavements in Northern Ontario

MTO Pavement Design	Climate Station	MEPDG Predicted Transverse Cracking (m/km)	
		PGAC 58-34	PGAC 58-28
5416-06-00(01)	North Bay	6.50	170.43
4504-02-00(01)	Petawawa	4.38	295.30
5404-05-00(01)	Sudbury	4.07	144.98
5404-05-00(02)	Kapuskasing	4.06	113.90
5055-06-00	Sudbury	33.42	532.25
5203-06-00(01)	North Bay	4.45	498.61
5416-06-00(02)	Sudbury	6.60	273.99
5203-06-00(02)	Sudbury	4.11	215.01
5203-06-00(03)	Sudbury	4.09	176.19
5203-06-00(04)	Sudbury	4.09	169.54

As shown in Table 4-2, the change in PGAC grade had a very significant impact on the MEPDG predicted transverse cracking, with many of the pavement sections significantly exceeding the MEPDG transverse cracking performance criteria threshold of 190 m / km.

4.2.1.3.2 Discussion

Based on the results of this analysis, it appears that MEPDG predicted transverse cracking in new flexible pavements is negligible provided the temperatures experienced by the pavement do not fall outside the bounds of the selected PGAC grade. However, if an inadequate PGAC grade is selected for the asphalt layers, the new flexible pavement will likely fail to meet the MEPDG transverse cracking performance criteria.

The results of State DOT verification studies are mixed regarding the accuracy of the nationally-calibrated MEPDG transverse cracking model. Von Quintus et al. (2007) have stated that the nationally-calibrated MEPDG transverse cracking model is reasonably accurate for flexible pavements in northern climates. However, Mallela et al. (2009) have stated that the MEPDG transverse cracking model generally overestimates asphalt creep compliance of HMA mixes and consequently underestimates thermal cracking, especially in colder climates. The very low values of thermal cracking observed in the new flexible pavements analyzed in this research suggests that the model may under-predict transverse cracking in new flexible pavements in Ontario. This could be verified in future research using MTO PMS pavement performance data.

4.2.1.4 MEPDG Predicted IRI for AASHTO 1993 New Flexible Pavements

4.2.1.4.1 Results

Figure 4-14 shows the MEPDG predicted terminal IRI for the fifty new flexible pavement sections designed using the AASHTO 1993 method. The pavements sections are arranged in ascending order of pavement design life from left to right. In addition, the pavement designs are arranged based on traffic loading (MEPDG estimated ESALs) in ascending order from left to right within each pavement design life group.

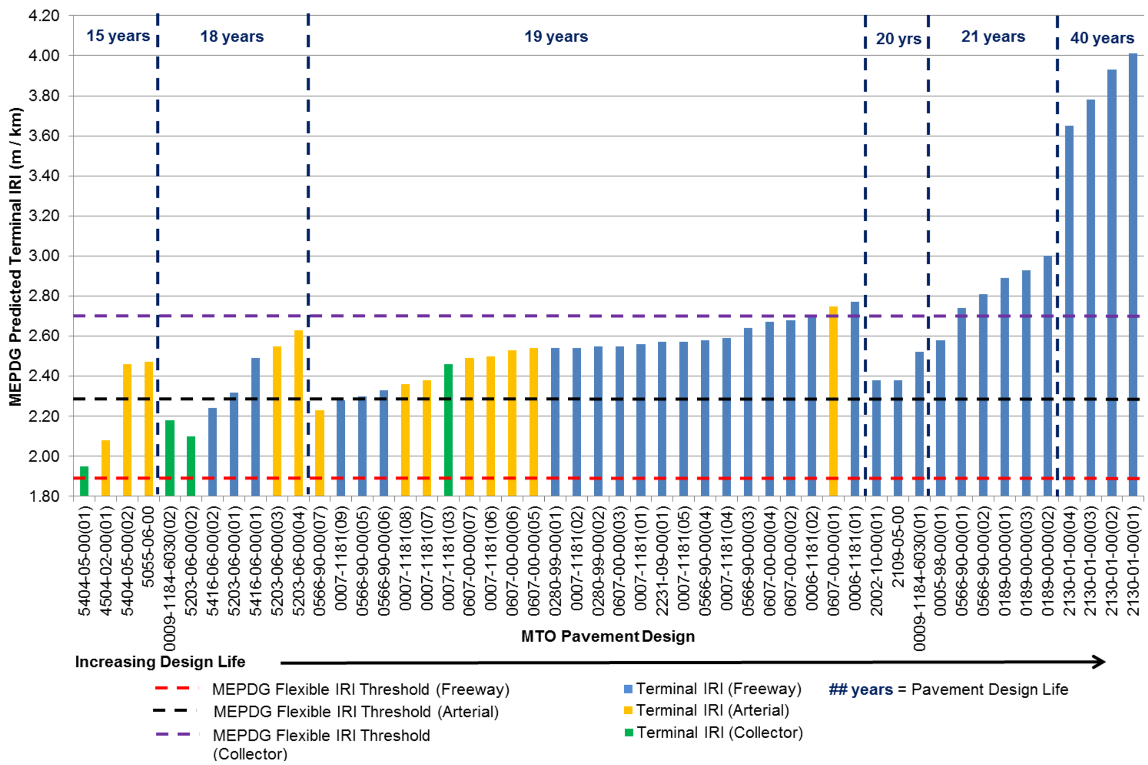


Figure 4-14: MEPDG Predicted Terminal IRI for AASHTO 1993 New Flexible Pavements (All Highway Functional Classes)

As shown in Figure 4-14, forty-two of the fifty new flexible pavements designed using AASHTO 1993 failed the MEPDG terminal IRI performance criteria threshold. There is a clear trend of increasing MEPDG predicted terminal IRI with increasing traffic loading among pavements with equal pavement design lives. There also appears to be a general trend of increasing MEPDG predicted IRI with increasing pavement design life.

Figure 4-15 shows MEPDG predicted terminal IRI plotted against MEPDG predicted total permanent deformation for new flexible pavements designed using the AASHTO 1993 method.

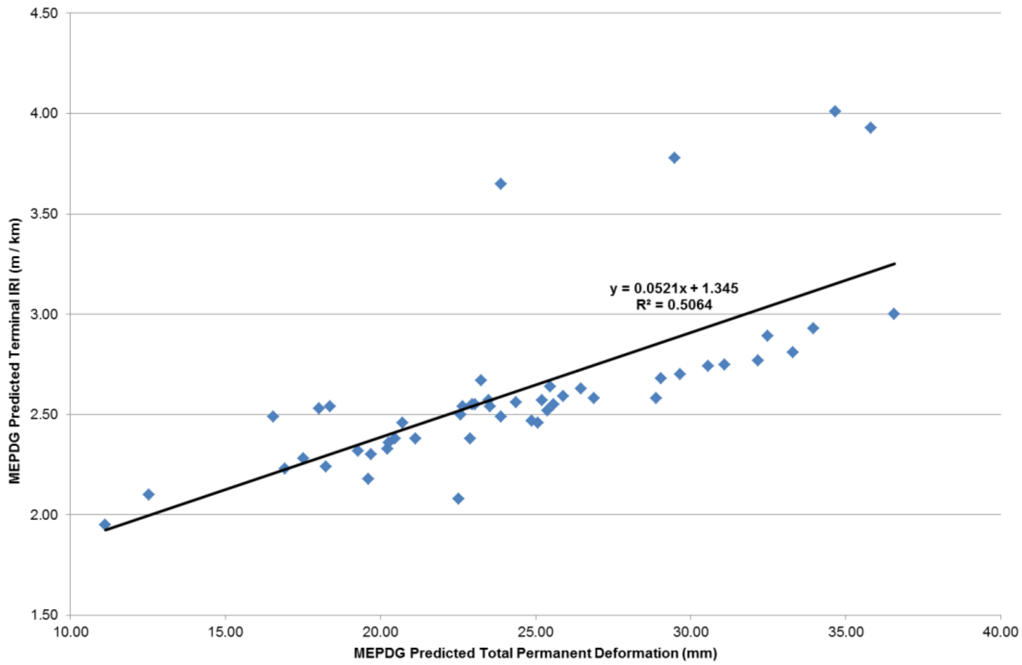


Figure 4-15: MEPDG Predicted IRI versus MEPDG Predicted Total Permanent Deformation for AASHTO 1993 New Flexible Pavement Designs

As shown in Figure 4-15, a very strong correlation was observed between MEPDG predicted terminal IRI and MEPDG predicted total permanent deformation in the new flexible pavements. Table 4-3 summarizes the results of regression analysis conducted for MEPDG predicted terminal IRI and the other MEPDG predicted pavement distresses for new flexible pavements.

Table 4-3: MEPDG Predicted Terminal IRI Linear Regression Analysis

Independent Variable	Slope	Y-intercept	R^2
Permanent Deformation – Total (mm)	0.0521*	1.3450*	0.5064
Permanent Deformation – Asphalt (mm)	0.0497*	2.0092*	0.4346
Bottom-Up Fatigue Cracking (% Total Area)	-0.0125	2.6448*	0.0018
Thermal Cracking (m / km)	0.0076	2.5674*	0.0054
Top-Down Fatigue Cracking (m / km)	-0.0004	2.6941*	0.0267

* Significant at the 95% confidence level

As shown in Table 4-3, a strong relationship was also observed between MEPDG predicted terminal IRI and MEPDG predicted asphalt layer rutting. In contrast, no relationship was observed between MEPDG predicted terminal IRI and MEPDG predicted bottom-up fatigue cracking, MEPDG predicted thermal cracking, or MEPDG predicted top-down fatigue cracking. In addition, the MEPDG fatigue cracking pavement performance measures were found to have a weak inverse relationship with MEPDG predicted terminal IRI.

Figure 4-16 shows MEPDG predicted terminal IRI plotted against MEPDG estimated ESALs for the new flexible pavements designed using the AASHTO 1993 method.

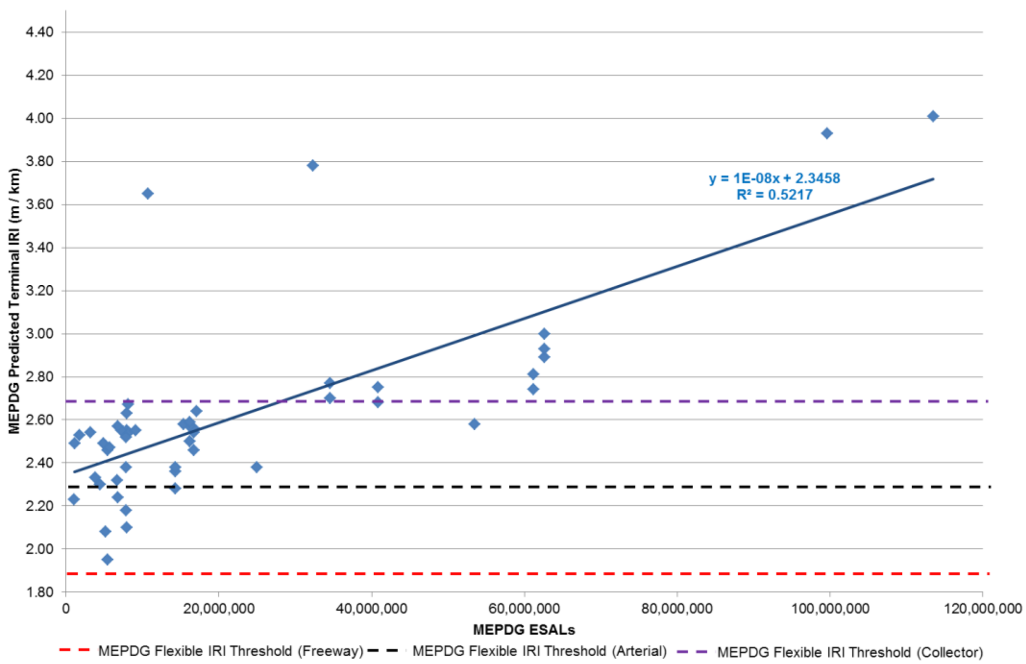


Figure 4-16: MEPDG Predicted Terminal IRI vs. MEPDG ESALs for AASHTO 1993 New Flexible Pavement Designs

As shown in Figure 4-16, a significant correlation was observed between MEPDG predicted terminal IRI and traffic loading in terms of MEPDG estimated ESALs.

Figure 4-17 shows the MEPDG predicted terminal IRI plotted against the design life of the new flexible pavement structures designed using AASHTO 1993.

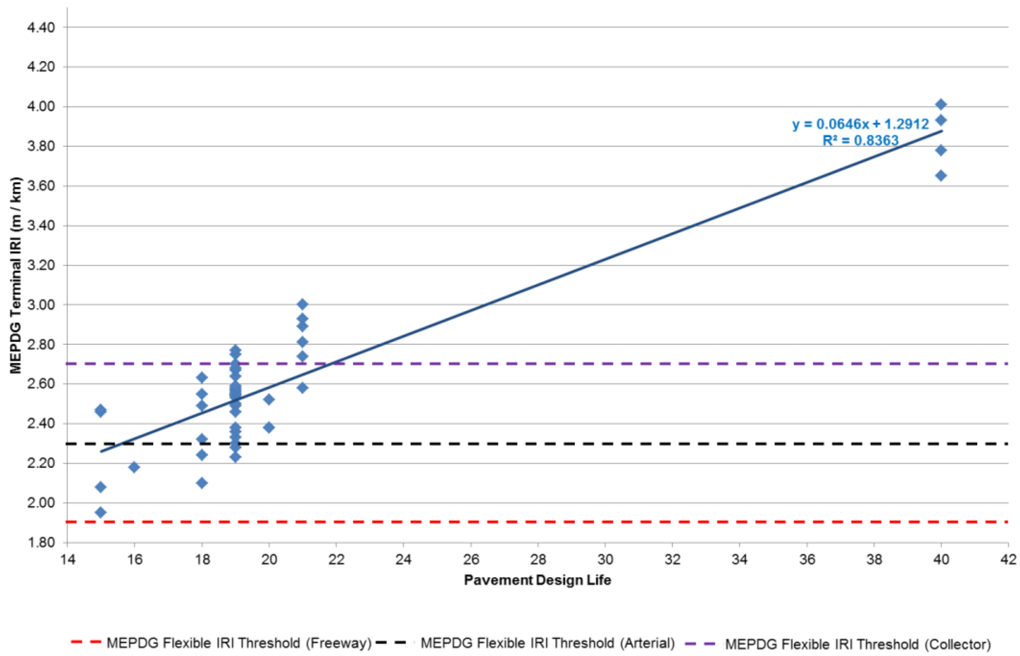


Figure 4-17: MEPDG Predicted Terminal IRI for AASHTO 1993 New Flexible Pavement Designs vs. Pavement Design Life

As shown in Figure 4-17, a very strong correlation was observed between MEPDG predicted terminal IRI and the pavement design lives of the new flexible pavements.

4.2.1.4.2 Discussion

MEPDG predicted terminal IRI was found to be a significant pavement distress contributing to the predicted failure of the new flexible pavements designed using the AASHTO 1993 method. Eighty-four percent (84%) of the new flexible pavements included in the analysis failed to meet the MEPDG performance criteria for IRI. This result demonstrates that the MEPDG generally over-predicts pavement roughness relative to the AASHTO 1993 method.

Unlike the other pavement distresses included in the MEPDG, the MEPDG IRI measure is a composite measure of the other MEPDG pavement distresses and a site factor. The results of this analysis indicated that MEPDG predicted permanent deformation was the pavement distress that most impacted MEPDG predicted IRI. This result is intuitive given the consistently high values of permanent deformation predicted in the new flexible pavements analyzed.

In terms of pavement design inputs, MEPDG predicted IRI was found to be most significantly influenced by traffic loading and pavement age. The MEPDG predicted IRI is a weighted composite measure of the other MEPDG predicted pavement distresses and a site factor. Therefore, MEPDG predicted IRI should be affected most significantly by the design inputs that have the greatest influence on the other pavement distress models. Since traffic loading was the most significant factor contributing to MEPDG predicted permanent deformation, and MEPDG predicted permanent deformation was the most significant pavement distress contributing to MEPDG predicted IRI, it was logical that traffic loading would have a significant effect on MEPDG predicted terminal IRI. The significant relationship observed between traffic loading and MEPDG predicted IRI has also also observed in many MEPDG sensitivity studies as cited in Section 2.4.5.1. As previously mentioned, the equation used by the MEPDG to determine flexible pavement IRI is a composite measure that includes a site factor. One of the variables accounted for in the site factor component of the MEPDG IRI equation is the age of the pavement structure. The strong correlation observed between MEPDG predicted terminal IRI and pavement design life suggests that the site factor exerts considerable influence on MEPDG predicted IRI. It should also be noted that a moderate correlation between pavement design life and traffic loading in terms of MEPDG estimated ESALs was observed, which may also account for some of the influence of pavement age on MEPDG predicted terminal IRI.

As stated in Section 2.5.5, verification studies conducted by State DOTs have generally found the nationally calibrated MEPDG IRI model to be the most accurate of the MEPDG flexible pavement distress models. However, some verification studies have found a very poor correlation with local PMS pavement performance data. Therefore, the accuracy of the nationally-calibrated MEPDG IRI model should be verified using pavement performance data from the MTO PMS. However, since the MEPDG IRI model is dependent on the output from the other flexible pavement distress models, this should not be completed until after work has been completed on the other models.

4.2.2 Comparison of MEPDG and AASHTO 1993 New Flexible Pavement Design Thickness

The purpose of this section is to compare the MEPDG and AASHTO 1993 new flexible pavement designs based on the asphalt concrete layer thicknesses required to satisfy their respective design criteria.

4.2.2.1 Results

Figure 4-18 shows the total asphalt layer thicknesses obtained for new flexible pavements using the MEPDG and AASHTO 1993 design methods.

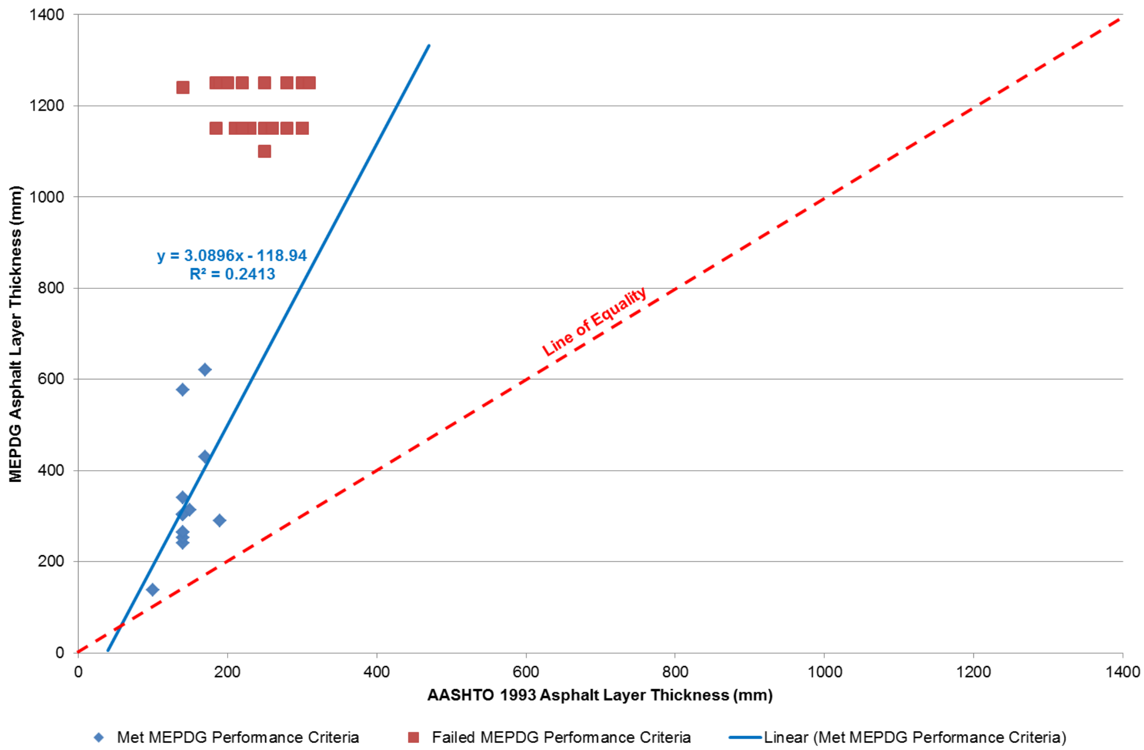


Figure 4-18: MEPDG versus AASHTO 1993 Asphalt Layer Thickness - New Flexible Pavements

As shown in Figure 4-18, the MEPDG resulted in thicker total asphalt layer thicknesses for all of the new flexible pavement structures that were capable of meeting the MEPDG performance criteria. This was expected given that forty-nine of the fifty new flexible pavements designed using the AASHTO 1993 method failed to meet the MEPDG performance criteria. Thirty of the fifty new flexible pavements were not able to meet the MEPDG performance criteria even with total asphalt layer thicknesses ranging from 1000 mm to 1300 mm. For these pavements, the thickness of the total asphalt layer was increased until the AASHTOWare Pavement ME software was not capable of analyzing the pavement structure due to the maximum number of pavement sub layers being exceeded.

The new flexible pavements that could not be re-designed to meet the MEPDG performance criteria failed based on asphalt layer permanent deformation and / or terminal IRI; the remaining pavement distresses could be reduced to meet the MEPDG performance criteria by increasing the asphalt layer thickness. Figure 4-19 shows the permanent deformation in the asphalt layer for the new flexible pavements designed using the MEPDG and AASHTO 1993.

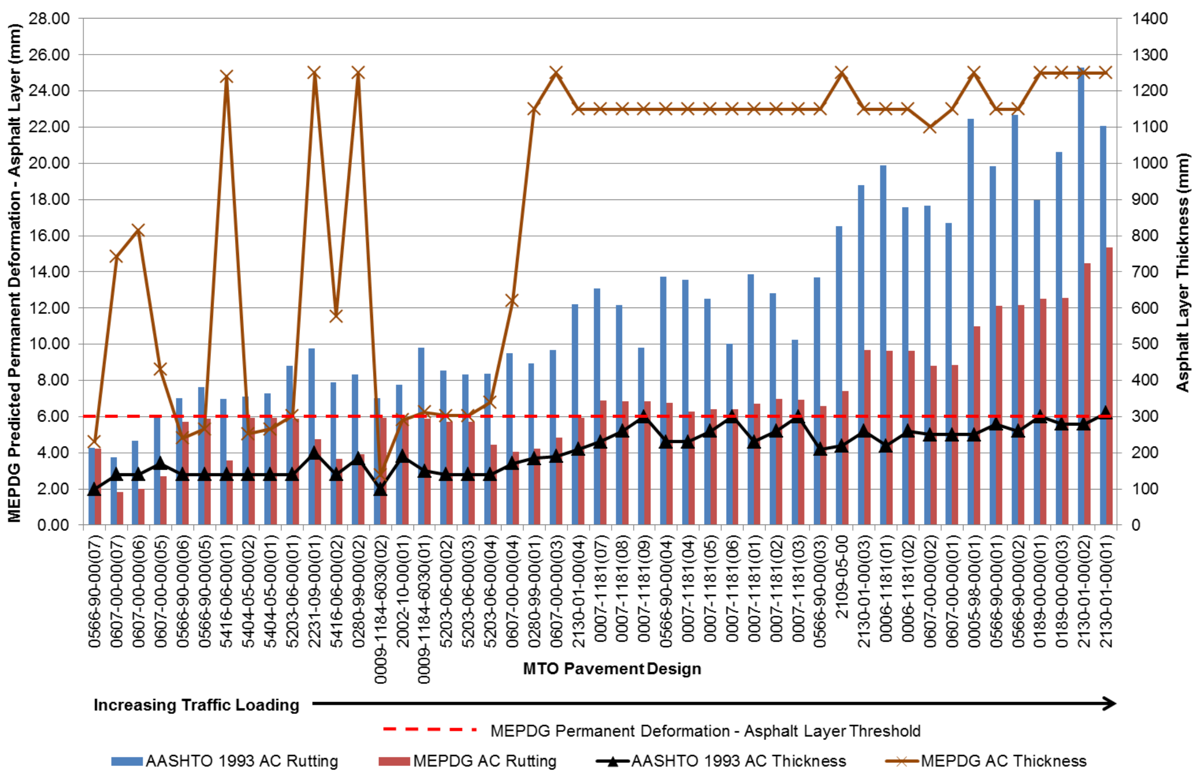


Figure 4-19: MEPDG versus AASHTO 1993 Asphalt Layer Permanent Deformation in New Flexible Pavements

As shown in Figure 4-19, increasing the thickness of the asphalt layer for the MEPDG design resulted in a corresponding decrease in asphalt layer permanent deformation. However, twenty-four of the new flexible pavements could not be designed to meet the MEPDG performance criteria for asphalt layer rutting based on increasing the thickness of the asphalt layer. In fact, no new flexible pavement with traffic loading in excess of 10 million ESALs, as estimated by the AASHTOWare Pavement ME software, was able to meet the MEPDG asphalt layer rutting performance criteria regardless of the thickness of the asphalt layer specified.

Figure 4-20 shows the terminal IRI for new flexible pavements design using the MEPDG and AASHTO 1993 methods.

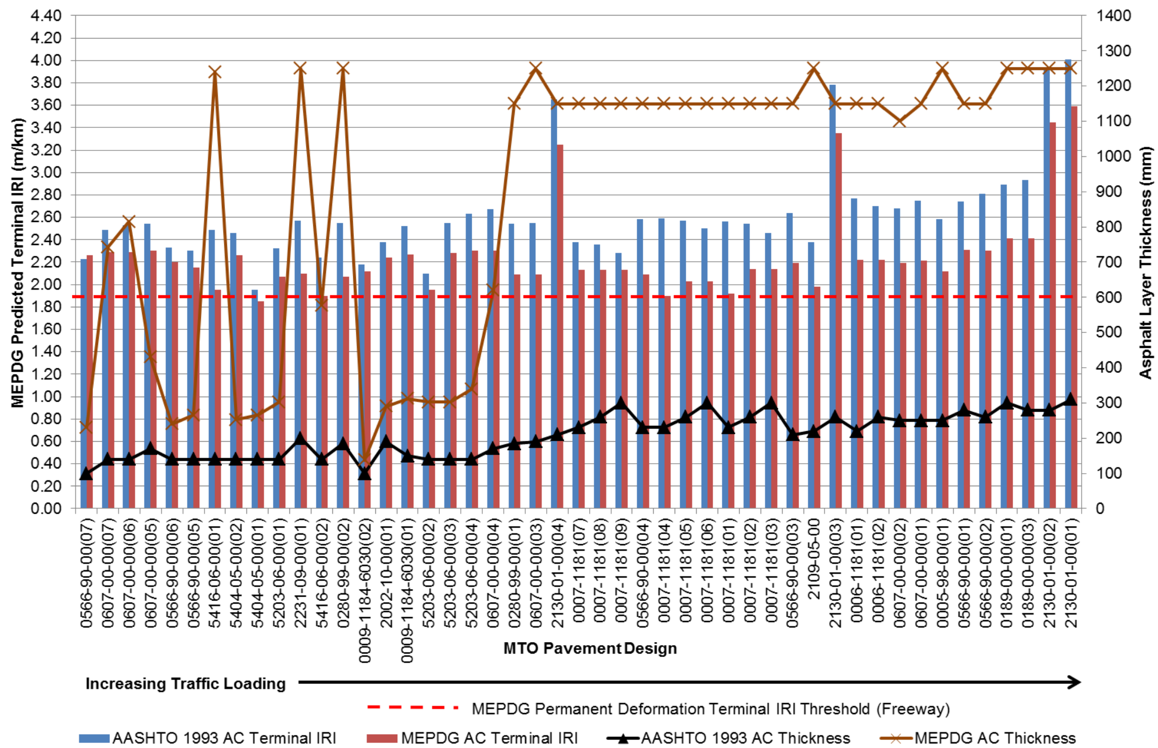


Figure 4-20: MEPDG versus AASHTO 1993 Terminal IRI in New Flexible Pavements

As shown in Figure 4-20, increasing the thickness of the asphalt layer for the MEPDG flexible pavement design resulted in a corresponding decrease in terminal IRI. However, twenty-eight of the new flexible pavements could not be re-designed to meet the MEPDG performance criteria for terminal IRI by increasing the thickness of the asphalt layer. In general, new flexible pavements with traffic loading in excess of 8 million ESALs were not able to be redesigned to meet the MEPDG performance criteria for terminal IRI.

To test the sensitivity of the MEPDG new flexible pavement designs to the specified MEPDG performance criteria, the new flexible pavements were re-designed using the MEPDG and the revised pavement performance criteria shown in Table 4-4.

Table 4-4: Revised MEPDG New Flexible Pavement Performance Criteria

Performance Measure	**Highway Functional Class**	**MTO Default MEPDG Performance Criteria**		**Revised MEPDG Performance Criteria**	
Permanent Deformation – AC Layer (mm)	All	6	95%	19	95%
Terminal IRI	Freeway	1.9	95%	1.9	50%
	Arterial	2.3	95%	2.3	50%
	Collector	2.7	95%	2.7	50%

The revised MEPDG performance criteria included in Table 4-4 were selected for the following reasons. First, MEPDG flexible pavement IRI is a composite performance measure that is calculated primarily based on the output of the other MEPDG flexible pavement distress models. As a result, the MEPDG IRI model accumulates the error inherent in each of the individual MEPDG flexible pavement distress models. For this reason, it has been recommended by some researchers that IRI not be used as a controlling pavement structural design criteria in MEPDG flexible pavement design (Wagner 2012). Given the above, predicting flexible pavement IRI at the 95% reliability level may be overly conservative given the high degree of error inherent in the way the composite measure is calculated. Second, the tolerable level of total permanent deformation in a flexible pavement structure is typically governed by safety concerns related to loss of vehicle control due to hydroplaning and / or pulling of the vehicle wheels into the rut path. The actual distribution of rutting within the pavement structure is of more concern when developing pavement rehabilitation strategies. Therefore, it was decided to limit the total permitted permanent deformation in the asphalt layer to the total permitted permanent deformation in the total pavement structure.

Figure 4-21 shows the thickness of the asphalt layers obtained using the revised MEPDG pavement performance criteria plotted against the original asphalt layer thicknesses obtained using the AASHTO 1993 method.

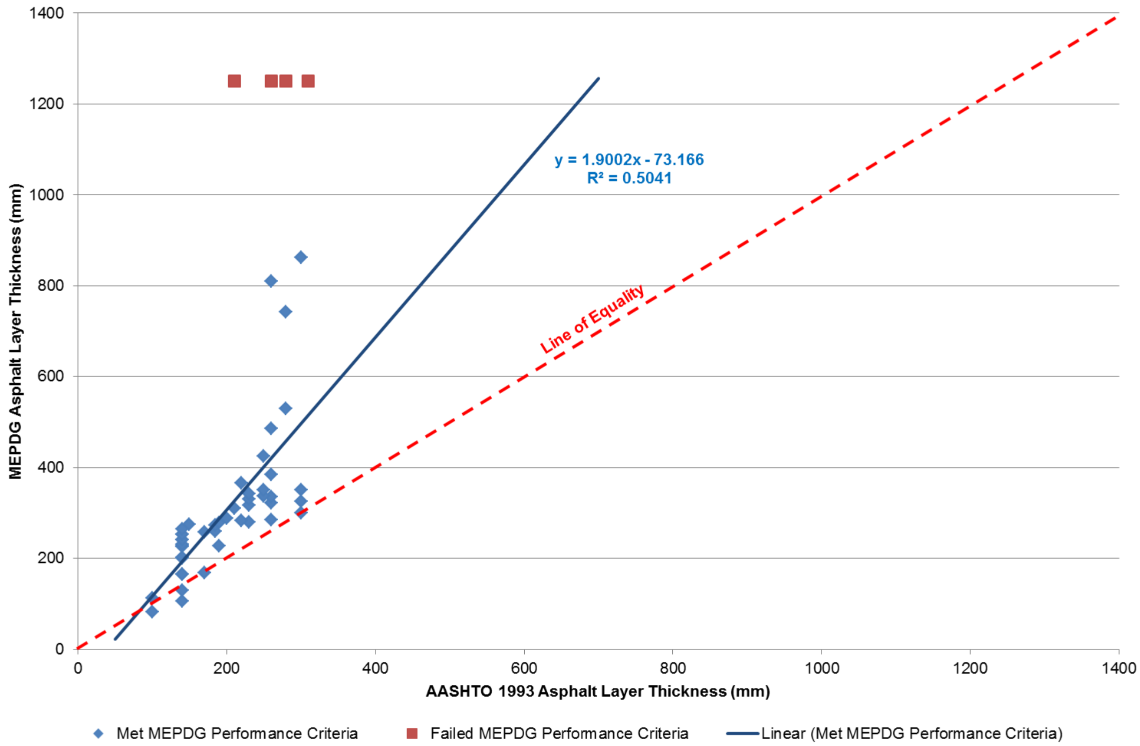


Figure 4-21: MEPDG versus AASHTO 1993 Asphalt Layer Thickness - New Flexible Pavements (Revised MEPDG Performance Criteria)

As shown in Figure 4-21, most of the new flexible pavement structures were successfully re-designed using the MEPDG based on the revised performance criteria. Only four flexible pavements could not meet the revised pavement performance criteria; all of these four pavements failed based on terminal IRI alone. The revised MEPDG pavement performance criteria resulted in a stronger correlation between total asphalt layer thicknesses obtained using the MEPDG and AASHTO 1993 methods. However, the MEPDG still generally resulted in thicker asphalt layer thicknesses relative to the AASHTO 1993 method.

4.2.2.2 Discussion

The results of this analysis show that the MEPDG, using the nationally-calibrated models and default performance criteria, resulted in significantly thicker asphalt layers than the AASHTO 1993 method. In addition, very poor correlation was observed between asphalt layer thicknesses obtained using the two pavement design methods. In addition, approximately 60% of the new flexible pavements analyzed could not be re-designed to meet the MEPDG performance criteria, regardless of the thickness of the asphalt layer.

As noted in Section 2.3, a number of studies have compared flexible pavement designs obtained using the MEPDG and AASHTO 1993 methods based on asphalt layer thickness. Many of these studies have found that the MEPDG produced flexible pavement designs with significantly thinner asphalt layer thicknesses relative to the AASHTO 1993 method. In addition, many of these studies have also found a strong correlation between the asphalt layer thicknesses obtained for new flexible pavements using the two pavement design methodologies. Both of these results conflict significantly with the results obtained in this study. The studies that have reported the above findings typically examined flexible pavements with much lower traffic loading (< 10 million ESALs) than examined in this study. In addition, some of these studies also used much lower reliability levels for the prediction of MEPDGH pavement distresses.

The results of this study correspond with the findings of Ahammed et al. (2011) in a recent study completed in the Province of Manitoba. The range of traffic loading examined in the study was 4.3 – 28.8 million ESALs. All of the flexible pavement structures designed using the AASHTO 1993 method were found to be unable to meet the MEPDG performance criteria using the nationally-calibrated MEPDG pavement distress models. In addition, terminal IRI and permanent deformation were found to govern flexible pavement design. The study also found that increasing traffic loading significantly reduced predicted pavement design life. In an earlier study, Carvahlo et al. (2006) also found that the AASHTO 1993 method overestimated pavement performance relative to the MEPDG for flexible pavements with high traffic loading (55 million ESALs). Therefore, the results of the new flexible pavement analysis completed as part of this study correspond with the findings of other studies that have examined flexible pavements with higher traffic loading.

4.3 Asphalt Concrete Overlay of Existing Flexible Pavement

4.3.1 MEPDG Predicted Performance of AASHTO 1993 Asphalt Concrete Overlay of Existing Flexible Pavement

The purpose of this section is to analyze the MEPDG predicted performance of asphalt overlays of flexible pavement designs completed using the AASHTO 1993 pavement design methodology.

4.3.1.1 MEPDG Predicted Permanent Deformation for AASHTO 1993 Asphalt Overlays of New Flexible Pavements

4.3.1.1.1 Results

Figure 4-22 shows the MEPDG predicted permanent deformation for asphalt overlays of existing flexible pavements designed using the AASHTO 1993 method.

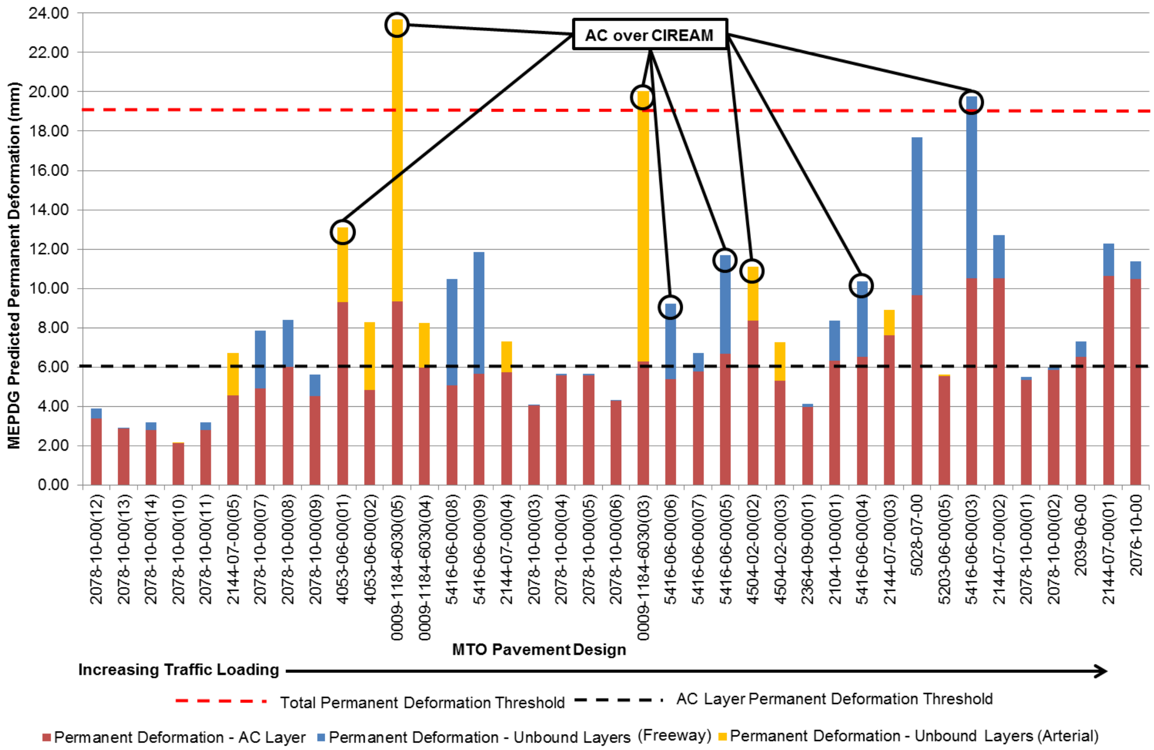


Figure 4-22: MEPDG Predicted Permanent Deformation for AASHTO 1993 Asphalt Overlays of Existing Flexible Pavements

As shown in Figure 4-22, twenty-five of the thirty-nine asphalt overlays of existing flexible pavements met the MEPDG performance criteria for permanent deformation. Of the fourteen asphalt overlays that did not meet the performance criteria, eleven failed based on asphalt layer rutting alone and three failed based on both asphalt layer and total pavement structure rutting.

All of the three asphalt overlays of existing flexible pavement that failed based on permanent deformation in the total pavement structure were placed over Cold-In Place Recycled Asphalt with Expanded Asphalt Cement (CIREAM). Figure 4-22 shows that the asphalt overlays placed over CIREAM generally experienced much higher permanent deformation in the unbound layers than the asphalt overlays placed over unmodified asphalt pavement layers. To examine the impact of CIREAM on asphalt overlay rutting performance, a sensitivity analysis was conducted using alternative asphalt overlay designs for the same MTO projects. The asphalt overlay rehabilitation treatments were designed using the same pavement design inputs and AASHTO 1993 structural number; one asphalt overlay was placed directly over existing asphalt pavement while the other asphalt overlay was placed over CIREAM. Table 4-5 shows the design inputs for the asphalt overlays of flexible pavement design scenarios examined for each MTO project.

Table 4-5: Asphalt Overlay of Flexible Pavement Design Scenarios - Design Inputs

MTO Pavement Design ID	Design Life (Years)	Initial IRI (m/km)	Design Reliability (%)	AADTT Base Year	MEPDG Est. Total ESALs	Climate Station	Subgrade Soil Classification
Project #1							
4053-06-00(01)	15	1.0	85%	836	4,766,784	Trenton	SM (Fine)
4053-06-00(02)	15	1.0	85%	836	4,766,784	Trenton	SM (Fine)
Project #2							
0009-1184-6030(05)	12	1.0	85%	1,875	7,900,000	Toronto	CL-ML
0009-1184-6030(04)	12	1.0	85%	1,875	7,900,000	Toronto	CL-ML
Project #3							
4504-02-00(02)	15	0.9	85%	711	4,603,249	Petawawa	SM (Coarse)
4504-02-00(03)	15	0.9	85%	711	4,603,249	Petawawa	SM (Coarse)

All of the asphalt overlays of new flexible pavements examined were arterial highways. Figure 4-23 shows the alternative pavement structure designs obtained using the AASHTO 1993 method.

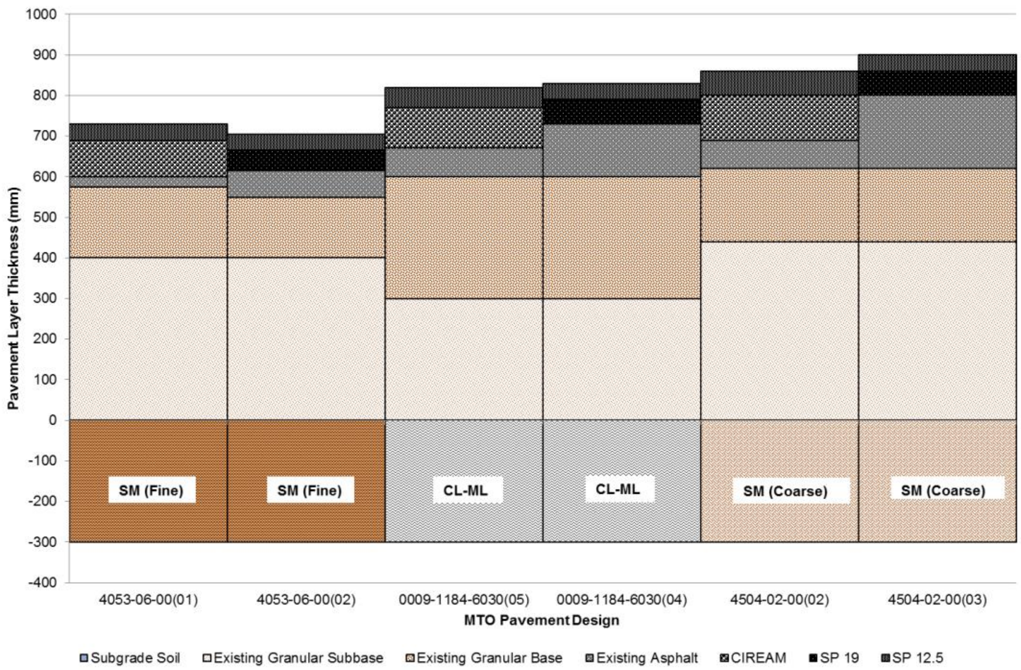


Figure 4-23: Asphalt Overlays of Flexible Pavement Design Scenarios – AASHTO 1993 Pavement Thickness

As shown in Figure 4-23, the alternative pavement structures for each project had the same materials and layer thicknesses for the base, subbase, and subgrade layers; only the asphalt layer thicknesses varied. Figure 4-24 shows the MEPDG predicted permanent deformation for the alternative asphalt overlay designs.

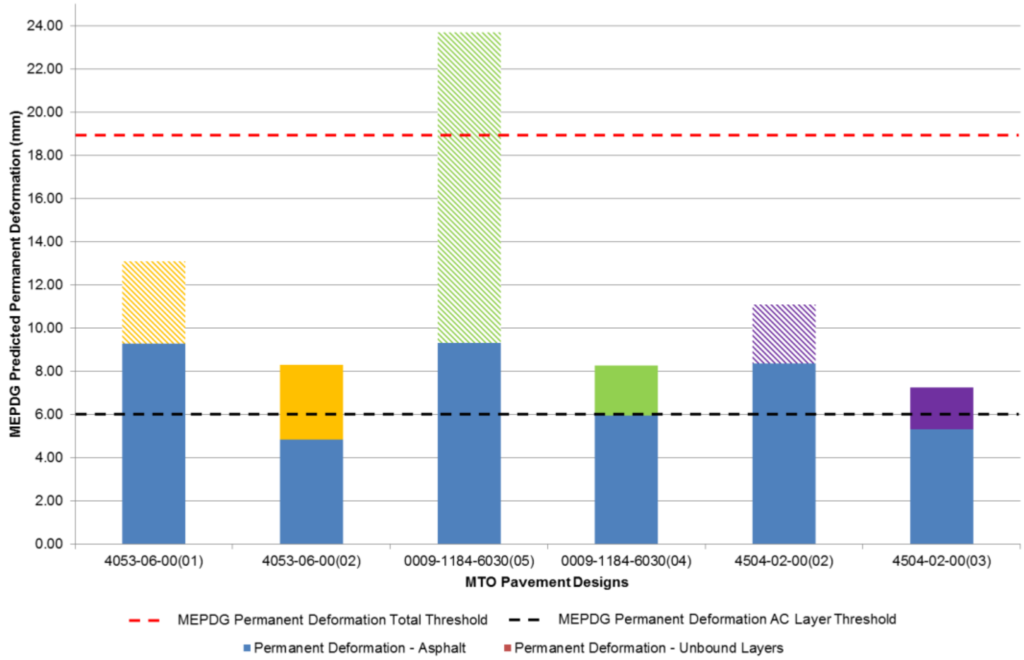


Figure 4-24: Asphalt Overlays of Flexible Pavement Design Scenarios – MEPDG Predicted Permanent Deformation

As shown in Figure 4-24, the asphalt overlays placed over CIREAM had significantly higher MEPDG predicted permanent deformation in both the asphalt and unbound layers compared to asphalt overlays placed over milled existing asphalt layers.

Figure 4-25 shows the average MEPDG predicted permanent deformation for asphalt overlays of existing flexible pavements designed using the AASHTO 1993 method.

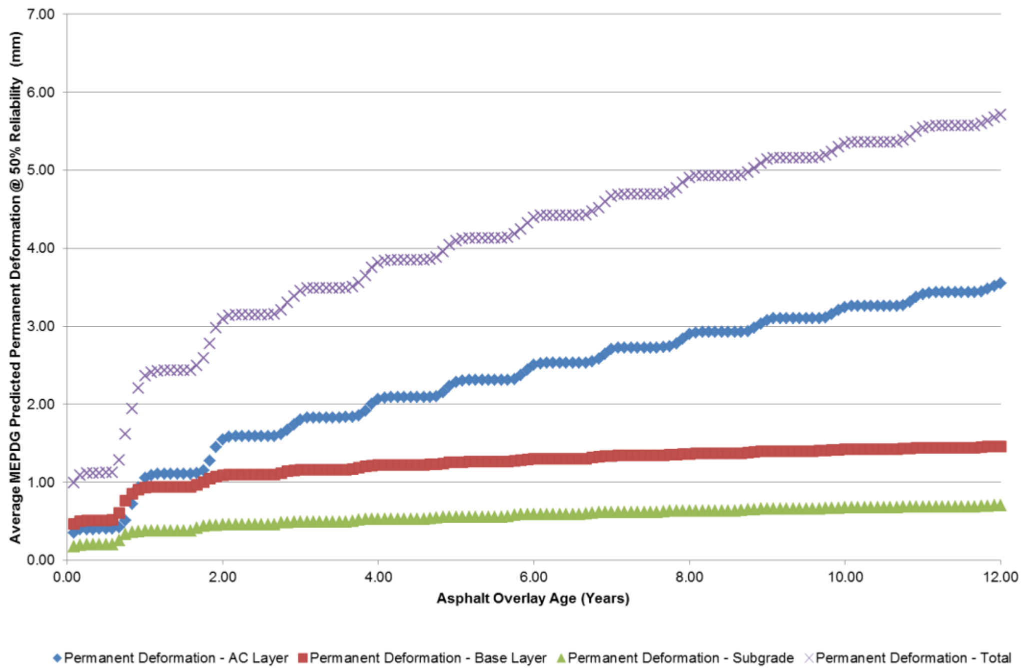


Figure 4-25: Average MEPDG Predicted Permanent Deformation at 50% Reliability versus Asphalt Overlay Age for AASHTO 1993 Asphalt Overlays of Existing Flexible Pavement

As shown in Figure 4-25, the MEPDG predicted minimal permanent deformation in the unbound layers and subgrade for the asphalt overlays of existing flexible pavements designed using the AASHTO 1993 method. In addition, no significant rutting in the unbound layers and subgrade was observed within the first year of the asphalt overlay design life.

Figure 4-26 and Figure 4-27 show MEPDG predicted permanent deformation in the total pavement structure and asphalt layers plotted against MEPDG estimated ESALs.

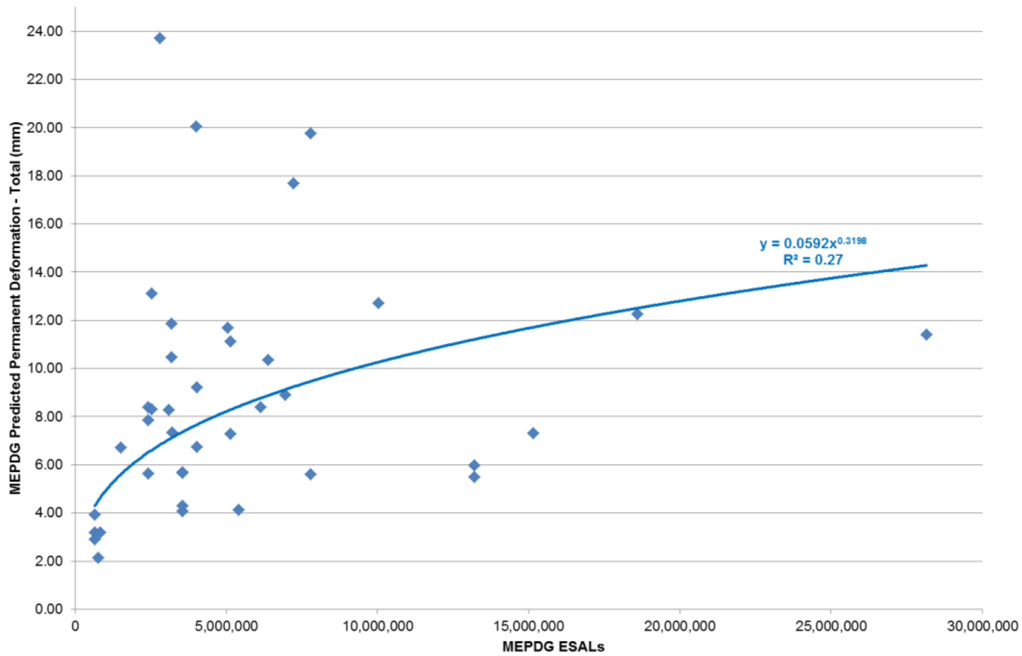


Figure 4-26: MEPDG Predicted Permanent Deformation in the Total Pavement Structure for AASHTO 1993 Asphalt Overlays of Existing Flexible Pavements

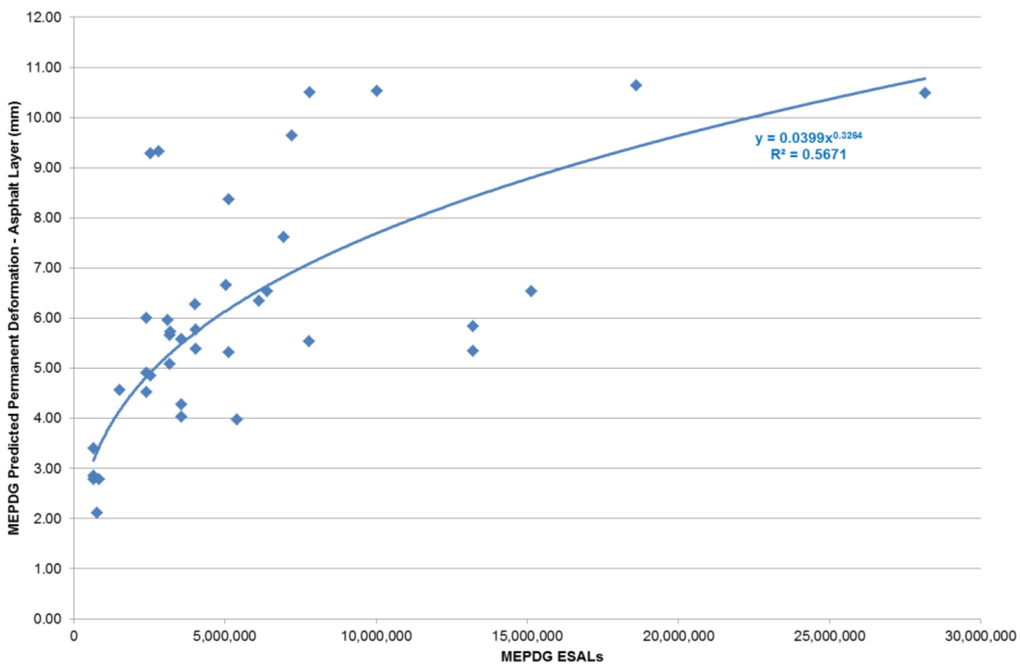


Figure 4-27: MEPDG Predicted Permanent Deformation in the Asphalt Layer for AASHTO 1993 Asphalt Overlays of Existing Flexible Pavements

As shown in Figure 4-26, a relatively weak relationship was observed between MEPDG predicted permanent deformation in the total pavement structure and traffic loading. However, Figure 4-27 shows that a significant relationship was observed between MEPDG predicted permanent deformation in the asphalt layers and traffic loading.

The MEPDG uses the same permanent deformation models for new flexible pavements and asphalt overlays of existing flexible pavements. However, the MEPDG includes an additional design parameter for asphalt overlays of existing flexible pavements not included in new flexible pavements, the existing pavement condition rating. Figure 4-28 and Figure 4-29 show MEPDG predicted permanent deformation in the total pavement structure and asphalt layer based on the existing pavement condition rating.

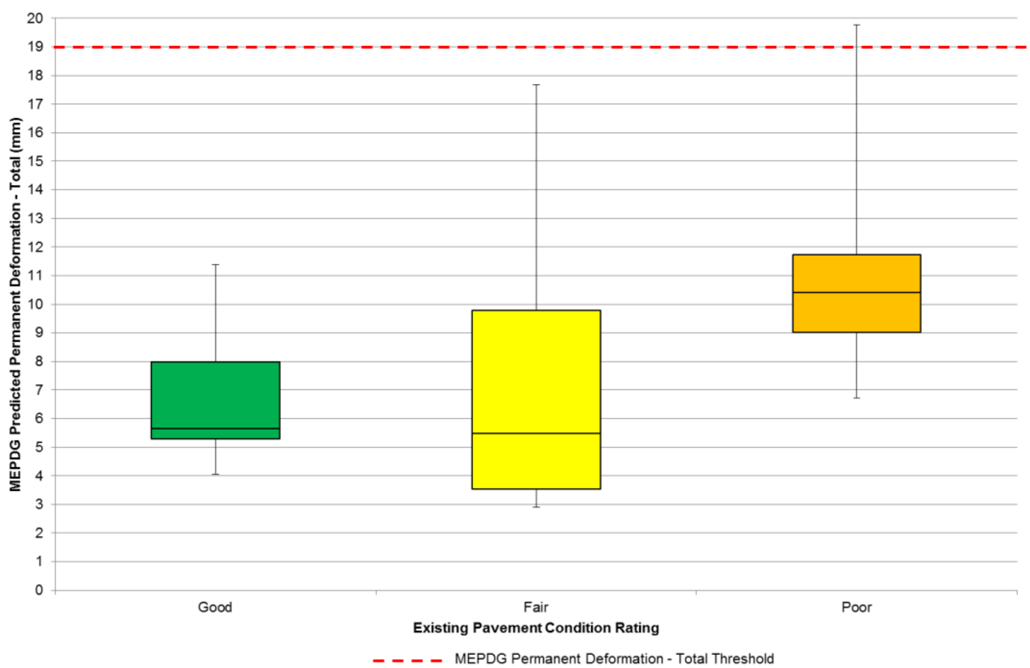


Figure 4-28: MEPDG Predicted Total Permanent Deformation by Existing Pavement Condition Rating for AASHTO 1993 Asphalt Overlays of Existing Flexible Pavement (Freeways)

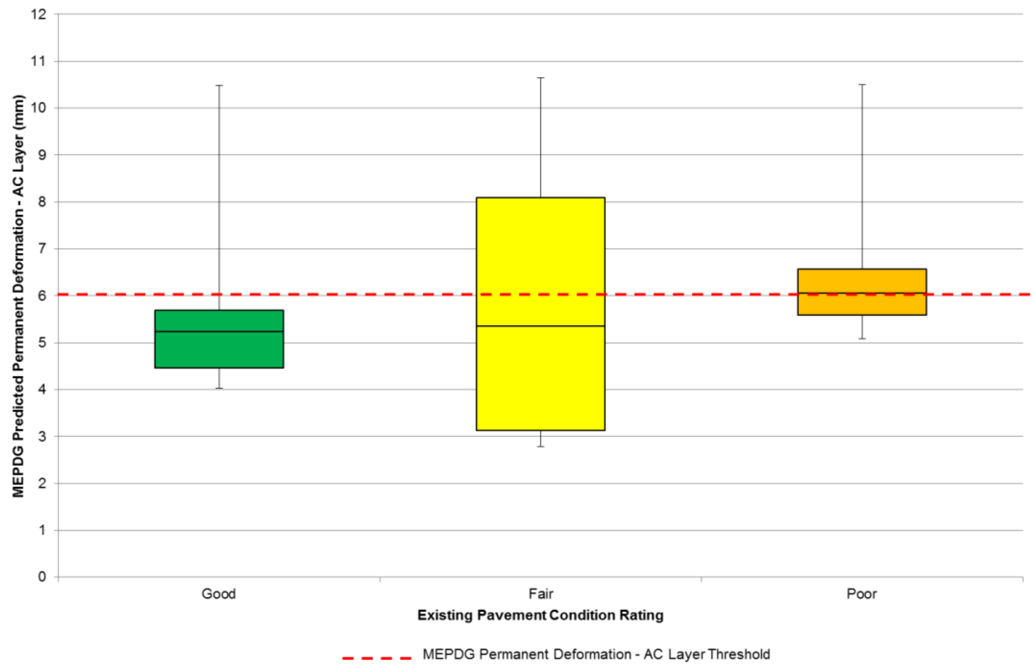


Figure 4-29: MEPDG Predicted Aspahlt Layer Permanent Deformation by Existing Pavement Condition Rating for AASHTO 1993 AC Overlays of Existing Flexible Pavement (Freeways)

As shown in Figure 4-28, asphalt overlays of flexible pavements in poor condition generally exhibited higher MEPDG permanent deformation in the total pavement structure. While the average MEPDG predicted permanent deformation in total pavement structure did not differ for asphalt overlays placed over exiting flexible pavement in fair or good condition, the variability in MEPDG predicted total permanent deformation was higher for existing flexible pavements in fair condition. Figure 4-29 shows that asphalt overlays of existing flexible pavements in poor conditions also exhibited greater rutting in the asphalt layer, although the relationship was less pronounced than for permanent deformation in the total pavement structure.

4.3.1.1.2 Discussion

Permanent deformation was one of the most significant pavement distresses contributing to the MEPDG predicted failure of the asphalt overlays of flexible pavements designed using the AASHTO 1993 method; approximately 36% of the designs examined failed to meet the MEPDG permanent deformation performance criteria.

MEPDG predicted permanent deformation in asphalt overlays of flexible pavement was influenced primarily by two design inputs: traffic loading; and, the existing pavement condition rating. As noted in Section 2.4.1.1 and Section 2.4.1.3, MEPDG sensitivity studies have consistently found these variables to have a significant influence on MEPDG predicted permanent deformation. MEPDG predicted permanent deformation in the asphalt layer was found to be primarily influenced by traffic loading for the asphalt overlays of flexible pavements; however, the relationship was not as strong as observed in new flexible pavements (see Section 4.2.1.1.2). This was likely due to the influence of the existing pavement condition rating variable, which was included in the asphalt overlay of flexible pavement analysis but not included from the new flexible pavement analysis. The MEPDG predicted permanent deformation in the total pavement structure was found to be less significantly influenced by traffic loading than permanent deformation in the asphalt layers; conversely, the influence of the existing pavement condition rating was found to be greater for MEPDG predicted permanent deformation in the total pavement structure than for MEPDG predicted permanent deformation in the asphalt layers.

The analysis also demonstrated that asphalt overlays placed over CIREAM generally experienced much higher permanent deformation than the asphalt overlays placed over unmodified asphalt pavement layers. The results of the sensitivity analysis show that, although the alternative pavement structures were designed using the same design inputs and AASHTO 1993 structural numbers, the asphalt overlays placed over CIREAM had higher MEPDG predicted permanent deformation compared to asphalt overlays placed over milled asphalt layers. This suggests that the MEPDG does not attribute the same structural performance to the CIREAM layer as the AASHTO 1993 method. This may be due to the fact that the MEPDG models asphalt overlays over CIREAM as new flexible pavement structures. As noted below, MEPDG predicted permanent deformation in new flexible pavements was significantly higher than for asphalt overlays of flexible pavements. Despite this, most of the asphalt overlays placed over CIREAM did meet the MEPDG performance criteria for permanent deformation in the total pavement structure.

Although the MEPDG uses the same permanent deformation models for new flexible pavements and asphalt overlays of flexible pavements, a significantly lower proportion of the asphalt overlay of flexible pavement designs failed to meet the MEPDG permanent deformation performance criteria. As discussed in Section 4.2.1.1.2, the significant MEPDG predicted permanent deformation in the new flexible pavements examined was found to be primarily due to the following three factors: very strong relationship between permanent deformation and traffic loading; unreasonably high predicted permanent deformation in the unbound layers within the first year of pavement design life; and, weak structural contribution assigned by the MEPDG to unbound granular layers. The results of the analysis show that these factors did not play as significant a role in the MEPDG predicted permanent deformation in asphalt overlays of flexible pavements. The relationship between traffic loading and MEPDG predicted permanent deformation was much weaker in asphalt overlays of flexible pavements compared to new flexible pavements. This was likely due to the influence of the existing pavement condition rating design input on MEPDG predicted permanent deformation, which was not included in the new flexible pavement analysis. The asphalt overlays of flexible pavements were also generally designed for shorter design lives than new flexible pavements, which reduced the total traffic loading experienced on these pavement structures. This resulted in lower MEPDG predicted permanent deformation in asphalt overlays of flexible pavements. The MEPDG also predicted relatively low permanent deformation in the unbound pavement layers and subgrades for asphalt overlays of existing flexible pavement relative to new flexible pavements. The reason for this was not apparent based on a review of the existing literature or analysis of data. In addition, no significant rutting in the unbound layers and subgrade was observed within the first year of the asphalt overlay design life as was the case for the new flexible pavement structures. Again, the reason for this was not determined through review of the existing literature or analysis of the data. As a result of all of the above factors, the MEPDG predicted permanent deformation in the asphalt overlays of flexible pavements was generally significantly lower than predicted for the new flexible pavements.

As stated in Section 4.2.1.1.2, almost all of the verification studies examined in the literature review found that the nationally-calibrated MEPDG permanent deformation models significantly over-predicted permanent deformation in new flexible pavements compared to pavement performance data recorded in the LTPP database or local State DOT PMSs (see Section 2.5.1 and Figure 2-9). Therefore, it is likely that the MEPDG over-predicted permanent deformation in AASHTO 1993 new flexible pavements observed in this analysis. Future studies could confirm this using pavement performance data from the MTO PMS or LTPP database.

4.3.1.2 MEPDG Predicted Fatigue Cracking for AASHTO 1993 Asphalt Overlays of New Flexible Pavements

4.3.1.2.1 Results

Figure 4-30 shows the MEPDG predicted bottom-up and top-down fatigue cracking in asphalt overlays of existing flexible pavement designed using the AASHTO 1993 method. The pavement designs are arranged in ascending order from left to right in terms of total asphalt layer thickness (i.e. the sum of existing asphalt thickness and new asphalt overlay thickness).

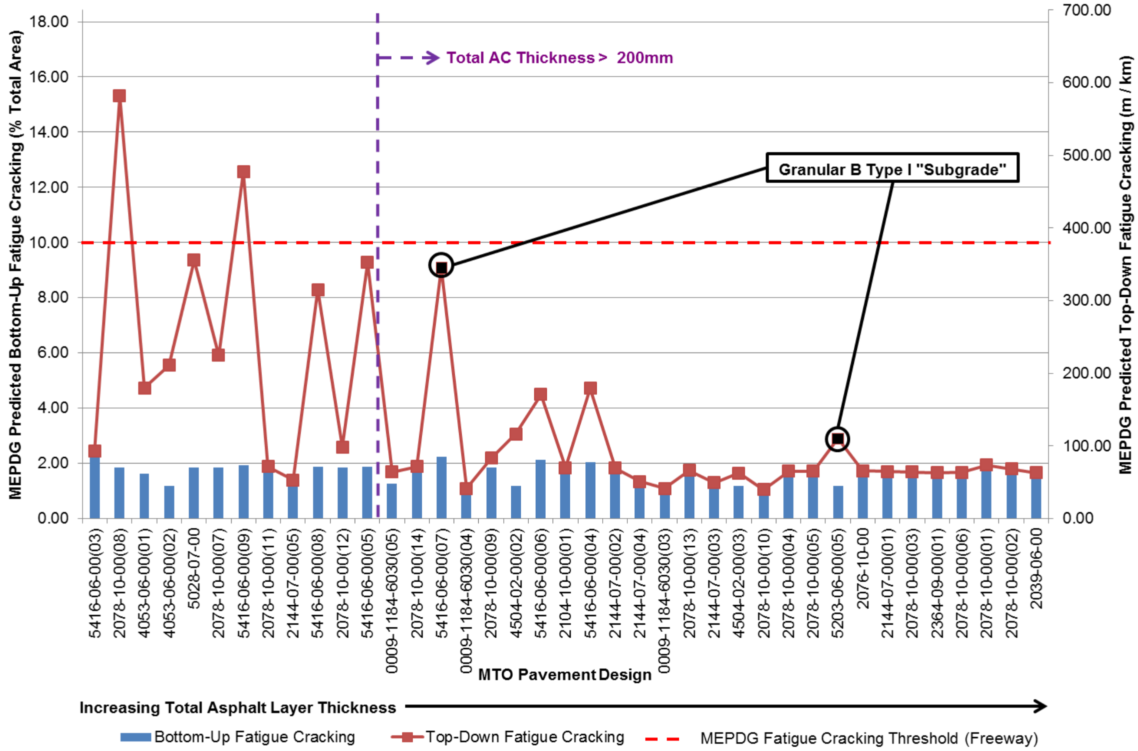


Figure 4-30: MEPDG Predicted Fatigue Cracking in AASHTO 1993 Asphalt Overlays of Existing Flexible Pavements

As shown in Figure 4-30, MEPDG predicted bottom-up fatigue cracking was very low and did not vary significantly among the pavement sections examined, ranging from only 1.17% to 2.24% of total pavement area.

In contrast to bottom-up fatigue cracking, Figure 4-30 shows that the MEPDG predicted top-down fatigue cracking did vary significantly among the pavement sections examined. MEPDG predicted top-down fatigue cracking appeared to have a significant inverse relationship with total asphalt thickness of the pavement structure. Figure 4-30 shows that, in general, the asphalt overlays of existing flexible pavements that exhibited significant top-down fatigue cracking were observed in pavement sections with total asphalt thickness less than 200 mm.

Figure 4-31 shows a box-and-whisker plot of MEPDG predicted top-down fatigue cracking in asphalt overlays of new flexible pavement designed using the AASHTO 1993 method based on total asphalt thickness.

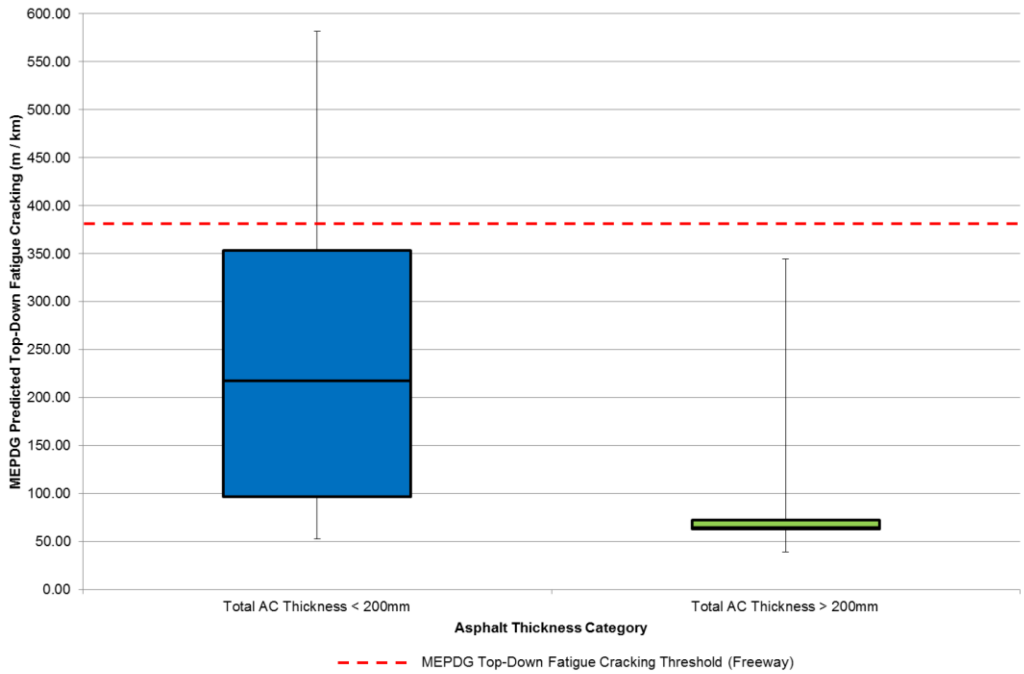


Figure 4-31: MEPDG Predicted Top-Down Fatigue Cracking in AASHTO 1993 Asphalt Overlays of Existing Flexible Pavement by Asphalt Layer Thickness

As shown in Figure 4-31, the MEPDG predicted top-down fatigue cracking in asphalt overlays of existing flexible pavement designed using the AASHTO 1993 method was much higher in pavement sections with a total asphalt thickness less than 200 mm. The range of MEPDG predicted top-down fatigue cracking was also much higher for pavement sections where the total asphalt layer thickness was less than 200 mm. With the exception of a few outliers, the MEPDG predicted top-down fatigue cracking in pavement sections where the total asphalt layer exceeded 200 mm was generally less than 100 m / km.

In addition to total asphalt layer thickness, the resilient modulus and coarseness of the subgrade was also found to exert a strong influence on MEPDG predicted top-down fatigue cracking in asphalt overlays of flexible pavements. Figure 4-32 shows MEPDG predicted top-down fatigue cracking in asphalt overlays of flexible pavements designed using the AASHTO 1993 method by subgrade type.

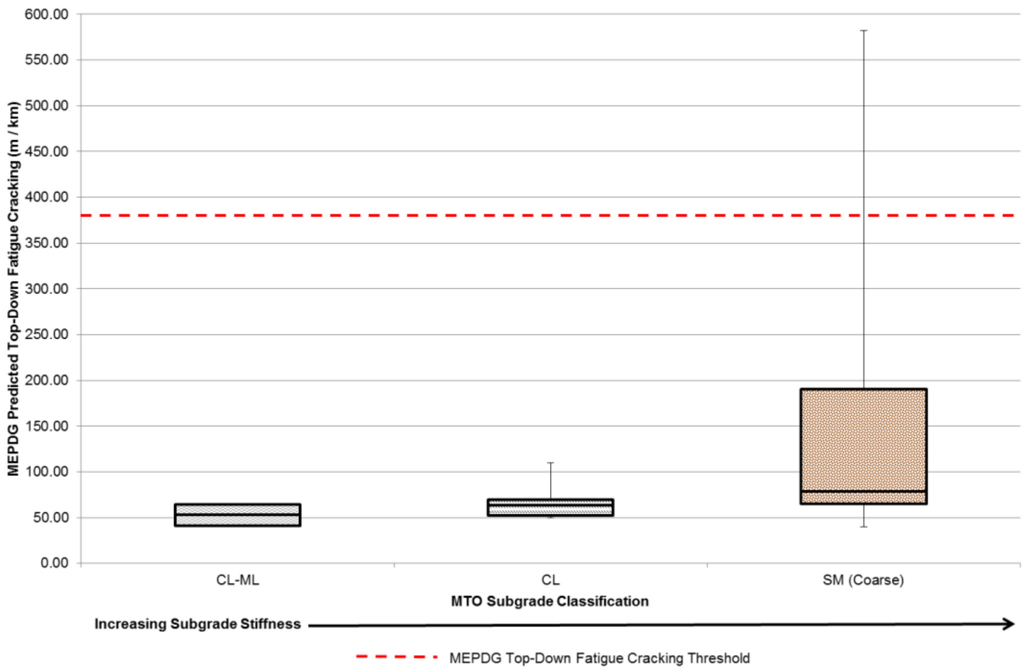


Figure 4-32: MEPDG Predicted Top-Down Fatigue Cracking in AASHTO 1993 Asphalt Overlays of Existing Flexible Pavements by Subgrade Type

As shown in Figure 4-32, MEPDG predicted top-down fatigue cracking increased significantly as the resilient modulus and coarseness of the subgrade material increased. No significant top-down fatigue cracking was predicted in asphalt overlays of flexible pavements constructed on clay / silt subgrades. The MEPDG predicted top-down fatigue cracking observed in pavement sections with the coarse sand subgrade was much higher and more variable.

Two of the asphalt overlays of flexible pavements had to be modelled using a Granular B Type I subbase. This was necessary due to the limitations of the AASHTOWare Pavement ME software, which is unable to produce analysis output for thick pavement structures where the number of sublayers exceeds nineteen (see discussion in Section 4.2.1.2.2). As shown in Figure 4-30, one of the pavement sections modelled with a Granular B Type I subbase had MEPDG predicted top-down fatigue cracking approaching the performance criteria threshold; however, the other pavement section, which had a much thicker total asphalt layer, had very low MEPDG predicted top-down fatigue cracking.

The MEPDG uses the same fatigue cracking models for new flexible pavements and asphalt overlays of existing flexible pavements. However, the MEPDG includes an additional design parameter for asphalt overlays of existing flexible pavements not included in new flexible pavements, the existing pavement condition rating. Figure 4-33 shows the MEPDG predicted permanent deformation in the total pavement structure and asphalt layer based on the condition rating of the existing flexible pavement.

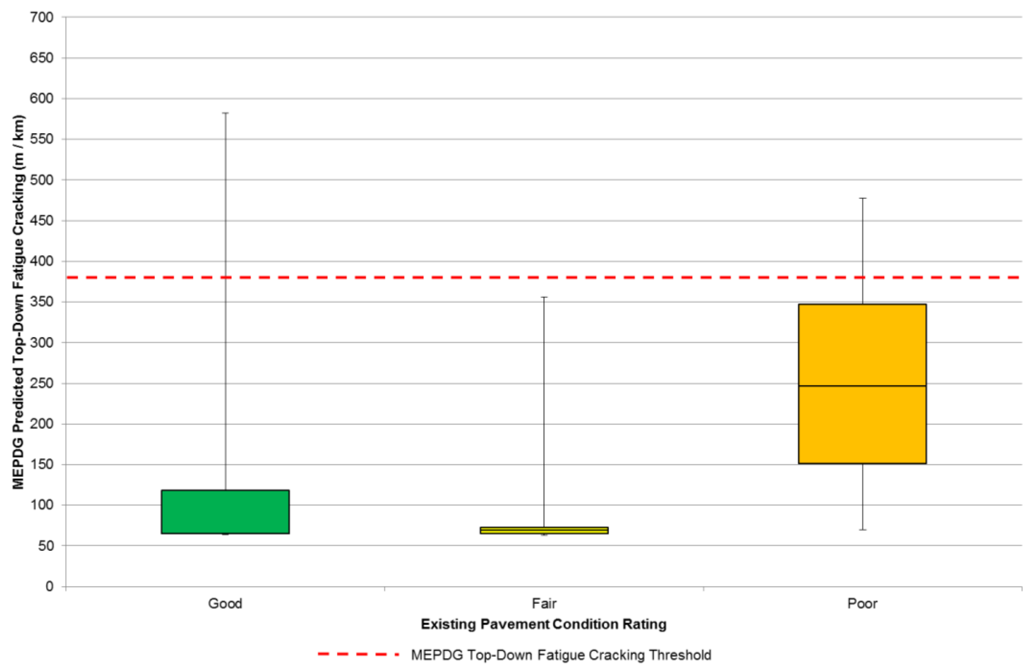


Figure 4-33: MEPDG Predicted Top-Down Fatigue Cracking in AASHTO 1993 Asphalt Overlays of Existing Flexible Pavements by Existing Pavement Condition Rating (Freeways)

As shown in Figure 4-33, the MEPDG predicted top-down fatigue cracking in asphalt overlays of flexible pavements designed using the AASHTO 1993 method varied considerably based on the condition rating of the existing flexible pavement. Asphalt overlays placed over existing flexible pavements in Poor condition experienced much higher overall MEPDG predicted top-down fatigue cracking; in addition, the range of MEPDG predicted top-down fatigue cracking was much higher in these pavements. MEPDG predicted top-down fatigue cracking was generally low in pavement sections where the existing pavement condition was either Good or Fair, although some outliers did experience significant MEPDG predicted top-down fatigue cracking.

4.3.1.2.2 Discussion

None of the thirty-nine asphalt overlays of existing flexible pavements designed using the AASHTO 1993 method failed based on MEPDG predicted bottom-up fatigue cracking. MEPDG predicted bottom-up fatigue cracking was very low and did vary significantly among the asphalt overlays of flexible pavement examined.

MEPDG predicted bottom-up fatigue cracking was also very low in asphalt overlays of flexible pavements that had total asphalt thicknesses less than 200 mm. As noted in Section 2.4.2.3, MEPDG sensitivity studies have generally found MEPDG predicted bottom-up fatigue cracking to be strongly related to asphalt layer thickness. In general, pavements with asphalt layer thicknesses less than 150 mm have been found to exhibit significant MEPDG predicted bottom-up fatigue cracking (see Figure 2-6). These findings also correspond with the observations for new flexible pavements in this study (see Section 4.2.1.2). However, the asphalt overlays of flexible pavements examined in this study did not experience any significant MEPDG predicted bottom-up fatigue cracking regardless of the thickness of the asphalt layer.

As stated in Section 2.5.2, most State DOT verification studies have found that the nationally-calibrated MEPDG bottom-up fatigue cracking model significantly under-predicts bottom-up fatigue cracking when compared to local pavement performance data in the LTPP database or State PMS. The very low values of MEPDG predicted bottom-up fatigue cracking observed in this analysis combined with the findings from these verification studies suggests that the MEPDG model may significantly under-predict bottom-up fatigue cracking; this could be verified for Ontario conditions in future research using pavement performance data from the MTO PMS and / or LTPP database.

Approximately 5% of the asphalt overlays of existing flexible pavements designed using the AASHTO 1993 method failed based on MEPDG predicted top-down fatigue cracking. In general, the asphalt overlays of existing flexible pavements that exhibited significant top-down fatigue cracking had total asphalt thicknesses of less than 200 mm. This corresponds with the relationship observed for new flexible pavements in Section 4.2.1.2 and reported in the MEPDG sensitivity studies (see Figure 4-9).

MEPDG predicted top-down fatigue cracking in asphalt overlays of existing flexible pavements was significantly higher and more variable in subgrades with higher resilient moduli and coarseness. This corresponds with the findings for new flexible pavements presented in Section 4.2.1.2.2, and reported in MEPDG sensitivity studies discussed in Section 2.4.3.4. The results of the analysis suggest that the presence of a coarse / stiff subgrade did not necessarily result in higher MEPDG predicted top-down fatigue cracking, however, it did increase the likelihood of higher values being predicted.

4.3.1.3 MEPDG Predicted Thermal Cracking for AASHTO 1993 Asphalt Overlays of New Flexible Pavements

4.3.1.3.1 Results

The MEPDG predicted thermal cracking was very low and did not vary significantly for the asphalt overlays of flexible pavements designed using the AASHTO 1993 method. The range of MEPDG predicted thermal cracking in these pavement sections was 4.05 m / km to 6.55 m / km, much less than the MEPDG performance threshold of 190 m / km.

Similar to the transverse cracking sensitivity analysis performed for new flexible pavements in Section 4.2.1.3, a sensitivity analysis was conducted to assess the response of the nationally-calibrated MEPDG transverse cracking model to a change in asphalt PGAC grade in asphalt overlays of flexible pavements. As per the recommendations in the MTO PDRs, asphalt mix designs in Southern Ontario used PGAC grade 58-28, while asphalt mix designs in Northern Ontario used PGAC grade 58-34. To assess the sensitivity of the MEPDG transverse cracking model, the PGAC grade of the new flexible pavements in Northern Ontario was changed to PGAC 58-28. The results of the sensitivity analysis are shown in Table 4-6.

Table 4-6: Effect of PGAC Grade on MEPDG Predicted Transverse Cracking in AASHTO 1993 Asphalt Overlays of Existing Flexible Pavements in Northern Ontario

MTO Pavement Design	Climate Station	MEPDG Predicted Transverse Cracking (m/km)	
		PGAC 58-34	PGAC 58-28
4504-02-00(02)	Petawawa	4.05	85.25
4504-02-00(03)	Petawawa	4.13	226.46
5028-07-00	Killaloe	6.42	53.78
5203-06-00(05)	Sudbury	4.08	278.74
5416-06-00(03)	North Bay	6.53	203.38
5416-06-00(04)	North Bay	6.38	47.01
5416-06-00(05)	North Bay	6.40	89.64
5416-06-00(06)	North Bay	6.37	29.05
5416-06-00(07)	North Bay	6.38	42.79
5416-06-00(08)	North Bay	6.39	25.58
5416-06-00(09)	North Bay	6.39	51.6

As shown in Table 4-6, the change in PGAC grade had a significant impact on the MEPDG predicted transverse cracking, with some of the pavement sections exceeding the MEPDG transverse cracking performance criteria threshold of 190 m / km. However, unlike the results observed in new flexible pavements, the MEPDG predicted transverse cracking in most of the pavement sections remained well below the performance criteria threshold.

4.3.1.3.2 Discussion

The results of the MEPDG predicted transverse cracking analysis demonstrate that MEPDG predicted transverse cracking in asphalt overlays of flexible pavements is negligible provided temperatures experienced by the pavement do not fall outside the bounds of the selected PGAC grade. If an inadequate PGAC grade is selected for the asphalt layers, the asphalt overlay of flexible pavement may fail to meet the MEPDG transverse cracking performance criteria.

As mentioned previously in Section 4.2.1.3.2, the results of State DOT verification studies are mixed regarding the accuracy of the nationally-calibrated MEPDG transverse cracking model. Von Quintus et al. (2007) have stated that the nationally-calibrated MEPDG transverse cracking model is reasonably accurate for flexible pavements in northern climates. However, Mallela et al. (2009) have stated that the MEPDG transverse cracking model generally overestimates asphalt creep compliance of HMA mixes and consequently underestimates thermal cracking, especially in colder climates. The very low values of thermal cracking observed in the asphalt overlays of flexible pavements analyzed in this research suggest that the model may under-predict transverse cracking in asphalt overlays of flexible pavements in Ontario. This could be verified in future research using MTO PMS pavement performance data.

4.3.1.4 MEPDG Predicted IRI for AASHTO 1993 Asphalt Overlays of New Flexible Pavements

4.3.1.4.1 Results

Figure 4-34 shows the MEPDG predicted terminal IRI in asphalt overlays of existing flexible pavement designed using the AASHTO 1993 method. The pavement sections are arranged in ascending order of overlay design life from left to right. In addition, the pavement sections are also arranged in ascending order of traffic loading, , based on MEPDG estimated ESALs, from left to right within each pavement design life group.

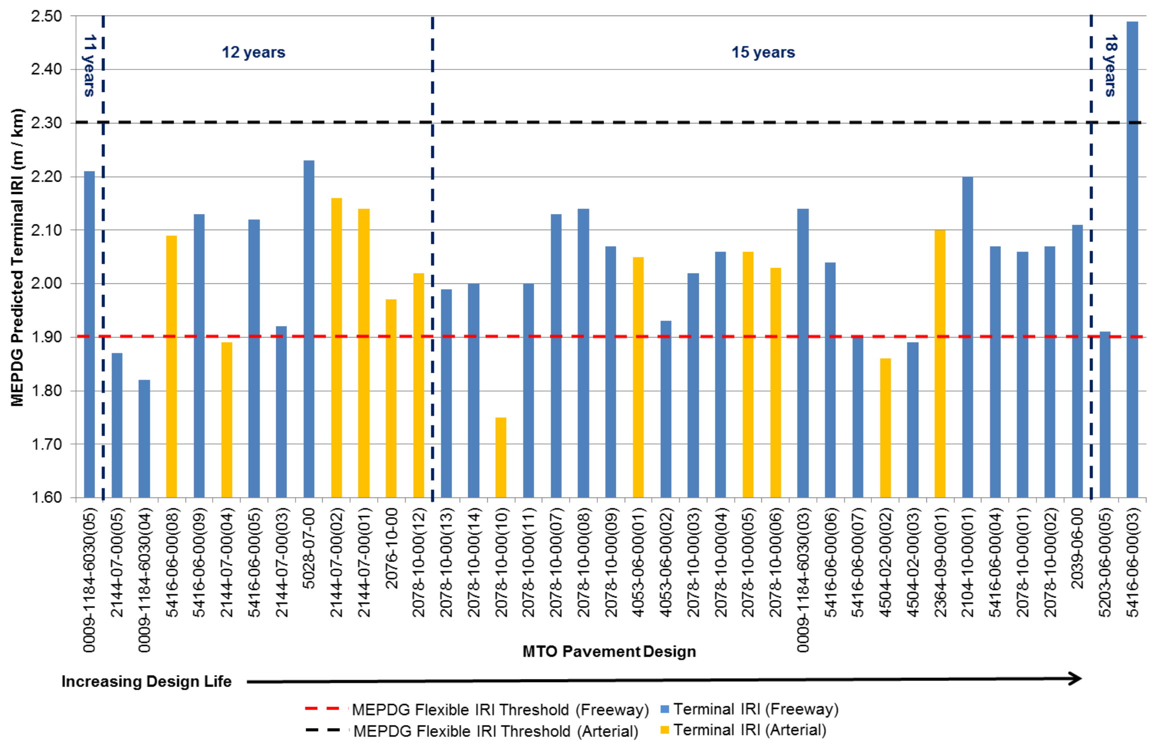


Figure 4-34: MEPDG Predicted Terminal IRI for AASHTO 1993 Asphalt Overlays of Existing Flexible Pavements

As shown in Figure 4-34, twenty-six of the thirty-nine asphalt overlays of existing flexible pavement designed using the AASHTO 1993 method failed the MEPDG performance criteria for terminal IRI. All of the freeway pavement sections failed the MEPDG performance criteria for terminal IRI except for MTO pavement design 5416-06-00(07), which had a MEPDG predicted terminal IRI of 190 m / km, exactly equal to the IRI performance threshold for freeways. In contrast, all of the arterial pavement sections met the MEPDG performance criteria for terminal IRI.

Figure 4-35 shows the MEPDG predicted terminal IRI plotted against MEPDG predicted permanent deformation in the total pavement structure for asphalt overlays of flexible pavement designed using the AASHTO 1993 method.

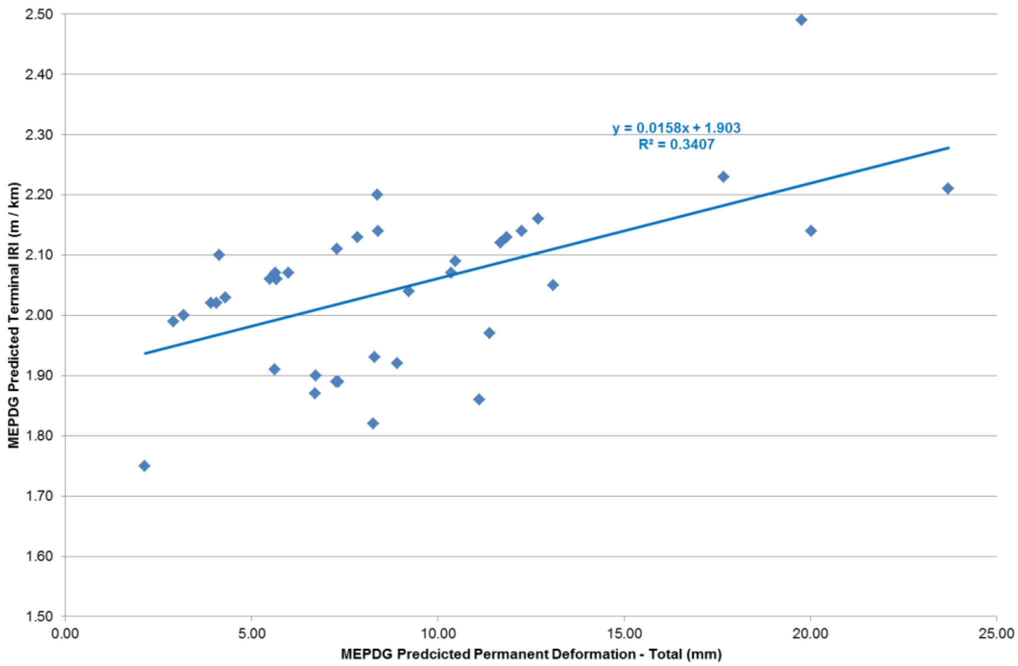


Figure 4-35: MEPDG Predicted IRI versus MEPDG Predicted Total Permanent Deformation for AASHTO 1993 AC Overlays of Flexible Pavement

As shown in Figure 4-35, a strong correlation was observed between MEPDG predicted terminal IRI and MEPDG predicted total permanent deformation in the asphalt overlays of flexible pavements examined.

Table 4-7 summarizes the results of regression analysis conducted for MEPDG predicted terminal IRI and the other MEPDG predicted pavement distresses for asphalt overlays of flexible pavements.

Table 4-7: MEPDG Predicted Terminal IRI Linear Regression Analysis for Asphalt Overlays of Flexible Pavement

Independent Variable	Slope	Y-intercept	R^2
Permanent Deformation – Total (mm)	0.0161*	1.8989*	0.3449
Permanent Deformation – Asphalt (mm)	0.0280*	1.8708*	0.2196
Total Cracking – Alligator + Reflective (% Total Area)	-0.0018	2.1060*	0.0237
Bottom-Up Fatigue Cracking (% Total Area)	0.2406*	1.6303*	0.3111
Thermal Cracking (m / km)	0.0703*	1.6348*	0.2601
Top-Down Fatigue Cracking (m / km)	0.0002	2.0104*	0.0516

As shown in Table 4-7, a moderate relationship was observed between MEPDG predicted terminal IRI and MEPDG predicted total permanent deformation, asphalt layer permanent deformation, bottom-up fatigue cracking, and thermal cracking. In contrast, almost no relationship was observed with either total cracking or top-down fatigue cracking.

Figure 4-36 shows MEPDG predicted terminal IRI plotted against traffic loading, expressed in terms of MEPDG estimated ESALs, for asphalt overlays of flexible pavement designed using the AASHTO 1993 method.

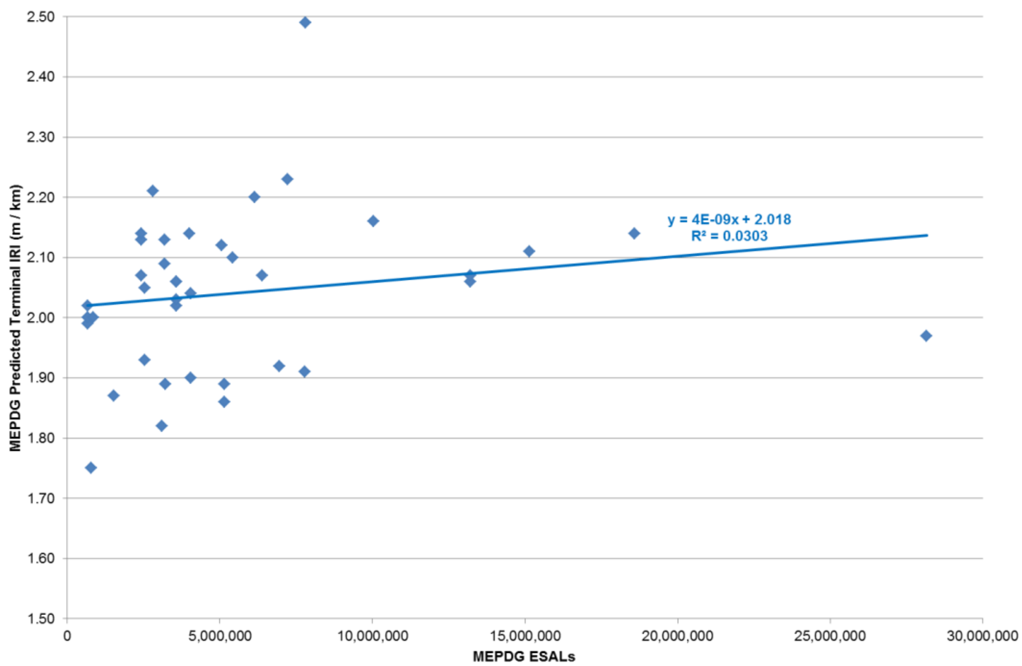


Figure 4-36: MEPDG Predicted Terminal IRI versus MEPDG ESALs for AASHTO 1993 Asphalt Overlays of Flexible Pavements

As shown in Figure 4-36, no relationship was observed between MEPDG predicted terminal IRI and traffic loading for the asphalt overlays of existing flexible pavement designed using the AASHTO 1993 method.

Figure 4-34 shows that no clear relationship was observed between MEPDG predicted terminal IRI and pavement design life in asphalt overlays of existing flexible pavements. Figure 4-37 and Figure 4-38 show MEPDG predicted terminal IRI for asphalt overlays of flexible pavement designed using AASHTO 1993 plotted against asphalt overlay design life and total pavement age, respectively.

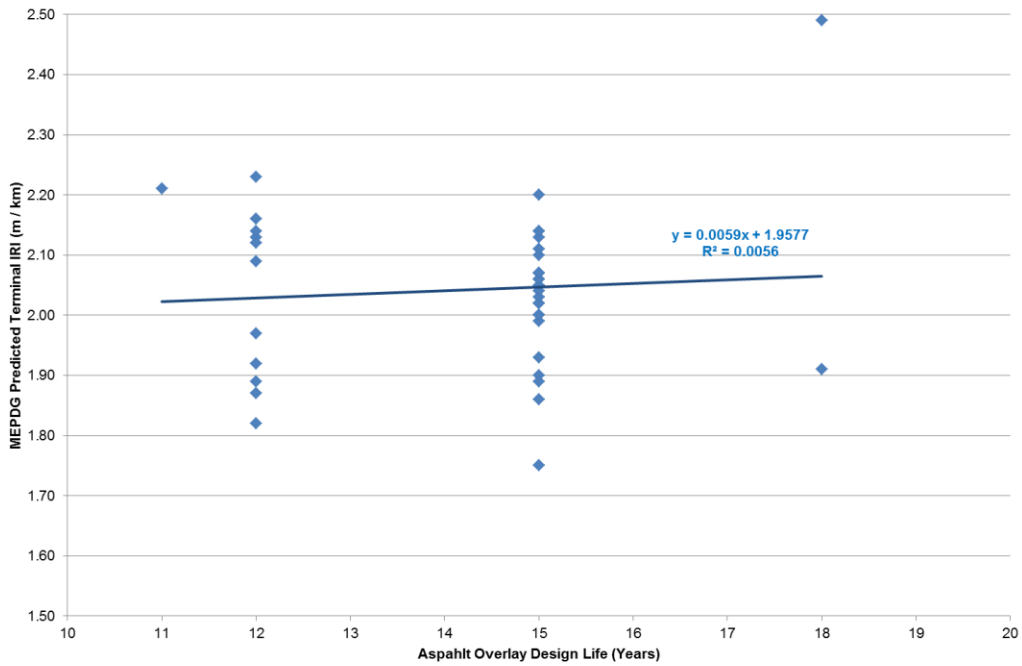


Figure 4-37: MEPDG Predicted Terminal IRI versus Pavement Design Life for AASHTO 1993 Asphalt Overlays of Flexible Pavement

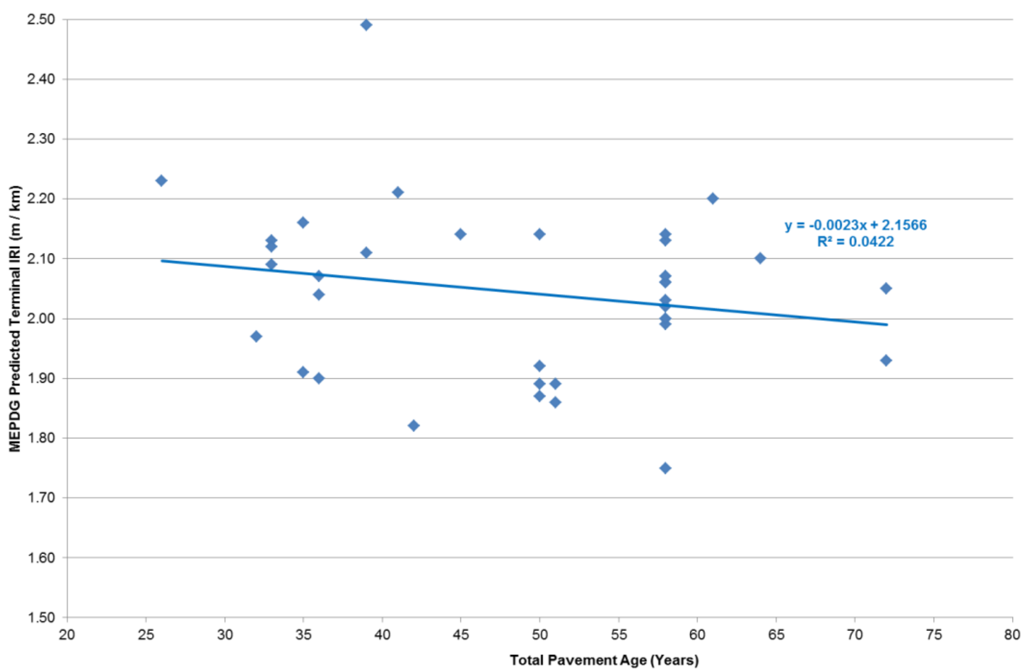


Figure 4-38: MEPDG Predicted Terminal IRI versus Pavement Age for AASHTO 1993 Asphalt Overlays of Flexible Pavement

As shown in Figure 4-37 and Figure 4-38, no relationship was observed between MEPDG predicted terminal IRI and either asphalt overlay design life or total pavement age for the asphalt overlays of flexible pavement examined.

Figure 4-39 and Figure 4-40 show MEPDG predicted terminal IRI for asphalt overlays of flexible pavement designed using the AASHTO 1993 method plotted against asphalt overlay thickness and total asphalt layer thickness, respectively.

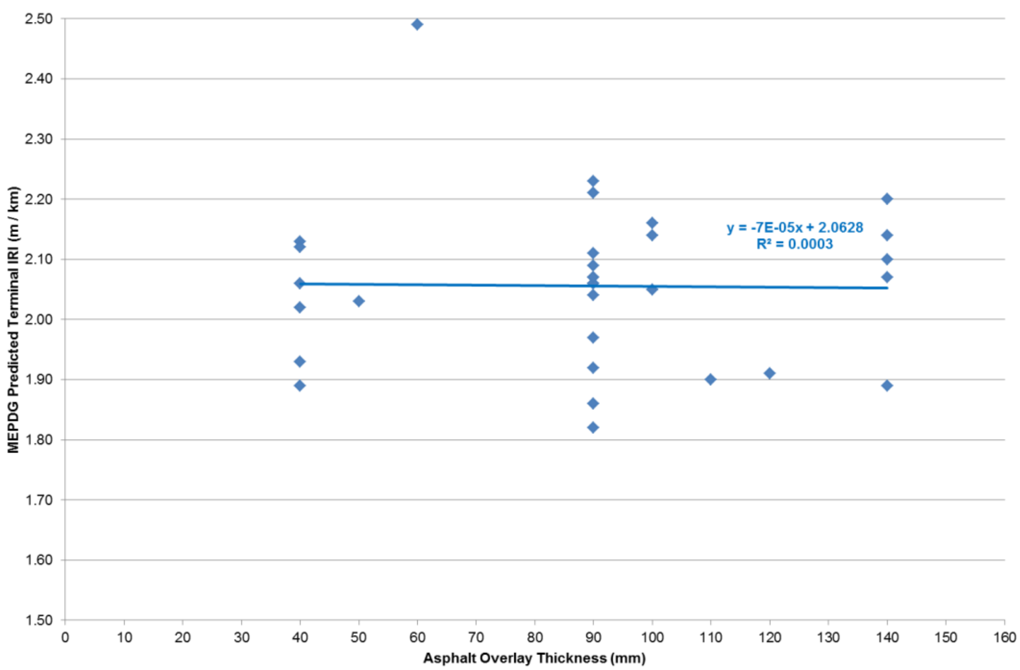


Figure 4-39: MEPDG Predicted Terminal IRI versus Asphalt Overlay Thickness for AASHTO 1993 Asphalt Overlays of Flexible Pavement

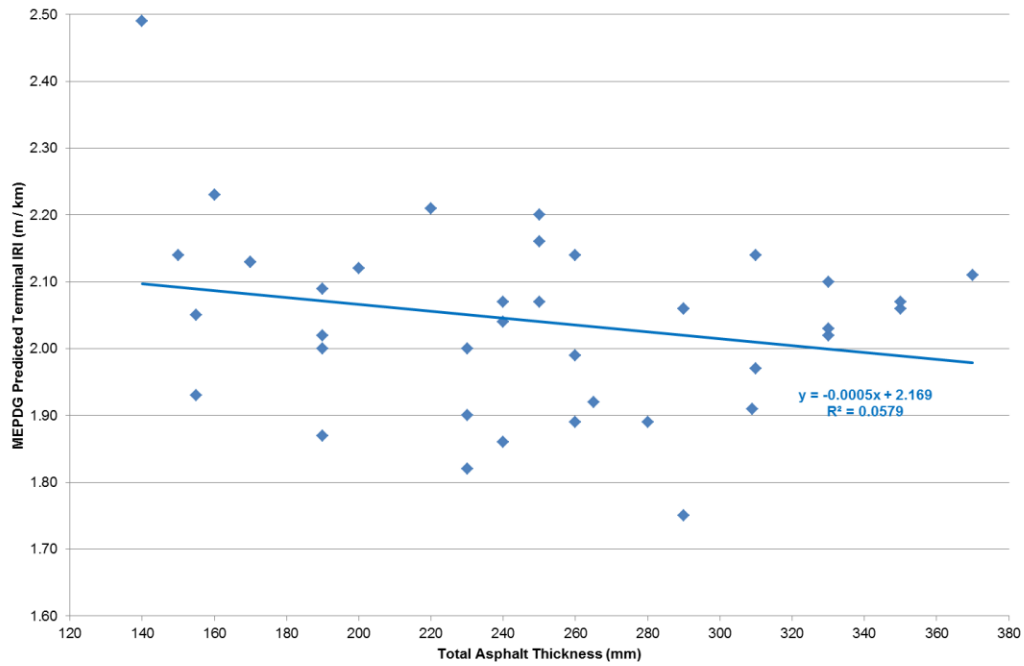


Figure 4-40: MEPDG Predicted Terminal IRI versus Total Asphalt Layer Thickness for AASHTO 1993 Asphalt Overlays of Existing Flexible Pavement

As shown in Figure 4-39 and Figure 4-40, no significant relationship was observed between MEPDG predicted terminal IRI and either asphalt overlay thickness or total asphalt layer thickness.

Figure 4-41 shows a box and whisker plot of MEPDG predicted terminal IRI based on the existing condition rating for asphalt overlays of flexible pavements designed using the AASHTO 1993 method.

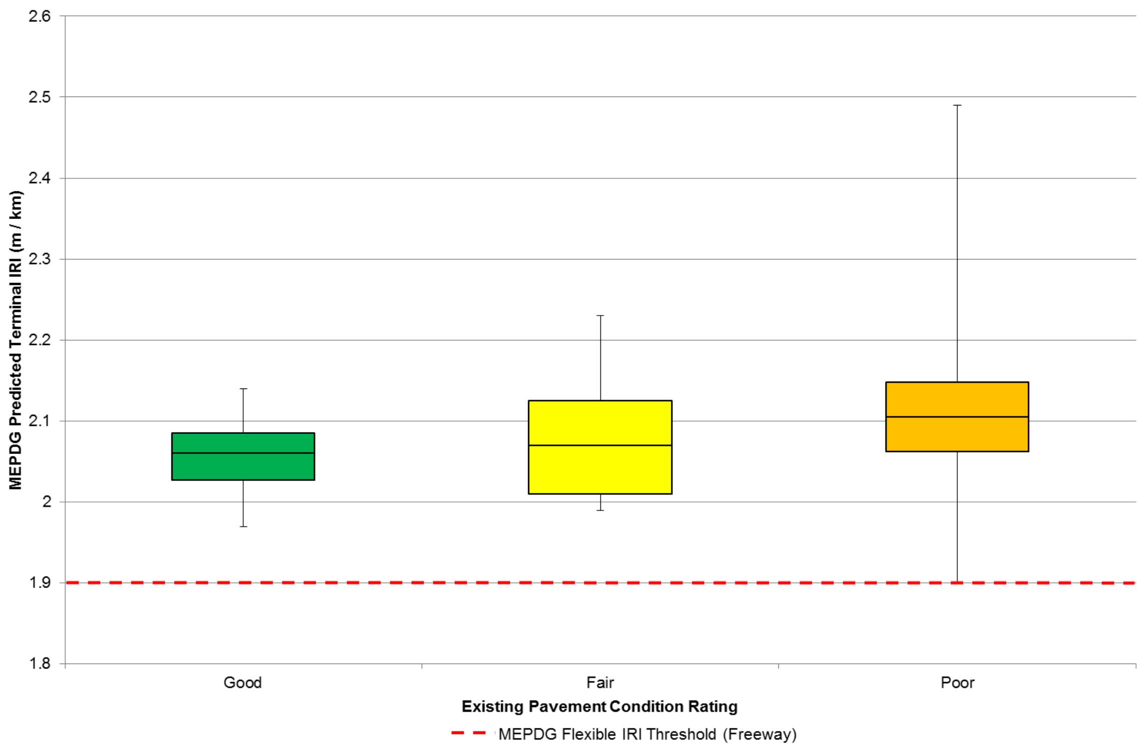


Figure 4-41: MEPDG Predicted Terminal IRI by Existing Pavement Condition Rating for Asphalt Overlays of Flexible Pavements (Freeways)

As shown in Figure 4-41, the MEPDG predicted terminal IRI increased moderately as the condition rating of the existing flexible pavement decreased from Good to Poor. The range in MEPDG predicted terminal IRI increased significantly as the condition rating of the existing flexible pavement decreased from Good to Poor.

4.3.1.4.2 Discussion

Approximately 67% of the asphalt overlays of flexible pavements examined failed the performance criteria for MEPDG predicted terminal IRI. MEPDG predicted IRI was the most significant pavement distress contributing to the predicted failure of asphalt overlays of flexible pavements designed using the AASHTO 1993 method. The results of this analysis indicate that the MEPDG over-predicts pavement roughness in asphalt overlays of flexible pavement relative to the AASHTO 1993 method.

The MEPDG predicted IRI was found to be most significantly influenced by highway functional classification, traffic loading, and existing pavement condition rating. In general, the asphalt overlays of flexible pavements that failed the performance criteria for MEPDG predicted terminal IRI were freeways, while the pavement designs that met the performance criteria were arterial highways. The arterial highways were more likely to meet the MEPDG performance criteria for MEPDG predicted terminal IRI for a number of reasons. First, the MEPDG terminal IRI performance threshold was higher for arterial highways compared to freeways. Second, the MEPDG predicted terminal IRI for arterial highways at a reliability level of either 90% for urban highways or 85% for rural highways. In contrast, MEPDG terminal IRI was predicted at a 95% reliability level for freeways. This made the IRI predictions for freeways higher than at the lower reliability level.

The MEPDG predicted terminal IRI in asphalt overlays of flexible pavement was also found to be significantly influenced by the existing pavement condition rating. The results of the analysis indicate that the condition rating of the existing flexible pavement exerts considerable influence on MEPDG predicted terminal IRI, with pavements in Poor condition having an increased likelihood of having very high MEPDG predicted terminal IRI. The significant influence of existing pavement condition rating on MEPDG predicted terminal IRI is consistent with the results of studies examined in the literature review (see Section 2.4.5.3). Since MEPDG IRI is a composite measure of the other pavement distresses, and since existing pavement condition rating exerts significant influence over the predicted value of the other pavement distresses, it is intuitive that existing pavement condition rating would have a significant influence on MEPDG predicted terminal IRI.

Unlike the MEPDG predicted values for terminal IRI in new flexible pavements, the MEPDG predicted values for terminal IRI in asphalt overlays of flexible pavements were not significantly affected by traffic loading or pavement age. The absence of any relationship between traffic loading and MEPDG predicted IRI was unexpected given the very strong relationship observed between these two variables in new flexible pavement structures designed using the AASHTO 1993 method (see Figure 4-16). This result may be due to both the reduced influence of traffic loading on permanent deformation and the reduced influence of permanent deformation on terminal IRI observed in asphalt overlays of flexible pavements relative to new flexible pavements. Similarly, the absence of a relationship between pavement age and MEPDG predicted IRI was inconsistent with the very strong relationship observed between these variables for new flexible pavements (see Figure 4-17). In addition, this result was counter-intuitive based on the linear relationship between MEPDG predicted terminal IRI and the site factor variable included in the MEPDG performance equation (AASHTO 2008).

As noted in Section 4.2.1.4, the MEPDG IRI model for new flexible pavements and asphalt overlays of flexible pavements predicts pavement roughness as a function of the other MEPDG predicted pavement distresses and a site factor variable. As such, MEPDG predicted IRI is a composite measurement primarily influenced by the predicted values of the other pavement distresses. The equation used in the MEPDG to relate predicted IRI to the other predicted pavement distresses is a linear model (AASHTO 2008). As such, it was expected that a strong linear relationship would be observed between the MEPDG predicted flexible pavement distresses and the MEPDG predicted terminal IRI. MEPDG predicted IRI in asphalt overlays of flexible pavements was strongly correlated with total permanent deformation, asphalt layer permanent deformation, bottom-up fatigue cracking, and thermal cracking. The value of the slope coefficients for MEPDG predicted total permanent deformation and asphalt layer permanent deformation indicated that the influence of these pavement distresses on MEPDG predicted terminal IRI was much less for asphalt overlays of flexible pavement than for new flexible pavements. The strong correlation between MEPDG predicted terminal IRI and MEPDG predicted bottom-up fatigue cracking and thermal cracking was odd given that very little variability was observed in the predicted values for these pavement distresses.

As stated in Section 2.5.5, verification studies conducted by State DOTs have generally found the nationally calibrated MEPDG IRI model to be the most accurate of the MEPDG flexible pavement distress models. However, some verification studies have found a very poor correlation with local PMS pavement performance data. Therefore, the accuracy of the nationally-calibrated MEPDG IRI model should be verified using pavement performance data from the MTO PMS. Since the MEPDG IRI model is dependent on the output from the other flexible pavement distress models, this should not be completed until after work has been completed on the other models.

4.3.2 Comparison of MEPDG and AASHTO 1993 Asphalt Concrete Overlay Thickness of Existing Flexible Pavement

The purpose of this section was to compare the MEPDG and AASHTO 1993 pavement design methods for asphalt overlays of flexible pavements in terms of the asphalt concrete overlay thickness and total asphalt layer thickness required to satisfy their respective design criteria.

4.3.2.1 Results

Figure 4-42 and Figure 4-43 show the asphalt overlay thickness and total asphalt thickness, respectively, obtained for the asphalt overlays of flexible pavements designed using the MEPDG and AASHTO 1993 methods.

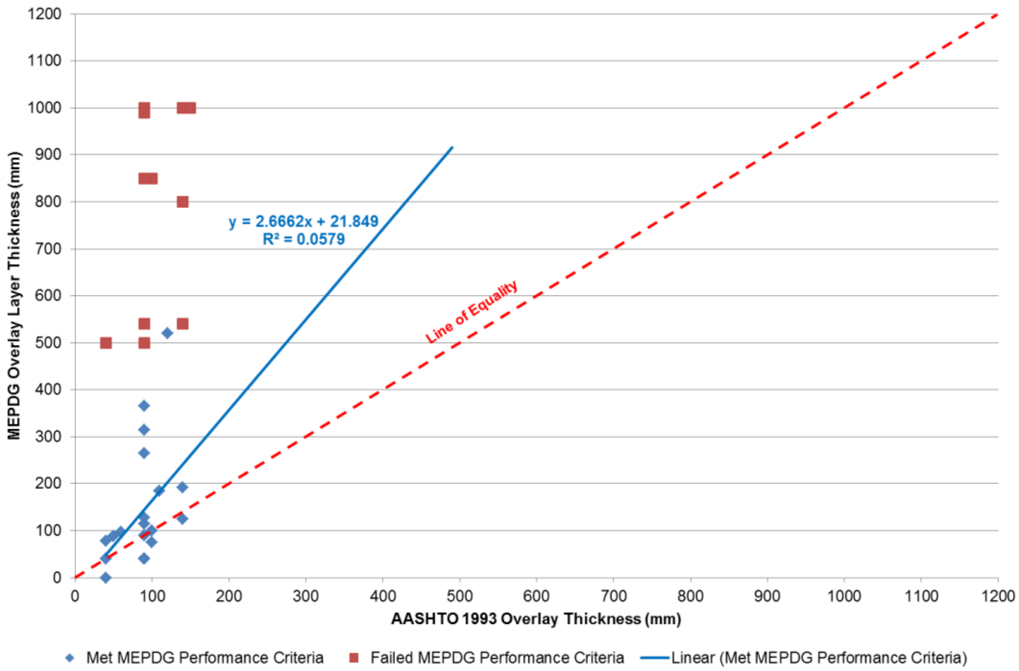


Figure 4-42: MEPDG versus AASHTO 1993 Asphalt Overlay Thickness - Asphalt Overlays of Flexible Pavement

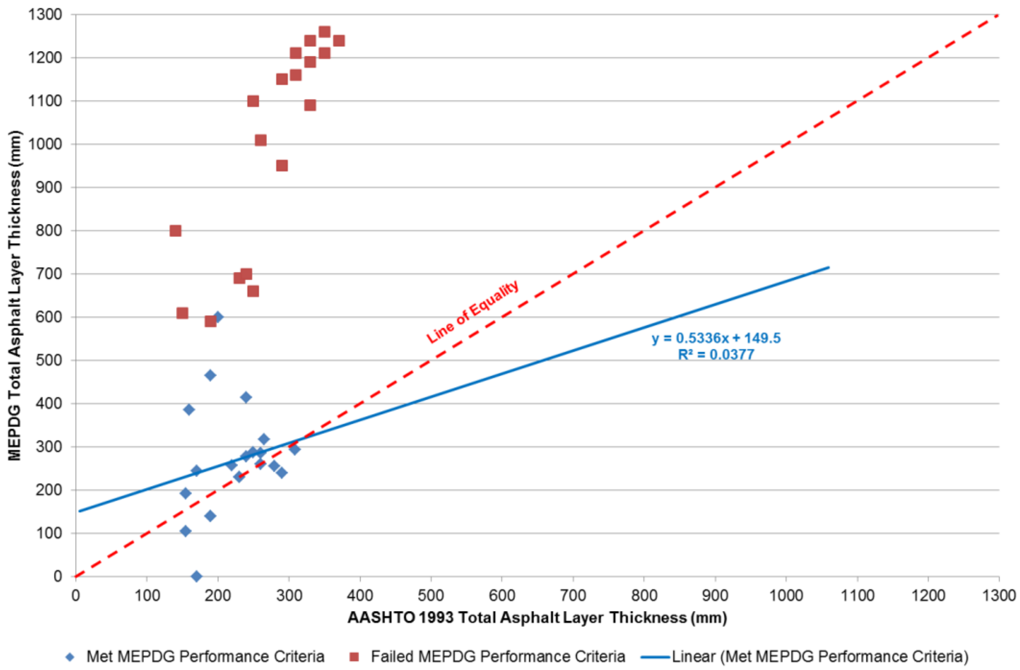


Figure 4-43: MEPDG versus AASHTO 1993 Total Asphalt Layer Thickness – Asphalt Overlays of Flexible Pavements

As shown in Figure 4-42 and Figure 4-43, the MEPDG generally resulted in much thicker asphalt overlay and total asphalt layer thicknesses than those obtained using the AASHTO 1993 method. However, eighteen of the thirty-nine asphalt overlays of flexible pavements were not able to meet the MEPDG performance criteria despite substantial increases in asphalt overlay thickness. The thickness of the asphalt overlays in the pavement sections unable to meet the MEPDG performance criteria was increased until the AASHTOWare Pavement ME software was not capable of analyzing the pavement structure. As described in Section 4.2.2, the AASHTOWare Pavement ME software is currently not capable of producing analysis output for thick pavement structures where the number of sub layers required to perform the analysis exceeds nineteen.

The asphalt overlays of flexible pavement that could not be re-designed to meet the MEPDG performance criteria failed based on MEPDG predicted terminal IRI and / or asphalt layer rutting. The remaining pavement distresses could be reduced to meet the MEPDG performance criteria by increasing the asphalt layer thickness. Of the eighteen asphalt overlays of flexible pavement that could not be re-designed to meet the MEPDG performance, fifteen failed based on terminal IRI alone, two failed based on both terminal IRI and asphalt layer rutting, and one failed based on asphalt layer rutting alone. The three asphalt overlays of flexible pavement that failed based on rutting in the asphalt layer were the only three pavement sections with MEPDG estimated traffic loading in excess of 15 million ESALs.

To test the sensitivity of the MEPDG asphalt overlay of flexible pavement designs to the specified MEPDG performance criteria, the new flexible pavements were re-designed using the MEPDG and the revised pavement performance criteria shown in Table 4-4. These were the same revised MEPDG performance criteria used in the sensitivity analysis for new flexible pavements completed in Section 4.2.2. The rationale for the selection of the revised MEPDG performance criteria is provided in that section.

Figure 4-44 and Figure 4-45 show the asphalt overlay thickness and total asphalt thickness, respectively, obtained using the revised MEPDG performance criteria plotted against those obtained using the AASHTO 1993 method.

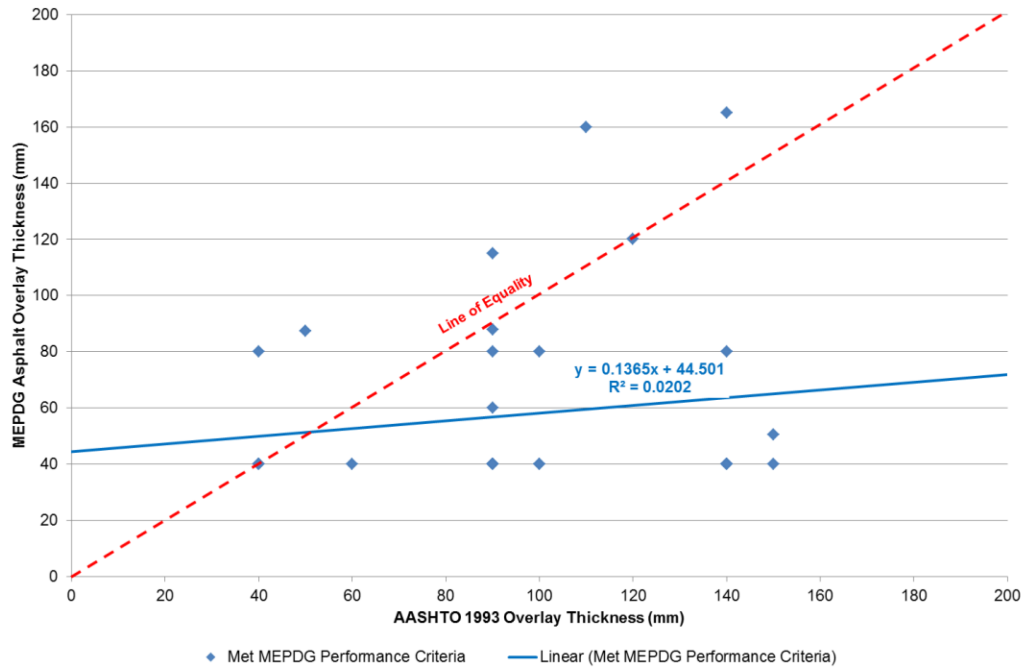


Figure 4-44: MEPDG versus AASHTO 1993 Asphalt Overlay Thickness – Asphalt Overlays of Flexible Pavement (Revised MEPDG Performance Criteria)

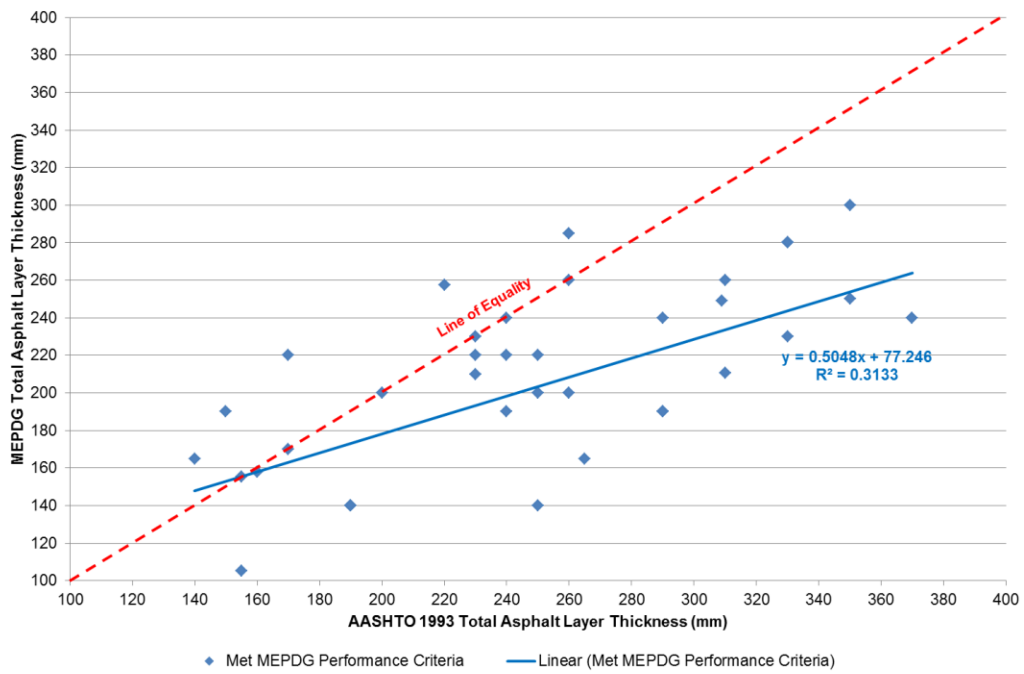


Figure 4-45: MEPDG versus AASHTO 1993 Total Asphalt Layer Thickness – Asphalt Overlays of Flexible Pavement (Revised MEPDG Performance Criteria)

As shown in Figure 4-44 and Figure 4-45, the asphalt overlay and total asphalt layer thicknesses obtained using the MEPDG revised performance criteria were generally less than those obtained using the AASHTO 1993 method. In addition, all of the asphalt overlays of flexible pavement were able to be re-designed to meet the revised MEPDG performance criteria.

4.3.2.2 Discussion

The results of this analysis show that the MEPDG, using the nationally-calibrated models and default performance criteria, resulted in significantly thicker asphalt overlay and total asphalt layers compared to the AASHTO 1993 method. In addition, no correlation was observed between asphalt layer thicknesses obtained using the two pavement design methods. Approximately 46% of the asphalt overlays of flexible pavements analyzed could not be re-designed to meet the MEPDG performance criteria, regardless of the thickness of the asphalt overlay.

As discussed in Section 4.2.2, these results generally correspond with the results of previous studies that have examined flexible pavements with high traffic loadings.

4.4 New Rigid Pavement

4.4.1 MEPDG Predicted Performance of AASHTO 1993 New Rigid Pavements

The purpose of Section 4.4.1 is to analyze the MEPDG predicted pavement performance of the new rigid pavements designed using the AASHTO 1993 method.

4.4.1.1 Results

MEPDG predicted transverse cracking in the new rigid pavements examined did not vary within each highway functional classification; the MEPDG predicted transverse cracking was always 4.92 % of total slabs for freeways (95% reliability) and 3.10% of total slabs for arterial highways (85% reliability), well below the MPEDG performance threshold of 10% of total slabs.

Figure 4-46 shows the MEPDG predicted mean joint faulting and terminal IRI for new rigid pavements design using the AASHTO 1993 method.

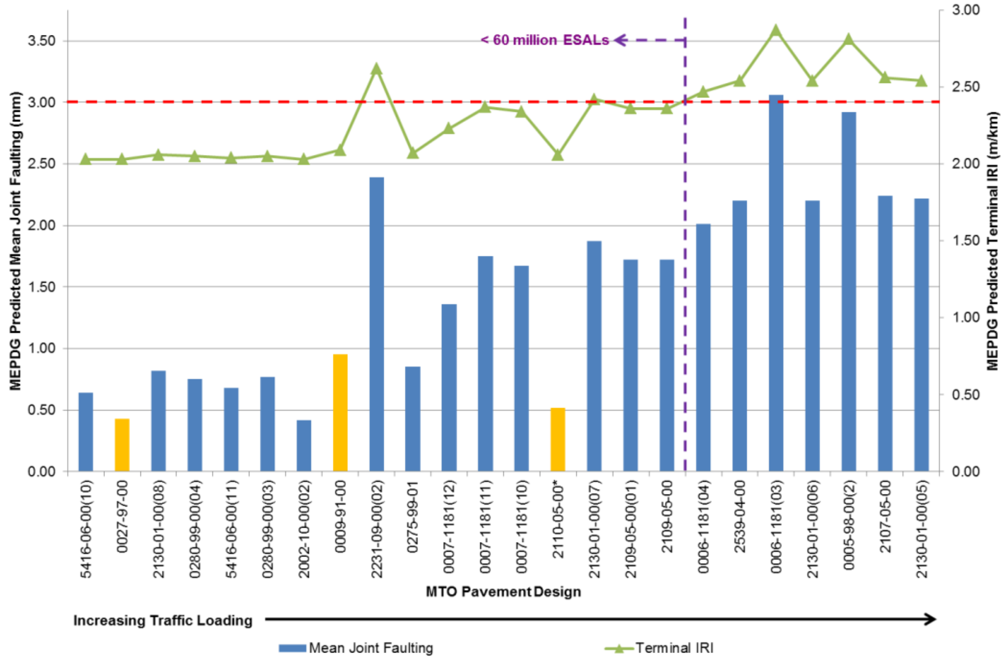


Figure 4-46: MEPDG Predicted Mean Joint Faulting and Terminal IRI in AASHTO 1993 New Rigid Pavements

As shown in Figure 4-46, only one of the twenty-four new rigid pavements designed using the AASHTO 1993 method failed to meet the MEPDG performance criteria for mean joint faulting. Figure 4-46 appears to show increasing MEPDG predicted mean joint faulting with increased traffic loading. Figure 4-47 shows MEPDG predicted mean joint faulting plotted against traffic loading, expressed in terms of MEPDG estimated ESALs, for new rigid pavements designed using the AASHTO 1993 method.

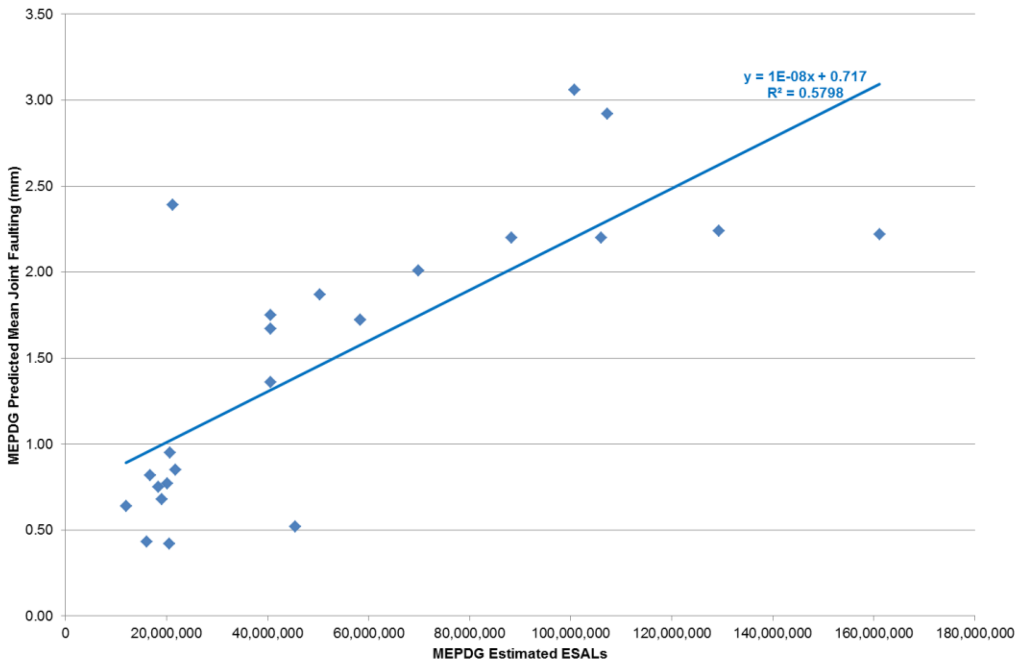


Figure 4-47: MEPDG Predicted Mean Joint Faulting versus MEPDG Estimated ESALs for AASHTO 1993 New Rigid Pavements

As shown in Figure 4-47, a strong correlation was observed between MEPDG predicted mean joint faulting and traffic loading expressed in terms of MEPDG estimated ESALs. MEPDG predicted mean joint faulting was also found to have a similarly strong positive correlation with PCC slab thickness.

As shown in Figure 4-46, nine of the twenty-four rigid pavements designed using the AASHTO 1993 method failed to meet the MEPDG performance criteria for terminal IRI. Figure 4-46 shows a trend of increasing MEPDG predicted terminal IRI with increasing traffic loading. None of the new rigid pavements that had MEPDG estimated traffic loading in excess of 60 million ESALs met the MEPDG performance criteria for terminal IRI. Figure 4-48 shows a plot of MEPDG predicted terminal IRI versus traffic loading in terms of MEPDG estimated ESALs for new rigid pavements designed using the AASHTO 1993 method.

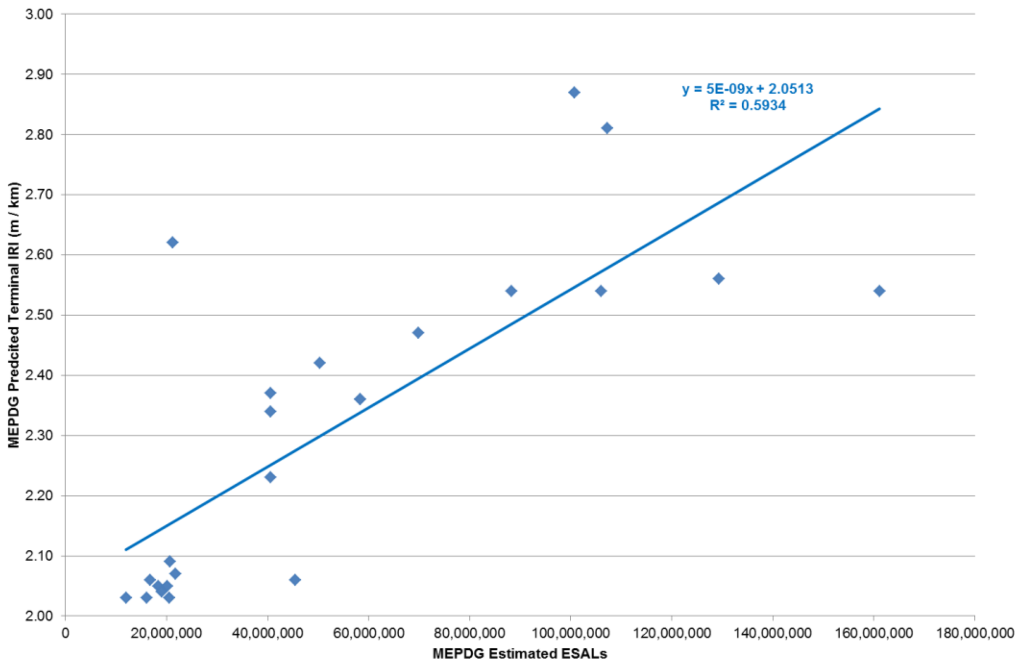


Figure 4-48: MEPDG Predicted Terminal IRI versus MEPDG Estimated ESALs for AASHTO 1993 New Rigid Pavements

As shown in Figure 4-48, a strong relationship was observed between MEPDG predicted terminal IRI and traffic loading. Figure 4-49 shows a plot of MEPDG predicted rigid pavement terminal IRI versus MEPDG predicted mean joint faulting.

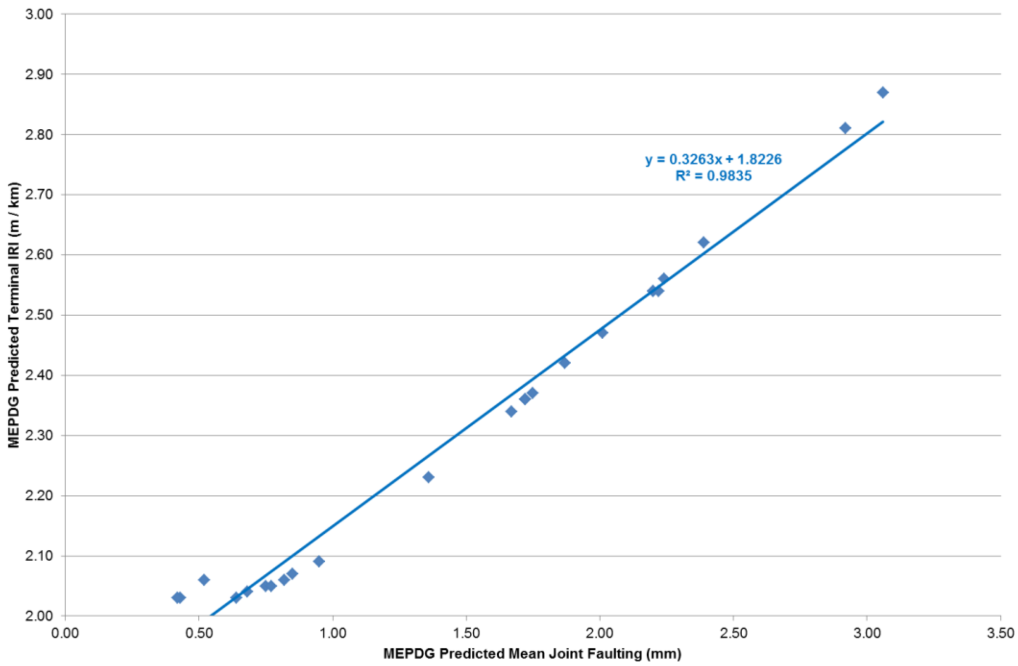


Figure 4-49: MEPDG Predicted Terminal IRI versus MEPDG Predicted Mean Joint Faulting for AASHTO 1993 New Rigid Pavements

As shown in Figure 4-49, approximately 98% of the variation in MEPDG predicted terminal IRI could be explained based on variation in MEPDG predicted mean joint faulting for the new rigid pavements examined in this study.

4.4.1.2 Discussion

Approximately 38% of the new rigid pavements designed using the AASHTO 1993 method did not meet the MEPDG performance criteria. Of the nine new rigid pavements that failed to meet the MEPDG performance criteria, nine failed based on MEPDG predicted terminal IRI and one failed based on mean joint faulting. The results of this analysis suggest that, in general, the AASHTO 1993 method under-predicts new rigid pavement performance relative to the MEPDG.

MEPDG predicted transverse cracking was found to not vary within highway functional classification for the new rigid pavements included in the analysis. Based on the results of MEPDG sensitivity studies cited in Section 2.4.7, rigid pavement transverse cracking has been found to be sensitive to a wide range of design input variables. Therefore, the lack of variability in MEPDG predicted transverse cracking observed in this study despite variable traffic, climate, and pavement structure inputs appears did not appear to be consistent with the results of MEPDG sensitivity studies.

This result may have been due to the significant impact of JPCP design and / or PCC material properties variables, which were held constant across the rigid pavement designs, on MEPDG predicted transverse cracking.

MEPDG predicted mean joint faulting was found to have a strong relationship with both traffic loading and PCC layer thickness. As stated in Section 2.4.6.1 and Section 2.4.6.3, some MEPDG sensitivity studies have also found MEPDG predicted mean joint faulting to be sensitive to these input variables. As stated in Section 2.4.6, the rigid pavement design input variables that have been consistently found to have the most significant effect on MEPDG predicted mean joint faulting are JPCP design variables and PCC material properties. Since these variables were held constant for all of the JPCP pavement designs examined in this study, it was not possible to examine their impact on MEPDG predicted mean joint faulting. As stated in Section 2.4.6.3, MEPDG sensitivity studies have found dowel diameter to be one of the most significant design inputs influencing MEPDG predicted mean joint faulting; in contrast, where PCC layer thickness has been found to have a significant effect on MEPDG predicted mean joint faulting, this has generally been attributed to its correlation with dowel diameter.

MEPDG predicted terminal IRI was found to be significantly influenced by traffic loading. This relationship was consistent with findings reported in MEPDG sensitivity studies (see Section 2.4.8.1), and was also expected given the strong relationship between MEPDG predicted mean joint faulting and traffic loading observed in the rigid pavements examined. Similar to flexible pavement IRI, rigid pavement IRI in the MEPDG is a composite measurement of the other rigid pavement distresses and a site factor. As such, it should be most significantly influenced by the pavement design input variables that have the most significant effect on the other rigid pavement distress models. In this analysis, since JPCP design variables and PCC material properties were constant across the pavement sections examined, the observed variation in rigid pavement distresses was primarily explained in terms of traffic loading. In the absence of any observed variation in MEPDG transverse cracking, variation in MEPDG predicted terminal IRI could be explained principally by the observed variation in MEPDG predicted mean joint faulting.

Several State DOTs have undertaken verification studies to assess the accuracy of the nationally-calibrated MEPDG rigid pavement distress models; these studies have typically been completed using local pavement performance data from the State PMS and / or the LTPP database. The verification studies have reported mixed results regarding the accuracy of the three MEPDG rigid pavement distress models. Therefore, it is recommended that the nationally-calibrated MEPDG rigid pavement distress models be verified using Ontario pavement performance data prior to implementation.

4.4.2 Comparison of MEPDG and AASHTO 1993 Thickness of New Rigid Pavements

The purpose of this section is to compare the MEPDG and AASHTO 1993 pavement design methods in terms of the Portland cement concrete layer thicknesses required to satisfy their respective design criteria.

4.4.2.1 Results

Figure 4-50 shows the total PCC layer thicknesses obtained for the new rigid pavements using the MEPDG and AASHTO 1993 design methods.

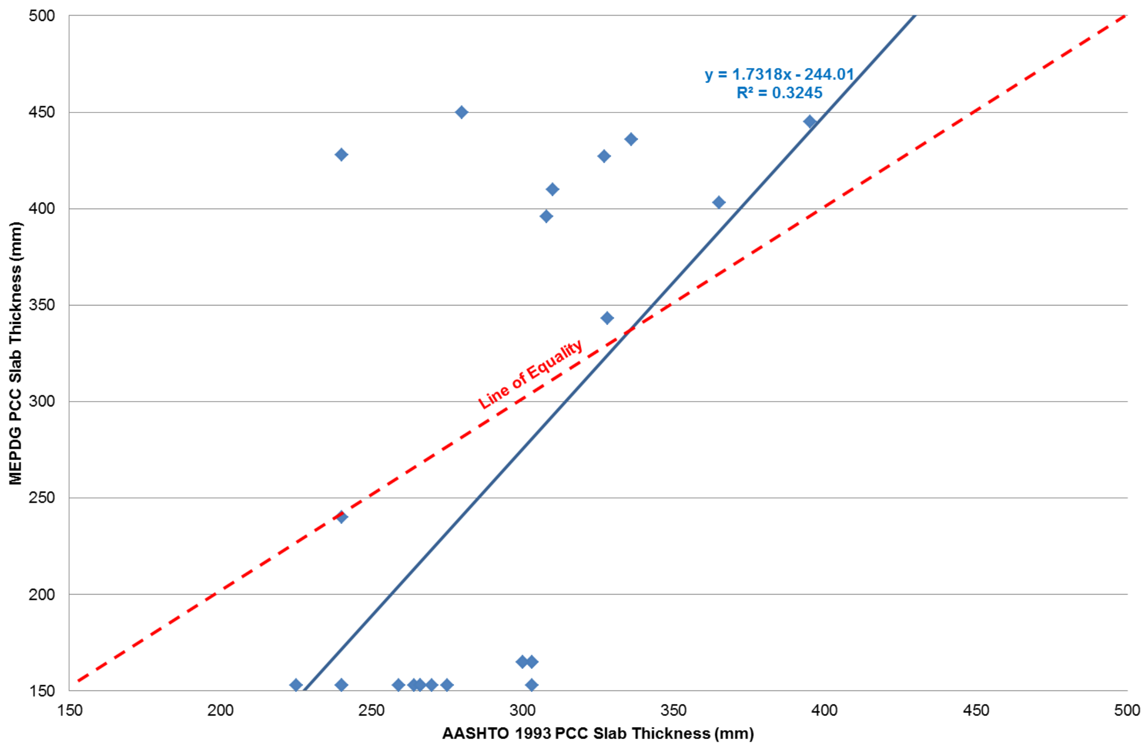


Figure 4-50: MEPDG versus AASHTO 1993 PCC Layer Thickness – New Rigid Pavements

As shown in Figure 4-50, the PCC layer thickness obtained using the MEPDG design method was thinner than the AASHTO 1993 method for fourteen of the twenty-four rigid pavements examined. The MEPDG resulted in a thicker PCC layer than the AASHTO 1993 method for nine of the new rigid pavements examined. One new rigid pavement had the same PCC layer thickness using both pavement design methods. Seven of the nine new rigid pavements designed using the MEPDG design method had the minimum PCC layer thickness that could be modelled by the AASHTOWare Pavement ME software (153 mm) and still met the MEPDG performance criteria.

Figure 4-51, Figure 4-52, and Figure 4-53 show the MEPDG predicted transverse cracking, mean joint faulting, and terminal IRI, respectively, for new rigid pavements designed using the MEPDG and AASHTO 1993 methods.

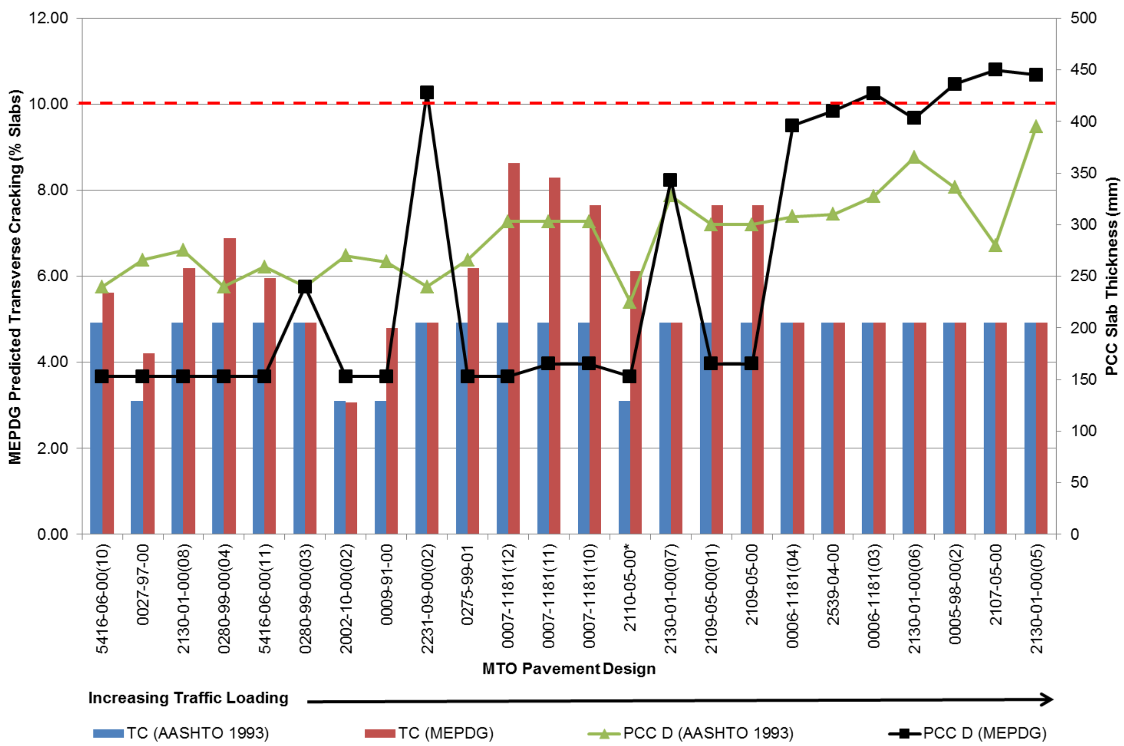


Figure 4-51: MEPDG Predicted Transverse Cracking and PCC Layer Thickness for New Rigid Pavements Designed Using the MEPDG and AASHTO 1993

As shown in Figure 4-51, a reduction in the thickness of the PCC layer for the MEPDG pavement designs resulted in an increase in MEPDG predicted transverse cracking in the new rigid pavements. Conversely, increasing the thickness of the PCC layer for the MEPDG pavement designs resulted in no change in MEPDG predicted transverse cracking.

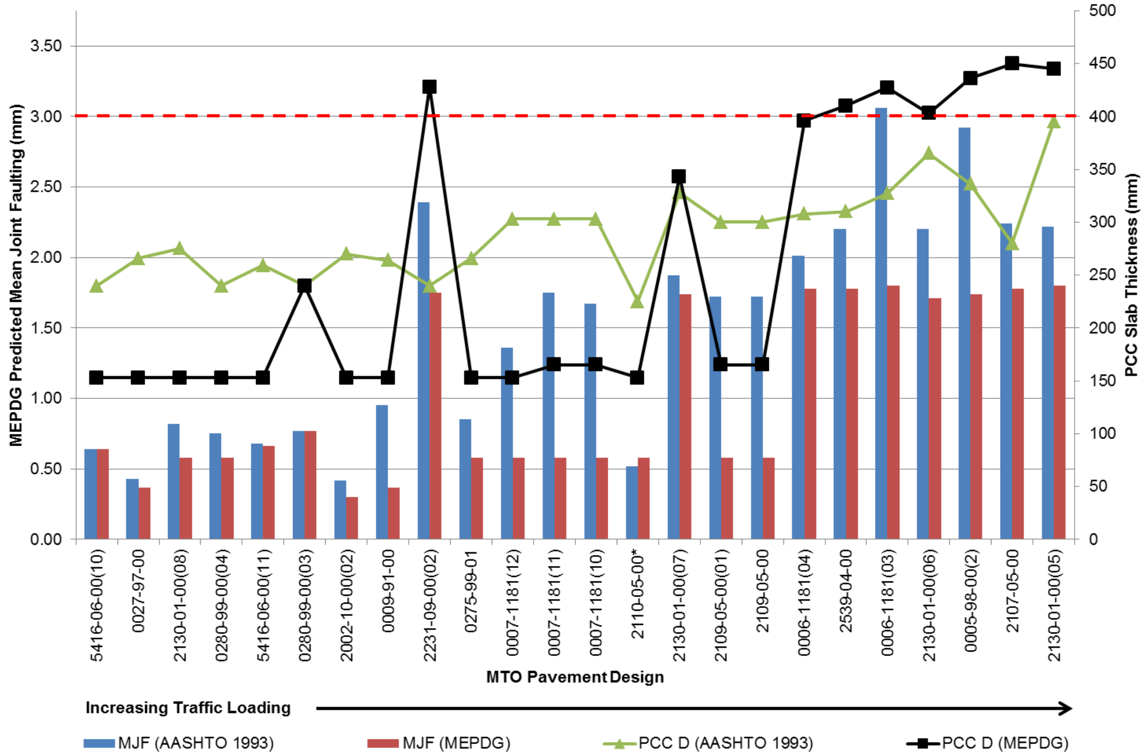


Figure 4-52: MEPDG Predicted Mean Joint Faulting and PCC Layer Thickness for New Rigid Pavements Designed Using the MEPDG and AASHTO 1993

As shown in Figure 4-52, MEPDG predicted mean joint faulting generally decreased for the MEPDG rigid pavement designs regardless of whether the thickness of the PCC layer was increased or decreased relative to the AASHTO 1993 design.

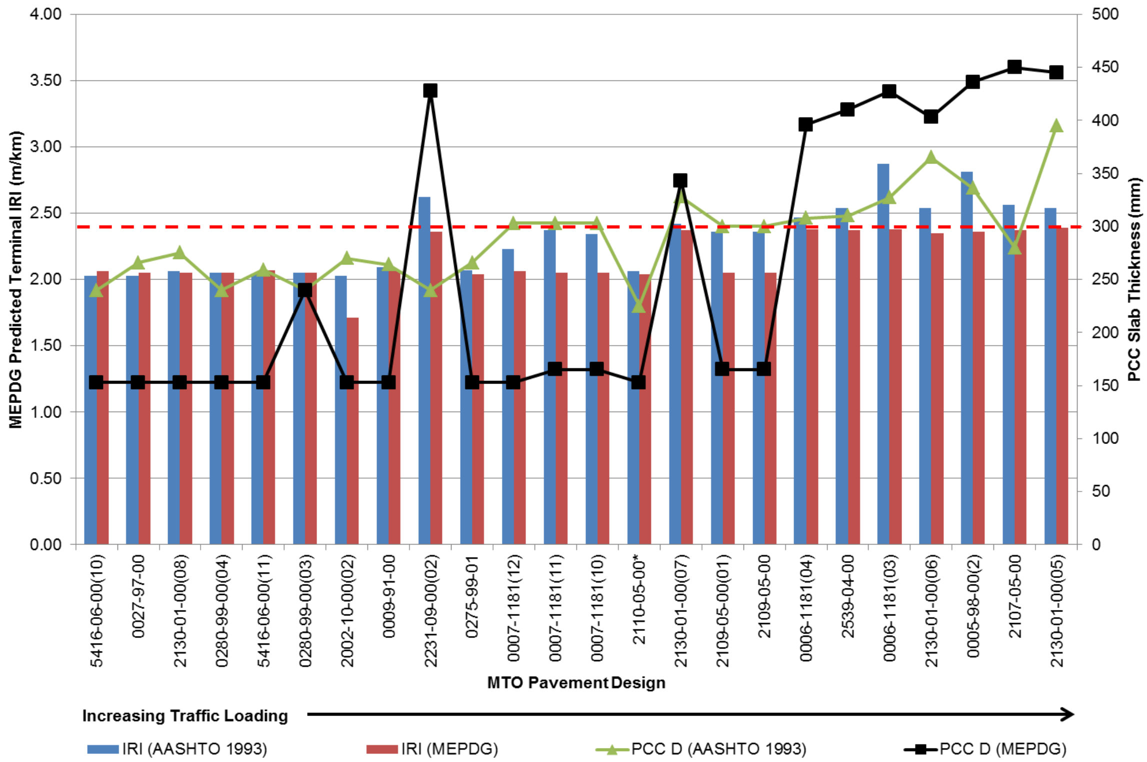


Figure 4-53: MEPDG Predicted Terminal IRI and PCC Layer Thickness for New Rigid Pavements Designed Using the MEPDG and AASHTO 1993

As shown in Figure 4-53, the MEPDG predicted terminal IRI generally decreased in the MEPDG rigid pavement designs when the thickness of the PCC layer was increased. However, a significant increase in PCC layer thickness was typically required to affect even a relatively moderate decreased in MEPDG predicted terminal IRI. Conversely, significant reductions in PCC layer thickness for the MEPDG rigid pavement designs resulted in either no change or a reduction in MEPDG predicted terminal IRI. Both of these results appear to be counter-intuitive, and were likely the result of the influence of MEPDG predicted mean joint faulting on MEPDG predicted terminal IRI.

4.4.2.2 Discussion

The results of this analysis show that the MEPDG, using the nationally-calibrated models and default performance criteria, resulted in a thicker PCC layer for 42% of the new rigid pavements compared to the AASHTO 1993 method. No significant correlation was observed between the PCC thicknesses obtained using the two pavement design methods.

As discussed in Section 2.3, a number of previous studies have compared the PCC layer thicknesses obtained for rigid pavement designs using the AASHTO 1993 and MEPDG design methods. These studies have generally found that the AASHTO 1993 method results in PCC layer thicknesses that are an average of 10% thicker that those obtained using the nationally-calibrated MEPDG. In addition, these studies generally found a good correlation between the PCC layer thicknesses obtained using both methods. However, the traffic loading examined in these studies was less than 10 million ESALs, which is very low relative to the typical traffic loading experienced on rigid pavements in Ontario. In contrast, the rigid pavement designs examined in this study experienced traffic loading ranging from approximately 12 million to 160 million ESALs as estimated by the MEPDG. Therefore, the results of this study suggest that the relationship observed between AASHTO 1993 and MEPDG rigid pavement PCC layer thickness may not be accurate beyond the relatively low traffic volumes observed in previous studies.

The results of the analysis show that reducing the thickness of the PCC layer resulted in an increase in MEPDG predicted transverse cracking, while increasing the thickness of the PCC slab resulted in no change to MEPDG predicted transverse cracking. In addition, MEPDG predicted transverse cracking in the AASHTO 1993 rigid pavement designs was found to be constant for pavements within the same highway functional classification. These results suggest that the MEPDG predicted transverse cracking in the new rigid pavements designed using the AASHTO 1993 method may have been a minimum threshold value predicted at the specified reliability level. This would explain why increasing the PCC layer thickness did not result in lower MEPDG predicted transverse cracking, but decreasing the PCC layer thickness did result in increasing MEPDG transverse cracking.

MEPDG predicted mean joint faulting generally decreased for the MEPDG rigid pavement designs regardless of whether the thickness of the PCC layer was increased or decreased relative to the AASHTO 1993 design. This result appears to be counter-intuitive. A reduction in mean joint faulting would be expected to result from an increase in PCC layer thickness, but would not be expected for a decreased in PCC layer thickness. This result was likely due to the use of a single dowel diameter (32 mm) in the MEPDG rigid pavement design analysis. As the PCC layer thickness decreased, the ratio of dowel diameter to PCC layer thickness increased, potentially resulting in greater stability at the slab joints. In reality, the dowel diameter would have to be decreased proportionately with PCC layer thickness beyond a certain point to ensure constructability and provide adequate concrete cover. For example, the AASHTO 1993 Guide recommends a ratio of dowel diameter to PCC layer thickness of

1:8, much less than the 1:5 ratio observed in several of the MEPDG new rigid pavement designs (AASHTO 1993). Given the significant influence of dowel diameter on MEPDG predicted mean joint faulting reported in the literature, it is recommended that future studies examine whether limiting the ratio of dowel diameter to PCC layer thickness produces different MEPDG PCC layer thickness designs for Ontario conditions.

4.5 Asphalt Concrete Overlay of Existing Concrete Pavement

4.5.1 MEPDG Predicted Performance of AASHTO 1993 Asphalt Concrete Overlay of Existing Rigid Pavement

The purpose of this section is to determine whether the AASHTO 1993 and MEPDG pavement design methodologies predict performance in a consistent manner for asphalt overlays of rigid pavements across a range of design conditions typical of Ontario.

4.5.1.1 Results

Figure 4-54 shows the MEPDG predicted top-down fatigue cracking, bottom-up fatigue cracking, and PCC transverse cracking in asphalt overlays of rigid pavement designed using AASHTO 1993.

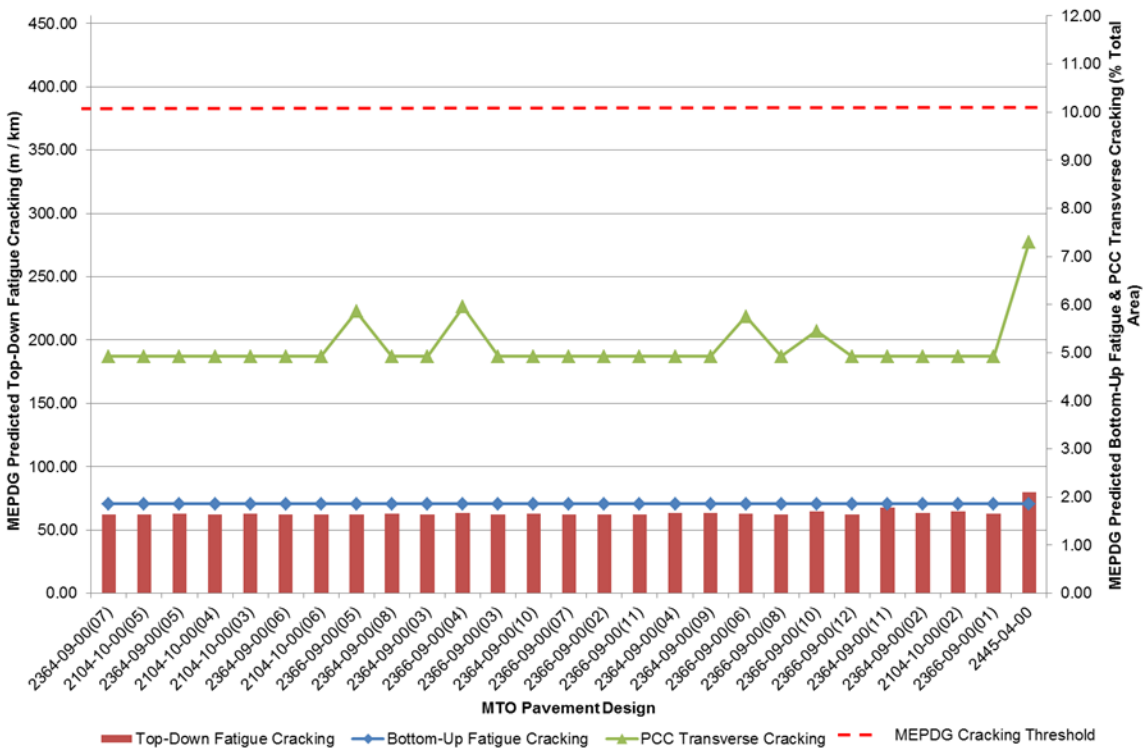


Figure 4-54: MEPDG Predicted Fatigue Cracking and Transverse Cracking in AASHTO 1993 Asphalt Overlays of Rigid Pavement

As shown in Figure 4-54, the MEPDG predicted top-down fatigue cracking, bottom-up fatigue cracking, and PCC transverse cracking did not vary significantly among the asphalt overlays of rigid pavement examined in this study. In addition, none of these predicted pavement distresses approached the MEPDG performance criteria threshold. Similar trends in MEPDG predicted pavement performance were observed for asphalt total cracking and asphalt thermal cracking. Asphalt total cracking ranged from 3.96% to 6.80% of total pavement surface area, consistently below the 10% performance criteria threshold. Similarly, MEPDG predicted asphalt thermal cracking ranged from 46.06 m / km to 46.18 m / km, consistently below the 190 m / km performance threshold. Figure 4-55 shows the MEPDG predicted asphalt layer permanent deformation and terminal IRI in asphalt overlays of rigid pavement.

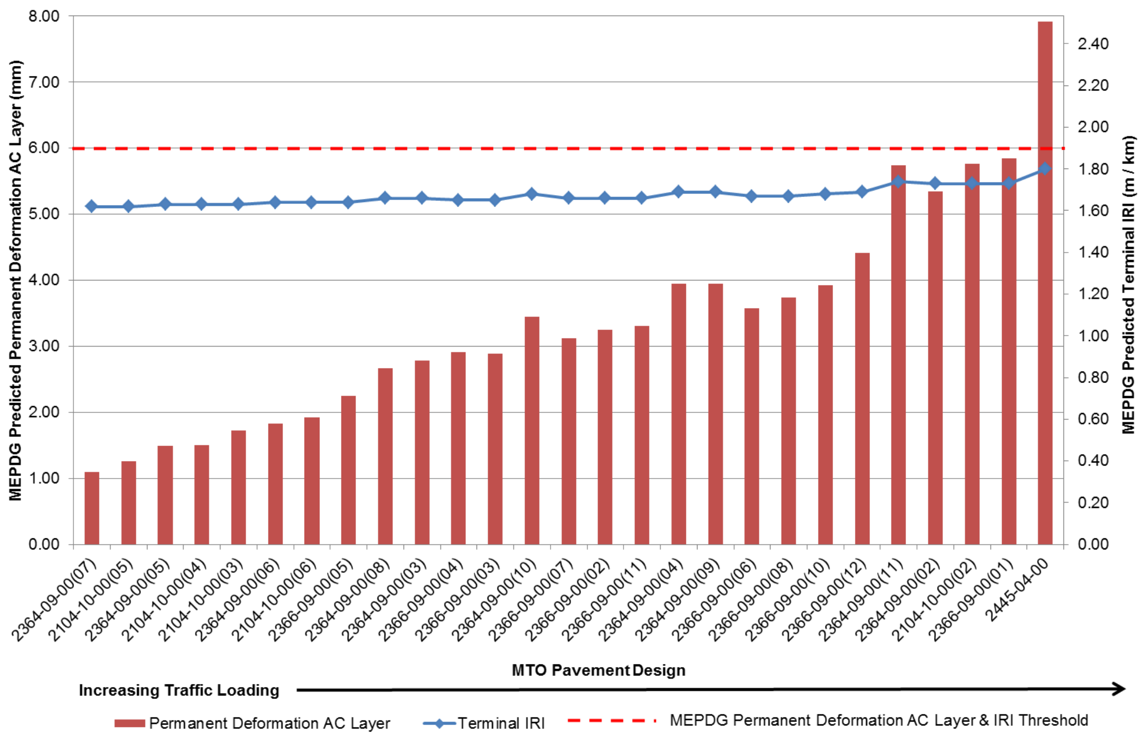


Figure 4-55: MEPDG Predicted Asphalt Layer Permanent Deformation and Terminal IRI for AASHTO 1993 Asphalt Overlays of Rigid Pavement

As shown in Figure 4-55, only one of the twenty-seven asphalt overlay of rigid pavement designs examined in this study failed to meet the MEPDG performance criteria for permanent deformation. It must be noted that for asphalt overlays of rigid pavement, no permanent deformation was observed in the unbound pavement layers; hence, MEPG predicted permanent deformation in the asphalt layers was equal to MEPDG predicted permanent deformation in the total pavement structure. A clear trend of increasing MEPDG predicted permanent deformation with increased traffic loading was observed. Figure 4-56 shows MEPDG predicted permanent deformation plotted against traffic loading, expressed in terms of MEPDG estimated ESALs.

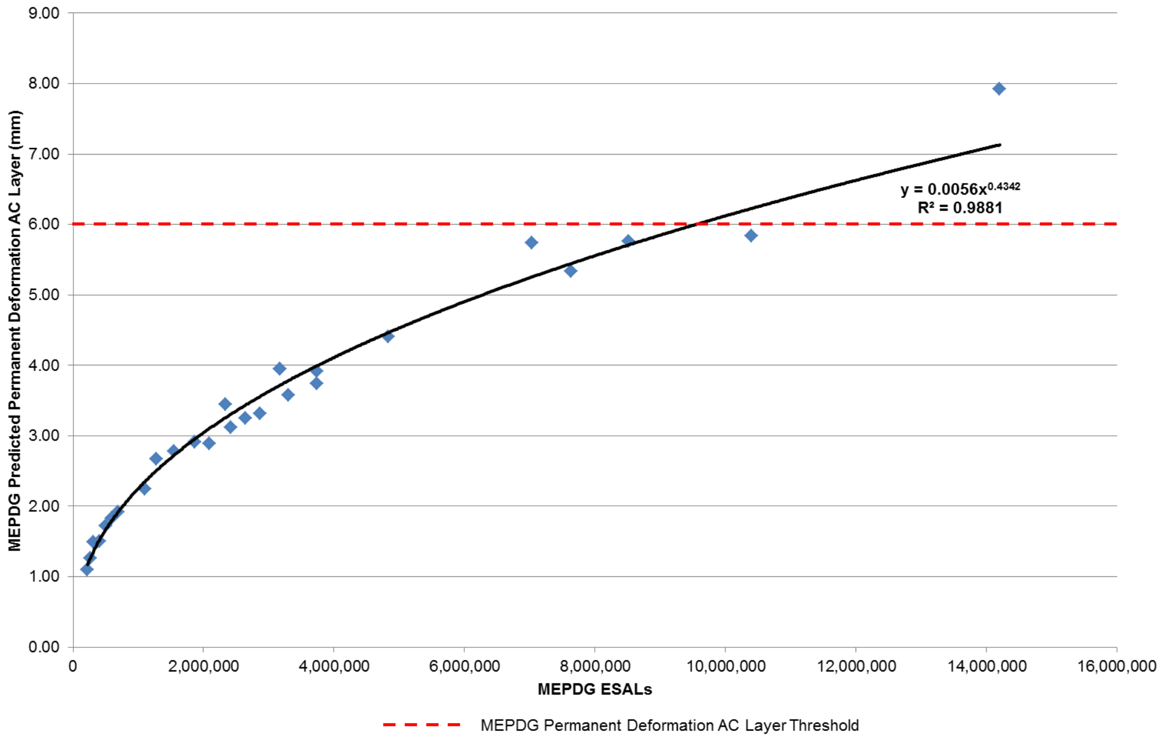


Figure 4-56: MEPDG Predicted Permanent Deformation versus MEPDG Estimated ESALs for Asphalt Overlays of Rigid Pavement

As shown in Figure 4-56, MEPDG predicted permanent deformation had a very strong relationship to traffic loading. The form and coefficient of the power relationship between traffic loading and MEPDG predicted permanent deformation closely mirrors the equation used to relate these two parameters in the MEPDG asphalt layer permanent deformation model (AASHTO 2008). In addition, the linear regression parameters show that almost all of observed variation in MEPDG predicted permanent deformation was accounted for by variation in traffic loading using the observed relationship.

Figure 4-55 also shows that the MEPDG predicted terminal IRI did not vary significantly across the asphalt overlay of rigid pavement designs examined; however, there did appear to be a trend of moderately increasing MEPDG predicted terminal IRI with increasing traffic loading. Figure 4-57 shows MEPDG predicted terminal IRI plotted against traffic loading expressed in terms of MEPDG estimated ESALs.

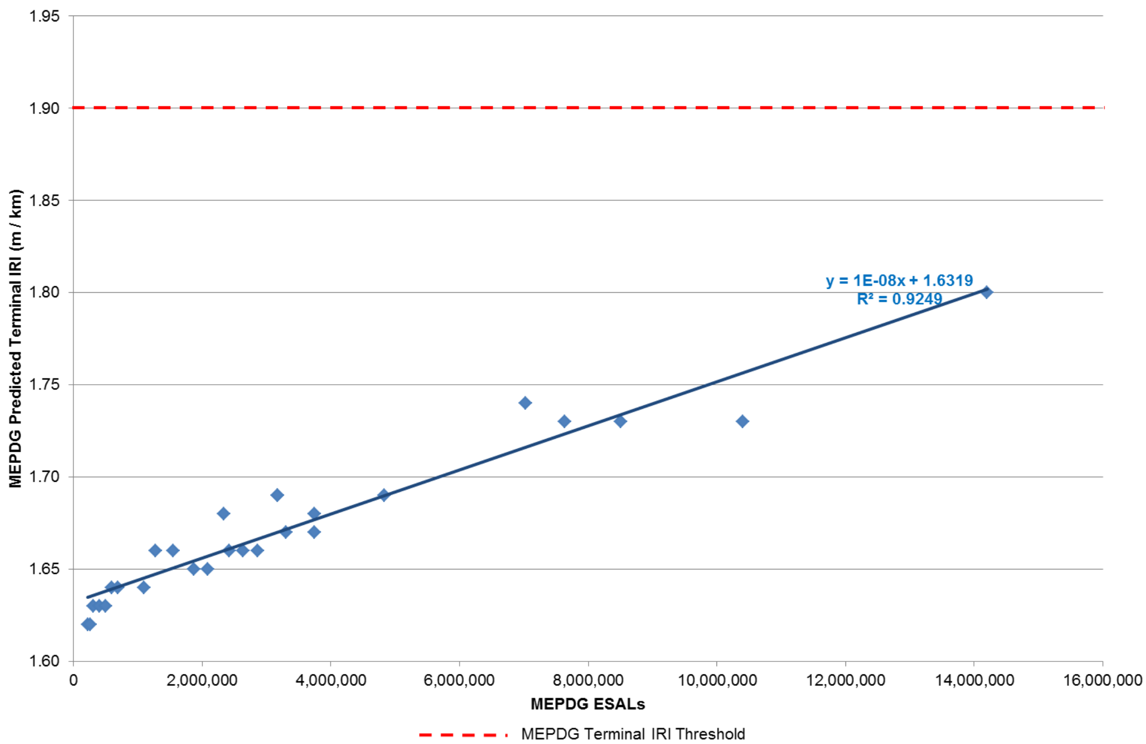


Figure 4-57: MEPDG Predicted Terminal IRI versus MEPDG Estimated ESALs for Asphalt Overlays of Rigid Pavement

As shown in Figure 4-57, a strong relationship was also observed between MEPDG predicted terminal IRI and traffic loading.

4.5.1.2 Discussion

To accurately interpret the MEPDG predicted performance of the asphalt overlays of rigid pavement, it important to note that many of the asphalt overlays of rigid pavements examined in this study actually exceeded the structural number required by the AASHTO 1993 design method. Most of the asphalt overlays of rigid pavements designed using the AASHTO 1993 method were actually rehabilitation treatments on existing composite pavements that involved milling off the existing asphalt layers, repairing of deteriorated underlying concrete pavement, and paving the asphalt overlays. As such, the asphalt overlays were typically required for the functional adequacy and not the structural adequacy of the pavement. In addition, the MTO typically requires that asphalt overlays of concrete pavement consist of one 50 mm asphalt binder lift and one 40 mm asphalt surface lift; this often exceeded the asphalt layer thickness required to achieve the AASHTO 1993 structural number, which was sometimes an impractically low number (i.e. 28mm). As a result, many of the asphalt overlays of rigid pavement designs do not represent a true comparison of the AASHTO 1993 and MEPDG design methods.

In addition, the MEPDG requires the pavement designer to input the percentage of concrete pavement slabs distressed / replaced before restoration and the percentage of concrete pavement slabs repaired / replaced after restoration. For the purposes of this analysis, it was assumed that all concrete pavement slabs distressed prior to restoration were repaired after restoration.

The asphalt overlays of rigid pavements generally met the MEPDG performance criteria. The MEPDG predicted cracking in the asphalt and concrete pavement layers was generally very low and did not vary significantly across the asphalt overlays of rigid pavements examined in this study. With the exception of one pavement design, the asphalt overlays of rigid pavements also met the performance criteria for MEPDG permanent deformation and terminal IRI. Traffic loading was the principal pavement design input influencing MEPDG predicted permanent deformation in asphalt overlays of rigid pavement. The strong relationship observed between MEPDG predicted terminal IRI and traffic loading was likely due to the strong relationship observed between MEPDG predicted permanent deformation and traffic loading. Based on the relationship observed between traffic loading and MEPDG predicted permanent deformation in the asphalt layer, no asphalt overlay of existing rigid pavement with traffic loading in excess of 10 million ESALs would be able to meet the MEPDG performance criteria for asphalt layer rutting. Since this study contained only one asphalt overlay of rigid pavement design with traffic loading in excess of 10 million ESALs, this result would need to be confirmed with additional pavement designs in future research.

No comparative analysis was completed to examine the thickness of asphalt overlays of rigid pavements for several reasons. First, since the historical MTO designs reflected the functional and not the structural requirements of the pavements, they were not truly representative of the thicknesses recommended by the AASHTO 1993 method. Second, since all of the asphalt overlays of exiting rigid pavements met the MEPDG performance criteria, and the thicknesses of the asphalt overlays could not be reasonably reduced, it was not possible to obtain alternative MEPDG designs for comparison. It is recommended that future studies conduct this comparative analysis for asphalt overlays of rigid pavements designed with thicker asphalt overlays and higher traffic loading.

Chapter 5
Conclusions and Recommendations

5.1 Conclusions

The purpose of this research was to conduct a comparative analysis of Ontario structural pavement designs using the AASHTO 1993 Guide for Design of Pavement Structures and the Mechanistic-Empirical Pavement Design Guide. Historical flexible, rigid, and asphalt overlay pavement designs completed using the AASHTO 1993 method were evaluated using a two-stage procedure. First, the nationally-calibrated MEPDG pavement distress models were used to predict the performance of the pavements designed using the AASHTO 1993 method. The purpose of this stage of the analysis was to determine whether the two methods predicted pavement performance in a consistent manner across a range of design conditions typical of Ontario. Finally, the AASHTO 1993 and MEPDG methods were compared based on the thickness of the asphalt concrete or Portland cement concrete layers required to satisfy their respective design criteria.

The MEPDG was found to over-predict pavement distresses in new flexible pavement relative to the AASHTO 1993 pavement design method. Approximately 98% of the new flexible pavement structures designed using the AASHTO 1993 method failed to meet the MEPDG performance criteria. The primary modes of failure predicted by the MEPDG for these new flexible pavements were pavement permanent deformation and roughness. Traffic loading was the most significant pavement design input influencing the MEPDG predicted failure of the new flexible pavements. The asphalt layer thicknesses produced using the MEPDG method were consistently higher than the asphalt layer thicknesses produced using the AASHTO 1993 method, and a very poor correlation was observed between the two methods. Sixty percent (60%) of the new flexible pavements designed using the AASHTO 1993 method could not be re-designed to meet the MEPDG performance criteria by increasing the asphalt layer thickness.

The MEPDG was found to over-predict pavement distresses in asphalt overlays of flexible pavements relative to the AASHTO 1993 pavement design method. Approximately 80% of the asphalt overlays of flexible pavements designed using the AASHTO 1993 method failed to meet the MEPDG performance criteria. The primary modes of failure predicted by the MEPDG for these asphalt overlays of flexible pavements were asphalt layer permanent deformation and roughness. Existing pavement condition rating and traffic loading were the most significant pavement design inputs influencing the MEPDG predicted failure of the new flexible pavements. The asphalt overlay and total asphalt layer thicknesses produced using the MEPDG method were consistently higher than those produced using the AASHTO 1993 method, and a very poor correlation was observed between the two methods. Forty-six percent (46%) of the new flexible pavements designed using the AASHTO 1993 method could not be re-designed to meet the MEPDG performance criteria through increasing the asphalt overlay thickness.

The results of the comparative analysis showed that the AASHTO 1993 method generally under-predicted rigid pavement performance relative to the MEPDG, although the results varied widely between alternative rigid pavement designs. The AASHTO 1993 rigid pavement designs that the MEPDG predicted would not meet the rigid pavement performance criteria generally failed due to pavement roughness. A very poor correlation was observed between the Portland cement concrete layer thicknesses obtained using the MEPDG and AASHTO 1993 design methods. The MEPDG predicted thinner Portland cement concrete layer thicknesses than the AASHTO 1993 design method for most of the rigid pavement designs.

5.2 Recommendations

The purpose of this research was to conduct a comparative analysis of Ontario structural pavement designs using the AASHTO 1993 Guide for Design of Pavement Structures and the Mechanistic-Empirical Pavement Design Guide. This was undertaken as a preliminary step in the MEPDG implementation process in Ontario. The following are recommendations for future research to adapt and validate the MEPDG for use in Ontario:

1. The accuracy of the nationally-calibrated MEPDG pavement distress models in Ontario conditions should be verified using measured pavement performance data from the MTOs PMS. The results of this analysis should be used to determine which MEPDG pavement distress models should be recalibrated and validated for Ontario conditions.

2. AASHTO has recognized that the MEPDG currently underestimates the structural contribution of high-quality granular materials in the pavement structure. This issue should be resolved prior to recalibrating and validating the MEPDG permanent deformation models for Ontario conditions. AASHTO has advised that there are currently several projects underway to resolve this issue. Given that Ontario pavement designs typically include thick layers of high quality granular materials, it would be prudent to wait until this issue has been

resolved prior to recalibrating and validating the MEPDG permanent deformation models for Ontario conditions. Otherwise, the MEPDG permanent deformation models may have to be recalibrated and validated again subsequent to their modification.

3. The MEPDG permanent deformation model needs to be assessed to determine whether the MEPDG predicted permanent deformation is reasonable at the high traffic loading typically experienced on Ontario freeways.
4. The MEPDG currently predicts unreasonably high permanent deformation during the first year of the pavement design life, particularly for new flexible pavements. This issue should be investigated and resolved in future research. The potential interim solution of subtracting the permanent deformation accumulated in the first year could be investigated to see if more reasonable results can be achieved.
5. The rigid pavement MEPDG analysis should be repeated to assess the impact of using a proportional relationship between dowel diameter and PCC slab thickness.
6. The MEPDG pavement distress models should be recalibrated and validated using Ontario pavement performance data from the MTO PMS.

Appendix A

Default Ontario Axle Load Distributions

Table A-1: Southern Ontario Single Axle Load Distribution Table

Axle Weight (kg)		Axle Load Distribution as Percentage per Truck Class									
Min	**Max**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
0	1,360	1.80	0.07	0.19	0.28	0.42	0.04	0.39	0.10	0.02	0.44
1,361	1,814	0.96	0.33	0.14	0.08	0.42	0.10	0.17	0.09	1.10	0.63
1,815	2,267	2.91	5.40	0.89	0.45	2.13	0.62	0.44	0.57	0.02	0.85
2,268	2,721	3.99	7.52	0.73	0.70	2.43	0.43	0.89	1.69	3.22	1.21
2,722	3,175	6.80	6.65	0.95	0.87	3.55	0.44	0.93	6.75	8.16	1.14
3,176	3,628	12.00	11.32	2.12	0.96	7.82	0.62	1.44	5.58	8.73	1.02
3,629	4,082	11.70	13.98	4.73	1.51	7.20	1.22	1.48	4.29	8.70	0.99
4,083	4,535	11.40	13.94	13.96	3.14	19.16	10.40	4.39	11.03	14.49	4.93
4,536	4,989	10.30	10.71	18.40	5.10	13.03	22.56	12.86	14.92	15.75	12.59
4,990	5,443	9.00	10.46	24.84	8.07	11.20	40.89	28.90	11.09	15.01	33.61
5,444	5,896	7.40	5.04	10.66	3.70	3.96	14.54	15.17	7.09	6.42	17.86
5,897	6,350	5.70	4.36	8.60	9.64	6.09	3.05	6.91	10.44	5.54	8.99
6,351	6,803	4.30	2.28	4.54	11.08	5.70	1.04	3.37	7.90	4.18	3.33
6,804	7,257	3.20	1.95	3.67	13.64	3.76	0.92	3.46	6.14	2.13	2.35
7,258	7,711	2.58	1.65	1.45	11.34	2.12	0.90	3.14	3.66	1.42	1.29
7,712	8,164	1.80	1.25	1.54	6.99	3.03	0.83	3.46	2.95	1.03	1.58
8,165	8,618	1.40	0.80	1.37	5.97	1.45	0.49	2.87	1.75	0.32	1.08
8,619	9,071	1.00	0.73	0.42	3.87	1.57	0.28	3.12	0.87	0.83	2.32
9,072	9,525	0.75	0.50	0.36	5.90	1.41	0.16	1.96	0.66	0.00	0.72
9,526	9,979	0.50	0.51	0.23	2.27	0.95	0.13	1.55	0.38	0.10	0.98
9,980	10,432	0.25	0.27	0.04	1.73	0.59	0.11	1.15	0.14	0.08	0.49
10,433	10,886	0.15	0.08	0.04	0.23	0.26	0.06	0.38	0.43	0.11	0.21
10,887	11,339	0.10	0.06	0.02	0.25	0.18	0.03	0.35	0.19	0.19	0.18
11,340	11,793	0.00	0.07	0.04	0.47	0.31	0.03	0.23	0.00	0.71	0.08
11,794	12,246	0.00	0.02	0.00	0.04	0.12	0.01	0.11	0.75	1.27	0.17
12,247	12,700	0.00	0.01	0.00	0.18	0.11	0.01	0.10	0.00	0.00	0.06
12,701	13,154	0.00	0.01	0.00	0.11	0.06	0.01	0.13	0.18	0.24	0.18
13,155	13,607	0.00	0.01	0.00	0.00	0.32	0.00	0.10	0.07	0.00	0.00
13,608	14,061	0.00	0.01	0.05	0.06	0.11	0.01	0.05	0.18	0.00	0.09
14,062	14,515	0.00	0.01	0.00	0.22	0.12	0.01	0.13	0.00	0.00	0.24
14,516	14,968	0.00	0.00	0.00	0.13	0.05	0.01	0.10	0.04	0.00	0.10
14,969	15,422	0.00	0.00	0.00	0.02	0.14	0.01	0.04	0.03	0.00	0.00
15,423	15,875	0.00	0.00	0.00	0.23	0.13	0.01	0.07	0.00	0.00	0.10
15,876	16,329	0.00	0.00	0.00	0.09	0.08	0.02	0.04	0.04	0.00	0.00
16,330	16,782	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.12
16,783	17,236	0.00	0.00	0.00	0.37	0.02	0.00	0.03	0.00	0.00	0.01
17,237	17,690	0.00	0.00	0.02	0.06	0.00	0.00	0.01	0.00	0.00	0.04
17,691	18,143	0.01	0.00	0.00	0.16	0.00	0.01	0.04	0.00	0.23	0.00
18,144	20,412	0.00	0.00	0.00	0.09	0.00	0.00	0.03	0.00	0.00	0.02
Total		**100**	**100**	**100**	**100**	**100**	**100**	**100**	**100**	**100**	**100**

Table A-2: Southern Ontario Tandem Axle Load Distribution Table

Axle Weight (kg)		Axle Load Distribution as Percentage per Truck Class									
Min	**Max**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
0	2,721	5.28	0.00	1.47	0.73	4.02	0.24	0.35	0.00	0.24	0.54
2,722	3,628	10.00	0.00	4.13	0.75	3.89	0.52	0.87	7.65	1.17	3.19
3,629	4,535	11.90	0.00	23.50	1.24	3.99	2.43	1.46	10.35	2.59	6.79
4,536	5,442	9.63	0.00	5.98	2.44	16.68	7.60	2.61	11.54	9.53	5.34
5,443	6,350	8.00	0.00	7.90	4.83	16.58	8.85	6.73	6.55	10.47	7.17
6,351	7,257	7.80	0.00	8.95	13.24	16.90	7.84	9.25	5.05	9.39	4.82
7,258	8,164	6.80	0.00	8.92	12.21	10.77	7.95	7.71	9.90	13.51	3.36
8,165	9,071	6.15	0.00	8.53	9.02	10.58	8.24	5.65	9.52	11.91	2.92
9,072	9,979	5.80	0.00	5.77	4.01	6.35	7.45	4.62	13.19	13.83	2.51
9,980	10,885	5.30	0.00	5.74	7.10	3.29	6.63	3.67	8.52	6.91	2.11
10,886	11,793	4.70	0.00	4.03	6.90	1.63	5.87	3.41	0.00	4.29	2.30
11,794	12,700	4.10	0.00	2.99	3.49	1.48	5.60	3.99	4.20	6.09	3.06
12,701	13,607	3.33	0.00	2.95	2.48	1.17	5.79	5.04	4.57	2.19	2.97
13,608	14,514	3.91	0.00	1.76	2.11	0.60	7.31	5.70	1.76	1.72	4.46
14,515	15,422	2.22	0.00	1.65	3.53	0.66	8.91	7.03	1.58	1.33	6.63
15,423	16,329	1.84	0.00	1.98	1.82	0.89	5.61	8.50	3.49	1.02	10.12
16,330	17,236	1.44	0.00	0.54	2.12	0.35	1.71	7.60	0.00	0.38	10.96
17,237	18,143	0.90	0.00	0.77	5.29	0.10	0.77	6.04	0.00	1.33	9.82
18,144	19,051	0.50	0.00	0.51	4.89	0.00	0.31	4.56	1.44	1.63	5.24
19,052	19,957	0.30	0.00	0.52	3.64	0.07	0.15	2.11	0.00	0.43	1.87
19,958	20,865	0.10	0.00	0.52	3.53	0.00	0.09	1.12	0.69	0.00	1.35
20,866	21,772	0.00	0.00	0.42	1.47	0.00	0.05	0.73	0.00	0.00	0.61
21,773	22,679	0.00	0.00	0.27	1.44	0.00	0.04	0.30	0.00	0.00	0.43
22,680	23,587	0.00	0.00	0.09	0.34	0.00	0.01	0.21	0.00	0.00	0.41
23,588	24,493	0.00	0.00	0.01	0.12	0.00	0.01	0.11	0.00	0.00	0.43
24,494	25,401	0.00	0.00	0.00	0.37	0.00	0.01	0.20	0.00	0.00	0.29
25,402	26,308	0.00	0.00	0.03	0.27	0.00	0.01	0.14	0.00	0.00	0.04
26,309	27,215	0.00	0.00	0.00	0.08	0.00	0.00	0.09	0.00	0.04	0.02
27,216	28,122	0.00	0.00	0.00	0.31	0.00	0.00	0.03	0.00	0.00	0.05
28,123	29,029	0.00	0.00	0.03	0.00	0.00	0.00	0.09	0.00	0.00	0.00
29,030	29,937	0.00	0.00	0.00	0.16	0.00	0.00	0.01	0.00	0.00	0.00
29,938	30,844	0.00	0.00	0.04	0.00	0.00	0.00	0.01	0.00	0.00	0.01
30,845	31,751	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.01
31,752	32,659	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00
32,660	33,566	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.03
33,567	34,473	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
34,474	35,380	0.00	0.00	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.01
35,381	36,287	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03
36,288	38,556	0.00	0.00	0.00	0.04	0.00	0.00	0.01	0.00	0.00	0.10
Total		**100**	**0**	**100**	**100**	**100**	**100**	**100**	**100**	**100**	**100**

Table A-3: Southern Ontario Tridem Axle Load Distribution Table

Axle Weight (kg)		Axle Load Distribution as Percentage per Truck Class									
Min	**Max**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
0	5,443	0.00	0.00	0.00	4.26	0.00	39.94	4.98	0.00	0.00	6.50
5,444	6,803	0.00	0.00	0.00	9.29	0.00	7.55	9.65	0.00	0.00	11.02
6,804	8,164	0.00	0.00	0.00	10.96	0.00	19.96	9.53	0.00	0.00	6.55
8,165	9,525	0.00	0.00	0.00	0.30	0.00	5.90	7.21	0.00	0.00	3.69
9,526	10,886	0.00	0.00	0.00	14.23	0.00	0.67	5.21	0.00	0.00	2.44
10,887	12,246	0.00	0.00	0.00	1.97	0.00	5.34	5.07	0.00	0.00	2.29
12,247	13,607	0.00	0.00	0.00	4.54	0.00	2.18	4.39	0.00	0.00	2.18
13,608	14,968	0.00	0.00	0.00	2.12	0.00	8.20	4.32	0.00	0.00	4.16
14,969	16,329	0.00	0.00	0.00	12.24	0.00	3.58	4.56	0.00	0.00	4.46
16,330	17,690	0.00	0.00	0.00	0.64	0.00	1.74	4.82	0.00	0.00	4.54
17,691	19,050	0.00	0.00	0.00	0.00	0.00	3.42	5.87	0.00	0.00	3.90
19,051	20,411	0.00	0.00	0.00	0.50	0.00	1.23	5.44	0.00	0.00	7.33
20,412	21,772	0.00	0.00	0.00	0.00	0.00	0.00	6.96	0.00	0.00	11.94
21,773	23,133	0.00	0.00	0.00	9.88	0.00	0.00	6.31	0.00	0.00	14.87
23,134	24,494	0.00	0.00	0.00	3.00	0.00	0.29	5.68	0.00	0.00	8.24
24,495	25,854	0.00	0.00	0.00	6.69	0.00	0.00	4.50	0.00	0.00	3.49
25,855	27,215	0.00	0.00	0.00	9.24	0.00	0.00	2.20	0.00	0.00	1.43
27,216	28,576	0.00	0.00	0.00	4.56	0.00	0.00	1.25	0.00	0.00	0.34
28,577	29,937	0.00	0.00	0.00	5.58	0.00	0.00	0.60	0.00	0.00	0.35
29,938	31,298	0.00	0.00	0.00	0.00	0.00	0.00	0.32	0.00	0.00	0.16
31,299	32,658	0.00	0.00	0.00	0.00	0.00	0.00	0.31	0.00	0.00	0.04
32,659	34,019	0.00	0.00	0.00	0.00	0.00	0.00	0.25	0.00	0.00	0.01
34,020	35,380	0.00	0.00	0.00	0.00	0.00	0.00	0.28	0.00	0.00	0.06
35,381	36,741	0.00	0.00	0.00	0.00	0.00	0.00	0.11	0.00	0.00	0.00
36,742	38,102	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.00
38,103	39,462	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.00	0.00	0.00
39,463	40,823	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.00	0.00	0.01
40,824	42,184	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
42,185	43,545	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
43,546	44,906	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
44,907	47,628	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total		**0**	**0**	**0**	**100**	**0**	**100**	**100**	**0**	**0**	**100**

Table A-4: Southern Ontario Quad Axle Load Distribution Table

Axle Weight (kg)		Axle Load Distribution as Percentage per Truck Class									
Min	**Max**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
0	5,443	0.00	0.00	0.00	0.00	0.00	0.00	1.25	0.00	0.00	4.32
5,444	6,803	0.00	0.00	0.00	0.00	0.00	0.00	4.16	0.00	0.00	8.96
6,804	8,164	0.00	0.00	0.00	0.00	0.00	0.00	6.17	0.00	0.00	13.83
8,165	9,525	0.00	0.00	0.00	0.00	0.00	0.00	6.06	0.00	0.00	5.35
9,526	10,886	0.00	0.00	0.00	0.00	0.00	0.00	4.70	0.00	0.00	0.75
10,887	12,246	0.00	0.00	0.00	0.00	0.00	0.00	5.89	0.00	0.00	0.00
12,247	13,607	0.00	0.00	0.00	0.00	0.00	0.00	3.56	0.00	0.00	2.19
13,608	14,968	0.00	0.00	0.00	0.00	0.00	0.00	2.04	0.00	0.00	2.96
14,969	16,329	0.00	0.00	0.00	0.00	0.00	0.00	2.87	0.00	0.00	13.84
16,330	17,690	0.00	0.00	0.00	0.00	0.00	0.00	2.37	0.00	0.00	0.82
17,691	19,050	0.00	0.00	0.00	0.00	0.00	0.00	3.58	0.00	0.00	3.16
19,051	20,411	0.00	0.00	0.00	0.00	0.00	0.00	3.03	0.00	0.00	8.64
20,412	21,772	0.00	0.00	0.00	0.00	0.00	0.00	5.41	0.00	0.00	2.03
21,773	23,133	0.00	0.00	0.00	0.00	0.00	0.00	6.94	0.00	0.00	5.77
23,134	24,494	0.00	0.00	0.00	0.00	0.00	0.00	8.55	0.00	0.00	11.63
24,495	25,854	0.00	0.00	0.00	0.00	0.00	0.00	6.94	0.00	0.00	7.89
25,855	27,215	0.00	0.00	0.00	0.00	0.00	0.00	4.36	0.00	0.00	0.24
27,216	28,576	0.00	0.00	0.00	0.00	0.00	0.00	3.84	0.00	0.00	0.38
28,577	29,937	0.00	0.00	0.00	0.00	0.00	0.00	3.72	0.00	0.00	0.00
29,938	31,298	0.00	0.00	0.00	0.00	0.00	0.00	3.79	0.00	0.00	0.00
31,299	32,658	0.00	0.00	0.00	0.00	0.00	0.00	3.12	0.00	0.00	3.09
32,659	34,019	0.00	0.00	0.00	0.00	0.00	0.00	3.61	0.00	0.00	4.15
34,020	35,380	0.00	0.00	0.00	0.00	0.00	0.00	1.50	0.00	0.00	0.00
35,381	36,741	0.00	0.00	0.00	0.00	0.00	0.00	0.79	0.00	0.00	0.00
36,742	38,102	0.00	0.00	0.00	0.00	0.00	0.00	0.35	0.00	0.00	0.00
38,103	39,462	0.00	0.00	0.00	0.00	0.00	0.00	1.02	0.00	0.00	0.00
39,463	40,823	0.00	0.00	0.00	0.00	0.00	0.00	0.16	0.00	0.00	0.00
40,824	42,184	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.00	0.00	0.00
42,185	43,545	0.00	0.00	0.00	0.00	0.00	0.00	0.16	0.00	0.00	0.00
43,546	44,906	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
44,907	47,628	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total		**0**	**0**	**0**	**0**	**0**	**0**	**100**	**0**	**0**	**100**

Table A-5: Northern Ontario Single Axle Load Distribution Table

Axle Weight (kg)		Axle Load Distribution as Percentage per Truck Class									
Min	**Max**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
0	1,360	1.80	0.20	0.22	0.00	2.14	0.06	0.63	5.59	0.59	0.15
1,361	1,814	0.96	0.61	0.00	0.00	1.88	0.09	0.20	0.00	0.00	0.46
1,815	2,267	2.91	11.58	0.47	0.26	5.38	0.61	0.66	0.00	2.59	0.58
2,268	2,721	3.99	10.37	0.35	0.00	6.19	0.42	0.66	0.00	1.27	0.61
2,722	3,175	6.80	8.26	0.09	0.03	7.42	0.22	1.61	5.59	2.50	1.04
3,176	3,628	12.00	11.40	7.08	0.17	9.96	0.77	2.06	0.00	6.41	1.13
3,629	4,082	11.70	11.52	8.11	0.32	13.50	1.20	2.21	1.96	4.29	1.47
4,083	4,535	11.40	12.33	10.21	3.28	13.60	4.72	3.17	6.93	12.67	3.71
4,536	4,989	10.30	8.79	14.42	5.52	7.22	11.71	9.34	16.96	5.81	12.37
4,990	5,443	9.00	8.64	30.26	3.80	8.18	42.47	27.56	4.48	22.17	33.59
5,444	5,896	7.40	3.72	9.15	9.29	2.61	23.52	19.40	10.05	14.30	25.58
5,897	6,350	5.70	2.32	5.20	23.71	4.02	4.64	8.64	1.96	6.63	10.57
6,351	6,803	4.30	3.04	4.34	9.42	3.75	2.47	3.75	13.96	8.89	1.60
6,804	7,257	3.20	1.53	3.12	17.49	4.88	1.94	3.57	13.47	1.44	1.41
7,258	7,711	2.58	0.62	2.29	4.60	3.01	1.40	3.00	0.00	0.00	0.91
7,712	8,164	1.80	1.66	1.45	2.23	1.26	0.66	3.31	7.03	1.04	1.67
8,165	8,618	1.40	1.14	1.62	4.85	0.74	0.69	3.19	0.00	3.26	0.84
8,619	9,071	1.00	0.90	1.41	4.02	1.42	0.38	2.37	7.03	0.00	0.91
9,072	9,525	0.75	0.51	0.00	6.21	0.17	0.24	1.10	3.03	0.00	0.22
9,526	9,979	0.50	0.12	0.00	1.78	0.00	0.25	1.19	0.00	0.00	0.21
9,980	10,432	0.25	0.05	0.00	1.16	0.79	1.20	0.76	0.00	3.26	0.00
10,433	10,886	0.15	0.42	0.21	0.29	0.74	0.08	0.27	0.00	1.25	0.06
10,887	11,339	0.10	0.15	0.00	0.25	0.00	0.04	0.10	1.96	0.59	0.00
11,340	11,793	0.00	0.12	0.00	1.15	0.00	0.06	0.29	0.00	0.00	0.07
11,794	12,246	0.00	0.00	0.00	0.00	0.00	0.00	0.35	0.00	1.04	0.00
12,247	12,700	0.00	0.00	0.00	0.00	0.00	0.02	0.17	0.00	0.00	0.00
12,701	13,154	0.00	0.00	0.00	0.00	0.00	0.01	0.07	0.00	0.00	0.00
13,155	13,607	0.00	0.00	0.00	0.00	0.82	0.02	0.04	0.00	0.00	0.28
13,608	14,061	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.00	0.00	0.00
14,062	14,515	0.00	0.00	0.00	0.00	0.32	0.01	0.09	0.00	0.00	0.00
14,516	14,968	0.00	0.00	0.00	0.02	0.00	0.00	0.05	0.00	0.00	0.11
14,969	15,422	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.00	0.00	0.00
15,423	15,875	0.00	0.00	0.00	0.14	0.00	0.02	0.05	0.00	0.00	0.12
15,876	16,329	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.00	0.00	0.23
16,330	16,782	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.03
16,783	17,236	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07
17,237	17,690	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00
17,691	18,143	0.01	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.00
18,144	20,412	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00
Total		**100**	**100**	**100**	**100**	**100**	**100**	**100**	**100**	**100**	**100**

Table A-6: Northern Ontario Tandem Axle Load Distribution Table

Axle Weight (kg)		Axle Load Distribution as Percentage per Truck Class									
Min	Max	4	5	6	7	8	9	10	11	12	13
0	2,721	5.28	0.00	0.00	0.08	5.81	0.10	0.51	0.00	0.00	0.92
2,722	3,628	10.00	0.00	2.55	2.82	3.76	0.29	1.20	0.00	1.13	4.36
3,629	4,535	11.90	0.00	24.63	0.32	12.00	1.26	1.78	0.00	0.00	6.47
4,536	5,442	9.63	0.00	9.79	0.81	16.34	3.61	2.37	39.95	3.70	4.46
5,443	6,350	8.00	0.00	3.94	24.47	27.43	4.77	3.98	60.05	6.17	7.05
6,351	7,257	7.80	0.00	8.59	10.08	12.08	5.48	7.60	0.00	7.23	5.43
7,258	8,164	6.80	0.00	10.85	6.24	0.81	4.86	6.11	0.00	10.13	1.86
8,165	9,071	6.15	0.00	10.84	19.07	6.21	6.40	6.43	0.00	17.36	1.75
9,072	9,979	5.80	0.00	3.29	2.01	4.91	6.58	3.44	0.00	19.40	1.45
9,980	10,885	5.30	0.00	2.27	0.78	1.98	8.89	4.85	0.00	6.54	1.70
10,886	11,793	4.70	0.00	0.67	1.69	1.98	8.71	3.85	0.00	3.84	1.33
11,794	12,700	4.10	0.00	5.02	1.16	0.64	8.43	3.85	0.00	5.44	2.28
12,701	13,607	3.33	0.00	2.54	0.84	0.00	6.32	5.20	0.00	5.34	3.17
13,608	14,514	3.91	0.00	1.36	1.19	0.00	8.48	5.62	0.00	0.00	4.45
14,515	15,422	2.22	0.00	0.83	0.66	5.54	10.65	6.54	0.00	6.26	10.30
15,423	16,329	1.84	0.00	3.29	3.59	0.00	7.85	9.18	0.00	0.00	11.82
16,330	17,236	1.44	0.00	2.65	5.49	0.51	3.73	7.84	0.00	6.26	14.14
17,237	18,143	0.90	0.00	1.23	1.82	0.00	1.71	6.42	0.00	0.00	9.13
18,144	19,051	0.50	0.00	1.65	3.33	0.00	0.61	5.47	0.00	0.00	3.66
19,052	19,957	0.30	0.00	1.86	3.68	0.00	0.34	2.61	0.00	0.00	1.32
19,958	20,865	0.10	0.00	0.70	2.58	0.00	0.23	1.34	0.00	0.00	0.67
20,866	21,772	0.00	0.00	0.32	0.26	0.00	0.23	1.65	0.00	0.00	0.37
21,773	22,679	0.00	0.00	0.77	2.59	0.00	0.23	0.37	0.00	0.00	0.32
22,680	23,587	0.00	0.00	0.36	1.19	0.00	0.08	0.41	0.00	0.00	0.13
23,588	24,493	0.00	0.00	0.00	0.05	0.00	0.11	0.21	0.00	0.00	0.33
24,494	25,401	0.00	0.00	0.00	2.53	0.00	0.01	0.59	0.00	0.00	0.07
25,402	26,308	0.00	0.00	0.00	0.27	0.00	0.02	0.33	0.00	0.00	0.85
26,309	27,215	0.00	0.00	0.00	0.19	0.00	0.01	0.00	0.00	1.20	0.05
27,216	28,122	0.00	0.00	0.00	0.00	0.00	0.01	0.10	0.00	0.00	0.09
28,123	29,029	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.06
29,030	29,937	0.00	0.00	0.00	0.21	0.00	0.00	0.06	0.00	0.00	0.00
29,938	30,844	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
30,845	31,751	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
31,752	32,659	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
32,660	33,566	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
33,567	34,473	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
34,474	35,380	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
35,381	36,287	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
36,288	38,556	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.00	0.00	0.00
Total		**100**	**0**	**100**	**100**	**100**	**100**	**100**	**100**	**100**	**100**

Table A-7: Northern Ontario Tridem Axle Load Distribution Table

Axle Weight (kg)		Axle Load Distribution as Percentage per Truck Class									
Min	Max	4	5	6	7	8	9	10	11	12	13
0	5,443	0.00	0.00	0.00	0.00	0.00	7.03	5.26	0.00	0.00	5.63
5,444	6,803	0.00	0.00	0.00	20.16	0.00	5.16	7.54	0.00	100.00	13.67
6,804	8,164	0.00	0.00	0.00	0.00	0.00	0.00	8.63	0.00	0.00	6.55
8,165	9,525	0.00	0.00	0.00	44.60	0.00	0.19	6.67	0.00	0.00	2.23
9,526	10,886	0.00	0.00	0.00	9.52	0.00	0.85	4.91	0.00	0.00	2.02
10,887	12,246	0.00	0.00	0.00	0.00	0.00	5.33	4.48	0.00	0.00	1.16
12,247	13,607	0.00	0.00	0.00	0.00	0.00	1.04	4.85	0.00	0.00	1.75
13,608	14,968	0.00	0.00	0.00	0.00	0.00	77.00	5.07	0.00	0.00	2.42
14,969	16,329	0.00	0.00	0.00	0.00	0.00	0.13	5.21	0.00	0.00	3.41
16,330	17,690	0.00	0.00	0.00	0.00	0.00	0.00	4.96	0.00	0.00	4.27
17,691	19,050	0.00	0.00	0.00	0.00	0.00	2.79	7.72	0.00	0.00	4.74
19,051	20,411	0.00	0.00	0.00	0.00	0.00	0.00	6.05	0.00	0.00	10.07
20,412	21,772	0.00	0.00	0.00	13.18	0.00	0.00	5.54	0.00	0.00	13.11
21,773	23,133	0.00	0.00	0.00	12.54	0.00	0.28	6.90	0.00	0.00	17.57
23,134	24,494	0.00	0.00	0.00	0.00	0.00	0.20	5.38	0.00	0.00	6.99
24,495	25,854	0.00	0.00	0.00	0.00	0.00	0.00	4.27	0.00	0.00	2.47
25,855	27,215	0.00	0.00	0.00	0.00	0.00	0.00	2.05	0.00	0.00	0.51
27,216	28,576	0.00	0.00	0.00	0.00	0.00	0.00	1.57	0.00	0.00	0.48
28,577	29,937	0.00	0.00	0.00	0.00	0.00	0.00	0.98	0.00	0.00	0.27
29,938	31,298	0.00	0.00	0.00	0.00	0.00	0.00	0.87	0.00	0.00	0.07
31,299	32,658	0.00	0.00	0.00	0.00	0.00	0.00	0.47	0.00	0.00	0.55
32,659	34,019	0.00	0.00	0.00	0.00	0.00	0.00	0.29	0.00	0.00	0.06
34,020	35,380	0.00	0.00	0.00	0.00	0.00	0.00	0.18	0.00	0.00	0.00
35,381	36,741	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.00	0.00	0.00
36,742	38,102	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.00
38,103	39,462	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
39,463	40,823	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
40,824	42,184	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
42,185	43,545	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00
43,546	44,906	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
44,907	47,628	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total		**0**	**0**	**0**	**100**	**0**	**100**	**100**	**0**	**100**	**100**

Table A-8: Northern Ontario Quad Axle Load Distribution Table

Axle Weight (kg)		Axle Load Distribution as Percentage per Truck Class									
Min	**Max**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
0	5,443	0.00	0.00	0.00	0.00	0.00	0.00	3.18	0.00	0.00	5.82
5,444	6,803	0.00	0.00	0.00	0.00	0.00	0.00	5.32	0.00	0.00	9.55
6,804	8,164	0.00	0.00	0.00	0.00	0.00	0.00	10.24	0.00	0.00	3.11
8,165	9,525	0.00	0.00	0.00	0.00	0.00	0.00	5.20	0.00	0.00	0.00
9,526	10,886	0.00	0.00	0.00	0.00	0.00	0.00	2.00	0.00	0.00	0.00
10,887	12,246	0.00	0.00	0.00	0.00	0.00	0.00	3.36	0.00	0.00	0.00
12,247	13,607	0.00	0.00	0.00	0.00	0.00	0.00	2.61	0.00	0.00	3.12
13,608	14,968	0.00	0.00	0.00	0.00	0.00	0.00	2.12	0.00	0.00	6.44
14,969	16,329	0.00	0.00	0.00	0.00	0.00	0.00	4.23	0.00	0.00	3.85
16,330	17,690	0.00	0.00	0.00	0.00	0.00	0.00	2.47	0.00	0.00	9.36
17,691	19,050	0.00	0.00	0.00	0.00	0.00	0.00	1.01	0.00	0.00	0.00
19,051	20,411	0.00	0.00	0.00	0.00	0.00	0.00	0.23	0.00	0.00	0.00
20,412	21,772	0.00	0.00	0.00	0.00	0.00	0.00	7.58	0.00	0.00	3.41
21,773	23,133	0.00	0.00	0.00	0.00	0.00	0.00	3.05	0.00	0.00	2.40
23,134	24,494	0.00	0.00	0.00	0.00	0.00	0.00	4.19	0.00	0.00	45.88
24,495	25,854	0.00	0.00	0.00	0.00	0.00	0.00	7.42	0.00	0.00	0.09
25,855	27,215	0.00	0.00	0.00	0.00	0.00	0.00	3.19	0.00	0.00	6.97
27,216	28,576	0.00	0.00	0.00	0.00	0.00	0.00	5.90	0.00	0.00	0.00
28,577	29,937	0.00	0.00	0.00	0.00	0.00	0.00	6.43	0.00	0.00	0.00
29,938	31,298	0.00	0.00	0.00	0.00	0.00	0.00	5.29	0.00	0.00	0.00
31,299	32,658	0.00	0.00	0.00	0.00	0.00	0.00	4.38	0.00	0.00	0.00
32,659	34,019	0.00	0.00	0.00	0.00	0.00	0.00	8.46	0.00	0.00	0.00
34,020	35,380	0.00	0.00	0.00	0.00	0.00	0.00	1.64	0.00	0.00	0.00
35,381	36,741	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
36,742	38,102	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
38,103	39,462	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
39,463	40,823	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.00	0.00	0.00
40,824	42,184	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
42,185	43,545	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
43,546	44,906	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
44,907	47,628	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total		**0**	**0**	**0**	**0**	**0**	**0**	**100**	**0**	**0**	**100**

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