DEVELOPMENT OF A SIMPLIFIED FLEXIBLE PAVEMENT DESIGN PROTOCOL FOR NEW YORK STATE DEPARTMENT OF TRANSPORTATION BASED ON AASHTO ME PAVEMENT DESIGN GUIDE

by

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Abstract

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New York State Department of Transportation (NYSDOT) has used the AASHTO 1993 Design Guide for the design of new flexible pavement structures for more than three decades. The AASHTO 1993 Guide is based on the empirical relationships developed for the data collected in the AASHO Road Test in the early 1960's. A newer pavement design method, called the Mechanistic-Empirical Pavement Design Guide (MEPDG) was developed by the National Cooperative Highway Research Program to provide a more efficient and accurate design method and based on sound engineering principles. The MEPDG models have been incorporated in the AASHTOWare Pavement ME 2.1 software program that can be purchased from AASHTO. Due to the advanced principles and design capabilities of the AASHTOWare program, NYSDOT decided to implement the MEPDG and calibrate the distress models included in the software for the conditions in the state.

The work conducted in this research included the local calibration of the distress models for the North East (NE) region of the United States. Design, performance and traffic

data collected on Long Term Pavement Performance (LTPP) sites in the NE region of the United States were used to calibrate the distress models. First, the AASHTOWare Pavement ME 2.1 with global calibration factors was used to compare the predicted and measured distresses, values that were used for model calibration. The local bias was assessed for all distresses models except for the longitudinal cracking model; it was found the bias existed for this model even after calibration. The thermal cracking model was not calibrated because of erroneous measured data. The calibration improved the prediction accuracy for the rutting, fatigue cracking and smoothness prediction models.

The AASHTOWare software was used to run design cases for combinations of traffic volume and subgrade soil stiffness (Mr) for twenty-four locations in New York State. The runs were performed for a road classified as Principal Arterial Interstate, the 90% design reliability level and 15 years design period. State-wide average traffic volume parameters and axle load spectra were used to define the traffic. The NYSDOT's Comprehensive Pavement Design Manual (CPDM) was initially used to obtain pavement design solutions. The thicknesses for the select granular subgrade materials and the asphalt layer thicknesses were varied to include several values higher and lower than the thickness recommended by CPDM. The thicknesses of asphalt surface and binder layers were kept constant; only the thickness of the asphalt base layer was changed. For each design combination, the design case with thinnest asphalt layer for which the predicted distress was less the performance criteria was selected as the design solution. The design solutions for each of the 24 locations were assembled in design tables.

The comparison of the design tables showed that some variation in the design thickness for the asphalt layers exists even, with thicker asphalt layers being needed for the locations in the Upper part of the New York State. The comparison between the new design tables and the table included in the CPDM proved that the new design tables require thinner asphalt layers at low AADTT and thicker asphalt layers at high AADTT than the corresponding design in the CPDM table. For stiff subgrade soil and low AADTT, the design thicknesses are almost the same in the new design tables and in the CPDM table.

Table of Contents

List of Illustrations

List of Tables

Chapter 1

Introduction

The road infrastructure network of the United States includes about 2.5 million miles of paved roads, out of which approximately 94% are flexible pavements. If we could extend the life of these pavements by only one percent, we could save more than 150 million dollars annually. This statistic alone indicates the major economic impact of the pavement design process: effective pavement design process results in significant financial savings in terms of road construction, maintenance and rehabilitation. Therefore, efforts have been taken for many years to develop and improve pavement design methods that can predict well the performance of pavement structures under the action of traffic and climate, for a given subgrade soil condition.

In 1958 American Association of State Highways Officials (AASHTO) sponsored a multi-million dollar project which was called AASHTO Road Test, in Ottawa, Illinois. The aim of this project was to study the pavement performance under different loads and speed, also to quantify the damage of trucks on pavement for tax purposes. The valuable outputs led AASHTO to develop an empirical tool to design the flexible and rigid pavement structures which is called AASHTO pavement design guide. The first version of this guide was developed on 1972. Then, subsequent versions were developed on (1986, 1993, and 1996). 1993 AASHTO pavement design guide was widely adopted by highway agencies in 48 states. The empirical key developed to reflect the materials properties, traffic, and climatic of the test location only. For this reason extrapolation was used to modify and improve the empirical equation.

In 1990's, AASHTO Joint Task Force and Pavement (JTFP) initiated an effort to develop AASHTO pavement design guide. In 1996, a workshop was sponsored by JTFP to develop the mean of the Mechanistic-Empirical Pavement Design Guide by 2002 as deadline for the participants. The conclusion of this meeting was NCHRP project 1-37A, Development of the 2002 Guide for Design of New and Rehabilitated Pavement Structures. Phase II was awarded in 1998 to the ERES Consultants Division of Applied Research Associates, Inc. The project called for the development of a design guide that employs existing state-of-the-practice mechanistic-based models and design procedures.

In 2004, The Mechanistic-Empirical Pavement Design Guide (MEPDG) became available. It was released for public for review and evaluation. Then, The National Cooperative Highway Research Program (NCHRP) conducted a formal review under project 1-40A. The result of NCHRP 1-40A was numerous numbers of improvements that incorporated into the MEPDG software version 1.1. MEPDG 1.1 was developed under NCHRP project 1-40D. On April 2007, MEPDG 1.1 was submitted into NCHRP, Federal Highway Administration (FHWA), and AASHTO. Later, MEPDG 1.1 was released to the public for implementation and evaluation purposes. MEPDG software was improved and modified into consequent several versions. Currently, the MEPDG software is called AASHTOWare Pavement ME or AASHTOWare. AASHTOWare Pavement ME version 2.1 is the last version of AASHTOWare serious.

All the traffic loadings including different traffic and axle load distribution can be used by AASHTOWare to design the pavements. Moreover, AASHTOWare employs the climate data to calculate and adjust the stiffness of the structural layers during the design life. Accordingly, the predicted distresses are dissimilar among the regions due to the difference in the climate data. AASHTOWare uses hierarchical design inputs; they are divided into three levels depending on the quality of the design inputs. Level 1 is the most reliable design inputs. AASHTO recommends using a combination of design input levels in designing the new flexible pavements. The performance models in AASHTOWare should be recalibrated to the local conditions; therefore, the distresses will be accurately predicted. Accurate distresses prediction leads to most reliable design solutions. Hence, less cost and last longer pavement section can be designed. For this reason, several highway agencies have conducted the effort of implementing AASHTO ME Pavement Design Guide in designing the new flexible pavements.

New York State (NYS) is divided into Upstate and Downstate New York. There are up to 7 regions in Upstate New York, and 4 regions in Downstate New York. Further, there is a difference in the climate among them. Consequently, New York State Department of Transportation (NYSDOT) uses different Performance Grade (PG) of bitumen for Upstate and Downstate New York.

NYSDOT currently has implement 1993 AASHTO pavement design guide. Since the design is performed in NYSDOT local offices of NYS, NYSDOT developed a simple design table to design the new flexible pavement for Interstate Highways in NYS. The design table was developed at 90% design reliability and 50 years design life. This design table is used to design the pavement in all regions of Upstate and Downstate New York. Likewise, NYSDOT developed a typical design section which is used as a guide to determine the number of pavement structural layers and their thicknesses.

1.1. Problem Statement

New York State Department of Transportation (NYSDOT) has embarked in such an effort to implement AASHTO ME Pavement Design Guide (MEPDG) due to the high accuracy in distresses prediction. To implement AASHTO ME pavement Design, NYSDOT needs to develop a simple design procedure which can be used by the regional offices to design the new flexible pavement. Besides, NYSDOT needs design tables that look similar in format to the current used design table. There are some challenges that face the implementation task. The implementation effort requires an extensive array of input data which must represent the specific local conditions such as, materials characteristics, traffic and climatic data as well as performance requirements. Moreover, the performance models were calibrated at national calibration level by using Long Term Pavement Database Program (LTPP), these models should be recalibrated into New York State conditions. Moreover, the traffic inputs are collected from WIM stations in New York State should be processed before being used in the design procedure.

1.2 Research Objective

The aim of this project is summarized as follow:

- Recalibrate the performance models of flexible pavement distresses in AASHTOWare Pavement ME 2.1 to the local conditions of North Eastern region of the United States.

- Develop a simple design procedure for New York State Department of Transportation (NYSDOT) to be used in designing the new flexible pavement structure in New York State,
- Develop design tables for NYSDOT local offices in each region of Upstate and Downstate New York based on AASHTO ME Pavement Design Guide.
- Identify the climate effects on the design by AASHTO ME Pavement Design Guide.

Chapter 2

Literature Review

2.1. New York State Department of Transportation Current Practice

Currently, the New York Department of Transportation (NYSDOT) performs the design of flexible pavement structures following the Comprehensive Pavement Design Manual (CPDM) (NYSDOT, 2001). The CPDM was first issued by NYSDOT on October 31, 1994 and it is based on 1993 AASHTO Pavement Design Guide. CPDM includes two methods for the design of flexible pavements; first, Conventional Pavement Design Method – for road sections shorter than 1.5 km, second, ESAL Pavement Design Method – for road sections longer than 1.5 km.

NYSDOT tabulated Table 2-1 to design the flexible pavements based on the conventional method. The designer should obtain the Annual Average Daily Traffic (AADT) and the estimated percent of trucks, to find the structural layers thicknesses.

However, to design the flexible pavements based on the ESAL pavement design method, Table 2-2 is used. Table 2-2 had been developed for 90% design reliability and 50 years design life. Based on Table 2-2, the designer needs to calculate the ESALs and the resilient modulus (Mr) of subgrade soil only, to obtain on the structural thickness of Hot Mix Asphalt (HMA) and select subgrade layers.

NYSDOT developed a typical design section of flexible pavement structure in order to be used as a guide for NYSDOT engineers. This research used it as reference in developing the design cases. Figure 2-1 shown NYSDOT flexible pavement typical section.

Table 2-1: Thickness Guide for Conventional Pavement Design

Annual Average Daily Traffic AADT[1]	Percent Trucks	Subbase Course (all Pavements)	Hot Mix Asphalt Pavement	
			Base Course	Top & Binder Courses Combined
Over 10,000 Vehicles	10% or more	300	150mm	90mm
	Less than 10%		125mm	
6,000 to 10,000	10% or more	300	125mm	90mm
	Less than 10%		100mm	
4,000 to 5,999	all	300	75mm	90mm
Under 4,000 Vehicles	all	300	75mm	80mm

Table 2-2: CPDM Flexible Pavement Design Tables in SI System Units

NYSDOT CPDM for Mr 28 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 2	165	0
2 < ESALs <= 4	175	0
4 < ESALs <= 8	200	0
8 < ESALs <= 13	225	0
13 < ESALs <= 23	250	0
23 < ESALs <= 45	250	150
45 < ESALs <= 80	250	300
80 < ESALs <= 140	250	450
140 < ESALs <= 300	250	600

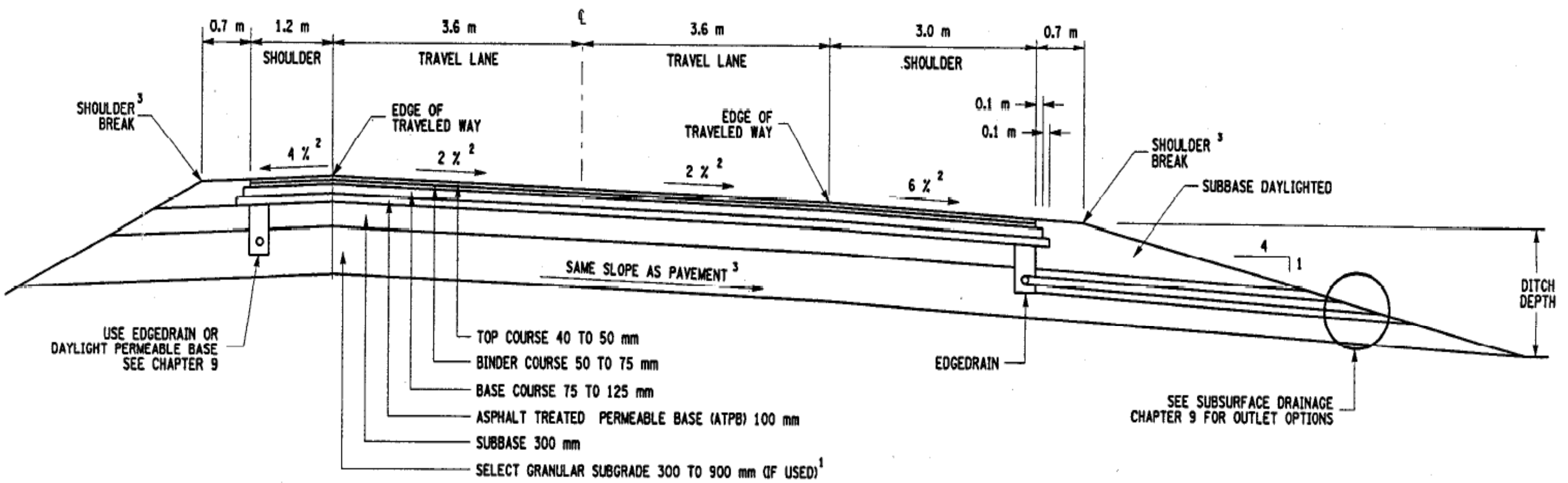
NYSDOT CPDM for Mr 34 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 4	165	0
4 < ESALs <= 7	175	0
7 < ESALs <= 13	200	0
13 < ESALs <= 23	225	0
23 < ESALs <= 40	250	0
40 < ESALs <= 70	250	150
70 < ESALs <= 130	250	300
130 < ESALs <= 235	250	450
235 < ESALs <= 300	250	600

NYSDOT CPDM for Mr 41 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 6	165	0
6 < ESALs <= 11	175	0
11 < ESALs <= 20	200	0
20 < ESALs <= 35	225	0
35 < ESALs <= 60	250	0
60 < ESALs <= 110	250	150
110 < ESALs <= 200	250	300
200 < ESALs <= 300	250	450

NYSDOT CPDM for Mr 48 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 8	165	0
8 < ESALs <= 16	175	0
16 < ESALs <= 30	200	0
30 < ESALs <= 50	225	0
50 < ESALs <= 85	250	0
85 < ESALs <= 160	250	150
160 < ESALs <= 300	250	300

NYSDOT CPDM for Mr 55 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 12	165	0
12 < ESALs <= 20	175	0
20 < ESALs <= 40	200	0
40 < ESALs <= 65	225	0
65 < ESALs <= 115	250	0
115 < ESALs <= 215	250	150
215 < ESALs <= 300	250	300

NYSDOT CPDM for Mr 62 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 15	165	0
15 < ESALs <= 30	175	0
30 < ESALs <= 50	200	0
50 < ESALs <= 90	225	0
90 < ESALs <= 150	250	0
150 < ESALs <= 300	250	150



THE DIMENSIONS SHOWN ARE TYPICAL ONLY AND THE DETAILS ARE SHOWN TO GIVE THE DESIGNER AN IDEA OF WHAT OVERALL SECTION LOOKS LIKE.

1. SEE CHAPTER 6 FOR MORE INFORMATION OR CONTACT REGIONAL GEOTECHNICAL ENGINEER FOR NECESSITY AND THICKNESS OF SELECT GRANULAR SUBGRADE. DAYLIGHT SELECT GRANULAR SUBGRADE ONTO EMBANKMENT SLOPES OR AT THE EXTENSION OF THE DITCH BACKSLOPE IN CUT SECTIONS.
2. REFER TO THE HIGHWAY DESIGN MANUAL, CHAPTERS 2 AND 3, FOR THE PROPER CROSS SLOPE AND FURTHER DEVELOPMENT OF TYPICAL SECTIONS.
3. REFER TO THE HIGHWAY DESIGN MANUAL, CHAPTER 3, FOR SUBGRADE GRADE CHANGES AND EDGE OF SHOULDER DETAILS.

Figure 2-1: NYSDOT Design Typical Section

SUperior PERerforming Asphalt PAVEments (SUPERPAVE) mixtures are currently used on the majority of the pavements built and maintained by NYSDOT since:

- The rehabilitation of these pavements is quick and easy.
- These pavements have a life span of 15 to 20 years for thicker overlay and 8 to 10 years for single course overlay if a proper maintenance is provided
- The life of the pavement foundation is 50 years
- Low Cost, especially in initial construction

To extend the performance of asphalt mixes, NYSDOT uses the performance graded binder (PG) specifications for asphalt binders; they were developed through research performed during Strategic Highway Research Program (SHRP) in the early 1990s. The CPDM recommends specific PG grades for the asphalt binder depending on the geographic location of the pavement research, as given in Table 2-3.

Table 2-3: Performance Graded Binder Selection

Location	Location by Counties	Standard PG Binder Grades (Material Designation)	Polymer Modified PG Binder Grades (Material Designation)
Upstate	All Counties not Listed under Downstate	64S-22 (702-64S22)	64V-221,2 (702-64V22)
Downstate	Orange, Putnam, Rockland, Westchester, Nassau, Suffolk Counties and City of New York	64H-22 (702-64H22)	64E-22 (702-64E22)

The NYSDOT recommends the aggregate in the asphalt mixes used for the top (surface) course to have the nominal maximum aggregate size (NMAS) of 12.5mm and 9.5mm. However, NYSDOT recommends the use of aggregates with NMAS of 9.5 mm where a lot of handwork is envisioned, if gravel aggregates are used, or on urban areas.

For the binder course, NYSDOT recommends the aggregate with NMAS of 19.0 mm and 25.0 mm. Normally, aggregates with NMAS of 19.0 mm are used for researchs where the 20 year ESAL count is less than 10 million. In addition, NYSDOT recommends using aggregate with NMAS of 25 mm if the HMA pavement is thicker or if the 20 year ESAL count is over 10 million.

The NYSDOT recommends the aggregate in the asphalt base course should have the NMAS of 37.5 mm, 75 mm, or 25.0 mm. Base course thickness is considered in choosing the aggregate size, also the calculated ESAL is considered too. NYSDOT recommends using nominal aggregate size 75mm or less if the base course is thick. However, aggregates with NMAS of 37.5 mm are the most common.

2.2. AASHTOWare Framework

In AASHTOWare Pavement ME computer software, the pavement responses, such as stresses and strains, are computed based on the materials and traffic loadings inputs by JULEA model for the analyzed pavement structure. To accurately compute the response of the pavement structures, the resilient modulus of unbound materials and the dynamic modulus of asphalt concrete layers are adjusted during the design life by the Integrated Climatic Model (ICM). The ICM uses the climatic inputs to perform this adjustment. Then, pavement response is used to compute the pavement damage. The computed pavement damage is exerted to predict the pavement distresses over the design life by the performance models.

The performance prediction models (transfer functions) have been embedded in AASHTOWare Pavement ME (AASHTOWare). AASHTOWare Pavement ME 2.1 is the latest version of AASHTOWare software series. These models are the key components of the design and the analytical process because they compute the distresses over the design life.

The empirical nature of the design method stems from the fact that the pavement performance predicted from the laboratory developed transfer functions. These developed functions are adjusted based on the observed field performance to reflect the differences between the actual field and computed distresses which refers to the calibration process (AASHTO, 2010).

The performance models embedded in the AASHTOWare have been calibrated globally. The calibration was accomplished by using measured construction and performance data on several hundred of representative pavement sections throughout North America, monitored as part of the Long Term Pavement Performance (LTPP) program (Jianhua, et al., 2009). The use of LTPP data was decided because of the consistency in the monitoring program over time and the diversity of test sections spread throughout North America. In addition to LTPP, other experimental test sections were included in this process; for example, MnRoad and Vandalia (AASHTO, 2010).

The term “model calibration” refers to the mathematical process which minimizes the total error Sum of Squared Errors (SSE) between the measured values and the values predicted by the model. The difference between the observed and the corresponding predicted value is called residual, or error. The major effect of the calibration process is that it also reduces the life-cycle cost (AASHTO, 2008). Without calibration, the default transfer functions cannot predict accurately the distresses; the design might be over conservative or insufficient for the local conditions. This leads to higher costs for construction, maintenance or rehabilitation of the road sections.

Local calibrations factors that can be included in the AASHTOWare models reflect the difference in construction, practices, maintenance policies, and material specifications across the United States (Mehta, 2008).

2.2.1 AASHTOWare Permanent Deformation Model

Rutting occurs due to the applied shear stress on the pavement with softened layers, when the pavement under traffic loading consolidates and/or there is a lateral movement of the Hot-Mix Asphalt (HMA). The hot mix asphalt movement may occur because of poor material properties or the effect of the environment (high temperature or moisture).

In AASHTOWare, permanent deformation is measured in absolute terms and not based on Miner’s law. In the national calibration permanent deformation data collected on 88 pavement sections in 28 states were used. Temperature and moisture content are included in the computation of permanent deformation through their effect on resilient modulus for granular layers and dynamic modulus for asphalt concrete. AASHTOWare calibrates specific form to calculate the accumulated damage in asphalt layer using Equation 2-1:

$$\Delta_{p(HMA)} = \beta_1 k_z \varepsilon_{r(HMA)}\ 10^{k_{1r}} n^{k2r\beta_{2r}} T^{k_3\beta_{r3}} \qquad \text{Equation 2-1}$$

Where:

$\Delta_p(HMA)$ = Accumulated permanent of plastic defomation in the HMA layer/sublayer, in.

$\varepsilon_{r(HMA)}$ = Resilient of plastic strain calculated by structure response model at the mid depth of each HMA sublayer, in/in

n = Number of axle - load repetitions

T = Mix or Pavement Temperature, F°

Kz = Depth confinement factor

$k1r, k2r, k3r =$ Global field calibration parermeters (from the NCHRP $1 - 40\text{D}$)

$B1r, B2r, B3$ = mixture field calibration constants for the global calibration

Where Kz is calculated as:

$$Kz = (C1 + C2D)0.328196^D$$

C1 and C2 are calculated as:

$$C1 = -0.1039\ (Hhma)^2 + \ 2.4868\ Hhma - 17.342$$

$$C2 = 0.0172(H_{hma})^2 - \ 1.7331\ H_{hma} + 27.428$$

Where:

D = Depth below the surface, in.,

H_{hma} = Total HMA thickness, in.

The mathematical filed calibration equation used by AASHTOWare to calculate the plastic vertical strain within all unbound pavement sublayers is:

$$\Delta_{p(soil)} = \boldsymbol{\beta_{s1}\, k_{s1} \varepsilon_v h_{soil} \left[\frac{\varepsilon_\circ}{\varepsilon_r}\right] e^{-\left[\frac{\rho}{n}\right]^{\beta}}} \qquad \text{Equation 2-2}$$

Where:

$\Delta_{p(soil)}$= Permanant or plastic deformation for the $\frac{\text{layer}}{\text{sublayer}}$, in.,

n = Number of axle − load application

$\varepsilon_\circ$ = Intercept determined from the laboratory repeated load permanent deformation tests in/in

ε_r = Resilient strain imposed in laboratory test to obtain materials properties $\varepsilon_\circ$, ε, and ρ , in/in

ε_v = Average vertical strain in layer/sublayer and calculated by structural response model.

h_{soil} = Thickness of the unbounded layer/sublayer, in

k_{s1} = Global calibration coefficients

$\log \beta = -0.61119 - 0.017638(W_C)$

$$\rho = 10^9 * \left(\frac{C_\circ}{(1-(10^9)^\beta)}\right)^\wedge \frac{1}{\beta}$$

$$C_\circ = Ln\left(\frac{a_1 * M_r^{b_1}}{a_9 * M_r^{b_9}}\right) = 0.0075$$

W_C = Water content, %

M_r = Resilient modulus of the unbound layer or sublayer, psi

$a_{1,9}$ = Regression constants; $a_1 = 0.15$ and $a_9 = 20.0$, and

$b_{1,9}$ = Regression constants; $b_1 = 0.0$ and $b_9 = 0.0$

2.2.2 AASHTOWare Alligator Cracking Model

Fatigue cracking is the major type of the cracking. Since wheel load repetitions generate tensile stresses in the bound layer, fatigue cracks initiated at the point where the highest tensile stresses are. Then, the cracks propagate through the entire layer. AASHTOWare considers the initiated bottom up cracking as fatigue cracking.

AASHTOWare calculates this type of cracking as a percent of total lane area. Incremental damage index is calculated first to predict the fatigue cracking. The allowable numbers of axle-load applications, *Nf*, are estimated to calculate the incremental damage index. Since the applied load causes the tensile strains, AASHTOWare calculates them using Equation 2-3:

$$Nf = kf1 * (C) * (Ch) * \beta f1 * (\varepsilon t)^{kf2 * \beta f2} * (Ehma)^{kf3 * \beta f3} \qquad \text{Equation 2- 3}$$

Where:

Nf = Allowable number of axle-load applications for a flexible pavement and HMA overlays

ε_t = Tensile strain at critical location

E_{hma} = Dynamic modulus of HMA measured in compression, psi

$\beta f1, \beta f2, \beta f3$ = Local specific field calibration constants, for global calibration efforts. Set to 1.0

$k1, k2, k3$ = Global calibration parameters

$C = 10^M$

$$M = 4.84 * \left(\frac{Vbe}{Va + Vbe} - 0.96\right)$$

Vbe = Effective asphalt contenet by volume, %

Va = Percent air voids in HMA mixture

CH = Thickness correction term, depends on the type of cracking

Where CH is calculated if the cracks for bottom-up by equation as:

$$CH = \frac{1}{0.000398 + \frac{0.003602}{1 + e^{(11.02 - 2.49Hhma)}}}$$

Where CH is calculated if the cracks for up-bottom by equation as:

$$CH = \frac{1}{0.01 + \frac{12.00}{1 + e^{(15.676 - 2.8186Hhma)}}}$$

$Hhma$ = Total HMA thickness layer, in

The incremental damage index is calculated by Miner's hypothesis. The damage is calculated using Equation 2-4 as the ratio of cumulative predicted wheel load repetitions to the allowable number of wheel load repetitions.

$$\boldsymbol{D} = \sum_{\boldsymbol{i=1}}^{\boldsymbol{T}} \frac{\boldsymbol{n_i}}{\boldsymbol{N_i}} \qquad \text{Equation 2-4}$$

Where:

D = Damage

n_i = Actual traffic for period i

Ni = Allowable failure repetitions under conditions prevailing in period i

AASHTOWare calculates the cumulative damage index by summing the incremental damage indices over the time, as:

$$DI = \sum \left(\frac{n}{N_{f-HMA}}\right)_{j,ml,p,t}$$

The fatigue damage transfer functions for longitudinal (top-down) and alligator cracking (bottom-up) are:

$$FC_{Top-Down} = 10.56 * \left(\frac{1000}{1+e^{c_1-c_2*logD}}\right) \qquad \text{Equation 2-5}$$

$$FC_{Bottom-Up} = \left(\frac{6000}{1+e^{\wedge}(c_1*c_1{'}+c_2*c_2{'}*log_{10}^{D*100})}\right) \qquad \text{Equation 2-6}$$

Where:

$FC_{Top-down}$ = Fatigue cracking (ft/mile)

$FC_{Bottom-up}$ = Fatigue cracking (% of total lane area)

*C'1 = -2*C'2*

C'2 = -2.40874-39.748(1+hac)-2.85609*

hac = asphalt layer thickness (inches)

D = damage in percentage

C1, C2= regression coefficients (7.0, 3.5) for top-down cracking

C1, C2= regression coefficients (1.0, 1.0) for bottom-up cracking

2.2.3 AASHTOWare International Roughness Index (IRI) Model

The degradation of the pavement due to the surface distresses is the reason of increasing the IRI distresses. The collected LTPP data were used to develop the IRI model as aforementioned. Equation 2-7 is embedded in the AASHTOWare to predict the IRI distresses.

$$IRI = IRI_O + 40 * RD + 0.4 * FC_{Total} + 0.008 * TC + 0.015 * SF \qquad \text{Equation 2-7}$$

Where:

IRI_O = Initial IRI after construction in/mile

RD = Average rut depth in

FC_{Total} = Area of fatigue cracking

TC = Length of transverse cracking ft/mile

SF = Site factor

SF is calculated based on the following equation as follows:

$$SF = Age\,[\,0.02003 * (PI + 1) + 0.007947 * (Precip + 1) + 0.000636 * (FI + 1)]$$

Where:

Age = Pavement age in years

PI = Percent of plasticity index of the soil

FI = Annual freezing average index in F°

$Precip$ = Average annual precipitation or rainfall in.

2.3 AASHTO Local Calibration Guide

The local calibration is vital for the implementation of the Pavement ME Design process; the local data set should be used to take into account local materials, traffic information, and the climatic conditions. In addition, the adoption of the calibrated models cannot be done without model validation. Validation is defined as "a systematic process that re-examines the recalibrated model to determine if the desired accuracy exists between the calibrated model and an independent set of observed data" (AASHTO, 2010); (Kim, et al., 2010). Separate and independent data should be used in the calibration and the validation.

American Association of State Highway and Transportation Officials (AASHTO) developed a guide to accomplish the local calibration and validation. This guide was developed under NCHRP (1-40B) research. The guide provides recommended steps to calibrate the performance models of AASHTOWare Pavement ME computer software. The recommended steps are explained as follow.

2.3.1 Select Hierarchical Input Level

The selection of the design input level has no affect on the predicted distresses and the smoothness at 50% design reliability (AASHTO, 2008). As result, the computation algorithm for damage and distresses are exactly the same. However, the Standard Error of Estimate (SEE) for each distress model is affected by the design input level. As result, SEE and design reliability are used to compute the distresses at the selected design reliability.

For example, if a designer decides using level three design inputs routinely, the standard error will be higher than if level one or two inputs are used. Thus, the computed distresses will be over estimated.

For each distress model, AASHTO developed an equation to compute SEE. Then, AASHTO embedded those equations in AASHTOWare to be used in predicting the distresses at design reliability. AASHTOWare uses Equation 2-7 to calculate the distress at selected reliability.

$$\boldsymbol{Y = X + SEE * Z} \qquad \text{Equation 2-7}$$

Where:

Y = Predicted distresses at design reliability

X = Predicted distresses at 50% design reliability

SEE = Standard Error of Estimate

Z = Standard normal distribution at selected design reliability

To start the calibration process, the design input level should be determined by the designer. The decision of the inputs levels is made based on the agency policy (AASHTO, 2010). Therefore, design inputs levels may vary throughout the U.S. In addition, AASHTO developed a list of predominated design inputs to be used in AASHTOWare if the inputs levels were determined by the agency as shown in Table 2-4.

Table 2-4: AASHTO Predominated Input Level

Input Group		Input Parameter	Recalibration Input Level Used
Truck Traffic		Axle Load Distributions (Single, Tandem, Tridem) Truck Volume Distribution	Level 1
		Lane and Directional Truck Distributions	Level 1
		Tire Pressure	Level 3
		Axle Configuration, Tire Spacing	Level 3
		Truck Wander	Level 3
Climate		Temperature, Wind Speed, Cloud Cover, Precipitation, Relative Humidity	Level 1 Weather Stations
Materials Properties	Unbbound Layers and Subgrade	Resilient Modulus-All Unbound Layers	Level 1
		Classification and Volumetric Proerties	Level 1
		Moisture-Density Relationships	Level 1
		Soil-Water Characteristic Relationships	Level 3
		Saturated Hydraulic Conductivity	Level 3
	HMA	HMA Dynamic Modulus	Level 3
		HMA Creep Compliance and Indirect Tensile Strength	Level 1, 2 and 3
		Volumetric Properties	Level 1
		HMA Coefficient of Thermal Expansion	Level 3
	PCC	PCC Elastic Modulus	Level 1
		PCC Flexture Strength	Level 1
		PCC Indirect Tensile Strength (CRCP Only)	Level 2
		PCC Coefficient of Thermal Expansion	Level 1
All Materials		Unit Weight	Level 1
		Posson's Ratio	Level 1 and 3
		Other Thermal Properties; Conductivity, Heat Capacity, Surface Absorptivity	Level 3
Existing Pavement		Condition of Existing Layers	Level 1 and 2

2.3.2 Develop Local Experimental Plan

The aim of this step is to refine the AASHTOWare distresses models and the IRI model based on the local materials, conditions, and policies. Preparation of the experimental plan or sample template is the first step in this process. Sample template parameters are categorized into two tiers: primary tier and secondary tier (AASHTO, 2010).

Primary tier parameter should be distress dependent. For example, subgrade soil type, pavement thickness, pavement type, and subsurface layer type. The secondary tier parameter should include: the traffic, climate and other design features that the pavement type depends on it.

Preparing the sample template is a sophisticated process. It requires the design of a fractional factorial matrix as much as possible. The matrix should be designed so it can be blocked because the design features. Blocking the fractional factorial will determine whether the bias and the standard error of the transfer functions are dependent on any of the primary tier parameters of the matrix (AASHTO, 2010). The sample template cells should include replicate segments.

2.3.3 Select Roadway Segments

The aim of this step is to select the roadway segments that maximize the benefits of local calibration and validation process, also reduce the cost of sampling and testing for test sections.

AASHTO recommends the experimental plan data should be obtained from the following experimental segments:

- Long Term (Full Scale Roadway Segments): They are categorized as, PMS segments, and the LTPP segments.
- Accelerated Pavement Testing (APT Segments): They are short pavement sections loaded with simulated truck loadings. Since the conditions are controlled, the bias and the standard errors are low. Therefore, APT sites can be used to supplement the roadway segments used in the calibration process. In addition, they can be used to determine the standard error of the estimate. APT sites should be used to determine the bias and quantify the variance of the transfer functions.

Likewise, AASHTO recommends using non complex road segments in the calibration and the validation process. The reason of this is to reduce the input parameters and testing. Moreover, it is suggested to use the roadway segments with or without overlay.

The segments that have a detailed prior history before and after the overlay, should give a high consideration. Moreover, the segments were paved with unconventional mixtures or design, should be included in the calibration and the validation effort because the model should include all unconventional design to simulate the current road construction practice.

2.3.4 Distress Evaluation and Extraction

It is important to evaluate and exam the collected measured distresses. Afterward, the measured distresses should be compared with the computed distresses as AASHTO recommends. The computed distresses are generated by running AASHTOWare design problems with global calibration factor. It is important to mention that local experimental plan data are the inputs for the design problems.

For each segment, the maximum measured value of each distresses type should be extracted and listed. Then, the average of the listed maximum observations is found for each distress type. It is preferable that the average distress value exceeds 50% of the design criteria (threshold value). The design criteria are selected by the designer in order to judge the acceptability of the trail design. The aim of extracting the maximum measured distresses value is to validate the accuracy and the bias of each distress model that will be well defined at the value that trigger major rehabilitation.

The design criteria are usually defined by the highway agency since they represent the maximum accepted distresses in the pavement structure before placing the overlay. Moreover, AASHTO developed a list of the design criteria which can be used by the designer when there are no threshold values defined by the highway agency. The suggested design criteria are tabulated in Table 2-5.

If the average less than 50%, there are two explanations, first, selected threshold value is high, so the agency should select a lower design criteria, second the flexible pavement was rehabilitated for other reasons (AASHTO, 2010).

Table 2-5: Suggested Design Criteria or Threshold Values by AASHTO

Pavement Type	Performance Criteria	Threshold Value
HMA Pavement and Overlays	Alligator Cracking	Interstate: 10% lane area
		Primary: 20% lane area
		Secondary: 35% lane area
	Rut Depth	Interstate: 0.4 in
		Primary: 0.5 in
		Other less than 45 mph: 0.65 in
	Thermal Cracking	Interstate: 500 ft/mi
		Primary: 700 ft/mi
		Secondary: 700 ft/mi
	IRI (Smoothness)	Interstate:160 in/mi
		Primary: 200 in/mi
		Secondary: 200 in/mi

Outliers in the measured data may reflect measurement errors for distress surveying; the Wisconsin Department of Transportation (WDOT) (Kang, et al., 2007) reports such errors. Therefore, as recommended by AASHTO, outliers in the measured data must be identified first. Due to the large volume of measured data, is recommended that a computer software that performs statistical analysis to be used for outlier identification.

Additionally, visual evaluation of the data can also be used for this purpose. For example, if the outliers can be explained and a result of non-typical condition, they must be removed. It should be noted that if outliers have been found, they should be removed. Then, the sample size should be estimated again to ensure its adequacy.

2.3.5 Sample Size Estimation for Each Performance Model

The purpose of this step is to determine the adequacy of the sample size to conduct the global calibration and then, determine the local calibration coefficients (AASHTO, 2010). The transfer functions are evaluated based on the Bias and the Precision. Consequently, the sample size is estimated based on the mean and variance. It is important to mention that the sample size relates to the bias. The bias is defined as the average of the residual errors (AASHTO, 2010).

The residual errors are used to compute the Standard Errors of Estimate (SEE) (Kim, et al., 2011). Therefore, the sample size relates to the variance too. The sample size can be estimated after defining Equation 2-8 and the following parameters:

$$\boldsymbol{Tolerable\ Bias(et) = SEE * Z_{\propto/2}} \qquad \text{Equation 2-8}$$

$SEE =$ Standard Error of Estimate, It is computed for each model based on the threshold value

$\sigma =$ Threshold Value (Varied based on the type of the distress model)

$Y =$ True Values (Observed Values from the Field)

$Sy =$ Standard Deviation of the True Value

$Se =$ Standard Devation of the Residuals

$X\alpha^2 =$ Chi − Square based on degree of freedom and level of significant

$t =$ T − Distribution Value based on degree of freedom and level of significant

AASHTO suggests the following equations to be used for sample size estimation based on the defined parameters. The use of equation depends on user decision either estimating bias or precision.

$$\boldsymbol{N} \geq \left(\frac{\boldsymbol{t*Sy}}{\boldsymbol{et}}\right)^{\boldsymbol{2}}$$ Equation 2-9

$$\boldsymbol{N} = \left(\frac{\frac{\boldsymbol{z\alpha*\delta}}{\boldsymbol{2}}}{\boldsymbol{E_T}}\right)^{\boldsymbol{2}}$$ Equation 2-10

$$\frac{\boldsymbol{Se}}{\boldsymbol{Sy}} \geq \left[\frac{\boldsymbol{X\alpha^2}}{\boldsymbol{n-1}}\right]^{\boldsymbol{0.5}}$$ Equation 2-11

Equations 2-9 and 2-10, estimate the sample size based on the mean or bias, but, Equation 2-11 estimates the sample size based on the variance or precision. In this step the maximum distress for each segment is extracted and listed. Then, the,(Se) and (Sy) can be computed. After that, Equations 2-9 to 2-11 can be used.

Three levels of significance can be used in estimating the sample size for each distress model: 75%, 90%, and 95% (AASHTO, 2010). However, 90% level of significant is suggested to be used.

In this research, same test sections will be used to estimate the sample size for all distress models because the coupling effect between different models (AASHTO, 2010). AASHTO suggests listed values of sample size to be used as guidance. Table 2-6 lists the minimum number of local test sections for each distress model.

Table 2-6: AASHTO Recommended Minimum Number of Test Sections

Distress Type	Min Number of Segments
Total Rutting	20
Load Related Cracking	30
Non Load Related Cracking	26
Reflection Cracking	26

2.3.6 Assess Local Bias

It is a vital step in the calibration process, the AASHTOWare outputs are compared with the full set of measured data. The purpose of this step is to compute the residual errors, bias, and SSE. Here the entire dataset is used unlike in the sample size estimation step where only the maximum values are used.

The local bias is evaluated by performing the null hypothesis that there is no bias which means that the difference between the computed and measured distresses is zero (AASHTO, 2010). Equation 2-12 showed the null hypothesis test as below:

$$\boldsymbol{H_\circ}: \sum(\boldsymbol{y\ measured} - \boldsymbol{y\ predicted}) = \mathbf{0} \qquad \text{Equation 2-12}$$

The intercept ($b_\circ$) and the slope (b_1) are used to evaluate the model bias by performing the regression between the measured and the computed values. The null hypothesis statement to test the intercept and the slope are:

$H_\circ: b_\circ = 0$

$H_a: b_1 = 0$

If the null hypothesis is rejected, the transfer function should be recalibrated to the local conditions. After calibrating to the local conditions, the local bias should be reassessed again and the null hypothesis studied again. If the null hypothesis is accepted, SEE for local data should be compared to the SEE for the global calibration dataset (AASHTO, 2010). Then, the null hypothesis is used to test the SEE. The assumption is that there is no significant difference between the SEE of the local and global dataset. However, if the null hypothesis is rejected, SEE of the local data set should be evaluated. In case, the SEE has higher values, it is recommended that distresses model should be recalibrated.

2.3.7 Validation of the Local Calibrated Models

Unfortunately, the common procedure for model development is by using all the obtained data in the calibration. Then the resulted Goodness - Of - Fit Statistics (GOFS) is considered as an indicator of the model accuracy (AASHTO, 2010). In fact, the calibration reflects the accuracy of the model for regenerating the calibration data but it does not reflect the accuracy of the model for the full population.

In other words, ignoring the validation process may result in misleading computed distresses. However, if the sample size is sufficiently large, the validation can be ignored (AASHTO, 2010). This scenario rarely occurs for pavement performance data.

The goal of the validation process is to demonstrate the capability of the local calibrated models to predict distresses (Kim, et al., 2011). In other words, the local calibrated performance models should predict a pavement performance close to or same as the field performance in the real world.

The model is successfully validated when the bias and (Se) are the same as those obtained from the calibrated models. For this purpose, a chi-square test is applied to determine if the (Se) of the validated and calibrated models are same. The test is performed at a recommended level of significance, α, of 0.05 (AASHTO, 2010).

There are two approaches can be used to achieve this goal, they are as follow:

1- Traditional Approach

2- Jack Knife Testing.

2.3.7.1 Traditional Approach

In this approach, the dataset used for calibration is divided randomly into two parts. The first part, typically at least half of the dataset, is used in the calibration process only while the second part is independently used to validate the model. It should be noted that if the dataset is small, these limitations cause misleading results. For this reason, they should be considered before applying this method. Small dataset is defined as a partial factorial with less than 25 percent of the cells filled with observations but without replications (AASHTO, 2010).

2.3.7.2 Jack Knife Testing

It is more reliable method to assess GOFS than the previous technique because it is estimated independently from the dataset used for calibration, so the size of the sample size has no effect on the validation process. For this reason, it works better for small datasets (AASHTO, 2010). In this approach, the standard errors are computed based on variables that have not been used in the calibration process, so the Jack Knife Testing is considered an independent measure of the model accuracy. The procedure consists of:

- Data set is divided into two groups, one for the calibration, and the other for prediction. These groups are randomly selected. Therefore, the prepared matrix will consist group X (as independent variables) and group (Y) as predicted variable, with (*i=1,,,,,, n),* sets of measured values.

- At the beginning of the process, one set of measured data is removed. By removing one set such as(x_1, y_1), the validation matrix will contain *(n-1*) sets of measured data to perform the calibration.
- After the calibration is performed, the calibrated coefficients are used to predict (y) which will be listed in a new group, it is called K_{th} group. So in this step, the predictor (y_{kt1}) will be produced.
- Then, the standard error (e_1) is found by the difference between the measured and computed value. For example$(\ y_{kth1} - y_1)$.
- The removed data set in step (2) is replaced with the second data set in the *(n-1)* validation matrix which is(y_2, x_2).
- Same previous process reapply to produce (y_{kth2}, x_2) and(e_2).
- The process of withholding, calibrating, and predicating is repeated until all (*n-1)* sets have been used for prediction.

As result, the limitation in the pavement performance availability, the sensitivity, or the stability, and the accuracy of the model to the sample size can be assessed which is one of the validation intent. Therefore, Multiple Jack Knifing approach is applied for this purpose. The procedure is achieved by applying same steps as the Jack Knife Testing. However, two sets of measured data are removed instead of only one. The process of the multiple jack knifing (withholding, calibrating, and predicting) is repeated until all *(n-2)* measured data have been used for prediction.

To identify the sensitivity of the model to the sample size, the *(SSE)* of jack knife testing and multiple jack knifing should be tested. If they are similar, the model is stable and not sensitive to the sample size, so the accuracy is confirmed.

2.4 Examples of Calibration of the Mechanistic-Empirical Pavement Design for Local Conditions

Once the MEPDG version 1.1 was released on 2007, the highway agencies started to work on the implementing effort of the ME design for flexible and rigid pavements (Khazanovich, et al., 2004). A key requirement was to recalibrate and validate the transfer functions. As result, the default regression coefficients of the performance models were calibrated with hundreds of selective pavement sections that were embedded in the MEPDG software version 1.1. Therefore, the efforts of developing local calibrated coefficients were conducted by numerous highway agencies as a first step in the implementation process (Halil Ceylan, 2013).

For this reason, AASHTO has released a guide to develop those regression coefficients as abovementioned. Some agencies efforts were initiated before releasing AASHTO local calibration guide. Consequently, the performance models were recalibrated by various procedures. However, After AASHTO released the local calibration guide earlier on 2010; several agencies conducted the calibration effort based on its procedure; for instance, of Utah, North Carolina, and Idaho.

It is significant to have a comprehensive background and knowledge about what highway agencies have done, so the calibration is professionally performed. Moreover, the calibration process should be continued when an updated version is released to ensure that the local bias elimination (Jianhua, et al., 2009). Thus, this chapter presents the local calibration efforts that were performed by other highway agencies.

2.4.1 North Carolina Local Calibration

North Carolina Department of Transportation (NCDOT) decided to adopt the Mechanistic Empirical Pavement Design Guide for future pavement design, and awarded several researches to the North Carolina State University. They are as follow:

- *HWY-2005-28* - Implementation Plan for the New Mechanistic Empirical Pavement Design Guide
- *HWY-2003-09* - Typical Dynamic Moduli for North Carolina Asphalt Concrete Mixes
- *HWY-2008-11* - Development of Traffic Data Input Resources for the Mechanistic Empirical Pavement Design Process
- *HWY-2007-07* - Local Calibration of the MEPDG for Flexible Pavement Design

For the purpose of calibrating local regression coefficients, HWY-2003-09 was dedicated to establish the material database. HWY-2008-11 focused on developing the traffic parameters that are required by the MEPDG 1.1. Then, HWY-2007-07 was dedicated to establish the local calibrated coefficients for the performance models of the MEPDG 1.1. The effort of HWY-2007-07 research is summarized as the follow.

2.4.1.1 Select Site Sections

NCDOT followed the steps recommended by the NCHRP (1-40B) panel, explained in detail in section 2.3 of this chapter. NCDOT has a total of twenty nine LTPP sites, eighteen flexible pavement sites and eleven rigid pavements. Those eighteen sites have twenty eight test sections, each five hundred feet long. These sections are divided further into twelve special pavement study (SPS) sections and sixteen general pavement study (GPS) sections. Six of SPS sections were retained for the calibration work; the rest had no sufficient data and were disregarded. LTPP sections were used for the calibration because LTPP have more complete distress and materials information available other than other sites.

In addition, NCDOT used data from twenty four PMS sections. Twelve of these sections were let in 1993 while the remaining twelve were let in 1999. It is important to mention that the asphalt concrete mixtures for the sections that were let in 1993, were designed using the traditional Marshall mix design method, whereas those that were let in 1999 were designed using the Superpave mix design method (Kim, et al., 2011). Based on what was mentioned earlier, NCDOT has a total of forty six test sections were selected for Calibration/Validation. For calibration, when data on these sections was missing, MEPDG default values were used instead (Kim, et al., 2011).

2.4.1.2 Selection of Hierarchical Input Level for each Input Parameter

NCDOT assured that the level of inputs for the calibration process should be similar to those are used by NCDOT personnel to design the new flexible pavement. NCDOT research program has no experiments results from the laboratory since the programs' intent is to design a new flexible pavement. For this reason, there was no Level (1) input for any layer of structure (Kim, et al., 2011). HMA material properties, including binder and mixture, were considered Level 2 inputs. Similarly, Level 3 and Level 2 inputs were used for the base and subgrade materials, respectively. Level 1 was used for the traffic inputs because actual data obtained from forty-four WIM stations was available.

2.4.1.3 Eliminating the Local Bias

NCDOT calibrated the permanent deformation and alligator cracking models. MEPDG 1.1 was employed for this reason. To calibrate the permanent deformation model, the twelve most commonly used Asphalt Concrete Mixture in North Carolina, were used to develop the design problems. For each design problem there was a specific mixture. In the MEPDG 1.1, there are options to input the (K) values for each Asphalt Concrete (AC) sublayer. The user can input up to three (K) values. NCDOT input those values based on the selected mixture, also NCDOT referred those values as (K_1', K_2', K_3'). The linear coefficients $(\beta_{r1}, \beta_{gb}, \beta_{sg})$ set as their default values.

Then, the MEPDG 1.1 was executed numerous times by using large set of (β_{r2}, β_{r3}) for the rutting model. The computed distresses were extracted and listed with the corresponded measured distresses of the selected roadway segments. The Microsoft solver had been used to optimize$(\beta_{r1}, \beta_{gb}, \beta_{sg})$, so minimum SSE was computed. Table 2-7 lists the optimized regression coefficients that produced the least SSE.

Table 2-7: Optimized Regression Coefficients for the Permanent Deformation Model

Parameter	Value
Br1	13.1
Br2	0.4
Br3	1.4
Bgb	0.303
Bsg	1.102

To calibrate the alligator cracking model, NCDOT calibrated Nf model by executing the MEPDG with numerous values of β_{f2}, and β_{f3} for each section which reached up to 181 values.

It is important to mention that the default (K) values in Nf model were replaced with the specific (K) values for unbounded layers which they were from the twelve commonly used mixtures in North Carolina. Furthers, NCDOT referred them as (K_1', K_2', K_3'). The computed distresses were extracted later to be compared with the measured distresses of the selected roadway segments. Then, Microsoft Solver was employed to optimize the rest of local coefficients to obtain a minimum Standard Squared of Errors (SSE). The optimized regression coefficients are listed in Table 2-8.

Table 2-8: Optimized Regression Coefficients for Alligator Cracking Model

Parameter	Value
βf1	3.878
βf2	0.8
βf3	0.8
C1	0.245
C2	0.245

2.4.1.4 Local Bias Assessment

NCDOT assessed the local bias for the rutting model by performing the null hypothesis test before and after the optimization or calibration. The test was performed at 95% confidence level with total a number of observations of 239. Bias was found to exist before and after the calibration. The standard errors(Se), (Se/Sy), and R^2 were computed too; they are listed in Table 2-9.

Table 2-9: Summary of Rutting Distress Model before and after Calibration

Distress type	Calibration	Total SSE	Bias	Se	Se\Sy	R^2	Hypothesis H0: ∑ (Meas-Pred) = 0
Total Rut	National	-0.031	0.129	1.027	1.027	Poor	Reject; p=0
	Local	3.803	-0.041	0.121	0.961	Poor	Reject; p=0
AC Rut	National	0.826	-0.017	0.057	1.005	Poor	Reject; p=0
	Local	0.731	-0.02	0.052	1.019	Poor	Reject; p=0
Base Rut	National	0.212	-0.004	0.03	0.845	0.16	Accepted; p=0.058
	Local	0.037	-0.003	0.012	0.81	Poor	Reject; p=0
Subgrade Rut	National	1.127	-0.01	0.069	0.695	0.39	Reject; p=0
	Local	1.534	-0.019	0.079	0.715	0.31	Reject; p=0

Table 2-9 showed that the calibration slightly improved the predictions as the total SSE decreased for the rutting in AC and Base layers but it increased for the subgrade and total rutting. However, the null hypothesis was rejected for all the calibrated rutting models which indicated the calibration did not adequately enhance the models' performance. According to NCDOT, the calibration did not improve the distress functions because:

- The lack of information due to the absence of forensic engineering. This reason led NCDOT to assume the contribution of each layer to the total measured rutting, in percentage, the same as the contribution to the distresses computed by the MEPDG.
- A small changed to $\beta_{f2},$ and β_{f3} , the exponents factors, can affect the overall prediction process.

Same process was applied to assess the local bias of the alligator cracking model. Table 2-9 shown the summary of the local bias assessment, the Total *SSE* and the bias decreased notably because the calibration. Nevertheless, (Se) did not show any significant decrease after the calibration.

Based on Table 2-10 the null hypothesis was rejected at the 95% confidence level, despite the improvement in the $p - value$. In other words, the differences between the measured and predicted alligator cracking values are still significant even after the calibration (Kim, et al., 2011).

Table 2-10: Summary of Alligator Cracking Model before and after Calibration

Distress type	Calibration	Total SSE	Bias	Se	Se\Sy	R^2	Hypothesis H0: ∑ (Meas-Pred) = 0
Alligator Cracking	National	56412	-11.034	19.498	1.022	Poor	Reject; p=0
	Local	41764	-4.836	19.852	1.041	Poor	Reject; p=0.008

2.4.1.5 Validation of Local Calibration for Rutting and Alligator Cracking

Traditional approach was applied by NCDOT to validate the calibrated models. Out of twenty-four PMS pavement sections selected for the validation, only fifteen sections were used due to the lack of the distresses data. For some sites, separate distress data sets were acquired for different travel directions (e.g., northbound vs southbound). Including the distress data for all directions, twenty five sets of distress data were used in the validation for both the 1993 and 1999 sections. To complement the data from 2008 and earlier, the 2010 collected data were obtained from NCDOT PMU.

To validate the permanent deformation model the null hypothesis at 95% confidence level was employed. The null hypothesis statement was, the difference between the predicted distress and the validation sections is zero.

In addition, the b$ias, (Se), \left(\frac{Se}{Sy}\right), and\ \ R^2$ were computed to evaluate the process statistically. They are listed in Table 2-10. The table shown that the validation statistics are in general, worse than the calibration statistics mainly for total rut depth and subgrade layers. On the other hand, the standard error of the estimate (Se) and the (Se/Sy) term both increased slightly especially for the total rutting, AC rutting, and base rutting values. In addition, Table 2-11 suggests that there are significant differences exist between predicted and measured rut depth values for validation sections. It is important to recall that the validation effort was achieved by using the PMS sections. However, the calibration effort was achieved by using the LTPP sections. Data from LTPP sections is more accurate than that from PMS section (Kim, et al., 2011).

To analyze the data more efficiently, predicted versus measured data (for validation sections) were plotted, clearly show that MEPDG over predicts the total rut depth, and the MEPDG predicts the AC rut depth better than the total rut depth, also some of the predicted base and subgrade rut depth values are in agreement with the measured subgrade rut depth values; but, in general, the trend was clear that the MEPDG over predicts both of these rut measurements for the PMS sections

Table 2-11: Comparison of Rutting Statistical Parameters between Calibration and Validation

Distress type	Analysis	Bias	Se	Se\Sy	R^2	Hypothesis H0: ∑ (Meas-Pred) = 0
Total Rut	Calibration	-0.041	0.121	0.961	Poor	Reject; P=0
	Validation	0.265	0.15	1.827	Poor	Reject; P=0
AC Rut	Calibration	-0.02	0.052	1.019	Poor	Reject; P=0
	Validation	0.088	0.052	1.287	Poor	Reject; P=0
Base Rut	Calibration	-0.003	0.021	0.81	Poor	Reject; P=0.001
	Validation	0.004	0.011	1.071	Poor	Reject; P=0
Subgrade Rut	Calibration	-0.019	0.079	0.715	Poor	Reject; P=0
	Validation	0.172	0.13	3.759	Poor	Reject; P=0

Same process was repeated for the alligator cracking model as shown in Table 2-12.

Table 2-12: Comparison of Alligator Cracking Statistical Parameters between Calibration and Validation

Distress type	Analysis	Bias	Se	Se\Sy	R^2	Hypothesis H0: ∑ (Meas-Pred) = 0
Alligator Cracking	Calibration	-4.836	19.852	1.041	Poor	Reject; p=0.008
	Validation	2.064	10.602	1.75	Poor	Reject; p=.032

The stated hypothesis that there is no difference between the predicted and measured was rejected. Moreover, the bias increased, but the standard error (Se) decreased. It is clear that the ratio of (Se/Sy) for the validation increased.

NCDOT personnel attributed the difference between the calibration and validation because the measured data were inaccurate. The alligator cracking data for the PMS sections were surveyed using the windshield method; it is very likely that less cracks than the true existing cracks were recorded. NCDOT assumed that some cracked areas were not caught by the camera while the car had been driven (Kim, et al., 2011). Thus, the unreliable survey technique may be the reasons that LTPP measured distress values are higher on average than the PMS-measured values.

Figure 2-1 illustrates the predicted versus measured alligator cracking for the PMS sections after the validation. The figure emphasizes that, on average, the alligator cracking measurements taken by the NCDOT are much lower than those predicted by the MEPDG after the local calibration. However, for a few cases the measured and predicted values were close.

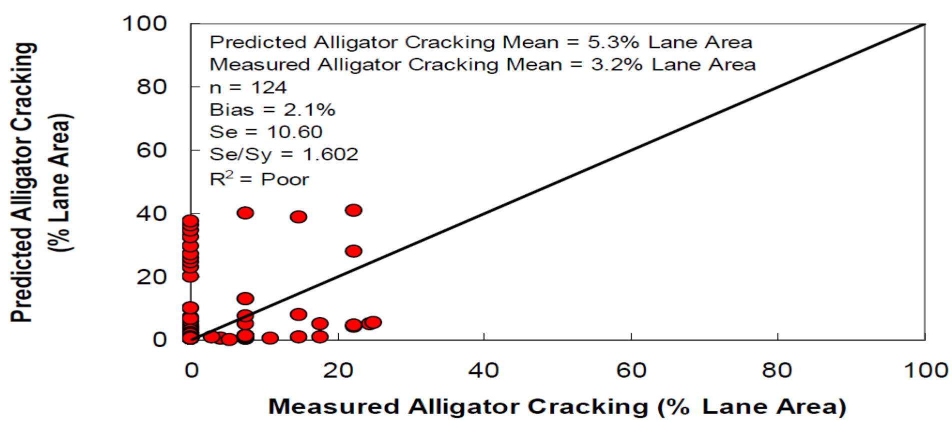


Figure 2-1: Predicted Alligator Cracking Vs Measured Alligator Cracking (Validation Sections)

2.4.2 New Mexico Local Calibration

New Mexico State Department of Transportation (NMDOT) believes that the 1993 AASHTO pavement design guide is inaccurate and the current design method does not reflect their actual conditions (Rafiqul A. Tarefder, 2012). Therefore, the effort of implementing the MEPDG design was conducted. NMDOT had decided to employ the MEDPG Version 1.1 for this purpose. A first step of this process was to establish an extensive pavement performance database to recalibrate the performance models. NMDOT is currently in the process of developing the design solutions based on the MEPDG after the performance models have been recalibrated for MEPDG 1.1. The performance models for total rutting, alligator cracking, longitudinal cracking and IRI were calibrated to the local conditions of New Mexico as follows:

2.4.2.1 Data for Local Calibration

The internal pavement management databases of NMDOT were the first source for traffic, climatic, structural, and performance data. Thirteen pavement segments were obtained (Tarefder, et al., 2013); they are shown in Table 2-12. Moreover, the LTPP database was used to provide performance data for an additional eleven segments, as show in Table 2-13. Six segments were new flexible pavements and the rest are rehabilitated pavements.

NMDOT has used the new and rehabilitated flexible pavement segments to recalibrate the fatigue cracking model. As a result, fewer pavement data are available. It is also important to note that NMDOT collected alligator cracking data from rehabilitated pavements without considering reflective cracking.

Table 2-13: NMDOT Flexible Pavement Sections in New Mexico

State Code	Road	Mile Post	Functional Class	Type of Experiment	Construction Date
NMDOT2	I-10	148	1	Rehabilitated	Jul,1984
NMDOT4	I-40	183	1	New	Jun,1999
NMDOT5	I-40	187	1	New	Jun,1999
NMDOT6	I-40	243	1	Rehabilitated	Jun,1986
NMDOT10	I-25	252	1	New	Jul, 1982
NMDOT12	US-54	82	2	New	Jun, 1977
NMDOT15	US-62	35	2	New	May, 1992
NMDOT19	US-64	97	2	Rehabilitated	Oct, 1983
NMDOT20	US-64	205	2	New	Oct, 1971
NMDOT21	US-70	254	2	New	Oct, 1986
NMDOT23	US-82	135	2	New	Sep, 1994
NMDOT25	US-84	183	2	Rehabilitated	Jul, 1985
NMDOT27	US-180	114	2	New	Sep, 1994

Table 2-14: LTPP Flexible Pavement Sections in New Mexico

State Code	SHRP ID	Road	Mile Post	Functional Class	Type of Experiment	Construction Date
35	1,002	US-70	310.1	2	GPS-6A	May, 1958
35	1,003	US-70	320.9	2	GPS-1	May, 1983
35	1,005	I-25	263.8	1	GPS-1	Sep, 1983
35	1,022	US-550	125.1	2	GPS-1	Sep, 1983
35	1,112	US-62	81.3	2	GPS-1	May, 1984
35	2,006	US-550	89.5	2	GPS-2	Jun, 1982
35	2,007	US-550	106.2	2	GPS-6A	Jun, 1981
35	2,118	I-40	346.2	1	GPS-2	Dec, 1979
35	6,033	I-25	159.3	1	GPS-6A	May, 1981
35	6,035	I-40	96.7	1	GPS-6A	May, 1985
35	6,401	I-40	107.7	1	GPS-6A	May, 1984

2.4.2.2 Calibrating and validating the permanent deformation model

The permanent deformation model was recalibrated by selecting nineteen pavement segments randomly. The rest were used for the validation purpose. The process was achieved by conducting two steps. First step was performed by optimizing B_{r2} and B_{r3} which are the non-linear calibration factors. This was done by varying them while the other three, B_{r1}, B_{gb}, and B_{Sg}, were set to the default value of 1.0. For every set of the non-linear calibration factors, the Sum Squared of Errors (SSE) and Mean Residual Errors (MRE) were computed.

In the second step, the sets of B_{r2} and B_{r3} which produced the minimum SSE and MSE were selected and fixed to their values. Then, the other three coefficients were recalibrated iteratively. Finally, the local calibration coefficients of B_{r1}, B_{r2}, B_{r3}, B_{gb}, and B_{Sg} were 1.1,1.1, 0.8, 0.8, and 1.2 respectively.

The measured and predicted rutting values were plotted before and after the calibration as show in Figures 2-2 and 2-3. Based on the plots, most of the LTPP data located below the equality line while the values most NMDOT PMS sections located above the equality line. NMODT attributed this dispersion to the different procedure of distresses measurement. After the calibration, data became less scattered, suggesting that the calibration factors should be adopted.

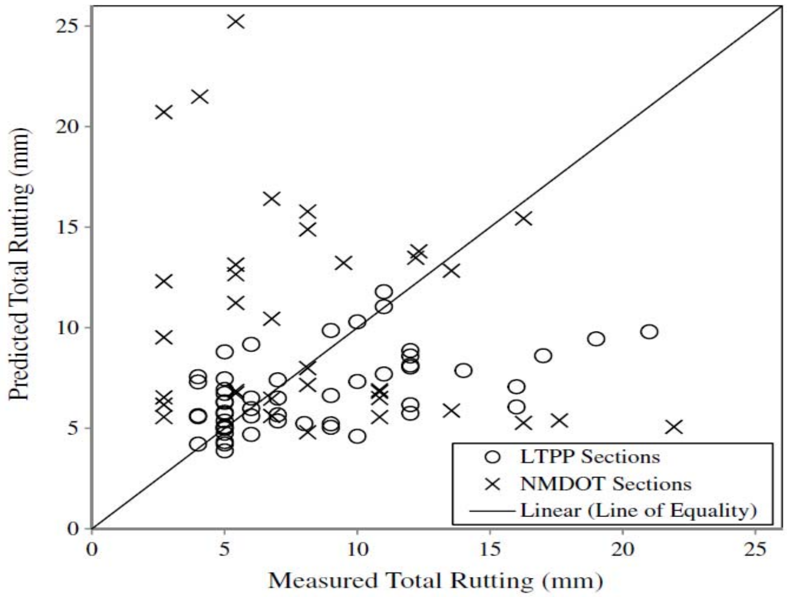


Figure 2-2: Predicted Versus Measured Total Rutting Before Calibration

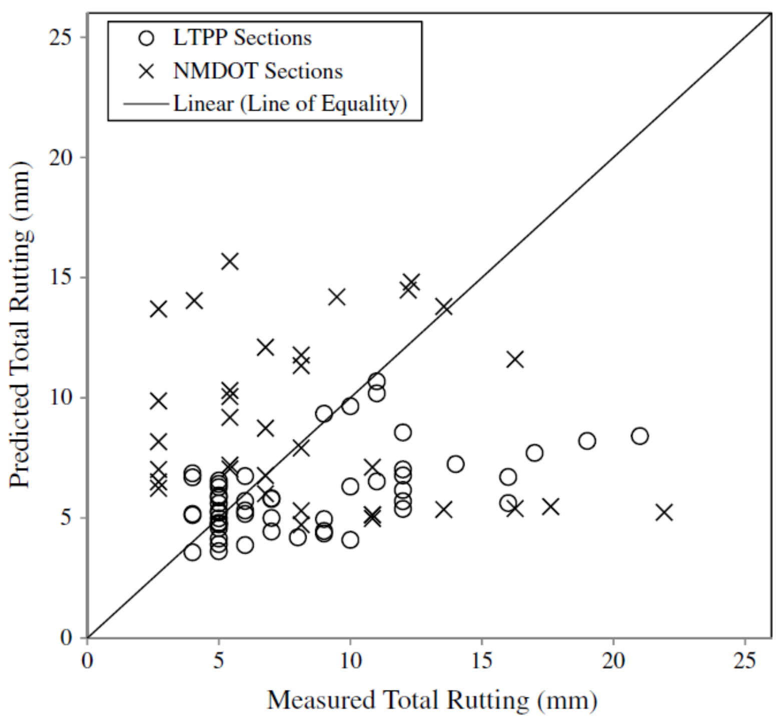


Figure 2-3: Predicted Versus Measured Total Rutting After Calibration

To validate the model with the calibrated coefficients, the traditional approach was used. Therefore, the MEPDG run with new set of calibrated coefficients were performed for the following sections SHRP 2006, SHRP 6033, NMDOT 15, NMDOT 21, and NMDOT 25. Then, the residual errors were checked by calculating the SSE and MRE. It was found that the model satisfied.

2.4.2.3 Calibrating and Validating the Alligator Cracking Model

The MEPDG was used to run with varied and permuted values of $C_1$and C_2 in order to find combination values that will reduce the SSE. C_3 was fixed to be 6,000 which is the default value. It was found that at C_1= 0.625 and C_2= 0.25, the minimum SSE and MRE obtained. Thus, they were considered to be the local calibration coefficients. The plots of predicted versus measured alligator cracking, before and after calibration are shown in Figures 2-4 and 2-5. NMDOT could not calibrate the coefficients $\beta_{f1}, \beta_{f2}, and\ \beta_{f3}$ of the fatigue cracking model since there was no available data to compare with.

The MEPDG runs with the locally calibrated models were conducted to predict the distresses for the validations' sections. These sections are SHRP 1002, SHRP 1022, NMDOT19, NMDOT20, and NMDOT27. The calibrated coefficients showed an improvement in the prediction of the distresses, the calibrated model was considered to be valid.

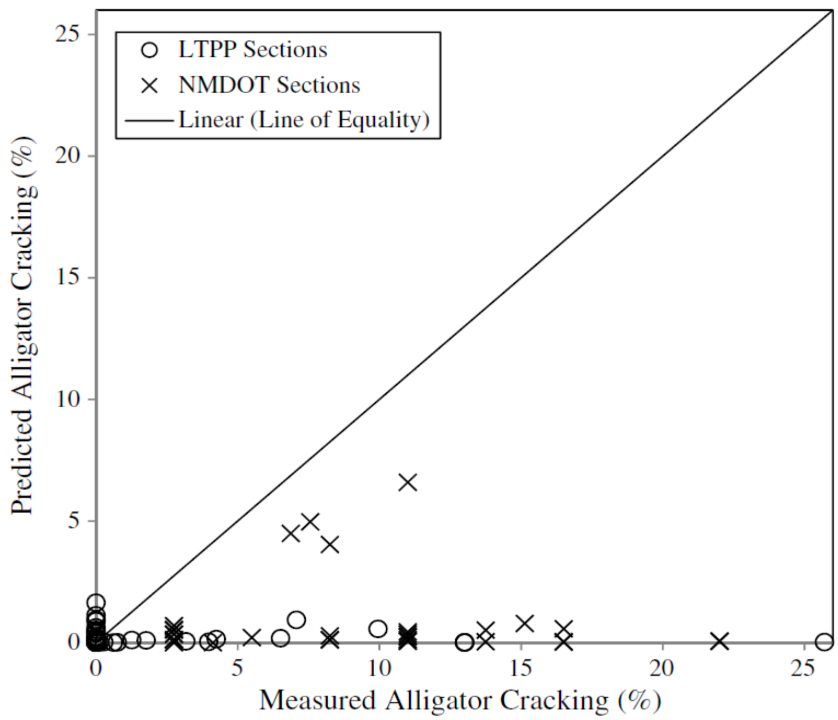


Figure 2-4: Predicted Versus Measured Alligator Cracking Before Calibration

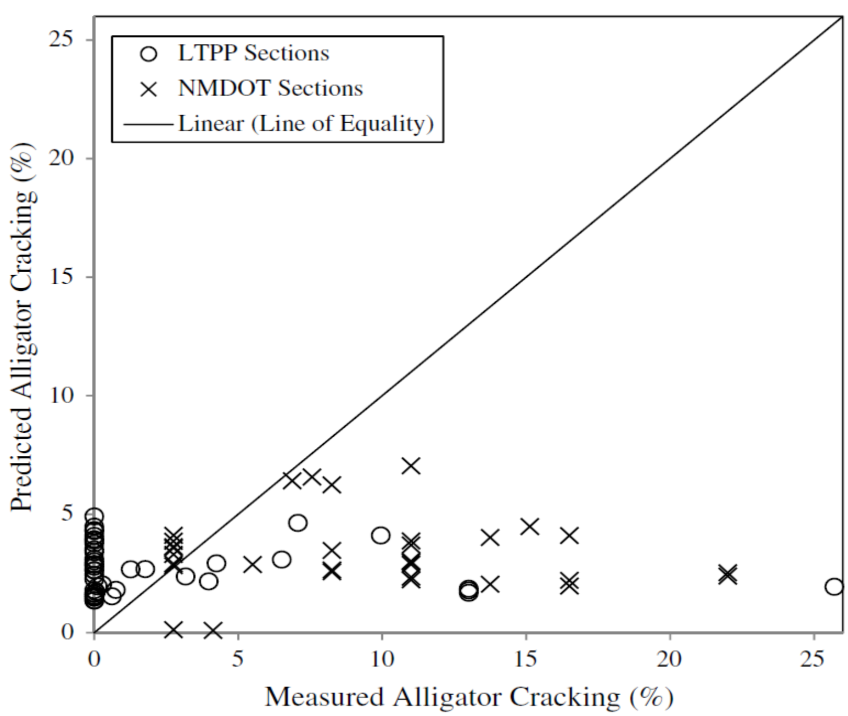


Figure 2-5: Predicted Versus Measured Alligator Cracking After Calibration

2.4.2.4 Calibrating and Validating the Longitudinal Cracking Model

The Top Down cracking model was calibrated by running the MEPDG with different combinations of C_1 and C_2. However, C_3 was kept constant as 1,000 as the default value. The *SSE* and *MRE* were computed for each combination

The local calibration coefficients were the combination of coefficients that lead to the lowest *SSE* and *MRE*. Thus, the calibrated coefficients were C_1= 3, C_2= 0.3, and C_3 = 1000. Figures 2-6 and 2-7 illustrate the predicted versus measured longitudinal cracking values before and after the calibration respectively.

The validation process was performed based on the traditional approach. The validated sections were SHRP 1003, SHRP I6035, NMDOT 2, NMDOT 12, and NMDOT 23. As result, the new calibration improved the prediction accuracy, thus, SSE reduced from 407,098,600 to 802,600. Therefore, the model was considered validated.

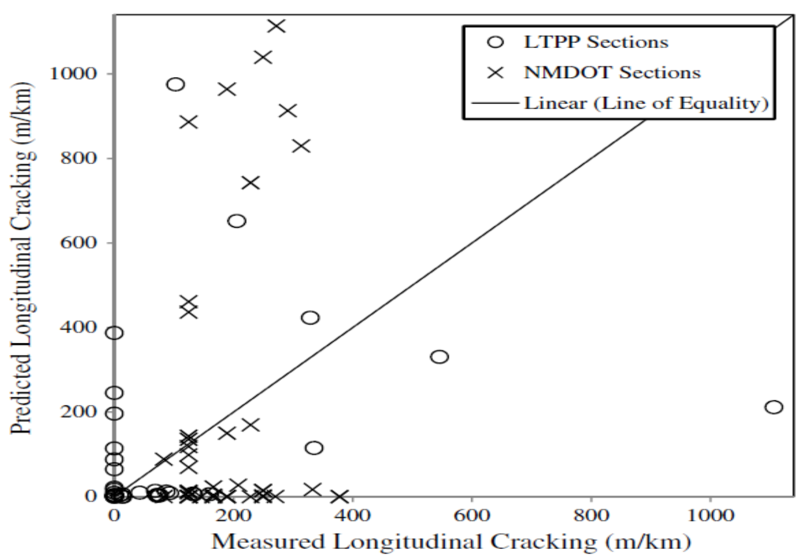


Figure 2-6: Predicted versus Measured Longitudinal Cracking before the Calibration

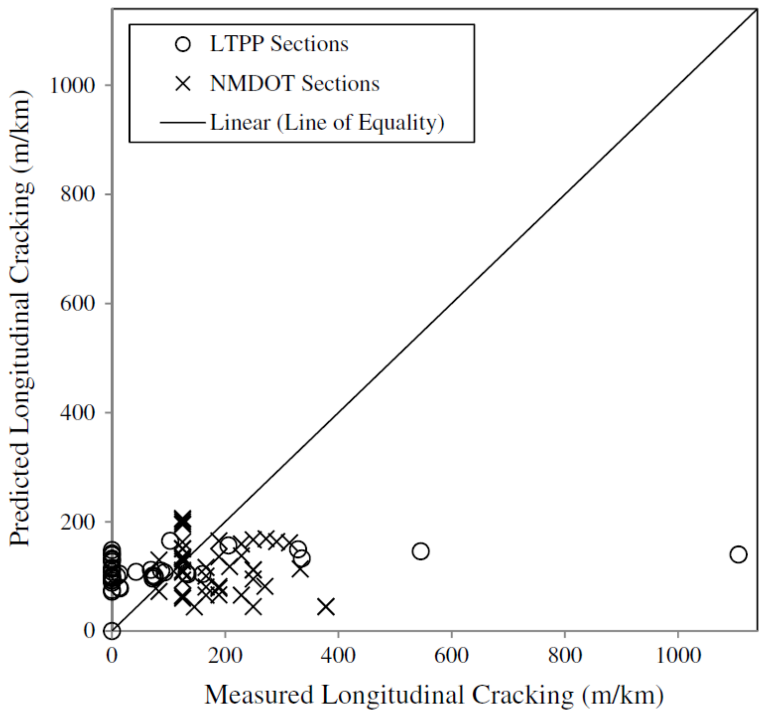


Figure 2-7: Predicted versus Measured Longitudinal Cracking after the Calibration

2.4.2.5 Calibrating and Validating the International Roughness Index (IRI) Model

NMDOT calibrated the IRI model by using the new calibrated coefficients of the rutting and alligator cracking models, also varied site factor was used to run the MEPDG. NMDOT used same procedure that was used for the previous models to calibrate the Site Factor (SF). Since several site factor values were employed, several SSE and MRE were computed. The minimum SSE and MRE indicated the successful site factor. The Site Factor values, SSE, and MRE are given in Table 2-15. The predicted versus measure IRI values before and after calibration are plotted in Figures 2-8 and 2-9, respectively.

The validation was performed based on the traditional approach. For this reason, SHRP 1112, SHRP 2007, NMDOT 5, NMDOT 6, and NMDOT 10. The MEPDG was run for those sections with the calibrated IRI model. The model was considered valid although the SSE increased for section NMDOT 6.

Table 2-15: SSE and MRE of the Different ‘’ Site Factors’’

Set Numbers	Site Factor	SSE	MRE
1	Default	265,638.14	4.96
2	0.001	308,583.03	5.34
3	0.015	268,903.98	4.99
4	0.1	919,555.57	9.22
5	1	101,768,736.60	97
6	0.01	278,288.97	5.07

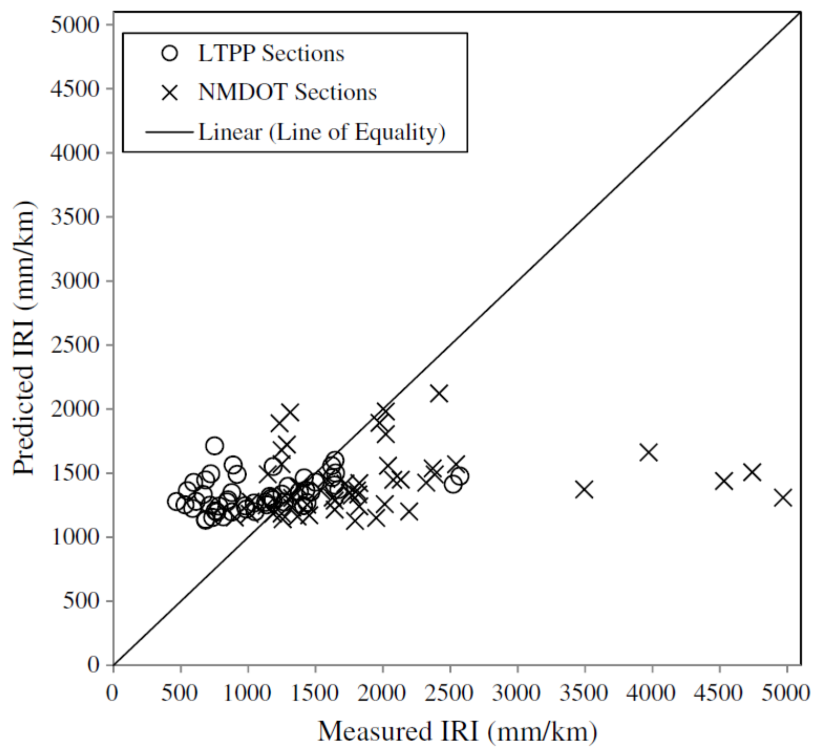


Figure 2-8: Predicted versus Measured IRI Distresses data before the Calibration

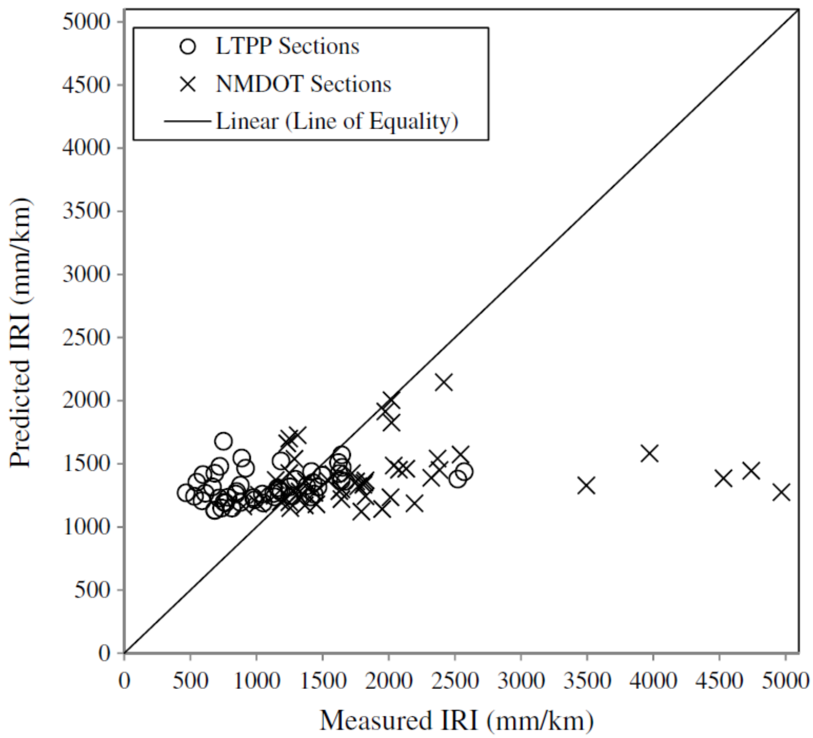


Figure 2-9: Predicted versus Measured IRI Distresses data after the Calibration

2.4.3 Washington State Local Calibration

Washington State Department of Transportation (WSDOT) started working on evaluating and calibrating the performance models of the released MEPDG in 2004. First, WSDOT calibrated the rigid portion of the MEPDG version 0.6 in 2005 (Jianhua, et al., 2009). At that time, the major findings of the software deficiencies were submitted to refine the software functions and predictabilities. Later in 2007, the new MEPDG version 1.1 was released, and many of those deficiencies were corrected (AASHTO, 2008). Thus, WSDOT employed the MEPDG1.1 to calibrate the flexible pavement performance models as a first step of Mechanistic Empirical (ME) pavement design implementation. The calibration process of the flexible portion is summarized in the following pages.

2.4.3.1 Local Calibration Data Assembly

WSDOT decided to use data of pavement sections in Washington State to calibrate the transfer functions since sufficient structural, performance, climatic, and traffic data were available in the WSDOT PMS database. The pavement sections were selected to be located in three climatic regions: Eastern Washington, Western Washington, or mountains pass. Most WSDOT pavements were constructed on subgrade soil with resilient modulus (Mr) from 15,000 to 17,500 psi in spite of some sections have a modulus as low as 5,000 psi (Jianhua, et al., 2009). Based on traffic level, the road sections used for calibrations were divided into three categories as follow:

- High Traffic Level (AADTT>2000)
- Medium Traffic Level (AADTT 200-2200)
- Low Traffic Level (AADTT < 200)

Eighteen pavement sections were nominated for the calibration and the validation process. However, ten of them have light traffic volume because of their location in sparsely populated areas, so they were disregarded. The pavement sections with medium traffic levels located in Western and Eastern Washington State were selected for the calibration (Jianhua, et al., 2009). All other sections were used for the validation since WSDOT intended to use the traditional approach.

The structural data of layer thicknesses, material and asphalt concrete type were collected from Washington State Pavement Management System (WSPMS) (Pierrce, et al., 2008). However, the asphalt binder and the aggregate gradation were obtained from the

WSDOT Standard Specifications for Road, Bridge and Municipal Construction (WSDOT, 2006). In addition, the performance data of the selected sections were assembled from the WSPMS too.

The annual average daily truck traffic (AADTT) and the traffic growth were collected from the Washington State Traffic Data Office. The axle load spectrum was obtained after conducting the cluster analysis. Thirty-eight WIM stations in different locations of the Washington state were selected to obtain the axle load spectrum data. When the load spectra patterns were compared with the MEPDG defaults, it was found one representative load spectrum can represent the traffic for the entire state (Jianhua, et al., 2009). Therefore, it was used in the calibration and validation process.

2.4.3.2 Local Calibration of the Flexible Pavements

WSDOT calibrated the MEPDG performance models according to the recommended practice for local calibration of ME pavement design guide (Quintus, et al., 2007). At the beginning, WSDOT performed bench testing approach to identify the prediction reasonableness of the models and the need of the calibration. By varying the design parameters, such as climatic, traffic, structural thickness, and materials properties, the reasonableness was checked by comparing the outputs with the acceptable pavement behaviors at different conditions, as shown in Table 2-16 (Jianhua, et al., 2009).

It was found, the MEPDG1.1 software predicts reasonable the distresses for Washington State. However, the calibration was needed since the defaults models underestimated the fatigue cracking, longitudinal cracking, and rutting. Thus, the calibration was being focused on fatigue damage, longitudinal cracking, alligator cracking, and rutting models.

Table 2-16: Input Sensitivity for Flexible Pavement Distress Conditions

Input Factor	Longitudinal Cracking	Transverse Cracking	Alligator Cracking	AC Rutting	IRI
Climate	Medium	High		High	High
PG Binder	High	Medium	Medium	Medium	
AC Thickness	High	Medium	Medium	High	
Base Type	Medium			High	Medium
AADTT	Medium			High	Medium
AC Mix Thickness			High		
Soil Type	Medium				

A sensitivity analysis was conducted to explore the effects of the calibration coefficients on the models. WSDOT used Equation 2-13 for this purpose:

$$\boldsymbol{E_{distress}^{C_i}} = \frac{\boldsymbol{\partial(distress)/distress}}{\boldsymbol{\partial\ (C_i)/C_i}} \quad \text{Equation 2-13}$$

Where the parameters are defined as:

$E_{distress}^{C_i}$ = elasticity of calibration factor C_i for the associated distress condition,

$\partial(distress)$ = change in esitmated distress associated with change in factor C_i

$\partial(C_i)$ = change in the calibratiobn factor C_i

$distress$ = estimated distress using default calibration factors

C_i = defualt value of C_i

The elasticity value indicates the influence of the coefficient. For example, the zero value indicates that there is no influence on the model. However, the positive value implies that the overestimation as the factors increase, but the negative elasticity value indicates underestimation as the factors increase (Greene, 2003). Numerous MEPDG runs were performed to compute the elasticity for each model.

The calibration sections were used for this reason. The results of this process indicted that the asphalt fatigue model should be calibrated before the longitudinal and fatigue cracking models. Moreover, the calibration coefficients of the rutting model β_{r2} and β_{r3} should be adjusted before β_{r1}. Table 2-17 shown the elasticity computed for each calibration factor.

Table 2-17: Elasticity of MEPDG Calibration Factors for WSDOT Flexible Pavements

Calibration Factor	Elasticity	Related Variable in MEPDG Distress Models
AC Fatigue		
Bf1	-3.3	Effective binder content, air voids, AC thickness
Bf2	-40	Tensile strain
Bf3	20	Material stiffness
Longitudinal Cracking		
C1	-0.2	Fatigue damage, traffic
C2	1	Fatigue damage, traffic
C3	0	No related variable
C4	0	No related variable
Alligator Cracking		
C1	1	AC Thickness
C2	0	Fatigue damage
C3	0	No related variable
Rutting		
Br1	0.6	Layer thickness, layer reliant strain
Br2	20.6	Temperature
Br3	8.9	Number of load repetitions
IRI		
C1	N/A	Rutting
C2	N/A	Fatigue cracking
C3	N/A	Transverse cracking
C4	N/A	Site factor

Then, the MEPDG1.1 was run with default calibration factors. The predicted distresses were compared with the measured performance data. After that, the coefficients were adjusted in order decreasing order based on their sensitivity. The process was repeated for each calibration section until the convergence occurred. The final calibration factors are tabulated in Table 2-18.

Table 2-18: Washington State Final Calibration Factors

Calibration Factor	Default	Calibrated
AC Fatigue		
Bf1	1	0.96
Bf2	1	0.97
Bf3	1	1.03
Longitudinal Cracking		
C1	7	6.42
C2	3.5	3.596
C3	0	0
C4	1000	1000
Alligator Cracking		
C1	1	1.071
C2	1	1
C3	6000	6000
AC Rutting		
Br1	1	1.05
Br2	1	1.109
Br3	1	1.1
Subgrade Rutting		
Bs1		0
International Roughness Index (IRI)		
C1	40	N/A
C2	0.4	N/A
C3	0.008	N/A
C4	0.015	N/A

It is important to mention that the IRI model was not calibrated. However, the IRI distresses were plotted after all other models were calibrated. Subsequently, the calibrated factors were validated by running the MEPDG1.1 for the validation sections. It was found that the calibrated models predicted reasonably well the pavement performance. Thus, the calibrated models are valid.

2.4.4 Local Calibration of MEPDG in Iowa

On 2005, Iowa Department of Transportation (IDOT) decided to implement the Mechanistic-Empirical Pavement Design Guide (MEPDG) to design the new flexible and rigid pavements (Kim, et al., 2010). Since this research focused only on the design of the new flexible pavement in New York State, the calibration of new flexible pavement performance models will be explained.

2.4.4.1 MEPDG General Implementation Plan for Iowa

First, a general strategic plan was conducted to demonstrate the benefits of the ME pavement design in Iowa. Therefore, the MEPDG1.1 inputs were examined and the sensitivity analyses were employed. Then, the national calibrated performance models of the MEPDG1.1 were investigated to explore the accuracy of the distresses predictions for Iowa conditions (Ceylan, et al., 2006).

As a part of this plan, Iowa State University (ISU) performed a research to evaluate the type, accuracy, and timeliness of information collected in the IDOT's Pavement Management Information System (PMIS) data regarding the MEPDG1.1 inputs and outputs information, and to determine whether the nationally calibrated distress models provide acceptable predictions and the desired accuracy (Kim, et al., 2010).

To achieve the research's aim, the PMIS data of IDOT for the interstate and primary roads had been retrieved from the database and evaluated. Only sixteen pavement sections were selected. Five out of sixteen sections were flexible pavements. It is important to mention that these sections were not used in the national calibration of the MEPDG1.1 performance models. Therefore, the actual performance data of the selected sections were used to verify the performance models of MEPDG 1.1.

It was found that PMIS of IDOT should be converted to the same measured unites system of the MEPDG 1.1 outputs, also ISU research team found that only total rutting data was available in Iowa. Since the MEPDG 1.1 predicts the rutting for each layer (AASHTO, 2008); thus, the research team expected some difficulties in local calibration effort.

The average annual daily traffic of the base year was obtained from the PMIS of IDOT. The base year is defined as the first calendar year that the road segment under the design is opened to the traffic (Kim, et al., 2010). However, the other defaults traffic inputs were used in the MEPDG1.1 due to the unavailability in the PMIS of IDOT. The materials inputs were extracted from the PMIS of IDOT and several other researches reports (Kim, et al., 2005)

In addition, the team extrapolated the weather data from that provided by weather station away from the calibration pavement sections. The MEPDG 1.1 has the flexibility of selecting and extrapolating the weather station by importing them from the MEPDG database in the software (AASHTO, 2008). The team used the default water table level which was suggested by the computer software for each segment.

After all the required inputs of the MEPDG were compiled, the verification of the performance models was achieved by simulating the design solutions of the selected pavements sections by the MEPDG1.1 with the default calibration factors. Then, the predicted distresses of each selected section were compared with the actual performance distresses. The statistics approach was implemented in this step to explore the considerable difference between the predicted versus the actual distresses. This approach has been recommended by NCHRP research (1-40B) related to the local calibration and verification of the MEPDG performance models (AASHTO, 2010). It is known that the statistics technique has been conducted by several highway agencies to verify and calibrate the performance models of the MEPDG1.1 (Michael, et al., 2009).

After plotting the predicted versus actual measured distresses, the location of the observations points was compared and investigated. Then, the null hypothesis was conducted at 95% confidence level. Figure 2-10 shown the null hypothesis results and comparison of the rutting distresses, while Figure 2-11 shown the null hypothesis results and the comparison of the IRI distresses. The null hypothesis was rejected for both rutting and the IRI distresses. This indicates that there was a significant difference between the predicted and the measured distresses. Thus, the performance models should be recalibrated.

The distresses of the longitudinal cracking were excluded from the verification process due to the inaccuracy of the model. Only the IRI and the rutting models were verified.

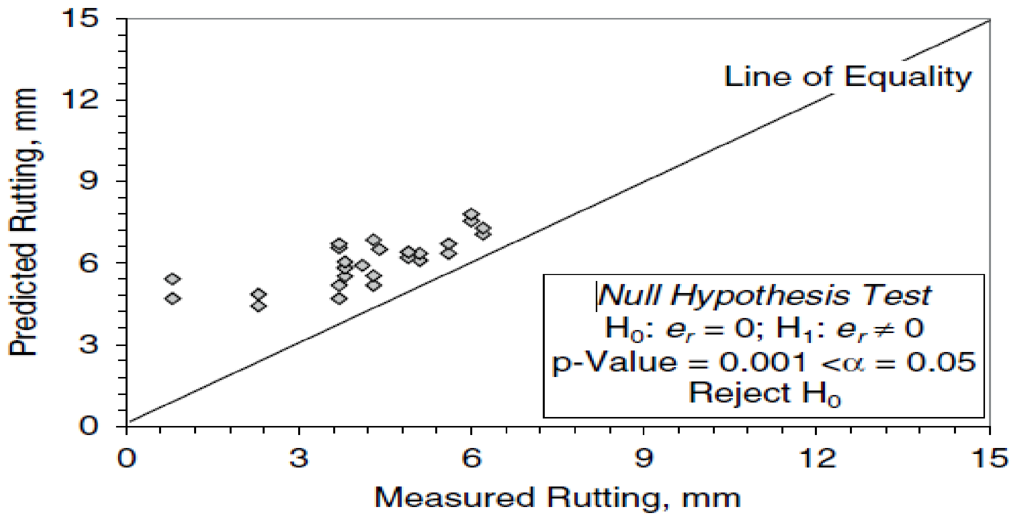


Figure 2-10: Null Hypothesis Test Results and Predicted Vs Measured Rutting

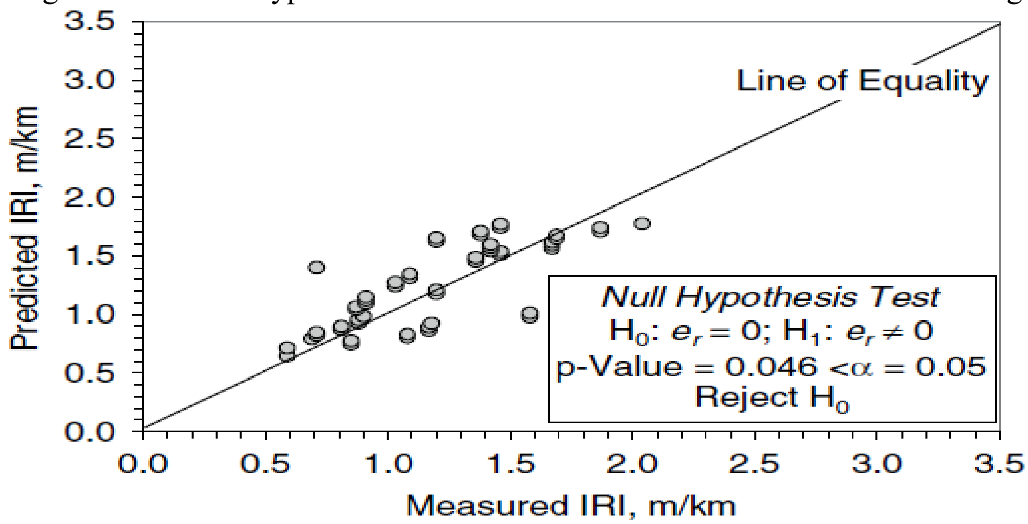


Figure 2-11: Null Hypothesis Test Results and Predicted Vs Measured IRI

2.4.4.2 Assessment the Accuracy of the Performance Models for Iowa

Since the predicted distresses were different from the measured distress, the local calibration was recommended. To achieve it, a newer version of ME pavement design software (DARWin ME), that replaced the MEPDG version 1.1, was used (Kim, et al., 2013).

Thirty-five flexible pavement sections in Iowa were selected; twenty-five sections were used for model calibration while ten sections were used for model verification. It is clear that traditional approach in verifying the performance models was used. However, according to the Calibration Manual (AASHTO, 2010) this technique should not be used for such a small sample size; Jackknife testing should be used instead.

To improve the confidence in the process, the pavement sections which had been used for MEPDG evaluation were not used anymore, despite the relative small total number of sections. The team tried to select sections at different locations with a wider spectrum of traffic characteristics in order to obtain calibration coefficients more representatives for the entire Iowa State. However, it was found that the majority of flexible pavements located on roads with low traffic volume, with an AADTT of less than 500.

The main source of the structural and materials inputs was the PMIS IDOT database. In addition, some of the previous reports and researches were used to obtain the required data (IDOT, 2014); (Halil Ceylan, 2009). For HMA materials properties in Iowa, IDOT HMA mix design database containing more than 4000 construction researches were reviewed and utilized. In cases where the HMA mix design information was not available for specific sections selected, the asphalt binder grade was determined from the LTPPBind program and the typical aggregate gradation of Iowa HMA mixture was obtained by averaging HMA aggregate gradation reported in the IDOT HMA mix design database (Halil Ceylan, 2013).

Then, the sensitivity analysis was conducted to explore the effect of changing the calibration coefficients on predicting the distresses, also to reduce the range of subsequent calibration coefficients. The sensitivity analysis saves the time and effort to achieve the aim of the calibration. One flexible pavement section in Iowa was used to perform the sensitivity analysis.

After that, multiple runs of DARWin ME were performed. The predicted distresses were plotted versus the corresponding measured distresses later. The statistical approach was employed to explore the relation between the predicted and the measured distresses. The high values of bias and standard error indicated the need of new runs of DARWin ME design cases with the amended calibration coefficients. The process continued until low values were obtained for the bias and standard errors. Thus, the obtained local calibrated coefficients are tabulated in Table 2-19.

Table 2-19: Calibration Coefficients of Flexible Pavement for Iowa

Model Type	Regression Coefficient
HMA Rut	β1= 1
	β2 = 1.15
	β3 = 1
Granular Base Rut	βgb= 0.001
Subgrade Base Rut	βsg=0.001
Longitudinal Cracking	C1 = 0.82
	C2 = 1.18
	C3 = 1000
Fatigue Cracking	C1 = 1
	C2 = 1.18
	C3 = 6000
Thermal Cracking	K = 1.5
IRI	C1 = 40
	C2 = 0.4
	C3 = 0.008
	C4 = 0.015

The rutting models were recalibrated due to the plots trend between predicted versus measured rutting. The plot of the predicted versus measured longitudinal cracking showed there was unsymmetrical trend related to the line of equality, so the model had to be recalibrated.

After plotting the predicted versus measured fatigue cracking, a symmetrical trend around the line of equality was obtained. Therefore, it was concluded that the fatigue cracking model did not require a recalibration. Figure 2-12 shown the predicted versus measured fatigue cracking when the national calibrated model was used.

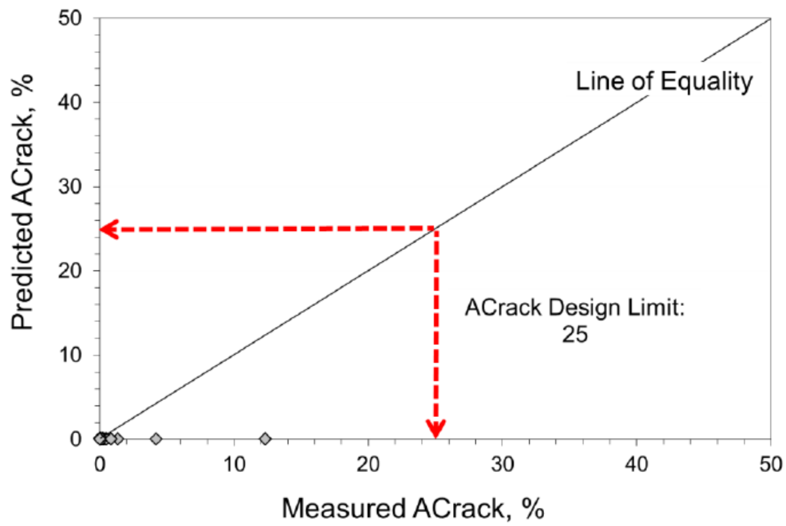


Figure 2-12: Predicted Vs Measured Fatigue Cracking Nationally Calibrated

The thermal cracking model was not calibrated to the Iowa conditions. As results, the research team found from previous research reports that there is no or little thermal cracking in flexible pavement in Iowa, mainly because the proper binder grade was used (Hall, et al., 2011); (Schwartz, 2012).

The predicted and measured IRI values showed a symmetrical trend around the line of equality. Therefore, the team concluded that there is no need to recalibrate the IRI model. Figure 2-13 shown the predicted versus the measured IRI distresses when national calibrated models were used.

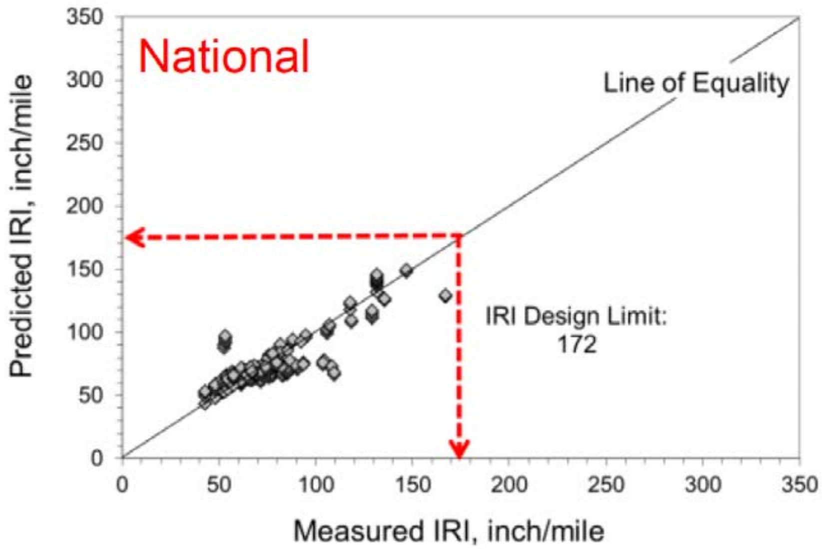


Figure 2-13: Predicted Vs Measured IRI Distresses Nationally Calibrated

The process concluded with the verification step. For this, the recalibrated models were verified by running DARWin ME with the local calibration coefficients for validation sections. The predicted distresses were plotted versus the measured distresses. A symmetrical trend between the predicted and the measured was found so the validation was successful. Therefore, the local calibrated coefficients were adopted for Iowa.

2.4.5 Local Calibration of MEPDG in Wisconsin

The calibration effort of the climate is rigorously different among those three states. Therefore, in this calibration the climate models were made based on Wisconsin region. The load related cracking models which include (longitudinal and alligator cracking) were calibrated based on cumulative damage concept; the predicted percentage damage was compared to actual percent damage. Moreover, the best fit was used to minimize the difference between MEPDG prediction and observed performance. In the longitudinal fatigue cracking model, the damage transfer function was used (Myungook, et al., 2007).

Microsoft solver program was used to evaluate the regression coefficients of damage transfer function, however, the default values of these coefficients were used in this study. After nine runs of each section, outputs were plotted by the MEPDG for different combination of calibration factors, these outputs were evaluated by Sum of Squared Errors (SSE).

2.4.6 Local Calibration of MEPDG in Michigan and Ohio

The calibration efforts of Michigan and Ohio were limited due to the budget, time limitation, and lack of information. Therefore, the calibration coefficients of Wisconsin were used in prediction distresses for Michigan and Ohio, however, the climate model was specified basis on the region of the study (Myungook, et al., 2007).

The MEPDG 1.1 was run to predict the pavement performance for each state. The predicted longitudinal cracking model for Michigan was compared with field collected performance data; it showed that prediction model was poorly for Michigan as shown in Figure 2-14.

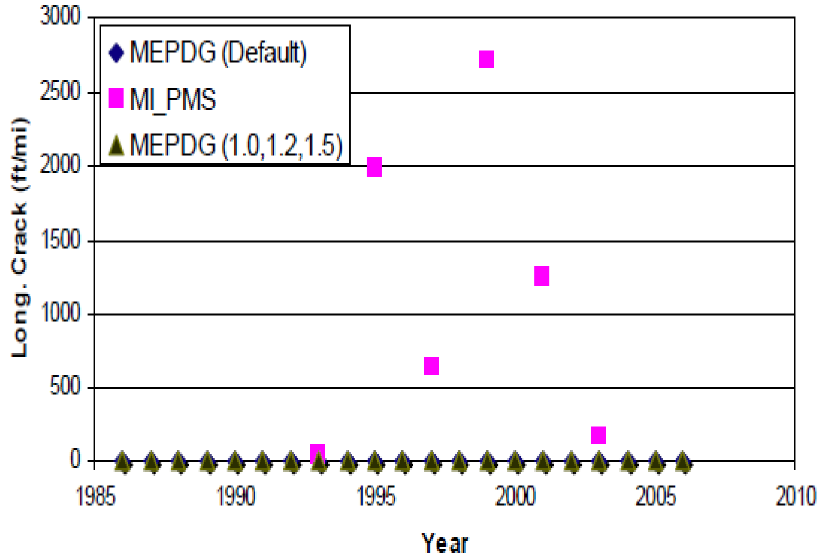


Figure 2-14: Longitudinal cracking in Michigan

Unlike with longitudinal cracking, the calibrated MEPDG1.1 model predicted alligator cracking well for Michigan. Figure 2-14 showed that the calibrated prediction model can minimize the difference between the prediction and the field-collected data. Figure 2-15 suggests that the prediction using the default calibration model is better than that obtained using the calibrated values. However, if the deterioration rate of field data is considered, a prediction using calibrated values may match field data better than a prediction obtained using default values.

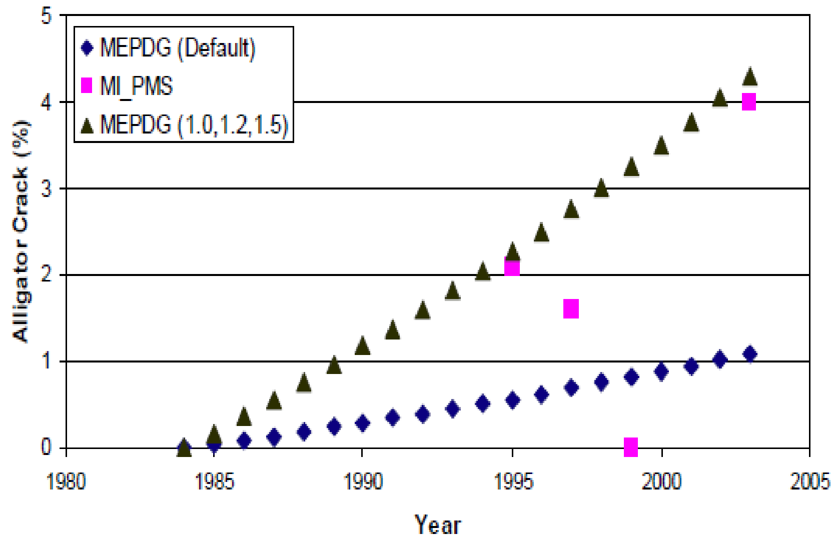


Figure 2-15: Alligator cracking in Michigan

Furthermore, if the deteriorated data field were considered in this model, the calibrated model may match the performance field data. The calibration effort for Ohio was difficulty to be predicted for the following reasons:

- The collected data of Ohio were not fit to be calibrated. The longitudinal cracks stay constant at zero, and then they rise quickly after couple years as shown in Figure 2-16
- The collected data of the alligator cracking were not adequately. Because the alligator cracking was increasing rapidly as shown in Figure 2-17.

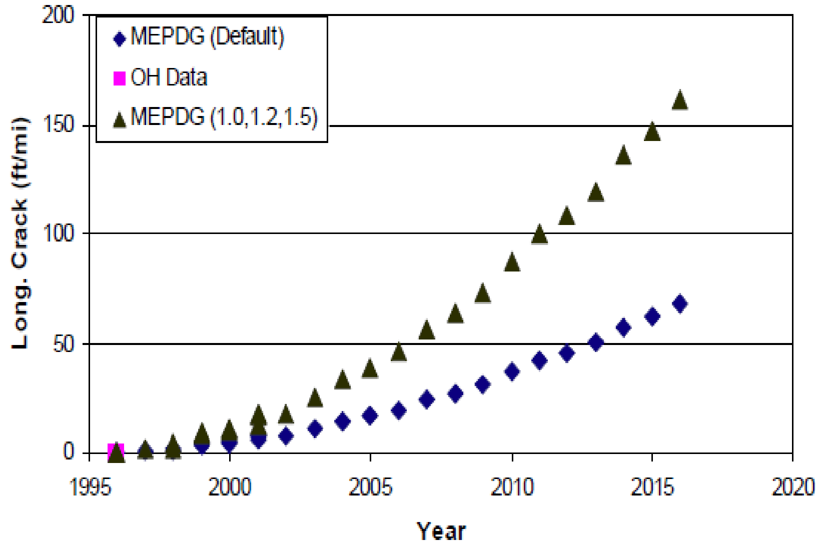


Figure 2-16: Longitudinal cracking in Ohio

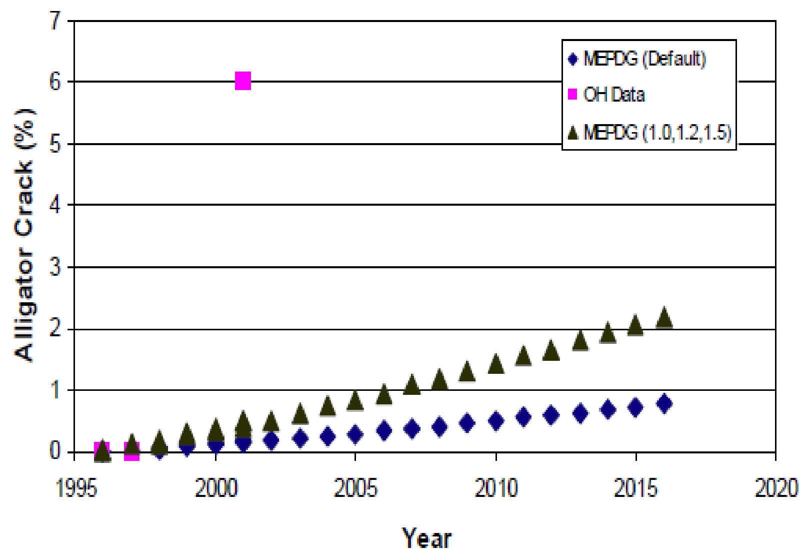


Figure 2-17: Alligator Cracking in Ohio

2.5 The ME Pavement Design Implementation in the United States

The Mechanistic-Empirical Pavement Design Guide (MEPDG) has not been fully implemented by all the states although it has been officially adopted by AASHTO, because the implementation process is a sophisticated and requires significant data and efforts.

Some of the national level and state highway agencies are in the process to implement the MEPDG in the design of new flexible pavement, but enhancing the performance model has not been performed yet. For example, Virginia Department of Transportation (VDOT) conducted a study on sensitivity analysis of the design materials inputs and their effects on the prediction of the rutting and fatigue cracking models to the local conditions of Virginia, the study concluded, design inputs database should established, also the models should be recalibrated before implementing the MEPDG (Diefenderfer, 2010). Later, VDOT performed a study to develop a design inputs catalog to be used in the MEPDG (Apeagyei, et al., 2011). On 2010, Nebraska Department of Roads (NDOR) initiated the process of implementing the MEPDG to design the flexible pavements; NDOR developed a database of dynamic modulus, resilient modulus, and creep compliance of various pavement materials, the database contains the three levels design inputs (Im, et al., 2010). Moreover, the state of California, Florida, and Mississippi are in the process of implementing the MEPDG (Hall, 2012). North Dakota State University initiated the efforts to understand the MEPDG, so North Dakota Department of Transportation (NDDOT) will be able to design the flexible pavement by using AASHTOWare, but the local calibration of the performance models has not been performed yet (Lu, et al., 2014).

Several state highway agencies reports either full or partial local calibration of the performance models; for instance, Texas developed a flexible pavement local material database; then, the MEPDG 1.1 was used to calibrated the permanent deformation model locally for five regions in Texas, and for Texas in general (Banerjee, et al., 2010). In Arkansas, the MEPDG 1.1 was used to calibrate the alligator and longitudinal cracking models, but the transverse cracking model was not calibrated due to the lack of the measured distresses. Also, the IRI model was not calibrated since the IRI is the function of other models outputs (Hall, et al., 2011). Montana also employed the MEPDG 1.1 to calibrate the performance models of the rutting, alligator cracking, transverse cracking, and the IRI models; the longitudinal cracking model was not calibrated due to the lack of data (Von Quintus, et al., 2007). Minnesota Department of Transportation (MNDOT) sponsored a study which was performed by the University of Minnesota to calibrate the rutting model; the calibration led to improvement of the rutting model in MEPDG1.1 (Hoegh, et al., 2010). Arizona Department of Transportation (ADOT) sponsored a research to assess the available distresses data and collection procedure for the sake of obtaining the required data to conduct the local calibration (Mamlouk, et al., 2010). Later another research was performed to calibrate the distress models of fatigue and longitudinal cracking (Mena, et al., 2010). Utah Department of Transportation (UDOT) used the MEPDG 1.1 to calibrate the performance models of the flexible pavement distresses, only the rutting model was locally calibrated due to the fair prediction of other performance models with the global calibration factors (Darter, et al., 2009). Missouri Department of Transportation (MODOT) developed the local calibration for the transverse cracking and total rutting models but the alligator cracking model was not recalibrated due to the fair prediction when MEPDG 1.1 was run with the global calibration factors (Mallela, et al., 2009).

As a part of the implementation efforts in Idaho, Idaho Department of Transportation (IDOT) performed a comprehensive research to implement the ME pavement design by using DARWin. Thus, the local calibration effort was conducted for all the flexible pavement distresses models and the IRI (Bayomy, et al., 2012). Colorado Department of Transportation (CDOT) calibrated the total rutting, alligator cracking, transverse cracking, and the IRI models (Mallela, et al., 2013). It is important to mention that AASHTOWare has not been used by any national level and state highway agency yet to perform the local calibration. Also, only Missouri and Indiana have successfully implemented the MEPDG for the design of new flexible pavement structures (Hall, 2012).

Chapter 3

Enhancing the Performance Models of AASHTOWare Pavement ME

3.1. Overall Concept for Enhancing the Performance Models

This chapter explains the used procedure to calibrate the performance models of AASHTOWare Pavement ME 2.1. The work of Momin (2011), was used as the primary source to assemble the required data. Then, the assembled data were used to calibrate the performance models so the bias between the predicted and the measured performance data was eliminated or reduced. The performance models were developed based on AASHTO ME local calibration guide that was developed under NCHRP Research 1-40B.

It is important to mention that the local calibration deals only with the calibrated coefficients and exponents of the distresses models. The local calibration cannot change the supported mathematical equations that are employed by the performance models. Examples of the supported models as the follow:

- Structural Response Models
- The enhanced Integrated Model
- Time Dependent Property Model

3.2. Data Assembly

The process of assembling the essential data for calibration was done by following two stages. The first stage was evaluating the available data in the Pavement Management System (PMS) unit of New York State Department of Transportation (NYSDOT) to determine if such data can be used for the local calibration (NYSDOT, 2002). It was found that complete calibration data were not available even for a single flexible pavement section. Thus, the use of PMS data was abandoned.

Then the Long Term Pavement Performance Database (LTPP) was reviewed to assure the following data are available for new flexible pavements in North Eastern (NE) of the United States.

- Traffic Data
- Structural Data and Materials Properties Data
- Climatic Data
- Distresses Data

Unfortunately, there no GPS 1 and GPS 2 pavements sections in New York State were built as part of the LTPP program (Abdullah, et al., 2014). GPS pavement sections are monitored pavement sections that had been built 15 years prior to the implementation of the LTPP program (Elkins, et al., 2003). GPS 1 sections are flexible pavements with unbound granular base while GPS 2 are flexible pavement sections with bound base layers.

Since no complete calibration data was available for new flexible pavement structures in New York State, it was decided to use collected data from the LTPP program on flexible pavement sections in the neighboring states. This approach is reasonable since the states in the NE region of the United States have very similar climatic conditions and use similar pavement materials and structural configurations in the construction of flexible pavements.

It is important to mention that Momin (2011) conducted the regional calibration of the distress models in MEPDG 1.1 by using LTPP sites for 18 flexible pavement sites in the NE region of the United States. Momin (2011) performed an extensive effort to assemble vital data and create input files for the MEPDG 1.1 software. His effort was considered for this work, the extracted data from Momin's effort are tabulated in the following Appendices:

- Appendix A:Extracted Long Term Pavement Performance (LTPP) Traffic Design Inputs
- Appendix B: Extracted Long Term Pavement Performance (LTPP) Structural and Materials Properties Design
- Appendix C: Extracted Long Term Pavement Performance (LTPP) Performance Data

3.2.1. Selection of the LTPP Sites

Twenty-nine LTPP monitored flexible pavement sections in the NE region of the United States have very similar conditions as the flexible pavements sections in New York State. Only eighteen LTPP flexible pavement sections have the complete required data for calibration (Momin, 2011). Table 3-1 lists the LTPP pavement sections used for calibration in this research.

Table 3-1: North Eastern Selected LTPP Pavement Sections

State Code	State	SHRP ID	Total Lanes	Structural Type	Construction Date	
					1	2
9	Connecticut	1803	2	Flexible	7/1/1988	1/17/1995
23	Maine	1001	4	Flexible	7/1/1988	6/6/1995
23	Maine	1009	2	Flexible	7/1/1988	8/22/1993
23	Maine	1028	2	Flexible	7/1/1988	5/12/1992
25	Massachusetts	1003	2	Flexible	6/1/1988	6/7/1988
34	New Jersey	1003	4	Flexible	8/1/1988	4/8/1994
34	New Jersey	1011	4	Flexible	7/1/1988	4/28/1998
34	New Jersey	1030	4	Flexible	12/1/1988	2/24/1991
34	New Jersey	1031	4	Flexible	7/1/1988	4/4/1996
34	New Jersey	1033	4	Flexible	7/1/1988	9/11/1997
34	New Jersey	1034	4	Flexible	12/1/1988	-
34	New Jersey	1638	4	Flexible	12/1/1988	-
42	Pennsylvania	1597	2	Flexible	8/1/1988	6/12/1990
42	Pennsylvania	1599	2	Flexible	8/1/1988	6/1/1999
50	Vermont	1002	2	Flexible	8/1/1988	-
50	Vermont	1004	2	Flexible	8/1/1988	10/6/1998
50	Vermont	1681	2	Flexible	6/1/1989	9/8/1991
50	Vermont	1683	2	Flexible	6/1/1989	9/23/1991
Missed Traffic Data and Unreliable Performance Data						
23	Maine*	1012	4	Flexible	7/1/1988	-
23	Maine*	1026	2	Flexible	7/1/1988	9/26/1996
25	Massachusetts*	1002	6	Flexible	6/1/1988	6/5/1988
25	Massachusetts*	1004	4	Flexible	8/1/1988	6/1/2001
33	New Hampshire*	1001	4	Flexible	8/1/1988	8/1/2001
36	New York*	1008	4	Flexible	5/1/1989	8/25/1989
36	New York*	1011	4	Flexible	6/1/1988	9/14/1993
36	New York*	1643	2	Flexible	5/1/1989	10/12/1989
36	New York*	1644	2	Flexible	5/1/1989	6/19/1996
42	Pennsylvania*	1605	2	Flexible	8/1/1988	6/14/1995
42	Pennsylvania*	1618	2	Flexible	12/1/1988	8/27/1989

3.2.2. Traffic Data Assembly

Traffic data are necessary to run the AASHTOWare Pavement ME 2.1 so that the traffic loads during the design life is simulated for the sake of distresses prediction. The traffic inputs of the AASHTOWare are summarized in Figure 3-1.

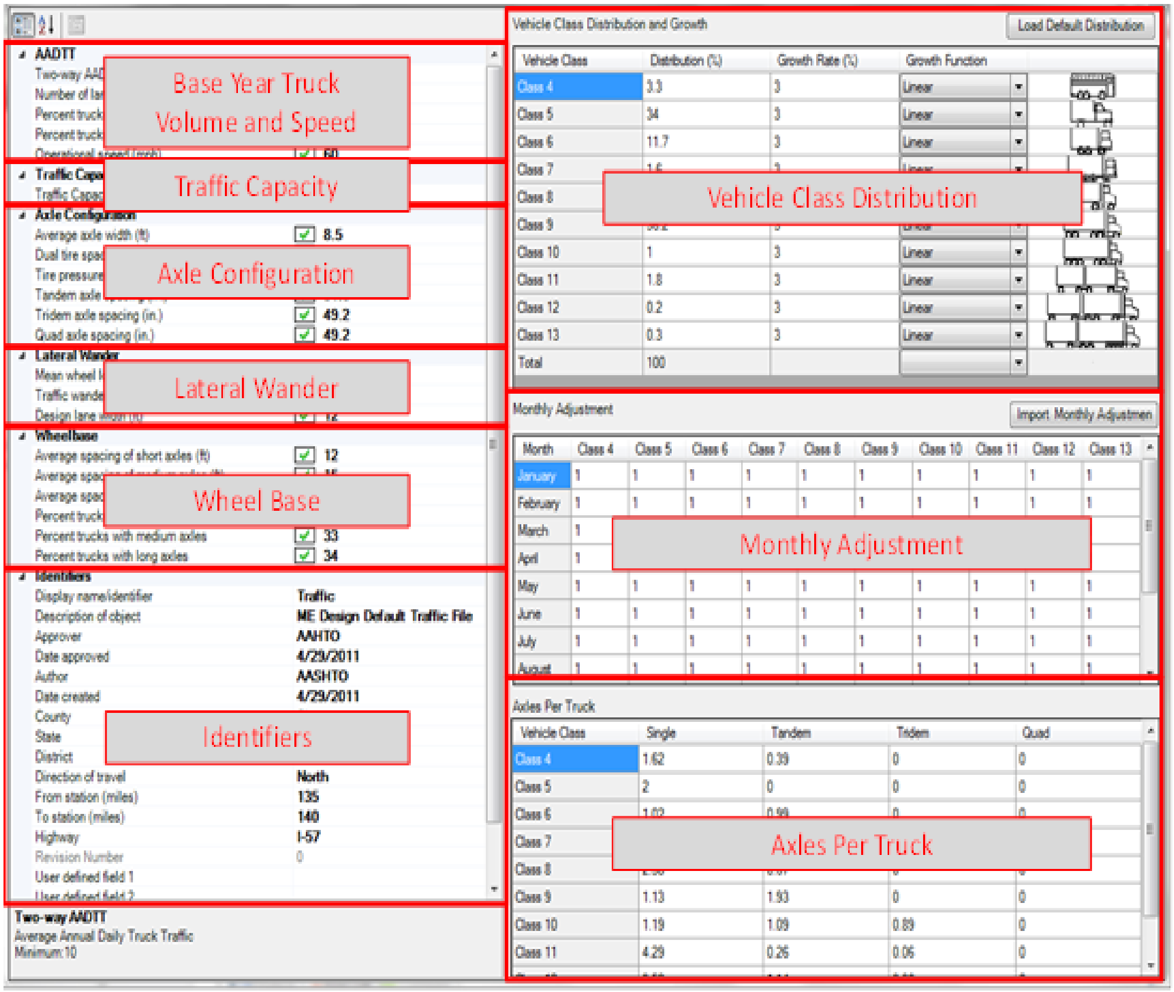


Figure 3-1: AASHTOWare Pavement ME 2.1 Traffic Inputs

For each selected LTPP site, the required traffic data at the base year were extracted from the traffic data tables assembled by Momin (2011). The extracted traffic data are tabulated in Appendix A. The traffic inputs are defined as the follow:

- *Average Annual Daily Truck Traffic (AADTT)*: It is defined as the total volume of truck traffic recorded on a highway segment during an entire year, divided by the number of days in the year (FDOT, 2002).

- *Vehicle Class Distribution (VCD):* It represents the percentage of each truck class (class 4 to class 13) in the total number of trucks. Federal Highway Administration (FHWA) has classified the vehicles into thirteen classes; out of each nine are truck classes, as illustrated in Figure 3-2.
- *Monthly Adjustment Factor (MAF):* Monthly adjustment factors represent the proportion of the annual truck traffic for a given truck class that occurs in a specific month. The monthly distribution factor for a vehicle class, for a specific month, is computed by dividing the monthly truck traffic for that vehicle class in that month by the total truck traffic for that vehicle class for the entire year (AASHTO, 2008).
- *Number of Axles per Truck:* It indicates the average number of axles for each truck class, and for each axle type (Single, Tandem, Tridem, Quad).
- *Axle Load Spectra:* It represents the axle load distribution for each axle type, for each month of the year and each vehicle class. It is the percentage of the total axle application within specified load intervals with respect into the axle type and vehicle class (Romanoschi, et al., 2011).
- *Growth Rate and Function:* Growth Rate represents the annual rate of truck traffic growth over time in the exponential growth model. The extracted growth rate for each of the eighteen LTPP selected sections was computed by Momin (2011), from the recorded truck traffic during the entire monitoring period and it is given in Table 3-2. AASHTOWare uses a single growth rate for all vehicle classes.

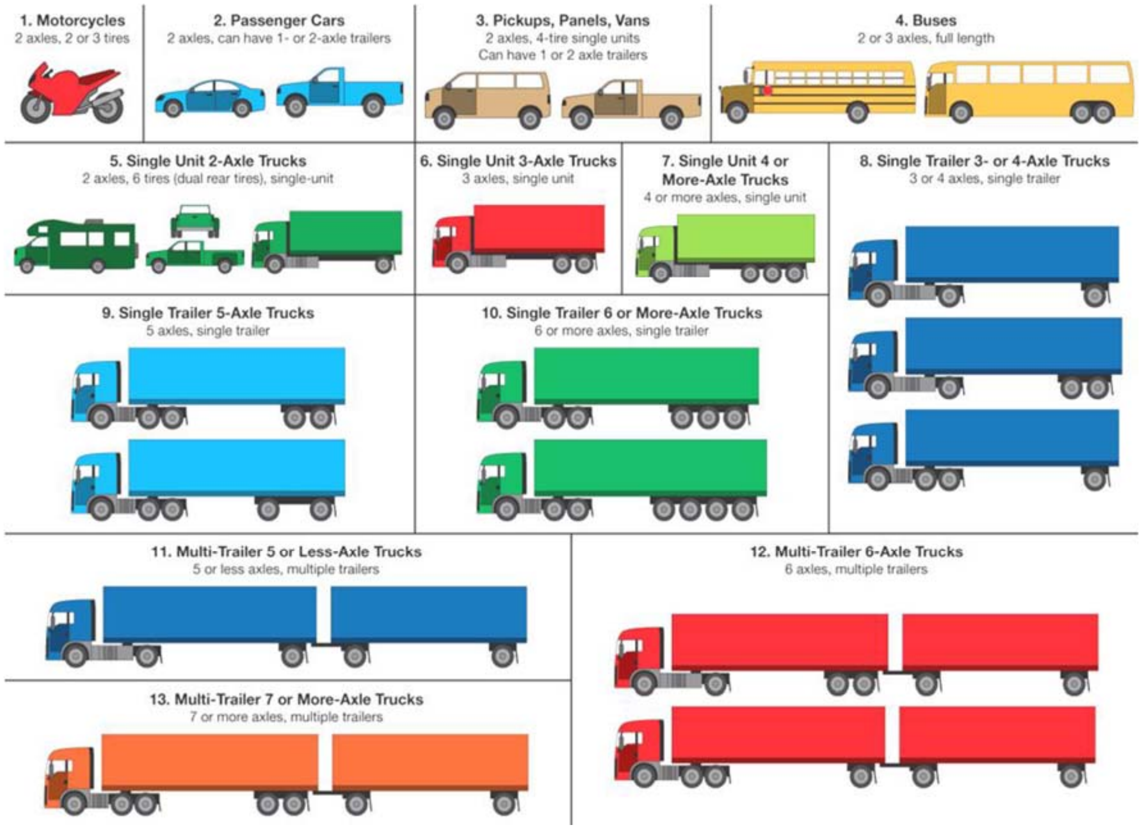


Figure 3-2: FHWA Vehicle Classification

Table 3-2: Exponential Traffic Growth Rate for the Selected LTPP Sections

SHRP ID	Traffic Growth Rate %	SHRP ID	Traffic Growth Rate %
091803	6.57	341033	-21.86
231001	1.15	341034	-0.83
231009	0.49	341638	-0.92
231028	5.9	421597	4.68
251003	-1.09	421599	-1.39
341003	-14.79	501002	3.33
341011	-6.5	501004	1.91
341030	0	501681	17.5
341031	9.59	501683	17.5

Additional traffic inputs are required by the AASHTOWare. Those traffic inputs are rarely available, so the suggested values by the AASHTOWare software are normally used instead. These default values are level 3 design inputs and are defined as:

- *Hourly Adjustment Factors*: It represents the ratio of the truck traffic in a given hour of the day divided by the total daily truck traffic. To ease the computation, it is not used for the design of flexible pavements.
- *Traffic Capacity:* Traffic capacity is an optional setting which allows a cap on forecasted traffic volume based on ME Design's internal capacity calculations which use the models included in the Highway Capacity Manual (HCM) 2000.
- *Axle Configuration*: It defines the inputs of average axle width and axle spacing. Also, dual tire spacing and tire inflation pressure.
- *Lateral Wander*: It includes the inputs of the mean wheel location, traffic wander standard deviation, and design lane width.
- *Wheel Base*: It includes the average spacing of short, medium, and long axles. In addition, it includes the percentage of trucks of those axles.
- *Identifiers:* It includes the source description of the traffic inputs.

3.2.3. Structural Layers and Materials Properties Data Assembly

Since the calibration of performance models relies on runs of the AASHTOWare software for the LTPP pavement sections selected, it is imperative that the structural configuration and material properties of the LTPP sections are used in the runs. Therefore, the required inputs were extracted to be used in the design problems for the sections listed in Table 3-1. The inputs are not existed in the LTPP database, were replaced with AASHTOWare default values. For example, default values were used for indirect tensile strength, reference temperature, and creep compliance, etc. The extracted data from the LTPP database are:

- *Layers Thickness:* The database of the selected LTPP sites contains adequate information regarding the number of the layers, type of the materials, and the thicknesses. Therefore, they were extracted to create AASHTOWare design problems for the selected LTPP sites. The extracted structural data are tabulated in Appendix B.
- *Gradation Data of Aggregate in the HMA*: Since the LTPP database does not contain dynamic modulus data for HMA mixes, as required for Level 1 input in AASHTOWare software, only Level 3 input values (aggregate gradation, binder grade, mix volumetric properties) could be extracted from the LTPP database. For Level 3 inputs, aggregate gradation is used in the Witczak's model to compute the dynamic modulus (E*) of the HMA layers. Figure 3-3 shown an example of the extracted aggregate gradation data for an HMA mix.

Gradation	Percent Passing
3/4-inch sieve	100
3/8-inch sieve	78
No.4 sieve	52
No.200 sieve	5

Figure 3-3: Aggregate Gradation of Surface HMA layer

- *Penetration\Viscosity Grade for Asphalt Binders*: Since the LTPP database contains only viscosity or penetration grades for asphalt binders and not actual viscosity or penetration values, as required for Level 2 design, the viscosity grade of each asphalt mixture was obtained from the LTPP database. The AASHTOWare assigns default values for the viscosity of the asphalt binders if the viscosity or penetration grades are selected. Figure 3-4 shown the screen capture for Level 3 inputs for asphalt binder. The extracted viscosity grades of selected LTPP sites are listed in Table 3-3.

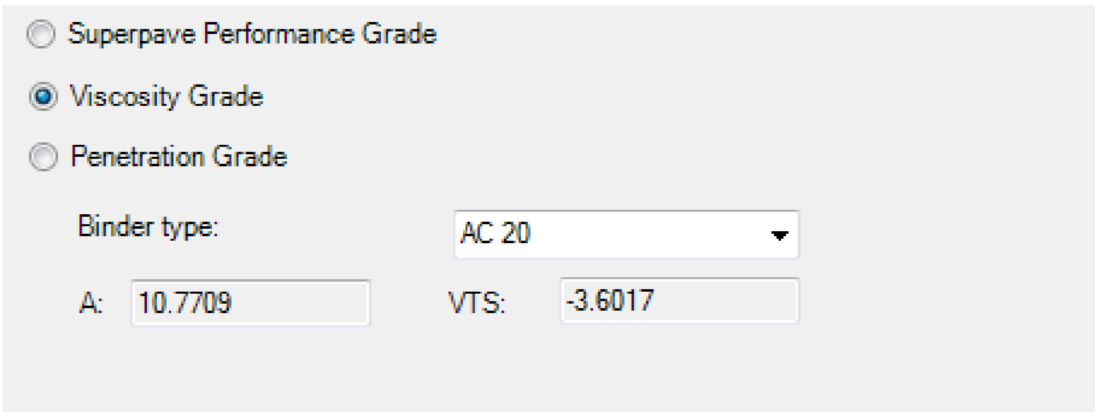


Figure 3-4: Level 3 Design Input Binder

Table 3-3: Viscosity Grades for the Selected LTPP Sites

LTPP Site	Viscosity Grade
091803	AC-20
231001	AC-10
231009	Pen 85-100
231028	AC-10
251003	AC-20
341003	AC-20
341011	Pen 85-100
341030	AC-20
341031	AC-20
341033	AC-20
341034	AC-20
341638	AC-20
421597	AC-20
421599	AC-20
501002	Pen 85-100
501004	Pen 85-100
501681	Pen 85-100
501683	Pen 85-100

- *HMA Volumetric Properties*: AASHTOWare Pavement ME 2.1 defines the volumetric properties of the HMA mixture based on the following inputs:
  - Effective binder content (%)
  - Air voids (%)
  - Unit weight
  - Poisson's ratio

The effective binder content and air voids were extracted by Momin (2011), for each selected LTPP site. The values are tabulated in Appendix B. Since no data was found for the unit weight and the Poisson's ratio of asphalt concrete, Level 3 design inputs were used.

- *Unbounded Layers properties*: Selected LTPP sites have limited data for the unbounded layers. For that reason, Level 3 design inputs were used to cover the missing data. Only ten sites have records about the base\subbase layers. The extracted data for the base\subbase layers are listed in Table 3-4. It is important to mention that the recommended resilient modulus values by AASHTO were used for base and subbase layers; these values depend on the AASHTO classification of the soil (AASHTO, 2008).
- *Subgrade Soil Type and Properties*: The LTPP sites have adequate records regarding the soil types. However, there are no available gradation data. Therefore, Level 3 soil gradation data were used to substitute for the missing data. It was noticed that the LTPP Site # 091803 has no information regarding the subgrade soil type; therefore, it was assumed to be A-4 because it is the predominated soil in Connecticut (Malla, et al., 2006) . The extracted data are listed in Table 3-5.

Table 3-4: Extracted Data of the Base\Subbase Layers for the Selected LTPP Sites

LTPP SITE	Construction #	Layer #	AASHTO Soil Classification	Plasticity Index	Max Dry Density in Lab	Optimum Moisture Content in the Lab	In Situ Dry Density (Mean)	In Situ Moisture Content (Mean)
231001	1	2	A-1-b	1	131	6.5	129	7
231001	1	3	A-1-a		139	6.1		
251003	1	2	A-1-a		125	8.4		
231009	1	2	A-1-b	1	133	10	126	3
231009	1	3	A-1-a		139	7.9	139	3
231028	1	2	A-1-a		142	6.2	141	4
231028	1	3	A-1-a		143	7.4	137	3
341031	1	2	A-1-a					7
341033	1	2	A-1-a					5
091803	1	2	A-1-a		137	7.6	138	5

Table 3-5: Subgrade soil Type and Properties for Selected LTPP Site

LTPP Site	Construction #	Layer#	AASHTO Soil Classification	CBR	Plasticity Index	Liquid Limit	Max Dry Density in Lab	Optimum Moisture Content in the Lab	In Situ Dry Density (Mean)	In Situ Moisture Content (Mean)
231001	1	1	A-4				135	6.7		
501002	1	1	A-7-6							
251003	1	1	A-2-4	10			114	12	106	
501004	1	1	A-6		0	0	112	12.6	102	82.1
231009	1	1	A-4							
231028	1	1	A-1-a		0	0	128	8.5		
501681	1	1	A-1-a		3	18				
501683	1	1	A-1-a		11	26				
091803	1	1	A-4				122	12.4		118.2
341003	1	1	A-7-6							
341011	1	1	A-7-6							
341030	1	1	A-4							
341031	1	1	A-7-6							
341033	1	1	A-2-4							
341034	1	1	A-1-a							
341638	1	1	A-1-b							
421597	1	1	A-7-5							
421599	1	1	A-7-5							

3.2.4. Selection of the Climatic Stations

The LTPP sites were monitored from 1986 to 1996. However, the stored climatic files in the AASHTOWare Pavement ME 2.1 contain the recorded weather data from 1996 to 2006. Momin (2011), generated the MEPDG 1.1 climatic files from 1986 to 1996 for the selected LTPP sites. The format of the generated climatic files is as same as the format of climatic files stored in the AASHTOWare. Therefore, the generated files by Momin (2011) were used in this study.

3.2.5. Pavement Performance Data

The accuracy of measured distresses for the selected LTPP sites has a significant impact on enhancing the predictions of the embedded performance models in the AASHTOWare Pavement ME 2.1. The actual distresses values were extracted from the LTPP database. Then, they were tabulated in Appendix C. The extracted distresses data are listed as below:

- Total Rutting
- Alligator Cracking
- Longitudinal Cracking
- International Roughness Index (IRI)

3.3. Developing the Performance Models for New York State Department of Transportation

Although New York State Department of Transportation (NYSDOT) only uses the IRI trigger value in the decision of placing the pavement overlay, the local calibration was performed for alligator cracking, total rutting and IRI. During the estimation of the local bias, it was found that the measured thermal cracking data were unreliable, so the calibration of this model was abandoned. The calibration of longitudinal cracking was not conducted due to the lack of accuracy observed while developing the calibration coefficients. It is important to mention that the lack of accuracy in the predicted longitudinal cracking distresses was also observed by Montana DOT in their effort to implement the MEPDG (VonQuintus, et al., 2007), and in Canada (Alauddin, et al., 2013).

3.3.1. Select Hierarchical Input Level

The design input level is selected by the designer based on the highway agency criteria. Nevertheless, the designer can use the design level inputs listed in Table 3-7, as recommended by AASHTO. NYSDOT has not developed a list of recommended design input level yet; Table 3-7 was used to select the design input level. It is recommended to use the same design inputs level in developing the design cases after calibrating the distresses models (Darter, et al., 2009).

Table 3-6: Recommended Design Levels Inputs by AASHTO

Input Group		Input Parameter	Recalibration Input Level Used
Truck Traffic		Axle Load Distributions (Single, Tandem, Tridem) Truck Volume Distribution	Level 1
		Lane and Directional Truck Distributions	Level 1
		Tire Pressure	Level 3
		Axle Configuration, Tire Spacing	Level 3
		Truck Wander	Level 3
Climate		Temperature, Wind Speed, Cloud Cover, Precipitation, Relative Humidity	Level 1 Weather Stations
Materials Propertie s	Unbbound Layers and Subgrade	Resilient Modulus-All Unbound Layers	Level 1; Backcalculation
		Classification and Volumetric Proerties	Level 1
		Moisture-Density Relationships	Level 1
		Soil-Water Characteristic Relationships	Level 3
		Saturated Hydraulic Conductivity	Level 3
	HMA	HMA Dynamic Modulus	Level 3
		HMA Creep Compliance and Indirect Tensile Strength	Level 1, 2 and 3
		Volumetric Properties	Level 1
		HMA Coefficient of Thermal Expansion	Level 3
	PCC	PCC Elastic Modulus	Level 1
		PCC Flexture Strength	Level 1
		PCC Indirect Tensile Strength (CRCP Only)	Level 2
		PCC Coefficient of Thermal Expansion	Level 1
All Materials		Unit Weight	Level 1
		Posson's Ratio	Level 1 and 3
		Other Thermal Properties; Conductivity, Heat Capacity, Surface Absorptivity	Level 3
Existing Pavement		Condition of Existing Layers	Level 1 and 2

3.3.2. Sample Size Estimation for Distress Prediction Models

In this research, the minimum number of the required road segments needed to calibrate the performance models was determined based on the mean and variance. When employing both of them, a significant variation in the estimations of the sample size was found. At the end, the most reliable estimated sample size was adopted. The sample size was estimated for the following models:

- Rutting Model
- Bottom Up Cracking Model
- Thermal Cracking Model
- International Roughness Index Model

To estimate the sample size, Equations 2-10 and 2-11 were employed (AASHTO, 2010):

$$N = \left(\frac{z_{\frac{\alpha}{2}} * \delta}{E_T}\right)^2 \qquad \text{Equation 2-10}$$

The sample size estimation based on the mean or bias is summarized in Table 3-7. The following steps were employed to develop Table 3-7:

- The Level of Confidence was selected as 90%.
- The design reliability was selected as 90% based on the CPDM (NYSDOT, 2014).
- The threshold value of each distress model was selected based on the recommended values by AASHTO (AASHTO, 2008). However, the IRI trigger value was

provided by the PMS unit of NYSDOT. NYSDOT uses IRI trigger value as a range from 200 to 250. Therefore, the mid-range value (225) was used.

- The Standard Error of Estimate (SEE) for each model was computed based on the trigger value of each distress model. For the IRI model, the SEE was selected to be 18.9 in/mile according to AASHTO (2008).
- The tolerable Bias (E_T) was estimated at 90% confidence level.

Table 3-7: Estimated Minimum Number of Sites Needed for Validation & Local Calibration Based on Bias

Pavement Type	HMA New Pavement			
Performance Model	Alligator Cracking	Rut Depth	Thermal Cracking	IRI
Perf. Indicator Threshold (@ 90 Percent Reliability) (δ)	10%	0.4 in	500 ft/mile	225 in/mile
Standard Error of Estimate (SEE)	5.30%	0.16 in	83 ft/mile	18.6 in/mile
Tolerable Bias (ET)	8.70%	0.27 in	136 ft/mile	31 in/mile
Minimum No. of Researchs Required for Validation & Local Calibration	4	6	36	74
Number of the LTPP Sections Used	17	18	17	17

$Z_{\propto/2} = 1.64$

$E_T = SEE * Z_{\frac{\alpha}{2}}$

As showed in Table 3-7, the estimated sample size satisfied the requirements for alligator cracking and rutting models. While the LTPP segments were the only segments that could be obtained, the estimated sample size for thermal cracking and IRI were not further considered and it was assumed that the 18 LTPP sites were sufficient.

The minimum sample size was estimated based on the variance also, as shown in Table 3-8. Equation 2-11 was used for this purpose.

$$\frac{Se}{Sy} \geq \left[\frac{X\alpha^2}{n-1}\right]^{0.5} \qquad \text{Equation 2-11}$$

Based on Equation 2-11, the sample size was estimated as follows:

- AASHTOWare Pavement ME 2.1 design problems were run with the global calibration coefficients. The computed distresses were extracted and tabulated based on the site number, date, and distress type; they are listed in Appendix D.
- The maximum measured distresses for each site were tabulated with the corresponded maximum computed distresses. Then, the residuals were computed as the difference between the measured and the computed values (Devore, et al., 1999). This process was repeated for each distress model. Tables 3-8 to 3-11 summarize the outputs of this step.
- Then, the standard deviation of the maximum measured distresses (Sy) was computed for each model. The standard deviation of the residuals (Se) was then computed for each distress model, as shown in Table 3-12.
- The same procedure was performed for the full set of measured distresses data instead of only for maximum distress values.

- Chi-Squared (X^2) values at 90% confidence level and (*n-1*) degree of freedom were computed. The parameter *n* represents the number of observations.

Equation 2-11 differs than Equation 2-10 as the local calibration guide defines it. Likewise, it should be noticed that ($\frac{S_e}{S_y}$) ratio compares the variability in the predicted performance to the measured performance. A ratio which is greater than 1.0 indicates that the variability of the residuals errors between the predicted and the measured values is larger than that in the variability of the measured values. Therefore, a ratio less than 1.0 is preferable.

Table 3-8: Extracted Maximum Measured data, computed data, and the residuals for the Fatigue Model

Bottom_Up Cracking Model				
Number	Segment ID	Max Measured	Computed	Residuals
1	231001	0.77	0.01	0.76
2	231009	1.60	0.02	1.58
3	231028	0.00	0.05	-0.05
4	251003	0.00	0.01	-0.01
5	341003	22.60	0.12	22.48
6	341011	22.60	0.04	22.57
7	341030	20.47	0.04	20.43
8	341031	10.15	0.06	10.10
9	341033	1.31	0.01	1.30
10	341034	0.16	0.02	0.14
11	341638	0.07	2.66	-2.59
12	421597	0.00	0.00	0.00
13	421599	0.00	0.00	0.00
14	501002	0.07	0.02	0.05
15	501004	4.11	0.00	4.11
16	501681	0.00	0.01	-0.01
17	501683	1.45	0.01	1.44

Table 3-9: Extracted Maximum Measured data, computed data, and the residuals for the Rutting Model

Rutting Model				
Number	Segment ID	Max Measured	Computed	Residuals
1	91803	0.24	0.20	-0.05
2	231001	0.25	0.51	0.26
3	231009	0.41	0.28	-0.14
4	231028	0.31	0.51	0.21
5	251003	0.30	0.18	-0.13
6	341003	0.42	0.83	0.40
7	341011	0.45	0.39	-0.05
8	341030	0.30	0.85	0.54
9	341031	0.48	0.55	0.07
10	341033	0.27	0.35	0.08
11	341034	0.35	0.28	-0.07
12	341638	0.39	0.32	-0.07
13	421597	0.17	0.22	0.05
14	421599	0.28	0.28	0.00
15	501002	0.33	0.62	0.29
16	501004	0.26	0.28	0.02
17	501681	0.19	0.49	0.30
18	501683	0.18	0.87	0.69

Table 3-10: Extracted Maximum Measured data, computed data, and the residuals for the Thermal Cracking Model

Thermal Cracking Model				
Number	Segment ID	Max Measured	Computed	Residuals
1	231001	2112	4010.94	1898.94
2	231009	2112	1780.33	-331.67
3	231028	1.14	1423.57	1422.43
4	251003	1594.43	3082.68	1488.25
5	341003	1461.87	3262.79	1800.92
6	341011	1552.86	6383.56	4830.70
7	341030	0.54	2895.64	2895.10
8	341031	24.99	6179.21	6154.22
9	341033	1908.68	2930.27	1021.59
10	341034	965.12	2885.25	1920.13
11	341638	1350.81	443.35	-907.46
12	421597	0.02	762.01	761.99
13	421597	0.02	762.01	761.99
14	501002	2112	4748.71	2636.71
15	501004	2112	2985.69	873.69
16	501681	2112	131.62	-1980.38
17	501683	2112	1517.09	-594.91

Table 3-11: Extracted Maximum Measured data, computed data, and the residuals for the Rutting Model

IRI Model				
Number	Segment ID	Max Measured	Computed	Residuals
1	231001	125.3	93.4	-31.88
2	231009	67.2	94.7	27.46
3	231028	91.7	88.1	-3.61
4	251003	122.6	83.6	-38.96
5	341003	124.5	85.9	-38.57
6	341011	115.7	101.3	-14.45
7	341030	252.9	75.7	-177.16
8	341031	144.7	90.2	-54.50
9	341033	199.1	96.1	-103.02
10	341034	96.3	98.7	2.38
11	341638	66.0	122.2	56.21
12	421597	107.0	67.9	-39.12
13	421599	93.8	90.9	-2.94
14	501002	93.5	114.3	20.79
15	501004	132.6	96.1	-36.50
16	501681	76.3	88	11.69
17	501683	142.6	87.3	-55.26

Table 3-12: Estimated Minimum of Sites Required for Validation & Local Calibration Based on Precision

Performance Models	Alligator Cracking	Rut Depth	Thermal Cracking	IRI
Based on Maximum Measured Values				
Sy	8.43%	0.23 in	1860 ft/mile	35 in/mile
Se	8.54%	0.24 in	1996 ft/mile	54 in/mile
Se/Sy	1.0	1.05	1.07	1.56
$(X^2\alpha/(n-1))$^0.5	1.0	1.05	1.07	1.28
Minimum No. of Researchs Required for Validation & Local Calibration	325	225	249	17
Number of LTPP sections used	17	18	17	17
Based on Full Set of Measured Data				
Sy	6.99%	0.17 in	1662.7 ft/mile	40.98 in/mile
Se	13.89%	0.24 in	80.3 ft/mile	320.59 in/mile
Se/Sy	1.99	1.37	0.05	7.82
$(X^2\alpha/(n-1))$^0.5	1.64	1.39	0.14	1.64
Minimum No. of Researchs Required for Validation & Local Calibration	2	5	10,000,000.00	2
Number of LTPP sections Used	17	18	17	17

Based on Table 3-12, when the maximum measured distresses were used; the estimated sample size of the IRI model was the only one that equals to the obtained LTPP sites. Nevertheless, the LTPP sites were insufficient for the other models. Thus, Table 3-7 was considered in this research since the estimated sample sizes of the performance models were less than or close to the LTPP sites. The estimated sample size based on the full set of measured data was abandoned due to the unreliable estimation, such as the estimated sample size for the thermal cracking model.

It is obvious; there is contrast in the estimation process between the two estimation methods although the sample size was estimated at the same confidence level. In Equation 2-10, the highway agency design criteria were used to compute SEE. Then, SEE was used to compute E_T, these two parameters have a great impact on the estimation process based on bias. However, the sample size was estimated in Equation 2-11 based on the predicted distresses of AASHTOWare with global calibration factors; as well as, the measured distresses. For this reason; Equation 2-10 is more reliable than Equation 2-11. It is important to mention that both equations estimated the sample size at one sided confidence level which makes the estimation process more precise. Statistically, the precision is defined for one sided confidence level (Devore, et al., 1999).

3.3.3. Extraction, evaluation and conversion of the measured data

Since Momin (2011) had extracted and converted the collected data, the data were checked and evaluated for use in this research. As mentioned earlier, the collected data are listed in Appendix C.

As a result, the obtained sample size is small; the existences of the outliers was not identified at the beginning of this effort, but only later, when bias was found after the local calibration. To identify the existence of the outliers, the SAS computer software was employed.

3.3.4. Assess Local Bias and Standard Error of the Estimate (SEE) from Global Calibration Factors

The full set of measured data, also the computed distresses from the previous runs of the AASHTOWare, were used to assess the local bias and SEE. The null hypothesis used for this purpose was as follow:

$H_o: Measured\ Distresess = Computed\ Distresess$

$H_a: Measured\ Distresess \neq Computed\ Distresess$

In addition, the plots of the measured versus computed distresses were prepared for each model to investigate the location of the points versus the line of equality. As previously mentioned, the computed distresses for the LTPP sites are tabulated in Appendix D.

3.3.4.1. Determine the Local Bias for Alligator Cracking Model

The null hypothesis was conducted to identify the existence of the local bias. Paired $t - test$ at 95% confidence level was used to determine if there is a significant difference between the measured alligator cracking and the computed alligator cracking. After the test was performed, the null hypothesis was rejected. Therefore, at 95% confidence level there is a significant difference between the measured and computed distresses. The Sum Squared Errors (SSE), Bias, and Correlation Coefficient (R^2) are given in Table 3-13.

Then, a plot of measured versus computed distresses was conducted as illustrates in Figure 3-5. The Figure 3-5 reveals that there is a poor exponential relationship between the measured and the computed distresses. Hence the local calibration effort is required for the alligator cracking model.

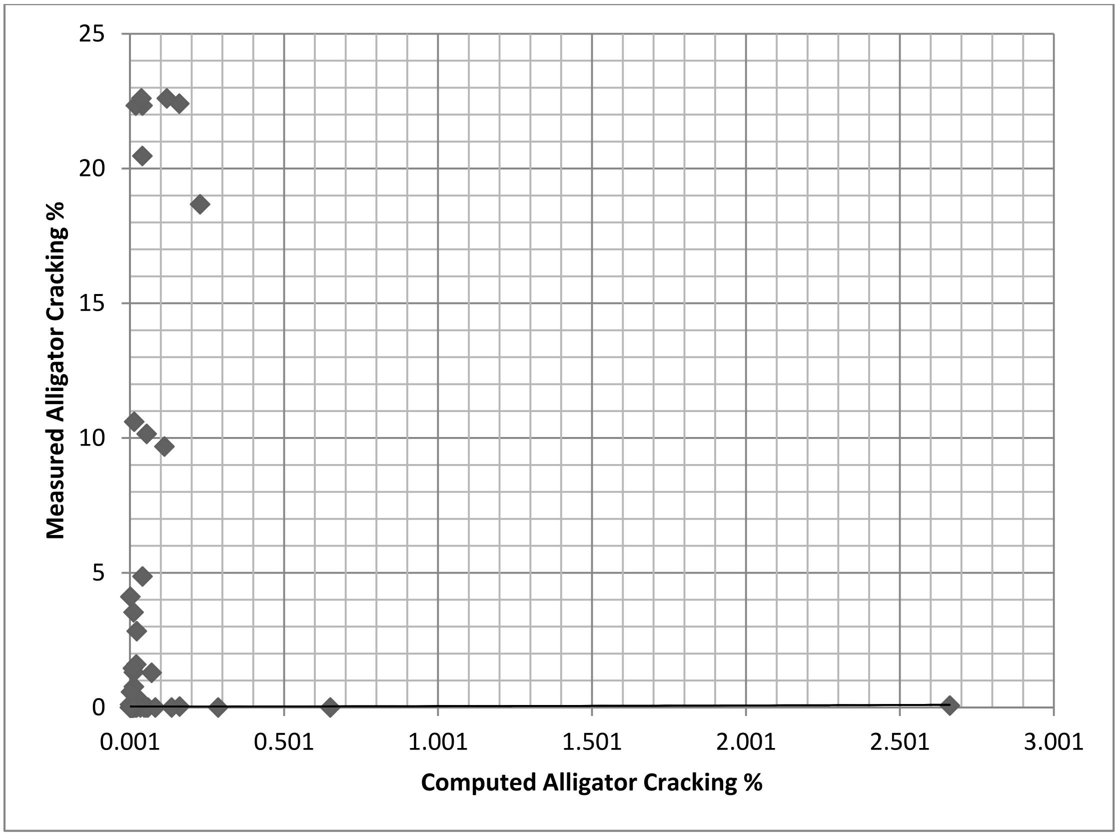


Figure 3-5: Measured Alligator vs Computed Alligator Cracking (Globally Calibrated)

<u>*3.3.4.2. Determine the Local Bias for Total Rutting Model*</u>

To determine the local bias, the computed total rutting was obtained by the summation the rutting of asphalt concrete (AC), base, and subgrade layers. Then, Paired $(t - test)$ was performed at 95% confidence level. The test concluded that at 95% confidence level, there is a significant difference between the measured and computed total rutting. The results of the statistical analysis are given in Table 3-13.

Additionally, the plot of measured versus computed total rutting was conducted, as illustrated in Figure 3-6. Figure 3-6 shown that there is a poor linear relationship between the measured and the computed total rutting. Therefore, the local calibration must be performed for this model. The plotted data show a funnel shape which suggests that the variance is not constant because the embedded performance models in the AASHTOWare were globally calibrated. However, it is not possible to eliminate the non constant variance by transformation techniques, as suggested by Kutner et al (2005) since only the calibration coefficients can be changed and not the variables themselves in AASHTOWare.

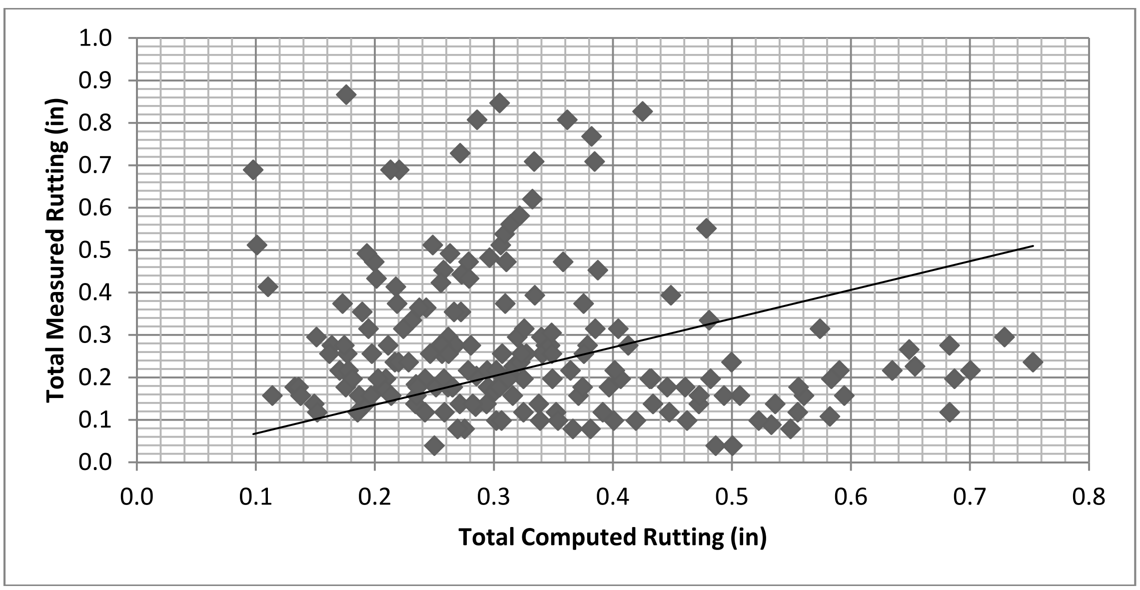


Figure 3-6: Measured vs Computed Total Rutting (Globally Calibrated)

3.3.4.3. Determine the Local Bias for Thermal Cracking Model

Local bias was determined by performing Paired $t-test$ at 95% confidence level. The null hypothesis was rejected, so there is a significant difference between the measured and computed performance data. Then, the measured versus computed thermal cracking is plotted in Figure 3-7.

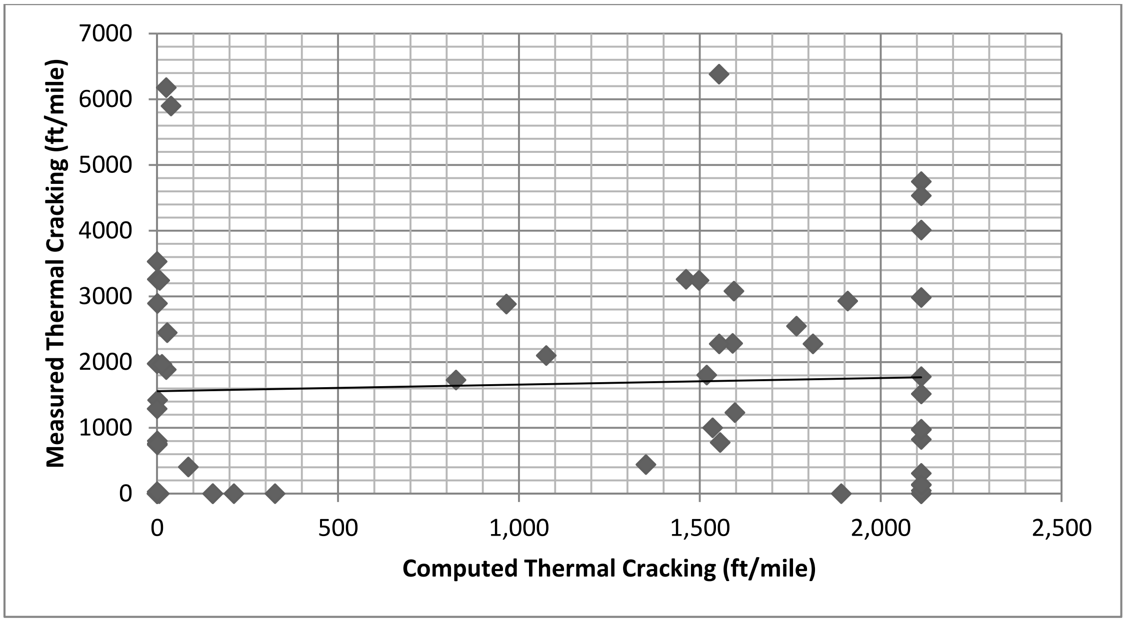


Figure 3-7: Measured vs Computed Thermal Cracking (Globally Calibrated)

Based on Figure 3-7, the points are scattered, also the trend of linear relationship between the measured and computed distress data is visibly poor. The existence of many outliers is also visible. Furthermore, Based on Table 3-13, the large value of the SSE and (R^2) indicate the unreliability of the measured data. Hence, the local calibration will not be performed for the thermal cracking model.

3.3.4.4. Determination the Local Bias for the IRI Model

The local bias was determined by the null hypothesis. For this purpose, Paired $t-test$ was employed. Based on the test, the null hypothesis was rejected. Therefore, at 95% confidence level there is significant difference between the measured and computed IRI. Then, SSE and (R^2) were computed and listed in Table 3-13. A plot of measured versus computed IRI is shown in Figure 3-8.

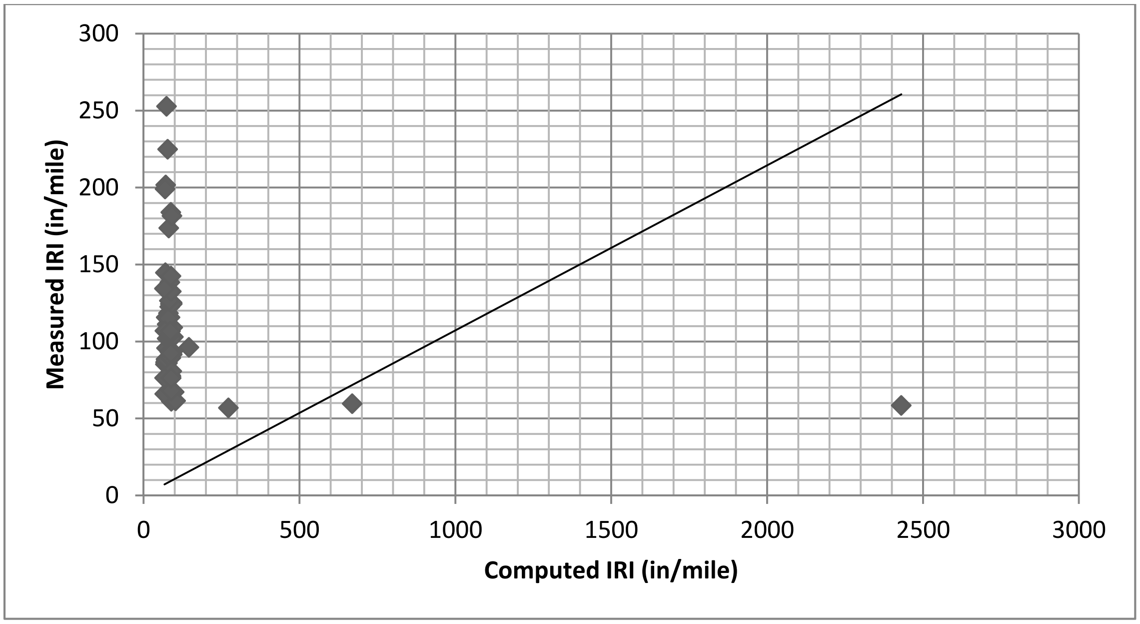


Figure 3-8: Measured IRI vs Computed IRI (Globally Calibrated)

Based on Figure 3-8, plotted points are poorly fit to the line of equality. Also the computed SSE and R^2 indicate there is a need to perform the local calibration to reduce the SSE and increase the coefficient of determination. Hence, the local calibration for the IRI model was performed. The required outputs needed to assess the local bias are given in Table 3-13.

Table 3-13: The summary of Local Bias Assessment

Model	Regression Coefficients	Bias	SSE	R^2	Se/Sy	P Value	Hypothesis; Ho:Σ(Meas.-Pred.) = 0
Alligator Cracking	C1=1 C2=1	-3.2	3,645	0.001	1	0.0006	Reject; P<0.05
Total Rutting	βr1=1 βr2=1 βr3=1	0.056	11.5	0.55	1.37	0.0013	Reject; P<0.05
Thermal Cracking	βt=1	129.1	234,373	0.31	1.116	0.0081	Reject; P<0.05
IRI	C1 =0.015 C2 = 0.4 C3 =0.008 C4 = 40	-24.7	754,583	0.09	7.82	0.02	Reject; P<0.05

3.3.5. Elimination of the Local Bias

To eliminate the bias, the Microsoft Excel solver was used to optimize the calibration coefficients of the performance models. The data in the Appendices A and B were used for this purpose. Table 3-14 lists the optimized calibration coefficients, Bias, SSE, (R^2), and the *P* value. The *P* value was used to judge the hypothesis. The following steps were performed to eliminate the local bias:

- The AASHTOWare Pavement ME 2.1 design problems were run with the global calibration coefficients to compute the distresses.

- The computed distresses were listed with the corresponding measured distresses for each segment and at same date. The required parameters to compute the distresses were listed too.
- The residual errors and the SSE were computed.
- The Microsoft Excel Solver was employed to adjust the regression coefficients, so that the minimum SSE is obtained.

3.3.5.1. Elimination of the Local Bias for the Alligator Cracking Model

First, the measured fatigue cracking data, the cumulative damages, and required parameters to compute the alligator cracking were extracted from Appendices A and B. The following steps explain the process in detail:

- A separate Excel spreadsheet file was prepared to list the extracted data.
- The regression coefficients (global calibration) of the alligator cracking transfer function were listed in the same file.
- Then, the transfer function was defined. The distresses were computed and were compared with the AASHTOWare computed distress. Since they were the same, the written equation was considered correct.
- The residuals errors were computed as the difference between the measured and the computed distresses. The Sum Squared of Errors (SSE) was computed from squaring the residuals, as shown in Table 3-14.
- Then, the Microsoft Solver was used to optimize the regression coefficients used to compute the alligator cracking to minimize the SSE. The optimized regression

coefficients defined as the local calibration coefficients. Then, SSE and (R^2) were computed; they listed in Table 3-14. SSE was reduced, but (R^2) was slightly improved.

- To identify the local bias, paired t-tests at 95% confidence level was conducted. At 95% confidence level, the null hypothesis was accepted as shown in Table 3-14. This indicates that the calibration improved the alligator cracking model.
- The measured versus computed alligator cracking is plotted in Figure 3-9. The plot shown an improvement in the data locations relative to the equality line. However, it is clear that outliers still exist. Due to the restrictions in obtaining the measured alligator cracking data, outlier analysis was not conducted.

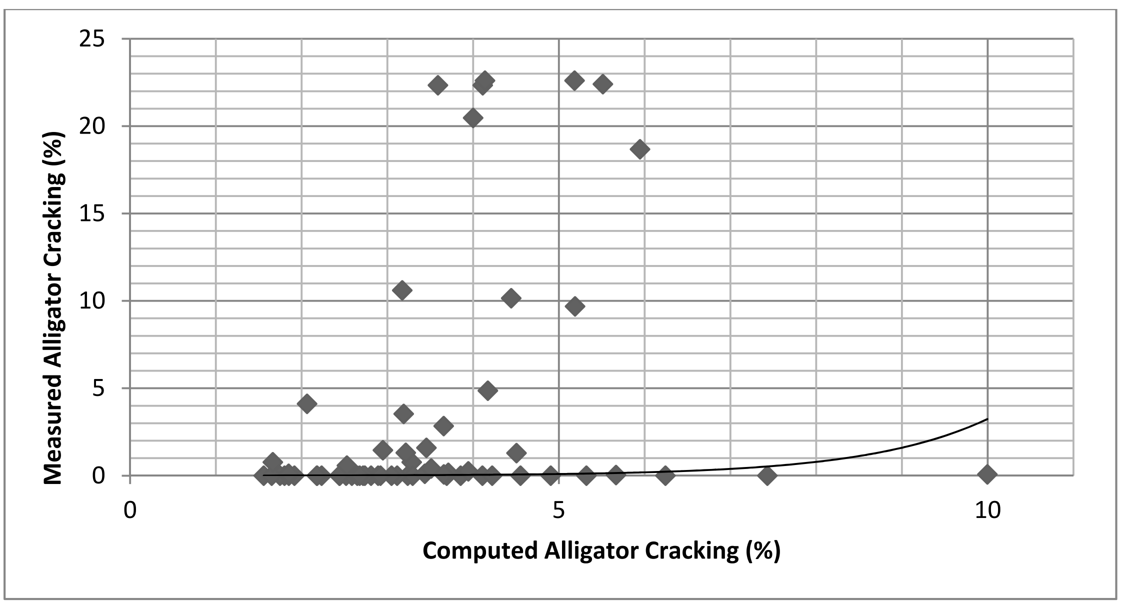


Figure 3-9: Measured vs Computed Alligator Cracking (Locally Calibrated)

3.3.5.2. Elimination of the Local Bias for the Rutting Models

The measured and computed total rutting data were extracted from Appendices A and B respectively in a separate Excel file. The bias for total rutting was eliminated by performing the optimization approach as mentioned earlier. The following steps explain the process:

- The equation of computing the total rutting was defined as a summation of the rutting in the subdivided layers (Asphalt Concrete (AC), Base and Subgrade layers).
- The computed rut depth in each layer was multiplied with the correspondent global regression coefficients such as, (βr1* AC Rutting), and so on.
- The residual errors for the full set of data were obtained as the difference between the computed total rutting and measured total rutting. From the residuals, SSE was obtained.
- The Microsoft Solver was employed to adjust the regression coefficients of the three subdivided layers to compute the distresses that give minimum SSE.
- The optimized regression coefficients were defined as the local calibration coefficients.
- Then the coefficient of determination, (R^2), and the Bias were computed as listed in Table 3-14. A slight improvement in the SSE and (R^2) was noticed.
- To identify the local bias, Paired $t-test$ at 95% confidence level was used for this purpose. After the test was performed, the null hypothesis was rejected, so at 95% confidence level there is a significant difference between the measured and the computed distresses (Table 3-14).

- The outlier analysis was performed by running SAS software. The existence of outliers in the measured distresses was determined by obtaining the absolute values of t-studentized $/t_i/$ for each segment. Then, Bonferroni test was applied. The test proved there are no outliers in the measured dataset.
- The measured total rutting versus computed total rutting plot was performed, as illustrates in Figure 3-10. The plot shown there is an improvement in the data location relative to the equality line, so the optimization improved the model.

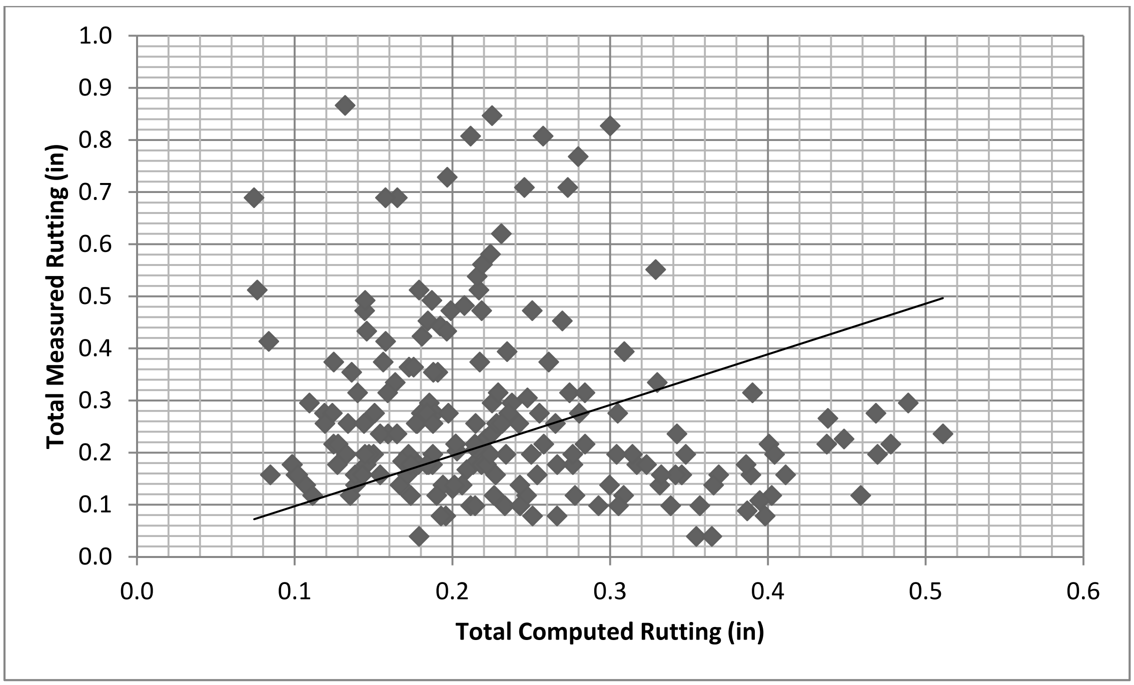


Figure 3-10: Measured vs Computed Total Rutting (Locally Calibrated)

3.3.5.3. Elimination of the Local Bias for the IRI Model

The bias of the IRI model was eliminated by performing the following steps:

- The IRI equation was defined as the summation of Site Factor (SF), Sum of alligator cracking and the thermal cracking (TC), and average total rut depth (RD).
- Each variable was multiplied with the corresponded global calibration coefficient.
- Then, the difference between the measured IRI and the initial IRI was found for each segment.
- The residual errors were obtained for all the listed data.
- Then, Microsoft Solver was employed to optimize the global calibration coefficients to obtain minimum SSE.
- To identify the local bias, Paired $(t - test)$ at 95%confidence level was used. After the test was performed, the null hypothesis was accepted, so at 95%confidence level there is no significant difference between the measured and the locally computed distresses, as shown in Table 3-14.
- Thus, the optimized regression coefficients were defined as the local calibration coefficients. Then, (R^2) and Bias were computed as shown in Table 3-14. It was noticed a significant improvement in the SSE , (R^2), and $(\frac{S_e}{S_y})$.
- The plot of measured versus computed IRI distresses was drawn in Figure 3-11. The plot shown there is improvement in the data locations, so the optimization improved the model.

Table 3-14: The Summary of Local Calibration and Elimination the Local Bias

Model Type	Regression Coefficients	Bias	SSE	R^2	Se/Sy	P Value	Hypothesis; Ho:Σ(Meas.-Pred.) = 0
Alligator Cracking	C1=0.501711	0.21	2,766	0.07	0.96	0.85	Accepted; P>0.05
	C2=0.227186						
Rutting	βr1=0.59	-0.04	8.80	0.56	1.21	0.008	Reject; P<0.05
	βr2=0.821						
	βr3=0.74						
IRI	C1 = 168.709	-6.0	115,777	0.87	1.053	0.33	Accepted; P>0.05
	C2 = -0.0238						
	C3 = 0.00017						
	C4 = 0.015						

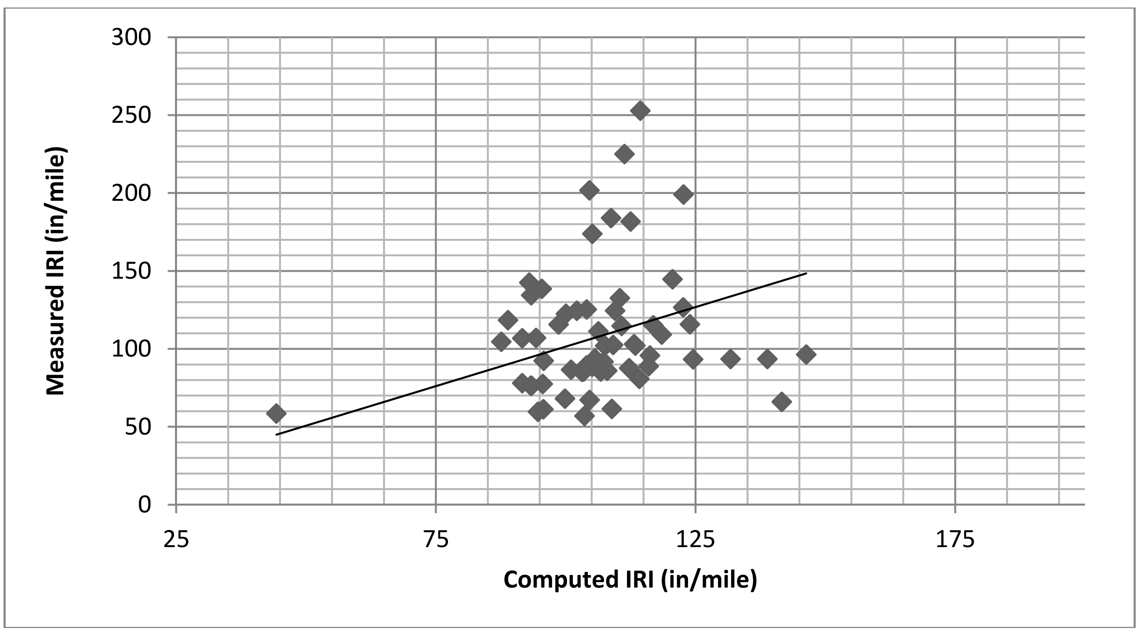


Figure 3-11: Measured IRI vs Computed IRI (Locally Calibrated)

3.4. Performance Models Validation

Jack Knife testing validation approach was only performed for the rutting and the IRI models since the models are linear thus making the process applicable. The validation of the alligator cracking model was not possible. The following steps were performed to validate the rutting model:

- Measured total rutting observations were extracted and listed in separate Excel files
- The extracted observations set was split into two groups independently, one for the calibration, and the other for prediction. These groups were randomly selected.
- Therefore, the prepared matrix consists group X (as variables) and group Y (as predictor), with (*i=1,,,,,, n),* sets of observations.
- At the beginning, one set of (X_i, Y_i) was removed. By removing one set such as (x_1, y_1), the validation matrix contained *(n-1*) observations to perform the calibration.
- After the calibration was performed on *n-1* matrix, the calibrated coefficients were used to predict (y) which listed in a new group, it is called K_{th} group.
- New the standard error $(new\ e_1)$ was found by the difference between the measured and computed distress value of the removed dataset. For example$(\ y_{kth1} - y_1)$.
- The removed dataset (x_1, y_1) was replaced with the second observations set in the *(n-1)* validation matrix which was (y_2, x_2).
- Same steps were repeated for all observations in the dataset.
- The F-Test at 95% confidence level was employed to identify if the new standard errors are significantly higher than the standard errors of the calibration. The test concluded

that at 95% confidence level, the new standard errors are not significantly higher than the standard errors of the calibration. Thus, the model is valid.

The abovementioned steps were repeated to validate the IRI model too. F-Test was used to test the validation at 95% confidence level. It was founs that IRI model is valid.

Chapter 4

AASHTOWare Pavement ME 2.1 Design Cases Development

4.1. Overall Concept for Developing the Design Cases

The design cases were developed to design the new flexible pavement structures based on the AASHTO ME Pavement Design Guide (MEPDG). For this purpose, AASHTOWare Pavement ME 2.1 was used with the local calibration factors. The design cases were made based on the combination of NYS traffic loads, climatic conditions, subgrade soil stiffness, pavement structure, and materials properties. The NYSDOT Comprehensive Pavement Design Manual (CPDM) was the main source providing the design inputs, along with NYSDOT standards and laboratory experimental data. The following conditions were considered when developing the design case:

- The pavement structures for new flexible pavement classified as Principal Arterial – Interstate.
- Design life of 15 years
- Design reliability of 90%
- Water table of 10 feet

The developed design cases simulated the current NYSDOT pavement configurations shown in Table 2-2. Thus, the following considerations were taken during the development process:

- Subgrade soil stiffness (Mr) 4.0, 5.0, 6.0, 7.0, 8.0 and 9.0 ksi (28, 34, 41, 48, 55 and 62 Mpa).

- Annual Average Daily Truck Traffic (AADTT) in one direction of 50, 100, 250, 500, 1,000, 2,000, 4,000, 5,000.
- Pavement structures starting with the design cases included in the NYSDOT Comprehensive Pavement Design Manual (CPDM). The thicknesses of base and subbase layers were kept the same as those in the recommended design template included in CPDM. Nevertheless, the thicknesses of asphalt concrete and selected subgrade soil layers were varied to optimize the design solutions.
- The climatic data for all 23 climatic stations available in AASHTOWare Pavement ME 2.1 for the New York State were considered.

A unique name format was used for each design case to distinguish the structure design components. The name format included the resilient modulus of subgrade soil layer (Mr), the total HMA thickness, the select subgrade layer thickness and the Annual Average Daily Truck Traffic. Accordingly, the name template of each design case was as below:

Mr (ksi)-HMA Thickness (in)-Select Subgrade Soil Thickness (in) - AADTT (one lane)

4.2. General Information

Since AASHTOWare runs were performed for hypothetical design cases, AASHTOWare default dates for construction and opening-to-traffic were used, as illustrated in Figure 4-1. They would represent a typical road construction schedule, where the unbound granular layers are placed in the Spring and early Summer, while the asphalt layers are placed in late Summer or early Fall.

The opening to traffic typically takes place in the Fall. The construction month in AASHTOWare refers to the month and year that the unbound layers have been compacted and finished in the month that the Hot Mix Asphalt (HMA) has been placed, while the traffic opening data represent the date of opening the road to the public.

AASHTOWare accounts the monthly traffic loading and climatic inputs based on the selected construction and opening-to-traffic dates. Therefore, the monthly modulus values of each layer are affected by the selected dates.

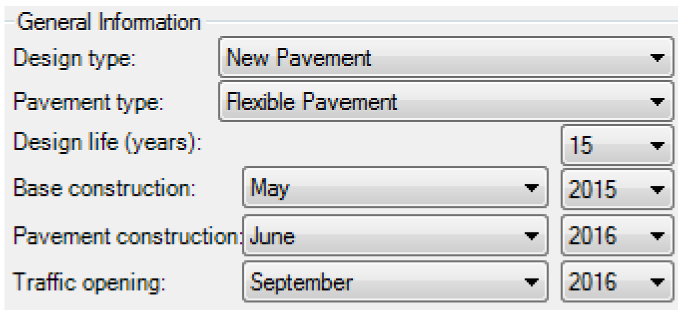


Figure 4-1: Selected Construction and Opening-To-Traffic Dates

4.3. Design Criteria and Reliability

In order to perform the design by AASHTOWare, the design criteria (trigger values) of flexible pavement distresses should be selected. The trigger values normally represent the distress values for which the asphalt pavement structure would be rehabilitated with an overlay. Thus, the NYSDOT Pavement Management Unite (PMU) was contacted and the typical distress values that trigger rehabilitation with an overlay were obtained. NYSDOT only uses IRI trigger values range of (200 - 250) in/mile to rehabilitate the flexible pavements. Table 2-5 was used to obtain the design criteria of the other distresses.

For this research, the trigger value of the IRI was selected as 225 in/mile according to AASHTO recommendations. AASHTO suggests using the average of the agency design criteria; the initial IRI value was selected 60 in/mile because AASHTO recommends this value for full depth asphalt pavements (AASHTO, 2008). Furthermore, NYSDOT approved using the total rutting in the design's judgment; the trigger value selected for total rutting was 0.75 in.

Therefore, the design solutions adequacy was determined based on the IRI and total rutting. It should be noticed that NYSDOT uses 90% design reliability to design the new flexible pavement structure. For this reason, same value of design's reliability was used. Table 4-1 gives the design criteria and reliability used for this study.

Table 4-1: Design Criteria and Reliability for this Study

Performance Criteria	Limit	Reliability
Initial IRI (In/mile)	60	-
Terminal IRI (In/mile)	225	90%
AC Longitudinal Cracking (ft/mile)	2000	90%
AC Fatigue Cracking (Percent)	10	90%
AC Thermal Cracking (ft/mile)	500	90%
Permanent Deformation-Total Rutting (in)	0.75	90%
Permanent deformation-AC only (in)	0.25	90%

4.4. Traffic Inputs

As aforementioned, AASHTOWare provides the designer a hierarchical design input level. Level 1 input data should be used when the traffic data specific for the design project is known. Level 2 presents the modest knowledge of the designer about the research site traffic data, and Level 3 presents the default or regional traffic inputs. The work of Intaj (2012) was reviewed and evaluated to use the traffic data for this research. Intaj (2012) recommended using average statewide traffic data to design the new flexible pavements. It was recommended that traffic data for 2010 was the most reliable data for the period 2007 to 2011 and therefore it should be used.

Intaj 2012 collected the traffic data of WIM stations throughout NYS from year 2007 to 2011 and processed the data with the TrafLoad computer software. It was found that the WIM stations collected sufficient traffic data for 2010 only. Thus, the year 2010 traffic data was considered the most reliable. After conducting a cluster analysis, Intaj found a small variation between specific site traffic data and average statewide data. Accordingly, average statewide traffic data of the year 2010 were adopted to develop the design cases for this research. The traffic data extracted by Intaj are summarized in:

- *Vehicle Class Distribution (VCD):* Table 4-2
- *Monthly Adjustment Factors (MDF):* Table 4-3
- *Number of Axles per Truck :* Table 4-4
- *Axle Load Spectra*

Table 4-2: Average Statewide VCD of Year 2010

Vehicle Class	Distribution (%)
Class 4	2.64
Class 5	27.3
Class 6	13.4
Class 7	3.04
Class 8	10.43
Class 9	36
Class 10	5.45
Class 11	0.79
Class 12	0.25
Class 13	0.7
Total	100

Table 4-3: Average Statewide MDF of Year 2010

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0.8	0.84	0.8	0.8	0.94	0.94	0.74	0.95	0.95	0.95
February	0.85	0.83	0.85	0.85	0.97	0.97	0.74	1.02	1.02	1.02
March	0.87	0.85	0.87	0.87	1.05	1.05	0.83	1.16	1.16	1.16
April	1.12	0.96	1.12	1.12	1.07	1.07	0.96	1.18	1.18	1.18
May	1.13	1.1	1.13	1.13	1.01	1.01	1.09	1.09	1.09	1.09
June	1	1.1	1	1	1.04	1.04	1.16	1.09	1.09	1.09
July	1.12	1.06	1.12	1.12	0.97	0.97	1.21	1.03	1.03	1.03
August	1.18	1.08	1.18	1.18	1.02	1.02	1.27	1.01	1.01	1.01
September	1.06	1.19	1.06	1.06	1.02	1.02	1.17	0.87	0.87	0.87
October	1.06	1.16	1.06	1.06	1.04	1.04	1.11	0.92	0.92	0.92
November	0.91	0.97	0.91	0.91	0.99	0.99	0.93	0.92	0.92	0.92
December	0.89	0.86	0.89	0.89	0.88	0.88	0.79	0.77	0.77	0.77

Table 4-4: Average Statewide Number of Axle per Truck of Year 2010

Vehicle Class	Single	Tandem	Tridem	Quad
Class 4	1.32	0.68	0	0
Class 5	2	0	0	0
Class 6	1	1	0	0
Class 7	1.32	0.28	0.64	0.05
Class 8	2.45	0.59	0.02	0
Class 9	1.23	1.89	0	0
Class 10	1.07	0.99	0.95	0.05
Class 11	3.7	0.27	0.25	0.01
Class 12	3.71	1.09	0.03	0
Class 13	2.11	0.76	0.28	0.32

Since AASHTOWare calls for additional traffic inputs, CPDM was reviewed and the appropriate values were selected:

- *Annual Average Daily Truck Traffic (AADTT)*: AADTT was selected 50, 100, 250, 500, 1,000, 2,000, 4,000 and 5,000.
- *Number of Lanes in Design Direction*: It was assumed to be one lane in the design direction

- *Percentage of Trucks in Design Direction*: A 50% value has been used, as recommended by CPDM.
- *Percentage of Trucks in Design Lane*: Since only one lane was selected for the design direction, 100% value was used.
- *Operational Speed*: The default value of 65 mph was used.
- *Truck Traffic Growth Rate*: The exponential traffic growth model, with a growth rate of 2% was used. This is the value recommended by NYSDOT when research specific values are not available (CPDM).

The default values were used for the following traffic inputs that have not been found to be recommended by CPDM or NYSDOT specifications.

- *Axle Configuration*
- *Lateral Wander*
- *Wheel Base*
- *Identifiers*

4.5. Climatic Data

Climatic data has an enormous impact on the distresses prediction since the hourly basis records of temperature, precipitation, relative humidity, wind speed, and cloud cover are used by the Integrated Climatic Model (ICM). The temperature and the moisture are computed by ICM in each sublayer of the pavement structure. The dynamic modulus and the resilient modulus are adjusted and modified over the design life by ICM model. AASHTOWare contains climatic files for 851 climatic stations located throughout the United States.

In this research, all New York State (NYS) stored climatic stations in AASHTOWare were employed for the sake of developing the design cases, as listed in Table 4-5. The weather conditions for the same region were represented in at least one climatic station per region. It was found that there is no physical weather station in Region 9 which explains unavailability in AASHTOWare. Hence, a virtual climatic station was created as a combination of the following climatic stations:

- Albany (14735)
- Elmira (14748)
- Montgomery (04789)
- Syracuse (14771)
- Utica (94794)

As result, there was no mention of the in Since the CPDM and NYSDOT specifications do not provide recommendation on values to be used for the water table depth, it was decided in collaboration with NYSDOT to develop design table only for a water table depth value of 10 feet. It is important to mention that AASHTOWare uses the water table depth to calculate the moisture content in the unbounded layer which is used for the estimation of the resilient modulus of unbound materials during the design life. Previous work indicated that a water table depth higher than 10 feet has no effect on the predicted distresses (AASHTO, 2008). Figure 4-2 shown a screen capture of the AASHTOWare climatic data tab.

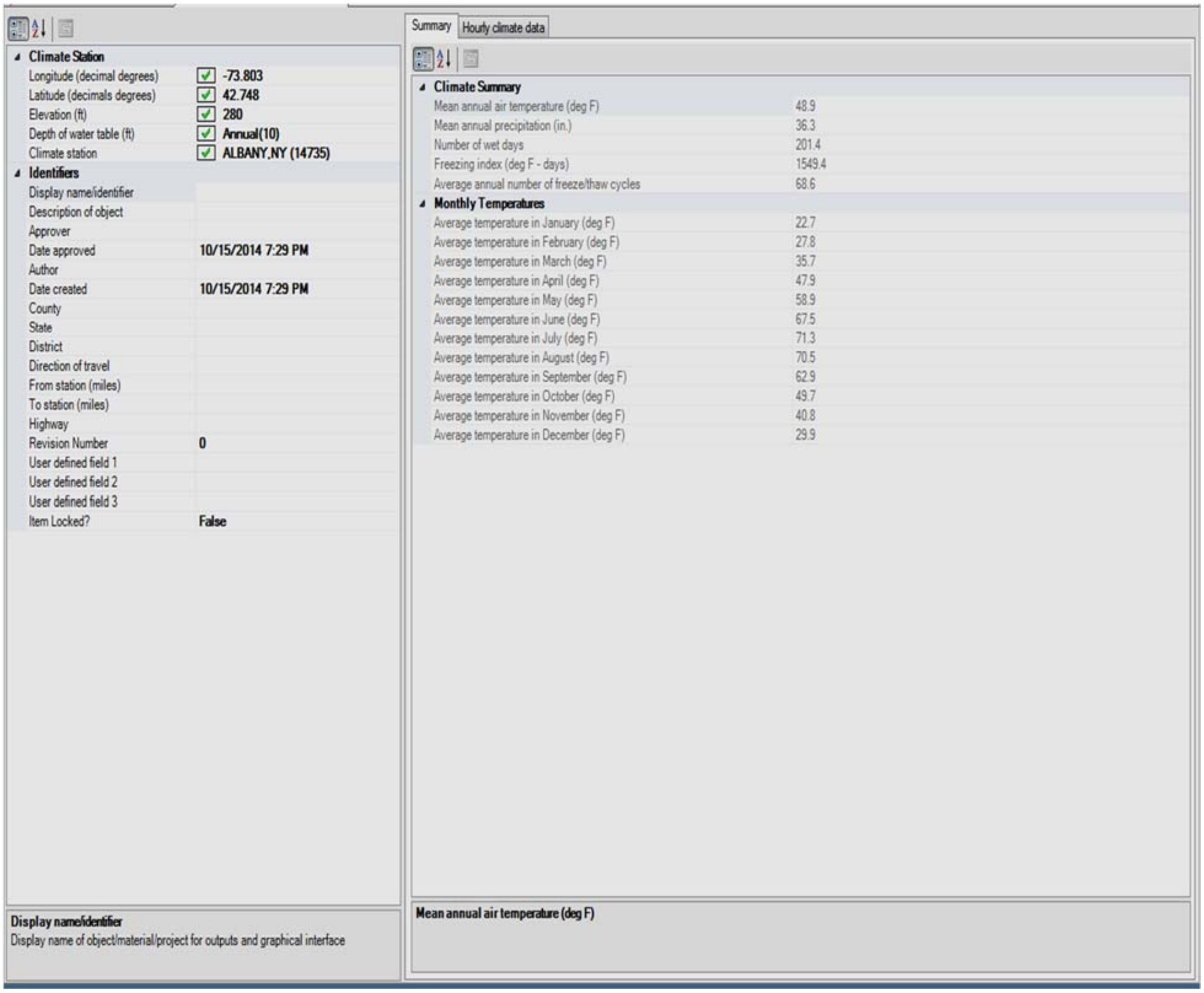


Figure 4-2: AASHTOWare Climate Tab

Table 4-5: AASHTOWare Climatic Stations Used for this Study

Climatic Stations					Annual Water Table (ft)
County	Station ID	Longitude (Decimal Degrees)	Latitude (Decimal Degrees)	Region	
Saratoge	Albany (14735)	-73.803	42.748	1	10
Warren	Glens Falls (14750)	-73.61	43.341	1	10
Oneida	Utica (94794)	-75.384	43.145	2	10
Onondaga	Syracuse (14771)	-76.103	43.109	3	10
Monroe	Rochester (14768)	-77.677	43.117	4	10
Erie	Buffalo (14733)	-78.736	42.941	5	10
Chautauqua	Dunkirk (14747)	-79.272	42.493	5	10
Niagara	Niagara Falls (04724)	-78.945	43.107	5	10
Steuben	Dansville (94704)	-77.713	42.571	6	10
Chemung	Elmira/Corning (14748)	-76.892	42.159	6	10
Allegany	Wellsville (54757)	-77.992	42.109	6	10
St. Lawrence	Massena (94725)	-74.846	44.936	7	10
Clinton	Plattsburgh (94733)	-73.523	44.687	7	10
Jefferson	Watertown (94790)	-76.022	43.992	7	10
Orange	Montgomery (04789)	-74.265	41.509	8	10
Dutchess	Poughkeepsie (14757)	-73.884	41.627	8	10
Westchester	White Plains (94745)	-73.708	41.067	8	10
Nassau	Farmingdale (54787)	-73.417	40.734	10	10
Suffolk	Islip (04781)	-73.102	40.794	10	10
Suffolk	Shirley (54790)	-72.869	40.822	10	10
New York	New York (94728)	73.967	40.783	11	10
Queens	New York (94789)	-73.796	40.655	11	10
Queens	New York (14732)	-73.881	40.779	11	10

4.6. Pavement Structure and Materials Data

The typical flexible pavement in New York State is a full depth asphalt pavement built with Superpave asphalt mixes. The full depth asphalt flexible pavement structure is divided into three layers:

- Top course layer
- Binder course layer
- Base course layer

In addition, the pavement structure is placed on Asphalt Treated Permeable Base (ATPB) layer. AASHTOWare has no direct tool to simulate ATPB layer. Therefore, a crushed stone layer with resilient modulus of 45000 psi was used to simulate the ATPB (AASHTO, 2008). The CPDM and NYSDOT specifications were used to assemble the required materials data (NYSDOT, 2008). The mix properties for several asphalt concrete mixes produced and compacted by the NYSDOT Asphalt Laboratory and tested at University of Texas at Arlington were used to assemble the aggregate gradation data of the asphalt mixtures.

4.6.1. Pavement Structure Layers Thicknesses

Based on the CPDM typical section, the structural layers thicknesses were assembled. As aforementioned various Hot Mix Asphalt (HMA) and select subgrade thicknesses were used to develop the design cases. The minimum allowed thickness by AASHTOWare is 1.0 inch. Thus, the minimum Asphalt Concrete (AC) thickness was 3 in since NYSDOT uses full depth flexible pavement structure. The thickness of the asphalt concrete base was gradually increased in 0.5 inch increments to obtain the design solutions.

This increase was only applied to the base course layer; the surface and the binder layers has a fixed thickness. However, were kept constant. The assembling of the structure layers thicknesses for all the design cases was done as follows:

- *Asphalt Concrete (AC) surface layer*: Two groups of thicknesses were used:
    - The thickness was select 1.0 in when total AC thickness less than 5.0 in.
    - The thickness was select 1.25 in when total AC thickness greater than or equal to 6.0 in.
- *Asphalt Concrete (AC) binder layer*: Two groups of thicknesses were used:
    - The thickness was select 1.0 in when total AC thickness less than 5.0 in.
    - The thickness was select 2.0 in when total AC thickness greater than or equal to 6.0 in.
- *Asphalt Concrete (AC) base layer*: The HMA thickness was gradually increased or decreased in 0.5 inch increments to reach the satisfied pavement structure layer thickness. In the initial step of this study, the CPDM tables given in Table 2-2 were used as a reference to calculate the base course thickness. Then, the base course layer thickness was increased and decreased from these values.
- *Asphalt Treated Permeable Base (ATPB) layer*: The selected thickness was 4.0 inches, according to Figure 2-1.
- *Subbase course layer*: The selected thickness was 12.0 inches, according to Figure 2-1.

- *Select granular subgrade layer*: The thickness was varied from 0.0 to 12.0 inches in 6.0 inch increments to obtain the optimized structure thickness.
- *Subgrade soil layer:* The thickness was assumed infinite.

4.6.2. Asphalt Concrete Volumetric Properties

The CPDM and NYSDOT specifications do not provide exact volumetric properties for the asphalt mixes since they vary from project to project. Because of this, the recommended volumetric properties inputs by AASHTO were used. The volumetric properties given in Figure 4-4 for the *Air Void content, Asphalt content, Unit Weight and Poisson's Ratio* were used; these are the values recommended for Level 3 design input as screen capture below:.

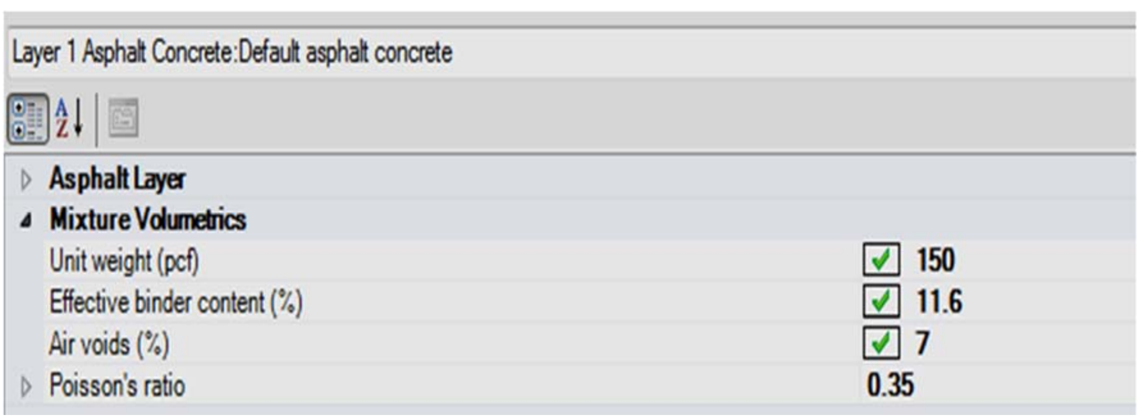


Figure 4-3: Asphalt Concrete Volumetric Properties

4.6.3. Asphalt Concrete Mechanical and Thermal Properties

Since NYSDOT does not have a database of dynamic modulus test results and could not provide Dynamic Shear Rheometer (DSR) test results for representative asphalt binders used in the state, specific (AASHTO, 2008), Level 3 design inputs were used. They are:

- *Hot Mix Asphalt (HMA) Aggregate Gradation:* The HMA aggregate gradations were not mentioned in the CPDM so the NYSDOT mix design specifications and the aggregate gradation for the asphalt concrete samples tested at the University of Texas at Arlington were used to obtain the information:
    - Asphalt concrete wearing course: NMAS 12.5 mm or 9.5 mm is recommended. The aggregate gradation data of this layer are summarized in Table 4-6.
    - Asphalt concrete binder course: NMAS 19 mm or 25 mm is recommended. The aggregate gradation data of this layer are summarized in Table 4-6.
    - Asphalt concrete base course: NMAS 19 mm or 25 mm is recommended. The aggregate gradation data of this layer are summarized in Table 4-6.
- *Select HMA E* Predictive Model:* Since the SuperPave asphalt mixture was used, the shear modulus of the asphalt binders (G*) were used for the equation to predict the dynamic modulus, as shown in Figure 4-5.
- *Reference Temperature:* Since the CPDM and NYSDOT specifications did not mention it, 70 F° was used, as recommended by AASHTO. Figure 4-4 illustrates the reference temperature value in AASHTOWare. It defines the baseline temperature that is used in deriving the dynamic modulus mastercurve (AASHTO, 2008).

Table 4-6: HMA Aggregate Gradation for Downstate and Upstate New York

Aggregate Gradation data for Upstate			
Sieve #	% passing	Layer	Nominal Maximum Aggregate Size
3/4"	100	Top	9.5mm
3/8"	100		
No.4	82		
No 200	4		
3/4"	92	Binder	19 mm
3/8"	67		
No.4	49		
No 200	2		
3/4"	86	Base	25mm
3/8"	67		
No.4	43		
No 200	5		
Aggregate Gradation data for Downstate			
Sieve #	% passing	layer	Nominal Maximum Aggregate Size
3/4"	100	Top	12.5mm
3/8"	89		
No.4	60		
No 200	4		
3/4"	78	Binder	19 mm
3/8"	63		
No.4	48		
No 200	5		
3/4"	65	Base	37.5mm
3/8"	56		
No.4	34		
No 200	4		

- *Asphalt Binder:* Once the Level 3 dynamic modulus was selected, AASHTOWare automatically defines same input level for the binder properties. Therefore, SuperPave performance grade (PG) was required to input. Table 2-3 was used for this reason. The listed PG values in Table 2-3 were according to the AASHTO M

332 binder classification. However, AASHTOWare requires the PG in the form of AASHTO M 320. Hence, NYSDOT suggested to substitute the values in Table 2-3 into AASHTO M 320 classification (NYSDOT, 2014). Table 4-7 compares the PG values according to AASHTO M 332 and 320.

Table 4-7: NYSDOT Binder Substitution Guidance

AASHTO M 320 PG Binder Grade	AASHTO M 332 PG Binder Grade
PG 64-22	PG 64S-22
PG 70-22	PG 64H-22

- *Indirect Tensile Strength:* There is no recommended value by the CPDM and NYSDOT specifications. For this reason, input Level 3 was used, as shown in Figure 4-4.
- *Creep Compliance:* Level 3 inputs were used for the creep compliance at -4°F, 14°F and 32°F due to the unavailability of laboratory measured creep compliance values, as given in Figure 4-5. AASHTOWare automatically calculates the creep compliance values based on the statistical relationship with other input values (AASHTO, 2008).
- *Thermal Properties:* The default values for the thermal conductivity and heat capacity of the asphalt materials were selected. In addition, the default coefficient of thermal contraction for HMA aggregates was selected. Figure 4-4 shown the Thermal properties inputs.

Mechanical Properties	
Dynamic modulus	Input level:3
Select HMA Estar predictive model	Use G* based model (nationally uncalibrated).
Reference temperature (deg F)	70
Asphalt binder	SuperPave:64-22
Indirect tensile strength at 14 deg F (psi)	361.14
Creep compliance (1/psi)	Input level:3
Thermal	
Thermal conductivity (BTU/hr-ft-deg F)	0.67
Heat capacity (BTU/lb-deg F)	0.23
Thermal contraction	1.301E-05 (calculated)
Is thermal contraction calculated?	True
Mix coefficient of thermal contraction (in./in./deg F)	
Aggregate coefficient of thermal contraction (in./in./deg F)	5E-06
Voids in Mineral Aggregate (%)	18.6

Figure 4-4: AC Mechanical and Thermal Properties

Creep compliance level: 3

Loading Time(sec)	Low Temp (-4 deg F)	Mid Temp (14 deg F)	High Temp (32 deg F)
1	2.93692E-07	4.788056E-07	6.546713E-07
2	3.227263E-07	5.589023E-07	8.376009E-07
5	3.655588E-07	6.857071E-07	1.160117E-06
10	4.016979E-07	8.004151E-07	1.484279E-06
20	4.414097E-07	9.34312E-07	1.899019E-06
50	4.999939E-07	1.14629E-06	2.630232E-06
100	5.494233E-07	1.338047E-06	3.365177E-06

Figure 4-5: Input Level 3 Creep Compliance

4.6.4. Aggregate Gradation of Unbound Granular Layers

The granular materials type, AASHTO soil classification, and aggregate gradation were assembled from the NYSDOT specifications. The extracted data of each unbounded layer are:

- *Asphalt Treated Permeable (ATB) Base Layer:* NYSDOT uses the ATPB as base layer, NYSDOT recommended aggregate gradation data given in Table 4-8. There are two types of aggregate gradation are recommended by NYSDOT, the selection is based on the site characteristics. For this research, Type 1 was used since the design cases are hypothetical. It should be noticed in Table 4-8 that all the percentages are based on total weight of aggregate and the asphalt content is based on the total weight of mix. A-1-a AASHTO soil classification was used for this layer because it would resemble the best the ATB material (Kass, et al., 2013).
- *Subbase Course Layer:* It is defined by NYSDOT as any materials that does not consist of concrete, asphalt, glass, brick, stone, sand, gravel or blast furnace slag, Four types of aggregate gradation are recommended by NYSDOT, as shown in Table 4-9. Type 2 was selected to be used in this research at the recommendations of NYSDOT. According to NYSDOT, Type 2 is defined as furnish materials consisting of approved Blast Furnace Slag or of Stone which is the product of crushing or blasting ledge rock, or a blend of Blast Furnace Slag and of Stone. A-1-a AASHTO soil classification was used for this layer.
- *Select granular subgrade layer:* NYSDOT recommended two options, either using well graded rock with particles greatest dimension of 12 inches or any other

materials except well graded rock with no particles greater than 6 in. NYSDOT recommended aggregate gradation is given in Table 4-10. A-1-a AASHTO soil classification was used for this layer.

- *Subgrade soil layer:* It is the surface ground area. There is no available information for this layer in the CPDM or the NYSDOT specification. Thus, A-7-6 AASHTO soil classification was used for this layer.

Table 4-8: ATPB Aggregate Gradation

Mixture Requirements	Permeable Base				Shim	
	Type 1		Type 2		Type 5	
Screen Sizes	General Limits % Passing	Job Mix Tolerance %	General Limits % Passing	Job Mix Tolerance %	General Limits % Passing	Job Mix Tolerance %
2 in	100	-	100	-	-	-
1 1/2 in	95-100	-	75-100	±7	-	-
1 in	80-95	±6	55-80	±8	-	-
1/2 in	30-60	±6	23-42	±7	-	-
1/4 in	10-25	±6	5-20	±6	100	-
1/8 in	3-15	±6	2-15	±4	80-100	±6
No. 20	-	-	-	-	32-72	±7
No. 40	-	-	-	-	18-52	±7
No. 80	-	-	-	-	7-26	±4
No. 200	0-4	±2	-	-	2-12	±2
Asphalt Content %⅔	2-4	NA	2.5-4.5	NA	7-9.5	NA
Mixing and Placing Temperature Range (F°)	225-300		225-301		250-325	

Table 4-9: Subbase Course Layer Aggregate Gradation

Sieve Size Designation	Type			
	1	2	3	4
4 in	-	-	100	-
3 in	100	-	-	-
2 in	90-100	100	-	100
1/4 in	30-65	26-60	30-75	30-65
No. 40	5-40	5-40	5-40	5-40
No. 200	0-10	0-10	0-10	0-10

Table 4-10: Select Granular Subgrade Layer

Sieve Size	Percent Passing by Weight
1/4 in	30 to 100
No. 40	0 to 50
No. 200	0 to 10

4.6.5. Granular Layers Materials Properties and Design Strategies

The CPDM and NYSDOT specifications were reviewed in order to assemble the mentioned properties for the granular layers. Since no available information was found, Level 3 inputs were used for:

- Liquid limit (L.L)
- Plasticity Index (P.I)
- Maximum unit weight (pcf)
- Saturated hydraulic conductivity (ft/hr)
- Specific gravity of the soil
- Optimum gravimetric water content (%)
- User-defined Soil Water Characteristic Curve (SWCC)
- Resilient Modulus (Mr)

Figure 4-6 illustrates an example of the granular materials properties inputs for the select subgrade layer.

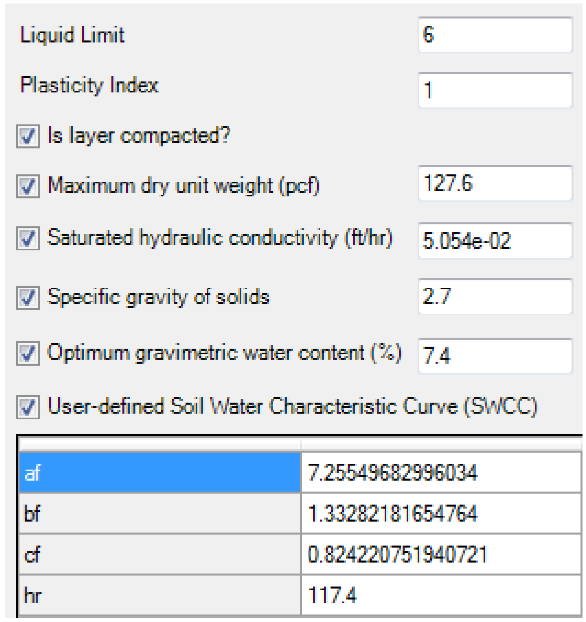


Figure 4-6: Select Subgrade Materials Properties

ATPB was simulated by using crushed stone layer with high quality aggregate according to AASHTO. Resilient modulus (Mr) of ATPB layer was selected to be 45,000 psi. Figure 4-7 shown an example of design input properties for the ATB layer.

The subbase layer was simulated by using crushed stone layer according to the recommended strategy. The resilient modulus (Mr) value was estimated based on Figure 4-8 since no specific value is recommended by NYSDOT. The design inputs and properties of the subbase layer are given in Figure 4-9.

Unbound	
Layer thickness (in.)	4
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5
Modulus	
Resilient modulus (psi)	45000
Sieve	
Gradation & other engineering properties	A-1-a
Identifiers	
Display name/identifier	Crushed stone
Description of object	Default material
Approver	
Date approved	1/1/2011
Author	AASHTO
Date created	1/1/2011
County	
State	
District	
Direction of travel	
From station (miles)	
To station (miles)	
Highway	
Revision Number	0
User defined field 1	

Figure 4-7: Simulated ATPB Layer in AASHTOWare

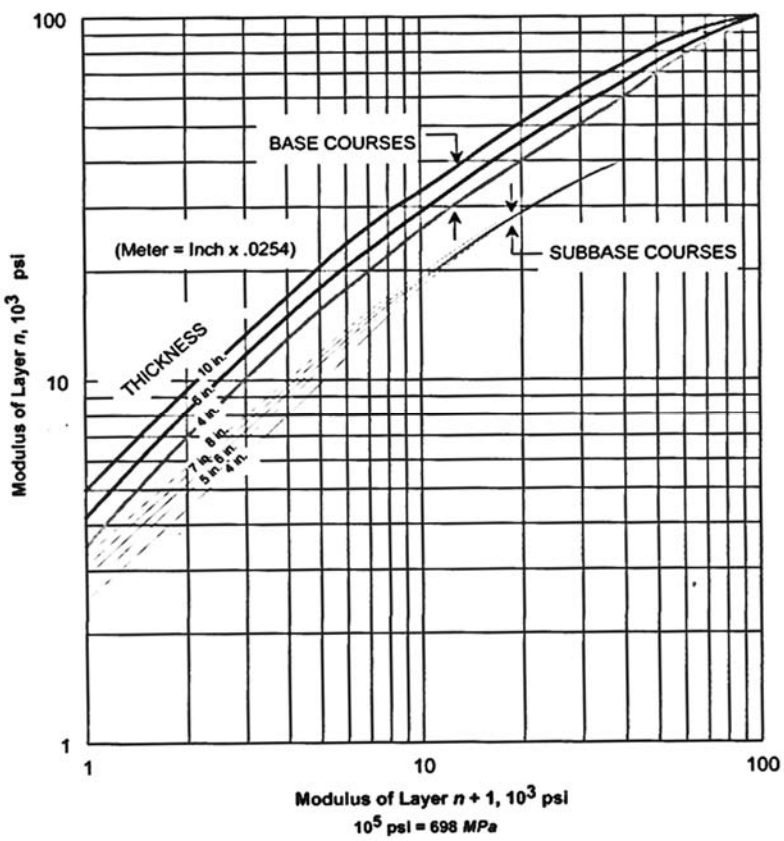


Figure 4-8: Modulus Criteria of Unbound Aggregate Base and Subbase Layers

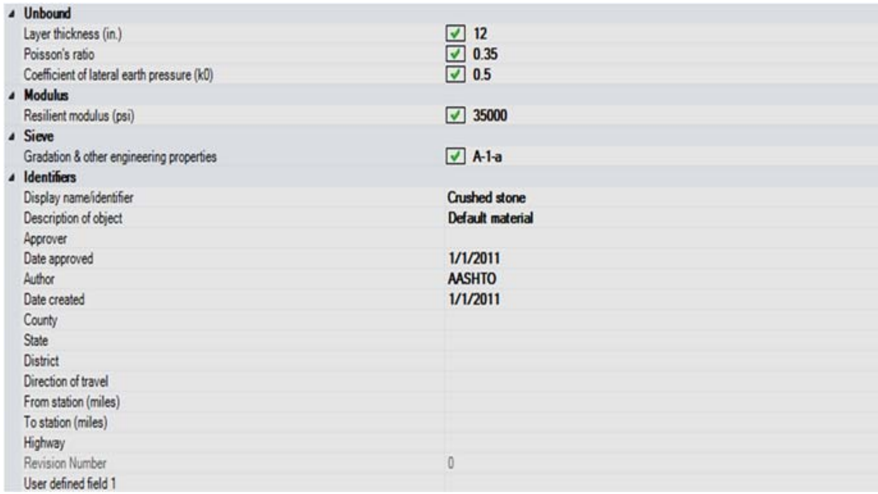


Figure 4-9: Simulated Subbase Course Layer in AASHTOWare

The select subgrade layer material was considered an A-1-a soil. The design inputs and properties of the select subgrade layer are given in Figure 4-10. AASHTO recommended that the resilient modulus of the top granular layer not to exceed three times the resilient modulus of the bottom layer. Figure 4-10 illustrates the materials input and properties of select subgrade layer.

Subgrade soil layer was considered an A-7-6 soil with resilient modulus values varying from 28 to 62 Mpa (4 to 9 ksi) to obtain design tables similar to those in CPDM. It is important to mention that AASHTOWare does not consider the frost susceptibility of the subgrade soil. Figure 4-11 gives the materials inputs and properties for subgrade soil layer. Table 2-2 was used as a reference to input resilient modulus for this layer.

Unbound	
Layer thickness (in.)	6
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5
Modulus	
Resilient modulus (psi)	25000
Sieve	
Gradation & other engineering properties	A-1-a
Identifiers	
Display name/identifier	A-1-a
Description of object	Default material
Approver	
Date approved	1/1/2011
Author	AASHTO
Date created	1/1/2011
County	
State	
District	
Direction of travel	
From station (miles)	
To station (miles)	
Highway	
Revision Number	0
User defined field 1	

Figure 4-10: Simulated Select Subgrade Soil Layer in AASHTOWare

Unbound	
Layer thickness (in.)	Semi-infinite
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5
Modulus	
Resilient modulus (psi)	4061
Sieve	
Gradation & other engineering properties	A-7-6
Identifiers	
Display name/identifier	A-7-6
Description of object	Default material
Approver	
Date approved	1/1/2011
Author	AASHTO
Date created	1/1/2011
County	
State	
District	
Direction of travel	
From station (miles)	
To station (miles)	
Highway	
Revision Number	0
User defined field 1	

Figure 4-11: Simulated Subgrade Soil Layer in AASHTOWare

4.7. Distress Models

In AASHTOWare the JULEA linear-elastic multi-layer model computes the response of the pavement under traffic loads throughout the design period. Then, the pavement response values are used to calculate the evolution of pavement distresses during the same design period. These distress models, also called performance models, were globally calibrated using a large set of the LTPP data for the national calibration. However, the distress models must be calibrated for local or regional conditions.

Momin (2011) successfully calibrated the distress models incorporated in the MEPDG 1.1 pavement design software for the North East (NE) region of the Unites States. The calibration factors he obtained are listed in Table 4-10. However, AASHTOWare Pavement ME 2.1 is the latest release of the Pavement ME Design computer software and it is the only version currently available to the public. Since this version of the software is greatly improved, the model calibration must be repeated for the AASHTOWare software. The calibration coefficients obtained by Momin for the MEPDG1.1 software cannot be used.

The set of calibration coefficients developed in this study for the AASHTOWare software and the calibration coefficients developed by Momin for the MEPDG1.1 software are in Table 4-11. In this research, the AASHTOWare local calibration factors were used to develop the design solutions.

Table 4-11: Calibration Coefficients Used for the Flexible Pavement Performance Models

Distress	Layer	Coeff.	Momin's Study	National	Obtained in this study
Permanent Deformation	HMA	$ß_{r1}$	1.308	1	0.59
		$ß_{r2}$	1	1	1
		$ß_{r3}$	1	1	1
	Base	$ßr_{GB}$	2.0654	1	0.82
	Subgrade	$ßr_{SG}$	1.481	1	0.74
Alligator Cracking	HMA	C_1	-0.06883	1	0.501711
		C_2	1.27706	1	0.227186
Longitudinal Cracking	HMA	C1	-1	7	7
		C2	2	3.5	3.5
		C3	1856	1000	1000
IRI	HMA	C1	51.6469	40	168.709
		C2	0.000218	0.4	-0.0238
		C3	0.0081	0.008	0.00017
		C4	-0.9351	0.015	0.015

Chapter 5

Development of Design Tables for New Flexible Pavement Structures based on AASHTOWare Pavement 2.1

5.1. Overall Concept for Developing the Design Tables

The design solutions were developed by running thousands of AASHTOWare design cases for Upstate New York and Downstate New York. For each case run, the computed distresses were extracted using a macro in Microsoft Excel and were tabulated in Excel spreadsheet files. The selection of the successful design solutions was based on the following design criteria:

- IRI = 225 in/mile
- Total Rutting = 0.75 in

The successful design solutions were determined, for each subgrade soil resilient modulus (Mr) and AADTT combination, as the run cases with the minimum select subgrade thickness and total asphalt layer thickness for which the IRI and total rutting were less than corresponding design criteria (225 in/mile for IRI and 0.75 total rut depth). It was found that the IRI design criteria (225 in/mile) was almost always reached before the design criteria for total rutting (0.75 inches) was reached. This is in agreement with NYSDOT practice of using the IRI as the trigger value for deciding when a distressed flexible pavement must be rehabilitated with an overlay.

Because of the very large number cases run, the results of the runs are available only in electronic form. The electronic file can be obtained from the author or from the

University of Texas at Arlington library. These design solutions were then assembled in separate design tables for each region, in a format similar to that in Table 2-2. The new design tables are given in Appendix E. The US customary units were used for layer thickness at the request of NYSDOT.

For some NYSDOT regions, more than one design table was obtained because more than one climatic station exists in that region, as indicated in Table 4-5. It was thus possible to compare the design tables for locations within the same NYSDOT region. A comparison was also made between the design tables obtained for the Upstate and the Downstate parts of New York State.

In addition, it was important to compare the new design tables derived with the locally calibrated AASHTOWare 2.1 models with the CPDM design table reproduced in Table 5-16. To allow the comparison, the AADTT values were converted into equivalent ESALs values and were added to the design tables. This conversion was possible since the statewide average values of the traffic volume characteristics and axle load spectra for 2010 were used for the AASHTOWare runs.

5.2. The Design Tables for Upstate New York

The design tables were developed for fourteen locations in the Upstate part of the state, as shown in Table 5-1. It can be observed that all regions have at least one location for which the design tables were developed.

Table 5-1: Climatic Stations in Upstate New York

County	Station ID	Region
Saratoge	Albany (14735)	1
Warren	Glens Falls (14750)	1
Oneida	Utica (94794)	2
Onondaga	Syracuse (14771)	3
Monroe	Rochester (14768)	4
Erie	Buffalo (14733)	5
Chautauqua	Dunkirk (14747)	5
Niagara	Niagara Falls (04724)	5
Steuben	Dansville (94704)	6
Chemung	Elmira/Corning (14748)	6
Allegany	Wellsville (54757)	6
St. Lawrence	Massena (94725)	7
Clinton	Plattsburgh (94733)	7
Jefferson	Watertown (94790)	7

5.2.1. Comparison of Design Tables for Region 1

In order to identify difference in the weather data among the studied locations in Region 1, the annual climate statistics are given in Table 5-2. To ease the comparison, the design tables for Region 1 are given in Table 5-3. The table suggests that, in general, the design solutions of Regions 1 for the same subgrade soil resilient modulus (Mr) and AADTT are similar.

Table 5-2: Region 1 Annual Statistics Climate Records

Region 1 Climatic Station	Albany	Glens Falls
Mean annual air temperature (F^0)	48.88	44.8
Mean annual precipitation(in)	35.53	37.27
Freezing Index (^{0}F-days)	1436.7	2667.9
Average annual number of freeze/thaw cycles	68.35	88.9

Table 5-3: Design Thickness of HMA and Select Subgrade Layers for Region 1 (in.)

AADTT	Albany	Glens Falls	Albany	Glens Falls
	Mr = 4 ksi		Mr = 5 ksi	
50	3 / 0	3 / 0	3 / 0	3 / 0
100	4 / 0	4.5 / 0	3 / 0	3 / 0
250	6 / 0	6.5 / 0	5 / 0	5 / 0
500	8.5 / 0	8.5 / 6	7 / 0	7 / 0
1,000	10.5 / 6	10.5 / 6	9.5 / 6	9.5 / 6
2,000	12.5 / 6	12.5 / 6	12 / 6	12 / 6
4,000	14 / 6	14.5 / 6	13.5 / 6	13.5 / 6
5,000	15 / 6	15 / 6	14 / 6	14 / 6
	Mr = 6 ksi		Mr = 7 ksi	
50	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0
250	4 / 0	4.5 / 0	3.5 / 0	3.5 / 0
500	6 / 0	6 / 0	5.5 / 0	5.5 / 0
1,000	8 / 0	8 / 0	7 / 0	7.5 / 0
2,000	11 / 6	11 / 6	10.5 / 6	10 / 0
4,000	13 / 6	13 / 6	12.5 / 6	12.5 / 6
5,000	13.5 / 6	13.5 / 6	13 / 6	13 / 6
	Mr = 8ksi		Mr = 9 ksi	
50	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	4.5 / 0	4 / 0	4 / 0
1,000	6.5 / 0	6.5 / 0	6 / 0	6 / 0
2,000	9 / 0	9 / 0	9 / 0	8.5 / 0
4,000	12 / 6	12 / 6	12 / 0	12 / 0
5,000	13 / 6	13 / 6	12.5 / 6	12.5 / 6

5.2.2. Comparison of Design Tables for Region 5

In order to identify difference in the weather data among the studied locations in Region 5, the annual climate statistics are given in Table 5-4. To ease the comparison, the design tables for Region 5 are given in Table 5-5. The table suggests that, in general, the design solutions of Regions 5 for the same subgrade soil resilient modulus (Mr) and AADTT are dissimilar. Few design solutions are found identical at low AADTT and stiffer soil.

Table 5-4: Annual Climate Statistics for Three Locations in Region 5

Region 5 Climatic Station	Buffalo	Dunkirk	Niagara Falls
Mean annual air temperature (F^0)	48.71	49.65	47.43
Mean annual precipitation(in)	37.62	34.59	31.1
Freezing Index (^{0}F-days)	1279.9	1099.5	1723.1
Average annual number of freeze/thaw cycles	47.36	55.98	52.94

Table 5-5: Design Thickness of HMA and Select Subgrade Layers for Region 5 (in.)

AADTT	Buffalo	Dunkirk	Niagara Falls	Buffalo	Dunkirk	Niagara Falls
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3.5 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
250	5.5 / 0	5.5 / 0	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0
500	7 / 0	7.5 / 0	7 / 0	6 / 0	6.5 / 0	6.5 / 0
1,000	8.5 / 6	9.5 / 6	9 / 6	8 / 0	8.5 / 0	8 / 0
2,000	11 / 6	12 / 6	11 / 6	10 / 0	11 / 6	10 / 6
4,000	13 / 6	13.5 / 6	12.5 / 6	12.5 / 6	13 / 6	12 / 6
5,000	13.5 / 6	14 / 6	13 / 6	13 / 6	13.5 / 6	12.5 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4 / 0	4 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
500	5 / 0	5 / 0	4.5 / 0	4 / 0	4.5 / 0	5 / 0
1,000	7 / 0	7.5 / 0	6.5 / 0	6 / 0	6.5 / 0	6 / 0
2,000	9 / 0	10 / 0	9 / 0	8 / 0	9 / 0	8 / 0
4,000	11.5 / 6	12 / 6	11 / 6	11 / 6	11.5 / 6	11 / 6
5,000	12.5 / 6	13 / 6	11.5 / 6	11.5 / 6	12.5 / 6	11.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	3.5 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	5 / 0	5.5 / 0	5 / 0	4.5 / 0	5 / 0	4.5 / 0
2,000	7.5 / 0	8.5 / 0	7 / 0	6.5 / 0	8 / 0	6.5 / 0
4,000	10 / 6	11 / 6	9.5 / 6	10 / 0	11 / 0	9.5 / 0
5,000	11 / 6	12 / 6	10.5 / 6	10 / 6	11.5 / 6	10 / 6

5.2.3. Comparison of Design Tables for Region 6

The difference in weather data among Region 6 locations was identified by listing the annual climate statistics as shown in Table 5-6. To fulfill the comparison, the design tables of Region 6 locations are listed in Table 5-7. Overall, the design solutions are varied although few cases are identical.

Table 5-6: Annual Climate Statistics for Three Locations in Region 6

Region 6 Climatic Station	Dansville	Elmira	Wellsville
Mean annual air temperature (F^0)	49.14	47.33	45.13
Mean annual precipitation(in)	30.24	31.54	35.87
Freezing Index (^{0}F-days)	1309.3	1611.9	2014.5
Average annual of freeze/thaw cycles	67.97	87.81	55.98

5.2.4. Comparison of Design Tables for Region 7

The weather data are listed in Table 5-8 to identify the difference Region 7 locations. To ease the comparison, the design tables of Region 7 locations are listed in Table 5-9. In general, the design solutions are almost identical due to the small variations in the weather data of Region 7 locations.

Table 5-7: Design Thickness of HMA and Select Subgrade Layers for Region 6 (in.)

AADTT	Dansville	Elmira	Wellsville	Dansville	Elmira	Wellsville
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3.5 / 0	4 / 0	3 / 0	3 / 0	3 / 0
250	5.5 / 0	5.5 / 0	6 / 0	4.5 / 0	4.5 / 0	4.5 / 0
500	8 / 0	7.5 / 0	7.5 / 0	6.5 / 0	6.5 / 0	6 / 0
1,000	9.5 / 6	9.5 / 6	9 / 6	9 / 0	8 / 6	8 / 6
2,000	12 / 6	11.5 / 6	11.5 / 6	11 / 6	11 / 6	10.5 / 6
4,000	13.5 / 6	13.5 / 6	13 / 6	12 / 6	12.5 / 6	12.5 / 6
5,000	14 / 6	14 / 6	13.5 / 6	13.5 / 6	13 / 6	13 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4 / 0	4 / 0	4 / 0	3 / 0	3.5 / 0	3.5 / 0
500	5 / 0	5 / 0	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0
1,000	8 / 0	8 / 0	7 / 0	7 / 0	6.5 / 0	6 / 0
2,000	10 / 6	10 / 0	9.5 / 0	9.5 / 0	9 / 0	8.5 / 0
4,000	12.5 / 6	12 / 6	11.5 / 6	12 / 6	11.5 / 6	11 / 6
5,000	13 / 6	12.5 / 6	12 / 6	12.5 / 6	12 / 6	11.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4 / 0	4 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	6 / 0	6 / 0	5.5 / 0	5 / 0	5 / 0	5 / 0
2,000	9 / 0	8 / 0	8 / 0	8 / 0	7.5 / 0	7 / 0
4,000	11.5 / 6	11 / 6	10.5 / 6	11 / 0	10.5 / 0	10 / 0
5,000	12 / 6	11.5 / 6	11 / 6	12 / 6	11.5 / 6	11 / 6

Table 5-8: Annual Climate Statistics for Three Locations in Region 7

Region 7 Climatic Station	Massena	Plattsburgh	Watertown
Mean annual air temperature (F^0)	44.06	44.92	46.03
Mean annual precipitation(in)	32.8	29.27	33.36
Freezing Index (^{0}F-days)	2866.4	2471.7	2208
Average annual of freeze/thaw cycles	71.95	74.78	71.7

Table 5-9: Design Thickness of HMA and Select Subgrade Layers for Region 7 (in.)

AADTT	Massena	Plattsburgh	Watertown	Massena	Plattsburgh	Watertown
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	4 / 0	4 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
250	6 / 0	6 / 0	6 / 0	5 / 0	5 / 0	5 / 0
500	8 / 0	8 / 0	8 / 0	7 / 0	7 / 0	6.5 / 0
1,000	10 / 6	10 / 6	9.5 / 6	9 / 0	9 / 0	9 / 0
2,000	12 / 6	12 / 6	12 / 6	11 / 6	11 / 6	11 / 6
4,000	14 / 6	14 / 6	13.5 / 6	13 / 6	13 / 6	13 / 6
5,000	14 / 6	14 / 6	14 / 6	13.5 / 6	13.5 / 6	13 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4.5 / 0	4.5 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
500	6 / 0	6 / 0	5.5 / 0	5 / 0	5 / 0	4.5 / 0
1,000	8 / 0	8 / 0	8 / 0	7 / 0	7 / 0	7 / 0
2,000	10 / 6	10 / 6	10 / 0	9.5 / 0	9.5 / 0	9.5 / 0
4,000	12.5 / 6	12.5 / 6	12 / 6	12 / 6	12 / 6	11.5 / 6
5,000	13 / 6	13 / 6	12.5 / 6	12.5 / 6	12.5 / 6	12.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	4.5 / 0	4.5 / 0	4 / 0	4 / 0	3.5 / 0
1,000	6 / 0	6 / 0	6 / 0	6 / 0	6 / 0	5 / 0
2,000	9 / 0	9 / 0	8.5 / 0	8 / 0	8 / 0	8 / 0
4,000	11.5 / 6	11.5 / 6	11 / 6	11.5 / 0	11.5 / 0	11 / 0
5,000	12 / 6	12 / 6	12 / 6	12 / 6	12 / 6	11.5 / 6

5.3. The Design Tables for Downstate New York

The design tables were developed for the listed Downstate New York regions and climatic stations as shown in Table 5-9. It is important to mention that a virtual climatic station was created for Region 9 since there is no data were available from a weather station in that region (see Table 4-5).

Table 5-9: Climatic Stations in Downstate New York

County	Station ID	Region
Orange	Montgomery (04789)	8
Dutchess	Poughkeepsie (14757)	8
Westchester	White Plains (94745)	8
Virtual Climatic Station	Combination of climatic Stations	9
Nassau	Farmingdale (54787)	10
Suffolk	Islip (04781)	10
Suffolk	Shirley (54790)	10
New York	New York City (94728)	11
Queens	New York City (94789)	11
Queens	New York City (14732)	11

5.3.1. Comparison of Design Tables for Region 8

The weather data of Region 8 locations are listed in Table 5-10 to identify variation. The comparison conducted by listing the design tables as shown in Table 5-11. It can be observed that the change in locations affected the design thicknesses for high traffic volumes. At low AADTT, the design solutions were identical in general. However, at high AADTT, the corresponding design solutions are different regardless the stiffness of the subgrade soil Mr.

Table 5-10: Annual Climate Statistics for Three Locations in Region 8

Region 8 Climatic Station	Montgomery	Poughkeepsie	White Plains
Mean annual air temperature (F^0)	49.43	50.42	51.26
Mean annual precipitation(in)	38.2	40.96	94.17
Freezing Index (^{0}F-days)	1274.8	1191.4	852.4
Average annual of freeze/thaw cycles	89.81	86.94	55.96

Table 5-11: Design Thickness of HMA and Select Subgrade Layers for Region 8 (in.)

AADTT	Montgomery	Poughkeepsie	White Plains	Montgomery	Poughkeepsie	White Plains
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	4 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
250	6 / 0	6 / 0	5.5 / 0	4.5 / 0	5 / 0	4.5 / 0
500	7.5 / 0	8 / 0	7 / 0	6.5 / 0	7 / 0	6 / 0
1,000	9.5 / 6	10 / 6	9 / 6	9 / 0	9 / 6	8 / 6
2,000	12 / 6	12 / 6	11 / 6	11 / 6	11 / 6	10 / 6
4,000	13.5 / 6	14 / 6	13 / 6	13 / 6	12.5 / 6	12 / 6
5,000	14 / 6	14.5 / 6	13.5 / 6	13.5 / 6	13 / 6	12.5 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3 / 0
500	5 / 0	5.5 / 0	5 / 0	4.5 / 0	5 / 0	4.5 / 0
1,000	7.5 / 0	8 / 0	7 / 0	7 / 0	7 / 0	6 / 0
2,000	10.5 / 6	10 / 6	9 / 6	9.5 / 0	9.5 / 6	8 / 6
4,000	12.5 / 6	12.5 / 6	11.5 / 6	12 / 6	12 / 6	11.5 / 6
5,000	13 / 6	13 / 6	12 / 6	12.5 / 6	12.5 / 6	12 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	5 / 0	4.5 / 0	4 / 0	5 / 0	4 / 0	4 / 0
1,000	6 / 0	6 / 0	5 / 0	5.5 / 0	6 / 0	5 / 0
2,000	8.5 / 0	9 / 0	7.5 / 0	9.5 / 0	8.5 / 0	7 / 0
4,000	11.5 / 6	11.5 / 6	10 / 6	11 / 0	11 / 0	9.5 / 0
5,000	12 / 6	12 / 6	11 / 6	11.5 / 6	12 / 6	10.5 / 6

5.3.2. Comparison of Design Tables for Region 10

The annual average statistics for the climatic indicator are listed in Table 5-12 for the three studied locations in Region 10.The comparison was fulfilled by tabulating the design tables of Region 10 as shown in Table 5-13. It is clear there are variations due to the climates alteration. However, at low AADDT and stiffer subgrade soil, the designs tend to be similar among the studied locations.

Table 5-12: Annual Climate Statistics for Three Locations in Region 10

Region 10 Climatic Station	Farmingdale	Islip	Shirley
Mean annual air temperature (F^0)	52.72	52.2	51.97
Mean annual precipitation(in)	39.22	39.18	42.09
Freezing Index (^{0}F-days)	637.686	672.3	702.414
Average annual of freeze/thaw cycles	52.18	64.17	73.17

Table 5-13: Design Thickness of HMA and Select Subgrade Layers for Region 10 (in.)

AADTT	Farmingdale	Islip	Shirley	Farmingdale	Islip	Shirley
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3.5 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
250	5 / 0	5 / 0	5 / 0	4 / 0	4 / 0	4.5 / 0
500	7 / 0	7 / 0	7.5 / 0	5.5 / 0	5.5 / 0	6 / 0
1,000	8.5 / 6	8.5 / 6	9.5 / 6	8.5 / 0	8 / 0	8 / 0
2,000	11 / 6	11 / 6	11.5 / 6	10 / 6	10 / 6	10.5 / 6
4,000	13 / 6	13 / 6	13.5 / 6	12.5 / 6	12.5 / 6	12.5 / 6
5,000	13.5 / 6	13.5 / 6	14 / 6	13 / 6	13 / 6	13.5 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3.5 / 0	3.5 / 0	4.5 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	4.5 / 0	6 / 0	4.5 / 0	4.5 / 0	5 / 0
1,000	7 / 0	7 / 0	8 / 0	6 / 0	6 / 0	6 / 0
2,000	9.5 / 6	9 / 6	10.5 / 6	8 / 0	8 / 0	9 / 6
4,000	11.5 / 6	11.5 / 6	12.5 / 6	11 / 6	11 / 6	11.5 / 6
5,000	12 / 6	12 / 6	13.5 / 6	11.5 / 6	11.5 / 6	12 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	3.5 / 0	3.5 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	5 / 0	5 / 0	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0
2,000	7.5 / 0	7.5 / 0	8 / 0	6.5 / 0	6.5 / 0	7 / 0
4,000	10.5 / 6	10.5 / 6	11 / 6	9.5 / 0	9.5 / 0	10 / 0
5,000	11 / 6	11 / 6	11.5 / 6	10.5 / 6	10.5 / 6	11.5 / 6

5.3.3. Comparison of Design Tables for Region 11

The differences in weather data of Region 11 locations was identified by tabulating the annual statistics data as shown in Table 5-14. To facilitate the comparison, the design tables of Region 11 locations are listed in Table 5-15. The listed values show the majority of the design solutions are dissimilar due to the climate variations.

Table 5-14: Annual Climate Statistics for Three Locations in Region 11

Region 11 Climatic Station	NYC 94728	NYC 94789	NYC 14723
Mean annual air temperature (F^0)	55.01	54.14	55.61
Mean annual precipitation(in)	44.39	39.58	42.39
Freezing Index (^{0}F-days)	429.48	429.444	384.084
Average annual of freeze/thaw cycles	31.86	41.74	29.24

Table 5-15: Design Thickness of HMA and Select Subgrade Layers for Region 11 (in.)

AADTT	NYC 94728	NYC 94789	NYC 14732	NYC 94728	NYC 94789	NYC 14732
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	5.5 / 0	5 / 0	5 / 0	4.5 / 0	4 / 0	4 / 0
500	8 / 0	6.5 / 0	7 / 0	6.5 / 0	5 / 0	5.5 / 0
1,000	9.5 / 6	8.5 / 6	8.5 / 6	9 / 0	7.5 / 0	8 / 0
2,000	12.5 / 6	11 / 6	11 / 6	11.5 / 6	9.5 / 6	10 / 6
4,000	14 / 6	12.5 / 6	13 / 6	13.5 / 6	12 / 6	12 / 6
5,000	14 / 12	13.5 / 6	13.5 / 6	14 / 6	12.5 / 6	13 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3.5 / 0	3 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
500	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0	4 / 0	4 / 0
1,000	8 / 0	6.5 / 0	6.5 / 0	7 / 0	5.5 / 0	5.5 / 0
2,000	10.5 / 6	8.5 / 6	9 / 6	10 / 6	7.5 / 6	8 / 6
4,000	13 / 6	11 / 6	11.5 / 6	12.5 / 6	10 / 6	10.5 / 6
5,000	13.5 / 6	11.5 / 6	12 / 6	13 / 6	11 / 6	11.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	6 / 0	4.5 / 0	5 / 0	5.5 / 0	4.5 / 0	4.5 / 0
2,000	9 / 0	7 / 0	7.5 / 0	8.5 / 0	6 / 0	6.5 / 0
4,000	12 / 6	9.5 / 6	10 / 6	11.5 / 0	9 / 0	9.5 / 0
5,000	12.5 / 6	10 / 6	11 / 6	12 / 6	10 / 6	10.5 / 6

5.4 Comparison of Design Tables for Upstate and Downstate New York

Because of differences in the aggregate gradation and PG binder grades for HMA mixes and the climatic conditions in the two parts of the State, it was expected that some differences in design solutions may exist. Tables 5-3, 5-5, 5-8 and Tables 5-11, 5-13 and 5-15 indicate that at low AADTT, the corresponding design solutions are the same for the Upstate and Downstate regardless the subgrade soil. However, at high AADTT and soft subgrade soil, the design solutions for the same traffic and subgrade soil are thicker for the Upstate part than for the Downstate part of New York State.

5.5. Comparison of ME and CPDM Design Tables

To facilitate the comparison, the design solutions of Table 2-2 were converted into US customary units system as shown in Table 5-16. The AADTT values were converted into equivalent ESALs values and were added to the design tables in the Appendix E. Figures 5-1, 5-2, 5-3, 5-4, 5-5, and 5-6 show the design thickness for the asphalt layer in the CPDM (Solid Line) and the newly developed tables (X-Y Scatters) for different traffic volumes, each chart separately for a subgrade layer resilient modulus.

The comparison reveals that for low traffic volumes, the design asphalt layer thickness in the CPDM table is bigger than the corresponding thickness in the newly developed tables. However, for high traffic volumes, the design asphalt layer thickness in the CPDM table is less than the corresponding thickness in the newly developed tables.

At average traffic volume (500 and 1,000 AADTT), the design thickness for the asphalt layers are about the same in both tables. The difference in the design solutions form the two methods can be attributed to the fact that the two methods use different design criteria, design inputs. Moreover, the two design methods rely on different principles and assumptions.

Table 5-16: Design Layer Thicknesses in CPDM Design Table in inches.

NYSDOT CPDM for Mr=4ksi		
ESALs (million)	HMA Thickness	Select Subgrade Thickness
ESALs <= 2	6.6	0
2 < ESALs <= 4	7	0
4 < ESALs <= 8	8	0
8 < ESALs <= 13	9	0
13 < ESALs <= 23	10	0
23 < ESALs <= 45	10	6
45 < ESALs <= 80	10	12
80 < ESALs <= 140	10	18
140 < ESALs <= 300	10	18

NYSDOT CPDM for Mr=5ksi		
ESALs (million)	HMA Thickness	Select Subgrade Thickness
ESALs <= 4	6.6	0
4 < ESALs <= 7	7	0
7 < ESALs <= 13	8	0
13 < ESALs <= 23	9	0
23 < ESALs <= 40	10	0
40 < ESALs <= 70	10	6
70 < ESALs <= 130	10	12
130 < ESALs <= 235	10	18
235 < ESALs <= 300	10	18

NYSDOT CPDM for Mr=6ksi		
ESALs (million)	HMA Thickness	Select Subgrade Thickness
ESALs <= 6	6.6	0
6 < ESALs <= 11	7	0
11 < ESALs <= 20	8	0
20 < ESALs <= 35	9	0
35 < ESALs <= 60	10	0
60 < ESALs <= 110	10	6
110 < ESALs <= 200	10	12
200 < ESALs <= 300	10	18

NYSDOT CPDM for Mr=7ksi		
ESALs (million)	HMA Thickness	Select Subgrade Thickness
ESALs <= 8	6.6	0
8 < ESALs <= 16	7	0
16 < ESALs <= 30	8	0
30 < ESALs <= 50	9	0
50 < ESALs <= 85	10	0
85 < ESALs <= 160	10	6
160 < ESALs <= 300	10	12

NYSDOT CPDM for Mr=8ksi		
ESALs (million)	HMA Thickness	Select Subgrade Thickness
ESALs <= 12	6.6	0
12 < ESALs <= 20	7	0
20 < ESALs <= 40	8	0
40 < ESALs <= 65	9	0
65 < ESALs <= 115	10	0
115 < ESALs <= 215	10	6
215 < ESALs <= 300	10	12

NYSDOT CPDM for Mr=9ksi		
ESALs (million)	HMA Thickness	Select Subgrade Thickness
ESALs <= 15	6.6	0
15 < ESALs <= 30	7	0
30 < ESALs <= 50	8	0
50 < ESALs <= 90	9	0
90 < ESALs <= 150	10	0
150 < ESALs <= 300	10	6

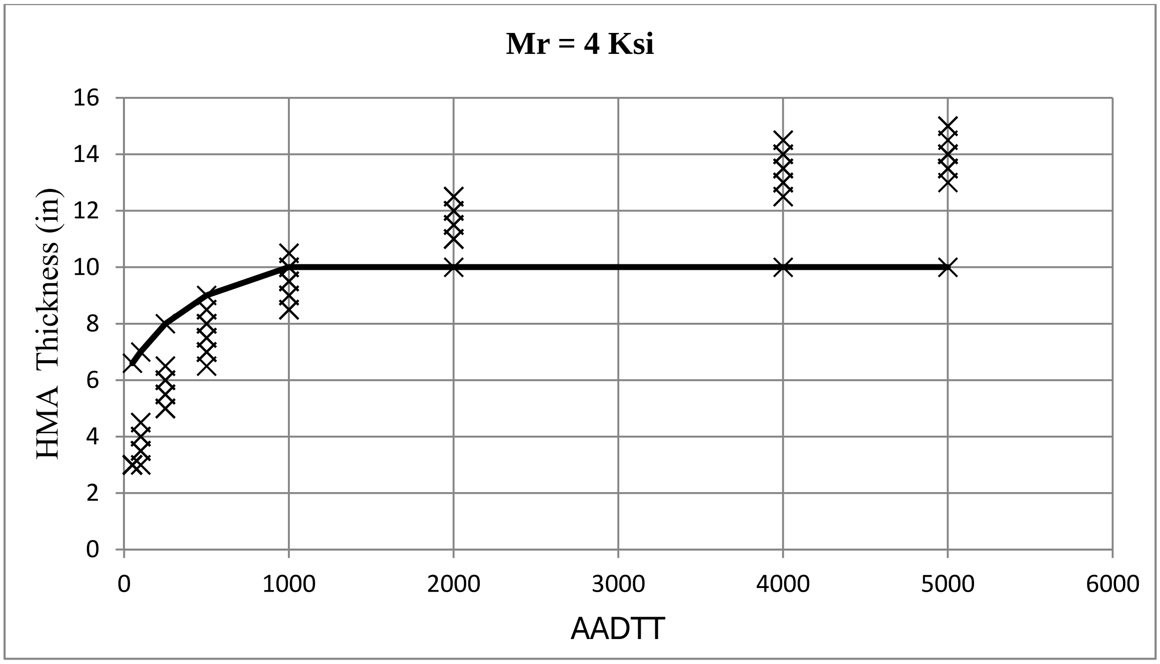


Figure 5-1: AADTT versus HMA Thickness (in) – Mr=4ksi

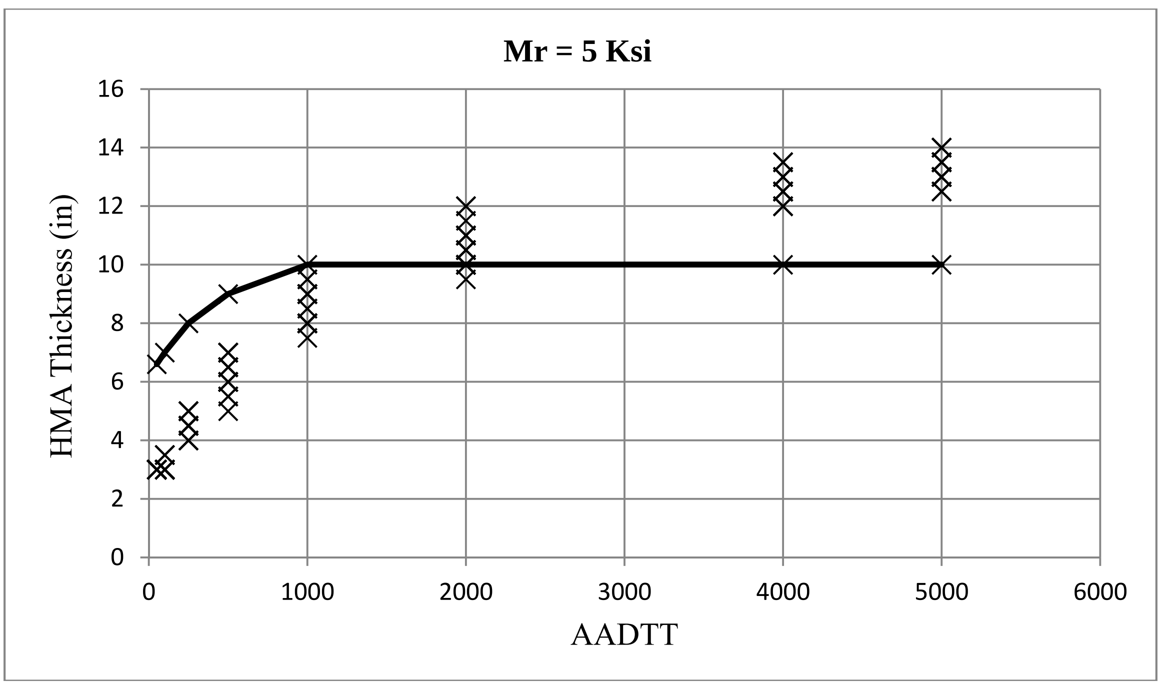


Figure 5-2: AADTT versus HMA Thickness (in) – Mr=5ksi

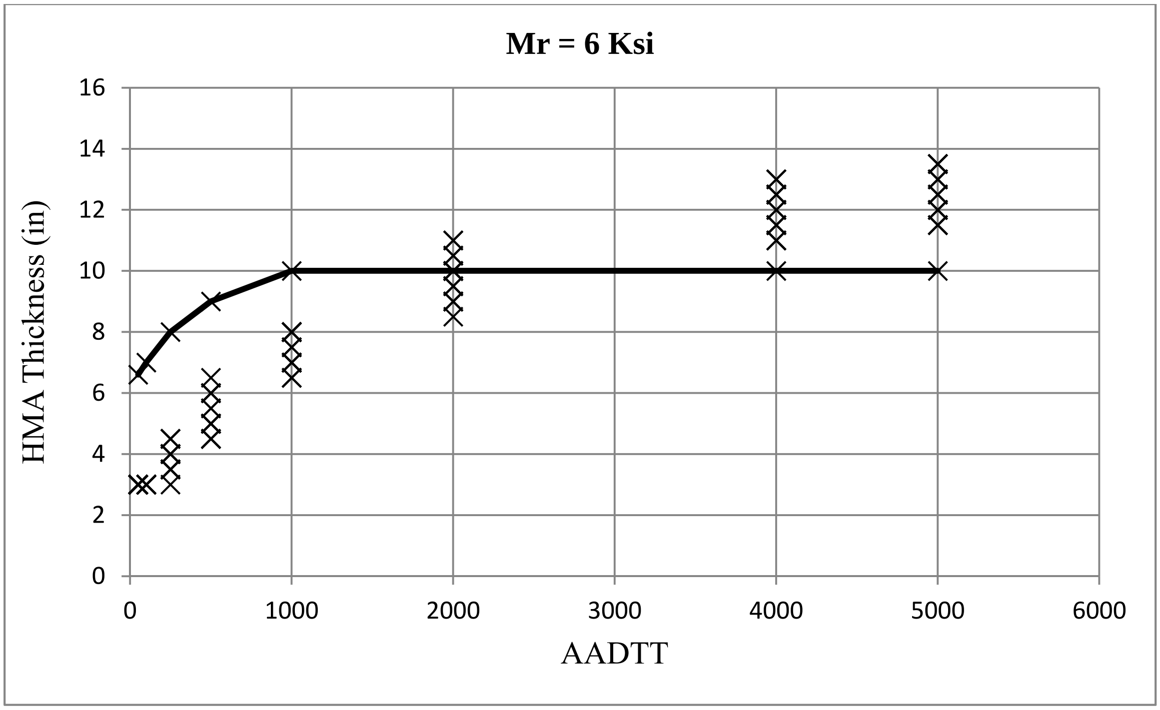


Figure 5-3: AADTT versus HMA Thickness (in) – Mr=6ksi

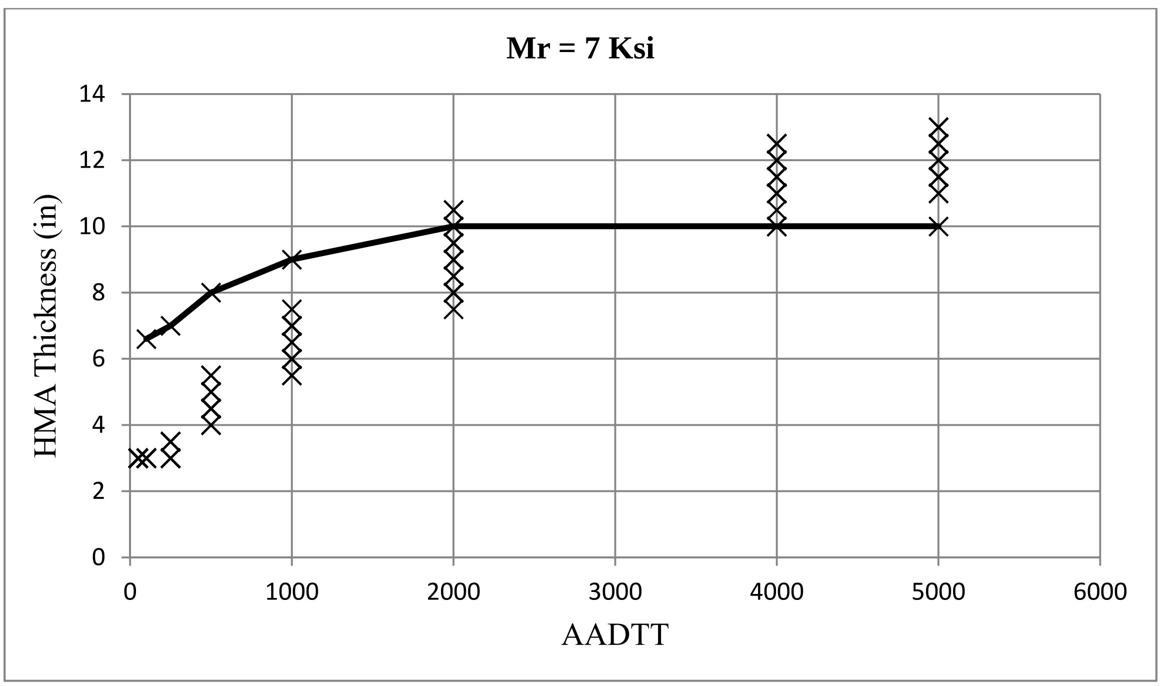


Figure 5-4: AADTT versus HMA Thickness (in) – Mr=7ksi

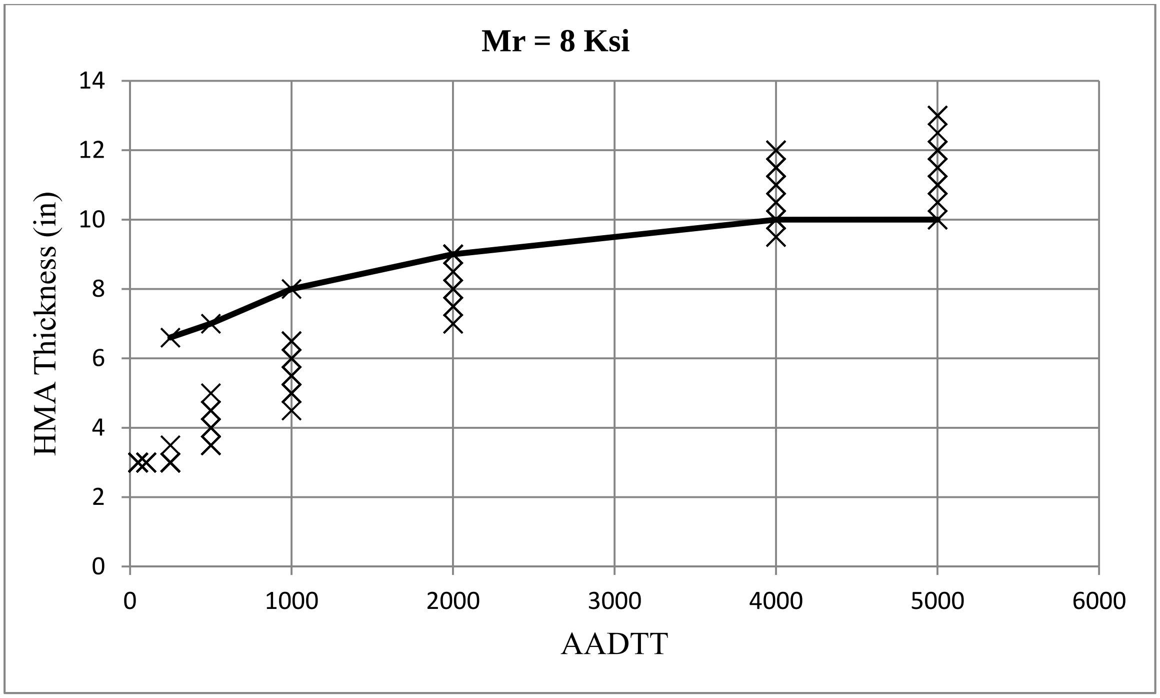


Figure 5-5: ESALs (million) versus HMA Thickness (in) – Mr=8ksi

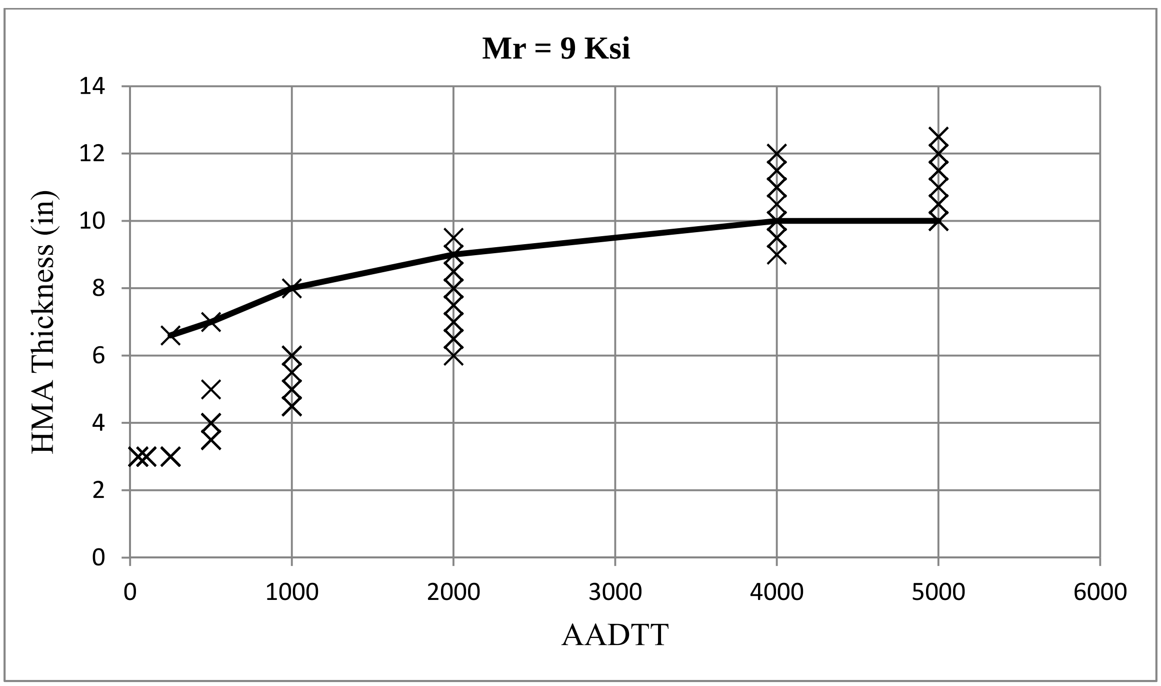


Figure 5-5: ESALs (million) versus HMA Thickness (in) – Mr=9ksi

Chapter 6

Conclusions and Recommendations

The New York State Department of Transportation has decided to use the Mechanistic Pavement Design Guide (MEPDG) for the design of new flexible pavement structures in the future. The process of implementing the use of MEPDG has commenced with the development of a database containing material and traffic inputs as well the calibration of the distress models to local conditions. Since the design of new and reinforced pavement structures is almost exclusively done in NYSDOT regional offices, that likely do not have designers with expertise in running the AASHTOWare Pavement 2.1 software, the most recent MEPDG software program, a more simple design method, based on MEPDG, is needed. This simplified design method could utilize design tables; the designer would need to select the design pavement structure based on a limited number of inputs, directly from these tables. Currently, NYSDOT is using only two tables for the design of new flexible pavements; the NYSDOT design engineers are very familiar with their use. However, these tables were developed based on the AASHTO 1993 Design Guide.

A first objective of this research work was to calibrate the AASHTOWare distress models for the local conditions in New York State. For this purpose, construction, traffic and performance data on 18 LTPP sites in the North East region of the United States were used. The alligator cracking, rutting and IRI models in AASHTOWare were successfully calibrated. The longitudinal cracking and low-temperature cracking models could not be calibrated because the field measured data were erroneous. The calibrated AASHTOWare model can be used for the design of the new flexible pavement structures in New York State.

The second objective of this research work was to develop design tables based on AASHTOWare, to be used by NYSDOT for the design of new flexible pavement structures. The development of the design tables was done by running the calibrated AASHTOWare software for combinations of: climatic conditions, traffic load level, subgrade soil stiffness and pavement structures. The runs were done only for the following conditions:

- Design pavement structure for a new flexible pavement classified as Principal Arterial - Interstate;
- Design reliability level of 90%;
- Analysis period of 15 year.
- Water table depth of 10 feet.
- At least one location for each of the eleven regions of NYSDOT. For Region 9 a virtual weather station was created. For all other regions, the AASHTOWare software contains climatic files for at least one location.
- Statewide average values for traffic volume parameters and for axle load spectra.

Design cases were established as combination of the following design situations:

- Subgrade soil resilient modulus of 28, 34, 41, 48, 55 and 62MPa (4, 5, 6, 7 8 and 9 ksi);
- AADTT in the design lane of 50, 100, 250, 500, 1,000, 2,000, 4,000 and 5,000 trucks
- Pavement structures starting with the design cases included in the Comprehensive Pavement Design Manual. The granular subbase materials and thicknesses recommended by CPDM were used but only the asphalt concrete layer thickness and the select subgrade layer thickness were varied to include several values higher and lower than those recommended by the CPDM. The thickness of the asphalt binder and surface layers were kept constant.

For each design case, the predicted distresses were compared to the corresponding performance criteria, 225 in./mile for IRI and 0.75 inch for total rutting. The design case with the thinnest asphalt base layer for which the predicted distresses were lower than the design criteria was selected as design solution. The design solutions were then assembled in design tables for each of the 24 locations.

The following conclusions were derived from this research:

- The calibration of the rutting, alligator cracking and IRI models was successful
- The methodology used to develop simple design tables was successful. The designer needs only AADTT and Mr to design the pavement structure
- The climates variations have an impact on the design thicknesses; the obtained design tables are different for different locations within the New York State.
- For high truck traffic volumes and soft subgrade soils, the design solutions vary from location to location, even within the same region,. For low traffic volumes, the design solutions are the same throughout the State.
- The design solutions for the Upstate part of New York State ask for thicker asphalt concrete layers than the corresponding design solutions for the Downstate part of the state. This may be explained by differences in binder grade and aggregate gradation for the asphalt mixes used in the two parts of the state and the difference in climatic conditions between the two parts of the state.
- At low AADTT, the new design tables recommend thinner asphalt concrete layers than those recommended in the CPDM table, while at high AADTT the design asphalt layer thickness is higher in the new design tables than in the CPDM table.

The following recommendations are resulting from this study:

- NYSDOT should develop a new flexible pavement performance database. It is recommended to monitor in-service or accelerated pavement structures in order to obtain a larger database of performance and construction data and thus, improve the calibration of the distress models.
- The flexible pavement performance models should be recalibrated if the new pavement performance database will be available or any of the distress models change.
- Additional design tables should be developed for water table depths of less than 10 feet.
- For high AADTT values, a life-cycle cost analysis (LCCA) should be conducted to determine the cost effectiveness of full-depth asphalt pavement designs included in the tables with rigid pavement designs.

APPENDIX A

Extracted Long Term Pavement Performance (LTPP) Traffic Design Inputs from Momin (2011)

Table A-1: Annual Average Daily Truck Traffic

SHRP ID	YEAR	AADTT	SHRP ID	YEAR	AADTT
091803	1992	100	341003	2003	790
091803	1993	110	341003	2004	870
091803	1994	170	341011	1993	1100
091803	1995	190	341011	1994	950
091803	2004	170	341011	1995	1000
091803	2005	170	341011	1996	1050
091803	2006	160	341011	1997	1220
231001	2001	660	341011	1999	1330
231001	2002	640	341011	2000	1340
231001	2003	630	341011	2001	1460
231009	2000	290	341011	2002	1510
231009	2002	290	341011	2003	1590
231009	2003	280	341011	2004	1600
231009	2006	300	341011	2005	1420
231028	2000	250	341011	2007	1230
231028	2001	270	341030	1993	360
231028	2002	310	341030	1994	360
231028	2003	290	341030	1995	350
231028	2004	320	341030	1996	320
231028	2005	300	341030	1997	330
231028	2006	360	341030	1999	390
231028	2007	400	341030	2001	360
251003	1992	100	341030	2006	390
251003	1993	90	341030	2007	330
251003	1994	120	341031	1994	1050
251003	1995	170	341031	1995	1120
251003	1996	230	341031	1996	1040
251003	1997	200	341031	1998	1310
251003	1994	120	341031	1994	1050
251003	1995	170	341031	1995	1120
251003	1994	120	341031	1994	1050

Table A-1: Continued

SHRP ID	YEAR	AADTT	SHRP ID	YEAR	AADTT
251003	1995	170	341031	1995	1120
251003	1996	230	341031	1996	1040
251003	1997	200	341031	1998	1310
251003	1998	200	341031	1999	1340
341003	1994	670	341033	1994	260
341003	1995	750	341033	1995	270
341003	1996	940	341033	2000	320
341003	1997	1520	341033	2002	300
341003	1998	1020	341033	2003	250
341003	1999	640	341033	2004	290
341003	2000	820	341034	1994	1190
341003	2001	830	341034	1995	1180
341003	2002	750	341034	1996	1230
341034	2004	1640	341034	1997	1290
341034	2007	1330	341034	1998	1340
341638	1994	1150	341034	1999	1310
341638	1995	1170	341034	2000	1370
341638	1996	1190	341034	2001	1450
341638	1997	1250	341034	2002	1560
341638	1998	1270	341034	2003	1570
341638	1999	1180	501002	2005	310
341638	2002	1610	501002	2006	490
341638	2003	1910	501002	2007	380
341638	2004	1960	501004	1992	170
341638	2005	1700	501004	1993	160
341638	2007	1350	501004	1994	170
361643	1995	770	501004	1996	210
421597	1998	90	501004	1997	210
421597	1999	90	501004	1998	210
421597	2004	150	501004	1999	180
421597	2005	130	501004	2000	200

Table A-1: Continued

SHRP ID	YEAR	AADTT	SHRP ID	YEAR	AADTT
421597	2006	160	501004	2001	200
421597	2007	130	501004	2002	190
421599	1998	450	501004	2003	180
421599	1999	470	501004	2004	190
421599	2000	510	501004	2005	200
421599	2001	490	501681	1992	400
421599	2003	490	501681	1993	390
421599	2004	490	501681	1994	400
421599	2005	490	501681	1995	400
421599	2006	500	501681	1996	410
421599	2007	530	501681	1997	440
501002	1992	240	501681	1998	490
501002	1993	220	501681	1999	530
501002	1994	220	501683	1992	390
501002	1995	220	501683	1993	380
501002	1996	250	501683	1994	400
501002	1997	260	501683	1995	400
501002	1998	260	501681	2006	710
501002	1999	380	501683	1996	410
501002	2000	370	501683	1997	430
501002	2001	320	501683	1998	470
501002	2002	290	501683	1999	510
501002	2003	300	501683	2000	520
501002	2004	280	501683	2001	550
501681	2000	540	501683	2002	630
501681	2001	560	501683	2003	490
501681	2002	520	501683	2004	510
501681	2003	570	501683	2005	570
501681	2004	660	501683	2006	480
501681	2005	710			

Table A-2: Vehicle Class Distribution

SHRP ID	Year	Vehicle Class										Total
91803	1992	0.87	50.95	17.49	12.75	2.75	14.52	0.51	0.14	0.01	0.01	100
231001	2002	3.45	20.82	2.32	0.05	3.47	57.85	11.03	0.87	0.13	0.01	100
231009	2000	9.49	35.83	10.71	2.05	8.17	23.71	9.89	0.15	0	0	100
231028	2000	6.71	18.51	7.85	2.29	2.5	24.48	37.61	0	0	0.05	100
251003	1993	1.75	56.96	24.43	0.31	7.37	8.88	0.26	0.04	0	0	100
341003	1994	1.05	61.56	9.98	0.24	4.95	21.62	0.5	0.1	0	0	100
341011	1993	1.6	31.16	17.69	1.64	8.9	36.53	1.13	1.19	0.06	0.1	100
341030	1999	1.82	62.91	12.14	4.9	3.95	13.82	0.46	0	0	0	100
341031	1998	1.74	28.45	5.25	9.68	7.25	44.94	1.96	0.7	0.02	0.01	100
341033	2002	2.54	48.96	14.17	1.23	6.12	25.95	0.7	0.26	0.05	0.02	100
341034	1997	2.23	41.07	9.47	3.58	7.68	34.19	1.19	0.55	0.02	0.02	100
341638	1996	1.59	37.31	6.4	3.38	9.68	39.95	1.05	0.61	0.02	0.01	100
421597	2004	4.69	42.94	14.61	3.43	8.21	23.62	0.35	2.11	0.01	0.03	100
421599	2001	1.02	15.98	9.49	9.13	4.55	58.67	0.45	0.54	0.03	0.14	100
501002	1992	3.45	32.84	18.81	1.26	8.21	33.28	0.77	0.74	0.63	0.01	100
501004	1994	1.91	53.98	10.32	0.19	10.21	22.59	0.51	0.1	0.19	0	100
501681	1992	2.52	26.82	8.2	0.39	8.81	50.24	2.24	0.76	0.02	0	100
501683	1992	2.52	26.56	8.62	0.52	9.7	49.86	1.72	0.45	0.04	0.01	100

Table A-3: Monthly Adjustment Factors

Site: 231001-2002

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.2	0.84	1.56	0	1.08	1.2	1.08	1.32	1.344	0
February	1.2	0.96	1.44	0	1.2	1.272	1.08	1.08	1.332	0
March	1.2	0.84	1.92	0	0.96	1.296	1.08	1.32	1.332	0
April	1.32	1.08	1.92	0	1.08	1.296	1.2	1.32	1.332	0
May	1.44	1.32	1.68	6	1.32	1.296	1.32	1.32	1.332	0
June	1.56	1.68	1.44	0	1.5	1.08	0.96	1.44	1.332	0
July	1.32	1.68	2.04	0	1.5	1.08	1.08	1.44	1.332	0
August	0	0	0	0	0	0	0	0	0	0
September	0	0	0	0	0	0	0	0	0	0
October	1.08	1.44	0	6	1.2	1.2	1.44	1.08	1.332	0
November	0.84	1.2	0	0	1.08	1.2	1.44	0.84	1.332	0
December	0.84	0.96	0	0	1.08	1.08	1.32	0.84	0	0

Site: 231009-2000

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	0	0	0	0	0	0	0	0	0
February	0.924	0.792	0.792	1.188	1.056	1.056	0.66	0	0	0
March	1.056	0.792	0.792	1.056	1.32	1.056	0.924	0	0	0
April	1.056	0.66	1.056	1.548	1.188	0.924	1.188	0	0	0
May	1.32	0.924	1.188	1.056	1.32	1.188	1.32	2.64	0	0
June	1.452	1.32	1.452	1.452	1.32	1.452	1.32	2.64	0	0
July	1.452	1.452	1.584	1.188	1.056	1.188	1.452	2.64	0	0
August	1.452	1.848	1.452	1.188	1.32	1.452	1.512	0	0	0
September	1.056	2.112	1.188	0.792	1.056	1.188	0.924	0	0	0
October	1.188	1.452	1.452	1.536	1.32	1.452	1.524	2.64	0	0
November	1.188	1.056	1.452	1.536	1.188	1.188	1.452	2.64	0	0
December	1.056	0.792	0.792	0.66	1.056	1.056	0.924	0	0	0

Site: 231028-2000

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.98	1.056	0.528	1.056	1.188	1.056	1.188	0	0	0
February	2.244	1.056	0.66	1.188	1.188	1.056	1.32	0	0	0
March	1.32	0.924	0.66	1.188	1.188	1.188	1.32	0	0	0
April	1.188	0.792	0.792	0.792	1.188	1.056	1.056	0	0	0
May	0.924	0.924	1.32	0.792	1.188	1.32	1.056	0	0	0
June	0.924	1.188	1.716	1.716	1.32	1.32	1.32	0	0	0
July	0.792	1.452	1.452	0.792	1.188	1.188	1.188	0	0	0
August	0.792	1.716	1.716	2.112	1.452	1.32	1.32	0	0	0
September	1.056	1.716	1.452	1.056	1.188	1.188	1.056	0	0	0
October	1.056	1.584	1.584	1.848	1.188	1.32	1.32	0	0	0
November	0.924	0.792	1.32	0.66	0.924	1.188	1.056	0	0	0
December	0	0	0	0	0	0	0	0	0	0

Site: 251003-1993

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.308	1.188	1.836	0	2.592	1.404	0	0	0	0
February	1.356	1.296	2.16	0	1.728	1.728	0	0	0	0
March	1.356	1.404	1.836	0	2.052	1.404	0	0	0	0
April	1.356	0.972	2.376	0	1.728	2.916	0	0	0	0
May	1.356	1.188	0.864	0	1.08	1.404	0	0	0	0
June	0	1.296	0.432	0	0.324	0.54	0	0	0	0
July	1.356	1.08	0.54	0	0.324	0.54	0	0	0	0
August	1.356	1.08	0.108	0	0.324	0.324	0	0	0	0
September	1.356	1.296	0.648	0	0.648	0.54	0	0	0	0
October	0	0	0	0	0	0	0	0	0	0
November	0	0	0	0	0	0	0	0	0	0

Site: 341003-1994

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	4.2	1.2	0.6	0	0.84	1.44	0.72	0	0	0
February	4.56	1.32	0.6	0	1.08	1.2	0.72	0	0	0
March	0.72	1.2	0.24	0	0.36	0.24	0	0	0	0
April	1.08	1.2	0.6	0	0.6	0.6	0	0	0	0
May	1.44	0.96	0.6	0	0.72	0.6	0.72	0	0	0
June	0	1.2	1.92	1.68	1.8	1.8	1.56	6	0	0
July	0	1.2	2.04	3.48	1.8	1.68	2.28	6	0	0
August	0	1.56	2.28	3.48	2.16	2.04	3	0	0	0
September	0	1.2	2.04	1.68	1.8	1.68	2.28	0	0	0
October	0	0.96	1.08	1.68	0.84	0.72	0.72	0	0	0
November	0	0	0	0	0	0	0	0	0	0
December	0	0	0	0	0	0	0	0	0	0

Site: 341011-1993

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	0	0	0	0	0	0	0	0	0
February	0	0	0	0	0	0	0	0	0	0
March	0	0	0	0	0	0	0	0	0	0
April	0	0	0	0	0	0	0	0	0	0
May	4.032	1.44	1.44	2.4	1.248	1.296	1.464	1.344	0	1.2
June	0	1.248	1.92	1.248	1.248	1.296	1.476	1.152	1.62	1.2
July	0	1.152	1.44	0.96	1.248	1.152	1.476	1.248	0	1.2
August	1.44	1.248	1.44	1.152	1.248	1.248	1.344	1.248	1.596	1.2
September	1.344	1.152	1.056	1.056	1.248	1.152	1.056	1.152	1.596	1.2
October	1.152	1.152	0.768	1.152	1.152	1.152	0.96	1.248	1.596	1.2
November	0.96	1.152	0.864	0.864	1.152	1.152	0.96	1.248	1.596	1.2
December	0.672	1.056	0.672	0.768	1.056	1.152	0.864	0.96	1.596	1.2

Site: 341030-1999

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.452	1.716	0.792	0.264	1.056	0.792	0	0	0	0
February	1.32	0.924	0.924	0.396	0.792	1.056	0.792	0	0	0
March	0	0	0	0	0	0	0	0	0	0
April	1.32	1.32	1.452	1.056	1.32	1.32	2.112	0	0	0
May	0.66	1.056	0.924	1.056	0.924	1.188	1.452	0	0	0
June	1.452	1.188	1.452	1.584	1.584	1.584	1.452	0	0	0
July	1.776	1.188	1.512	1.716	1.584	1.32	1.452	0	0	0
August	1.788	1.32	1.32	2.376	1.98	1.32	1.452	0	0	0
September	1.32	1.188	1.056	0.66	1.452	1.188	0.792	0	0	0
October	1.056	1.056	1.056	0.792	1.056	1.188	1.452	0	0	0
November	0.528	1.056	1.188	1.452	0.66	1.056	0.792	0	0	0
December	0.528	1.188	1.524	1.848	0.792	1.188	1.452	0	0	0

Site: 341031-1998

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	0	0	0	0	0	0	0	0	0
February	0	0	0	0	0	0	0	0	0	0
March	1.32	1.32	1.56	1.32	1.68	1.44	1.56	1.2	12	0
April	0.84	1.08	1.08	0.96	1.44	1.2	1.2	0.72	0	0
May	0.96	1.2	1.08	0.96	1.08	1.08	1.08	0.84	0	0
June	1.56	1.2	1.2	1.32	1.2	1.2	1.32	1.2	0	0
July	1.32	1.32	1.2	1.2	1.2	1.2	0.96	1.08	0	0
August	1.2	1.2	1.08	1.32	1.08	1.2	1.2	1.2	0	0
September	1.32	1.2	1.2	1.32	1.08	1.2	1.2	1.32	0	0
October	1.2	1.2	1.2	1.32	1.08	1.2	1.32	1.32	0	0
November	1.2	1.2	1.2	1.2	1.08	1.2	1.2	1.2	0	0
December	1.08	1.08	1.2	1.08	1.08	1.08	0.96	1.92	0	0

Site: 341033-2002

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.2	0.84	0.96	0.96	1.08	1.2	1.632	1.5	0	0
February	1.2	0.72	0.6	0.36	0.96	0.96	1.08	1.5	0	0
March	0	0	0	0	0	0	0	0	0	0
April	0	0	0	0	0	0	0	0	0	0
May	1.56	1.8	1.32	0.96	1.2	1.32	1.08	1.5	0	0
June	1.2	2.16	1.488	1.32	1.2	1.296	1.08	1.5	0	0
July	1.08	1.56	1.476	1.56	1.38	1.296	1.08	1.5	0	0
August	1.2	1.2	1.2	1.32	1.38	1.296	1.644	1.5	0	0
September	1.2	0.96	1.32	1.68	1.38	1.296	1.08	1.5	0	0
October	1.2	0.96	1.476	1.32	1.38	1.296	1.644	1.5	0	0
November	1.08	0.96	1.2	1.56	1.08	1.2	1.08	0	0	0
December	1.08	0.84	0.96	0.96	0.96	0.84	0.6	0	0	0

Site: 341034-1997

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.08	1.08	1.08	1.08	1.188	1.296	0.864	1.188	0	0
February	1.188	1.188	1.08	0.864	1.188	1.188	0.864	1.296	0	0
March	1.296	1.08	1.08	0.972	1.08	1.08	1.08	1.296	0	0
April	1.296	1.296	1.296	1.188	1.404	1.404	1.728	1.56	10.8	0
May	1.296	1.296	1.296	1.08	1.404	1.404	1.728	1.572	0	0
June	1.296	1.296	1.296	1.512	1.296	1.296	1.62	1.296	0	0
July	0	0	0	0	0	0	0	0	0	0
August	0	0	0	0	0	0	0	0	0	0
September	0	0	0	0	0	0	0	0	0	0
October	1.296	1.296	1.62	1.944	1.404	1.296	1.188	1.296	0	0
November	1.08	1.08	1.188	1.188	0.972	0.972	0.972	0.756	0	0
December	0.972	1.188	0.864	0.972	0.864	0.864	0.756	0.54	0	0

Site: 341638-1996

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	1.056	1.188	0.528	1.056	1.188	0.792	1.188	0	0
February	0	1.188	1.188	0.66	1.188	1.344	1.056	1.188	0	0
March	0.264	1.188	1.32	1.452	1.188	1.356	1.32	1.188	0	0
April	1.584	1.248	1.38	2.376	1.356	1.356	1.716	1.32	0	0
May	1.908	1.272	1.392	1.98	1.368	1.356	1.32	1.32	0	0
June	1.92	1.272	1.188	1.584	1.188	1.188	1.32	1.188	0	0
July	1.716	1.272	1.188	1.452	1.188	1.188	1.584	1.188	0	0
August	1.716	1.272	1.32	1.188	1.368	1.188	1.584	1.452	0	0
September	0	0	0	0	0	0	0	0	0	0
October	1.452	1.188	0.924	0.66	1.188	1.056	0.66	1.188	0	0
November	1.452	1.188	1.056	0.792	1.056	1.056	1.056	1.188	0	0
December	1.188	1.056	1.056	0.528	1.056	0.924	0.792	0.792	0	0

Site: 421597-2004

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.188	1.056	0.528	0.792	0.924	0.924	0	0.792	0	0
February	1.188	1.188	0.528	1.056	1.188	1.056	0	1.32	0	0
March	1.584	1.188	0.528	1.056	1.188	1.056	0	1.32	0	0
April	1.584	1.452	0.924	1.056	1.32	1.188	0	1.584	0	0
May	1.32	1.056	1.32	0.792	1.056	1.188	3.3	1.32	0	0
June	0.792	0.924	1.452	1.584	1.32	1.452	0	1.32	0	0
July	0.528	1.056	1.188	1.056	1.188	1.188	0	1.32	0	0
August	0.792	1.188	2.64	2.112	1.188	1.452	3.3	1.32	0	0
September	1.848	1.188	1.056	1.32	1.32	1.32	0	0.792	0	0
October	1.98	1.584	1.452	1.584	1.32	1.32	3.3	1.32	0	0
November	0.396	1.32	1.584	0.792	1.188	1.056	3.3	0.792	0	0

Site: 421599-2001

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0.72	1.2	0.84	0.96	1.2	1.08	0.6	0.96	0	0
February	0	0	0	0	0	0	0	0	0	0
March	0	0	0	0	0	0	0	0	0	0
April	1.08	1.2	1.2	1.08	1.2	1.2	1.2	1.38	0	1.5
May	1.32	1.272	1.56	1.512	1.2	1.2	1.2	1.356	0	1.5
June	1.32	1.2	1.44	1.524	1.32	1.2	1.8	1.356	0	1.5
July	0.72	1.08	1.32	1.32	1.2	1.2	1.8	1.356	0	0
August	1.08	1.284	1.2	1.524	1.2	1.32	1.2	1.356	0	1.5
September	1.32	1.2	1.2	1.2	1.08	1.2	1.2	1.356	0	1.5
October	1.56	1.284	1.32	1.2	1.32	1.32	1.8	0.96	0	1.5
November	1.32	1.2	0.96	0.84	1.2	1.2	0.6	0.96	0	1.5
December	1.56	1.08	0.96	0.84	1.08	1.08	0.6	0.96	0	1.5

Site: 501002-1992

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.152	0.72	0.864	1.44	1.008	1.296	1.584	0.864	1.776	0
February	1.44	0.576	0.864	1.44	1.008	1.152	0.72	1.536	1.008	0
March	0.864	0.432	1.008	0.864	1.008	1.152	0.72	0.864	1.008	0
April	1.008	1.152	0.864	1.44	1.296	1.296	1.584	1.536	1.008	0
May	1.152	1.296	1.008	0.864	1.152	1.296	0.72	1.536	1.008	0
June	1.44	1.584	1.296	1.44	1.728	1.368	1.584	0.864	1.008	0
July	1.152	1.584	2.16	1.44	1.728	1.296	2.16	1.536	1.008	0
August	1.44	1.44	2.016	0.864	1.44	1.296	1.584	1.536	1.776	0
September	1.44	1.584	1.872	2.88	1.152	1.368	1.584	1.536	1.776	0
October	1.44	1.44	1.008	0.432	1.152	1.008	0.72	0.864	1.008	0
November	0.864	1.296	0.72	0.432	0.864	0.864	0.72	0.864	1.008	0
December	1.008	1.296	0.72	0.864	0.864	1.008	0.72	0.864	1.008	0

Site: 501683-1992

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.296	0.576	0.864	1.152	1.008	1.008	1.152	0.72	0	0
February	1.296	0.576	0.864	1.152	1.008	1.008	1.152	0.72	0	0
March	1.296	0.576	0.864	1.152	1.008	1.152	1.152	0.72	0	0
April	1.296	0.576	0.864	1.152	1.152	1.152	1.152	0.72	0	0
May	0.864	1.296	1.152	1.152	1.152	1.152	1.152	0.72	0	0
June	1.152	1.44	1.296	1.152	1.152	1.152	1.44	1.44	0	0
July	1.296	1.44	1.44	2.016	1.344	1.44	1.872	1.44	0	0
August	1.152	1.512	1.44	1.152	1.356	1.296	1.152	1.44	0	0
September	1.008	1.512	1.296	1.152	1.356	1.296	1.008	1.44	0	0
October	1.584	1.44	1.152	2.016	1.356	1.296	1.152	1.44	0	0
November	1.44	1.44	1.728	1.152	1.356	1.152	1.008	1.44	0	0
December	1.008	1.296	1.296	0.576	1.008	1.152	1.008	1.44	0	0

Site: 501681-1992

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.152	0.576	1.008	0.864	1.008	1.152	1.152	0.864	0	0
February	2.016	0.576	0.864	0.864	1.152	1.152	1.152	0.864	0	0
March	1.008	0.576	1.008	1.536	1.152	1.152	1.152	0.864	0	0
April	0.864	1.152	1.152	1.536	1.296	1.152	1.008	1.296	0	0
May	0.864	1.44	1.296	1.536	1.152	1.152	1.008	0.864	0	0
June	1.152	1.632	1.44	0.864	1.296	1.296	2.016	1.296	0	0
July	1.296	1.632	1.296	1.536	1.296	1.296	1.296	1.296	0	0
August	1.152	1.632	1.44	1.536	1.44	1.296	1.008	1.872	0	0
September	1.44	1.44	1.296	0.864	1.296	1.296	1.296	1.296	0	0
October	1.44	1.44	1.44	1.536	1.152	1.152	1.296	1.296	0	0
November	1.008	1.152	1.152	0.864	1.008	1.152	1.008	1.296	0	0
December	1.008	1.152	1.008	0.864	1.152	1.152	1.008	1.296	0	0

Site: 501004-1994

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	0	0	0	0	0	0	0	0	0
February	0	0	0	0	0	0	0	0	0	0
March	0	0	0	0	0	0	0	0	0	0
April	0	0	0	0	0	0	0	0	0	0
May	1.44	1.056	0.96	0	1.056	1.152	1.92	0	4.8	0
June	1.44	1.152	1.248	0	1.44	1.32	1.92	0	4.8	0
July	1.152	1.272	1.248	0	1.248	1.152	0	0	0	0
August	0.768	1.272	1.248	0	1.248	1.308	1.92	0	0	0
September	1.44	1.272	1.248	0	1.248	1.308	0	0	0	0
October	1.44	1.272	1.248	0	1.248	1.248	1.92	0	0	0
November	1.152	1.152	1.152	0	1.056	1.152	1.92	0	0	0
December	0.768	1.152	1.248	9.6	1.056	0.96	0	0	0	0

Table A-4: Axles Per Truck

Site	Axles	Vehicle Class									
		4	5	6	7	8	9	10	11	12	13
091803	Single	1.84	2	1	1	2.36	1.05	1.01	2	4	1
	Tandem	0.67	0	1	0.11	0.72	1.96	0.99	0	1	0
	Tridem	0	0	0	1	0.82	0.08	0.99	1	0	2
	Quad	0	0	0	0	0	0	0.26	0	0	0
231001	Single	1.83	2.14	1	1.01	2.34	1.47	1.01	5	4	1.71
	Tandem	0.17	0.04	1	0.02	0.66	1.76	1.09	0	1	1.82
	Tridem	0	0	0	0.85	0	0	0.91	0	0	0.65
	Quad	0	0	0	0.13	0	0	0	0	0	0
231009	Single	1.76	2.11	1	1	2.19	1.21	1.03	5	4	0
	Tandem	0.24	0.03	1	0	0.81	1.89	1.22	0	1	0
	Tridem	0	0	0	1	0	0	0.78	0	0	0
	Quad	0	0	0	0	0	0	0	0	0	0
231028	Single	1.57	2.17	1	1	2.35	1.45	1.01	5	4	1.16
	Tandem	0.43	0.03	1	0	0.64	1.77	1.11	0	1	0.32
	Tridem	0	0	0	0.99	0	0	0.89	0	0	1.81
	Quad	0	0	0	0.01	0	0	0	0	0	0
251003	Single	1.87	2	1	1	2.18	1.04	1	2.75	0	0
	Tandem	0.64	0.04	1	0	0.83	1.96	1.47	1	0	0
	Tridem	0	0	0	1	0.11	0.17	0.97	0.25	0	0
	Quad	0	0	0	0	0	0	0	0	0	0
341003	Single	1.37	2	1	0.91	2.34	1.07	1.02	2.04	2.5	1
	Tandem	0.66	0.01	1	1.13	0.66	1.95	1.01	0.55	1	0.4
	Tridem	0	0	0	0.91	0	0.02	0.99	1	0.5	2
	Quad	0	0	0	0	0	0	0.31	0	0	0.4
341011	Single	1.33	2	1	0.99	2.11	1.08	1.04	4.12	3.86	1.02
	Tandem	0.67	0	1	0.14	0.89	1.95	1	0.11	1.05	0.9
	Tridem	0	0	0	0.99	0	0.01	0.96	0.37	0.51	1.35
	Quad	0	0	0	0	0	0	0.15	0	0	0.86
341030	Single	1.53	2	1	0.98	2.41	1.1	1.02	0	0	0
	Tandem	0.47	0	1	0.04	0.59	1.95	1.11	0	0	0
	Tridem	0	0	0	0.97	0	0	0.86	0	0	0
	Quad	0	0	0	0.01	0	0	0	0	0	0
341031	Single	1.4	1.99	1	1	2.17	1.09	1.01	4.86	2.53	1.24
	Tandem	0.6	0.01	1	0.02	0.83	1.95	1	0.09	1.16	1.12
	Tridem	0	0	0.01	1	0	0	0.99	0.12	1.09	1.9
	Quad	0	0	0	0	0	0	0.03	0	0	0.84

Table A-4: Continued

Site	Axles	Vehicle Class									
		4	5	6	7	8	9	10	11	12	13
41033	Single	1.61	2.04	1	0.91	2.27	1.16	1.01	4.34	1.33	1
	Tandem	0.39	0.01	1	0.45	0.67	1.91	1.47	0.27	1.08	0.79
	Tridem	0	0	0	0.63	0.02	0	0.52	0	0.72	1
	Quad	0	0	0	0	0	0	0	0	0	0.29
341034	Single	1.48	2	1	0.99	2.19	1.09	1.01	4.64	3.21	1.13
	Tandem	0.52	0	1	0.06	0.81	1.95	1	0.11	0.95	1.15
	Tridem	0	0	0	0.99	0	0.01	0.99	0.23	1.02	1.06
	Quad	0	0	0	0	0	0	0.07	0	0	1.04
341638	Single	1.51	2	1	1	2.19	1.08	1.01	4.69	3.18	1.24
	Tandem	0.49	0	1	0.05	0.81	1.95	1	0.1	1.31	1.86
	Tridem	0	0	0	1	0.01	0.01	0.99	0.19	0.72	1.81
	Quad	0	0	0	0	0	0	0.08	0	0	0.92
421597	Single	1.91	2	1	1	2.26	1.26	1.06	5	4	1.13
	Tandem	0.09	0	1	0	0.74	1.87	1.12	0	1	0.5
	Tridem	0	0	0	1	0	0	0.86	0	0	0.88
	Quad	0	0	0	0	0	0	0	0	0	0.63
421599	Single	1.94	2	1	1	2.33	1.23	1.02	5	4	2.65
	Tandem	0.06	0	1	0	0.67	1.89	1.16	0	1	1.65
	Tridem	0	0	0	1	0	0	0.83	0	0	0.38
	Quad	0	0	0	0	0	0	0	0	0	0.09
501002	Single	1.24	2	1	0.96	2.14	1.02	1.06	2.99	2	1.25
	Tandem	0.76	0.01	1	0.66	0.86	1.98	1.03	1.01	2	2.5
	Tridem	0	0	0	0.96	0	0.02	0.97	0	0.14	1
	Quad	0	0	0	0	0	0	0.39	0	0	0
501004	Single	1.71	2	1	0.89	2.24	1.12	1.07	2	1.54	1
	Tandem	0.42	0	1	1.67	0.77	1.93	1.07	1.08	1.49	1
	Tridem	0	0	0	0.89	0	0.03	0.94	1	0.72	0
	Quad	0	0	0	0	0	0	0	0	0	1
501681	Single	1.3	1.99	1	0.93	2.16	1.03	1.03	3.02	2.28	1
	Tandem	0.7	0.01	1	1.03	0.84	1.97	1.02	0.99	1.86	2.5
	Tridem	0	0	0	0.93	0.01	0.01	0.98	0	0	1.17
	Quad	0	0	0	0	0	0	0.12	0	0	0
501683	Single	1.36	2	1	0.97	2.14	1.02	1.1	3	2.02	2.08
	Tandem	0.64	0.01	1	0.78	0.85	1.98	1.08	1	1.89	1.33
	Tridem	0	0	0	0.97	0	0.01	0.94	0	0.82	1.08
	Quad	0	0	0	0	0	0	0.13	0	0	0

APPENDIX B

Extracted Long Term Pavement Performance (LTPP) Structural and Materials Properties Design Inputs from Momin (2011)

Table B-1: General Information on the Selected LTPP Sections

STATE CODE	SHRP ID	CONSTRUCTION DATE		NO. Of LTPP LANES	TOTAL LANES	Functional Class	Direction
		1	2				
9	1803	1-Jul-88	17-Jan-95	1	2	Rural Major Collector	N
23	1001	1-Jul-88	6-Jun-95	2	4	Rural Principal Arterial - Interstate	N
23	1009	1-Jul-88	22-Aug-93	1	2	Rural Principal Arterial - Other	N
23	1028	1-Jul-88	12-May-92	1	2	Rural Principal Arterial - Other	E
25	1003	1-Jun-88	7-Jun-88	1	2	Urban Other Principal Arterial	N
34	1003	1-Aug-88	8-Apr-94	2	4	Rural Minor Arterial	N
34	1011	1-Jul-88	28-Apr-98	2	4	Rural Principal Arterial - Interstate	E
34	1030	1-Dec-88	24-Feb-91	2	4	Rural Principal Arterial - Other	S
34	1031	1-Jul-88	4-Apr-96	2	4	Urban Principal Arterial - Other Freeways	N
34	1033	1-Jul-88	11-Sep-97	2	4	Rural Principal Arterial - Other	S

Table B-1: Continued

34	1034	1-Dec-88	-	2	4	Urban Principal Arterial - Other Freeways	S
34	1638	1-Dec-88	-	2	4	Urban Principal Arterial - Other Freeways	N
36	1008	1-May-89	25-Aug-89	2	4	Urban Other Principal Arterial	E
36	1011	1-Jun-88	14-Sep-93	2	4	Urban Principal Arterial - Interstate	S
36	1643	1-May-89	12-Oct-89	1	2	Rural Principal Arterial - Other	N
36	1644	1-May-89	19-Jun-96	1	2	Rural Minor Arterial	W
42	1597	1-Aug-88	12-Jun-90	1	2	Rural Minor Arterial	E
42	1599	1-Aug-88	1-Jun-99	1	2	Urban Other Principal Arterial	W
50	1002	1-Aug-88	-	1	2	Rural Principal Arterial - Other	N
50	1004	1-Aug-88	6-Oct-98	1	2	Rural Principal Arterial - Other	E
50	1681	1-Jun-89	8-Sep-91	1	2	Rural Principal Arterial - Other	N
50	1683	1-Jun-89	23-Sep-91	1	2	Rural Principal Arterial - Other	S

Table B-2: Gradation Data of HMA Aggregates

STATE CODE	SHRP ID	LAYER NO	1	7/8	3/4	5/8	1/2	3/8	#4	#8	#10	#16	#30	#40	#50	#80	#100	#200
			Percent Passing															
23	1001	1												63				45
23	1001	2							63					17				3
23	1001	3					51							8				3
23	1001	4						87	74	64					10			2
23	1001	5	88		62		49	44	36	31		27			11		3	2
23	1001	6	100		99		70		39	33		27			13		7	3
23	1001	7	100	100	100	100	100	98	41	18			9					4
50	1002	2	75				51		24									4
50	1002	3	75		60		52		31	23								1
50	1002	4	100		99		81	71	52	38		29	20		10			2
50	1002	5	100	100	100		99	82	64	48		34	23		12			3
25	1003	1												70				20.3
25	1003	2	83		77		71	66	56		47	31		16		6		3
25	1003	3	100	100	93		65		35		25			12				2
25	1003	4	100	100	100	100	100	88	60		39	26		18		10		4

Table B-2: Continued

STATE CODE	SHRP ID	LAYER NO	1	7/8	3/4	5/8	1/2	3/8	#4	#8	#10	#16	#30	#40	#50	#80	#100	#200
			Percent Passing															
34	1003	2			86				56						9			5
34	1003	3	98				70		50	40					16			
34	1003	4	100	100	100	100	100	98	69	50					19			7
50	1004	1												77				19.5
50	1004	2	69				46		30									5
50	1004	3	79		60		48		28	23								2
50	1004	4	100	100	100		83	72	55	40		29	20		13			3
50	1004	5	100	100	100	100	100	84	61	47		35	25		16			3
23	1009	2												16				2
23	1009	3					61							12				3
23	1009	4			64			47	42	34		25			10		8	3
23	1009	5	100	100	100	100	100	99	71	51		38	25		15		8	4
34	1011	2			91				73						18			4
34	1011	3			87				49	37					15			6
34	1011	4	100	100	100	100	100	98	72	46					18			6

Table B-2: Continued

STATE CODE	SHRP ID	LAYER NO	1	7/8	3/4	5/8	1/2	3/8	#4	#8	#10	#16	#30	#40	#50	#80	#100	#200
			Percent Passing															
23	1028	1												13				1.2
23	1028	2												16				5
23	1028	3					61							16				3
23	1028	4	100		96		77	59	40	32		26	18		12		6	2
23	1028	5	94		73		55	44	35	29		23	16		11		8	3
34	1030	2	100	100	100	100	100		95									6
34	1030	3			67				52					25				6
34	1030	4	7			3												
34	1030	5			83				48	42					17			6
34	1030	6	100	100	100	100	100	97	62	51					19			6
34	1031	2			94				69						12			6
34	1031	3	99				69		36	30					13			3

Table B-2: Continued

STATE CODE	SHRP ID	LAYER NO	1	7/8	3/4	5/8	1/2	3/8	#4	#8	#10	#16	#30	#40	#50	#80	#100	#200
				Percent Passing														
34	1031	4	100	100	100	100	100	93	60	48					18			6
34	1033	2			81				47						11			4
34	1033	3	100				77		49	40					15			7
34	1033	4	100				75		45	32					12			5
34	1033	5	100	100	100	100	100	98	70	51					18			7
34	1034	2	100				74		45	38					16			6
34	1034	3	100		98		82	71	46	40					16			5
42	1597	2	100		76			53	37	27		20						5
42	1597	3																
42	1597	4	100	100	100	100	100	90	63	45		33	23		15		9	7
42	1599	2			76			51	24			6						3
42	1599	3			90		69	57	36	25		16	11		8		6	4.5
42	1599	4	98				69	57	36	25		16	11		8		6	4.5
42	1599	5	100	100	100	100	100	95	60	42		26	17		11		8	5.5
34	1638	3	100				74		45	38					16			6

Table B-2: Continued

STATE CODE	SHRP ID	LAYER NO	1	7/8	3/4	5/8	1/2	3/8	#4	#8	#10	#16	#30	#40	#50	#80	#100	#200
			Percent Passing															
34	1638	4	100		98		82	71	46	40					16			5
50	1681	1												17.6				10.2
50	1681	3	69		66		61	57	47	36		26	18		10		5	3
50	1681	5	100	100	100		93	76	53	37		29	24		20			5
50	1683	1												51.7				41.5
50	1683	3	86		83		78	73	60	51		40	31		20		10	6
50	1683	5	100	100	100		92	79	54			29	23		19			6
9	1803	2						47			34			17			5	2
9	1803	3	100		72				35	30					14			4
9	1803	4	100	100	100		99	78	52	42					17			5

Table B-3: Binder Content

STATE CODE	SHRP ID	LAYER NO	MAX SP. GRAVITY	BULK SP.GRAVITY MEAN	ASPHALT CONTENT MEAN	PERCENT AIR VOIDS MEAN	VOIDS MINERAL AGGREGATE	EFFECTIVE ASPHALT CONTENT
23	1001	4		2.24	4			
23	1001	5	2.49	2.38	5.1	4.3	15	
23	1001	6	2.47	2.33	5.4	5.7	14.7	
23	1001	7	2.512	2.455	6.2	10.8	22.3	
33	1001	5	2.521	2.41	4.5	6.7	15.3	
33	1001	6	2.457	2.34	6.3	4.9	17.7	
25	1002	4	2.67	2.53	4.4	4.8		
25	1002	5	2.58	2.33	6.3	8.8		
50	1002	4	2.488	2.382	5.5	4.2	15.6	4.9
25	1003	3	2.45	2.27	5	6.5		
25	1003	4	2.39	2.26	6.4	5.3		
25	1004	4	2.63	2.54	4.5	3.6		
25	1004	5	2.63	2.54	4.5	3.6		
50	1004	3	2.502	2.389	5	4.5	14	4.1
50	1004	4	2.471	2.38	5.5	3.7	14.2	4.5
50	1004	5	2.439	2.359	6.2	3.1	15.4	5.3
23	1009	4	2.49	2.41	5.1		15.5	
23	1009	5	2.415	2.405	7.1	7.2	16.8	

Table B-3: Continued

STATE CODE	SHRP ID	LAYER NO	MAX SP.GRAVITY	BULK SP.GRAVITY MEAN	ASPHALT CONTENT MEAN	PERCENT AIR VOIDS MEAN	VOIDS MINERAL AGGREGATE	EFFECTIVE ASPHALT CONTENT
23	1012	4	2.448	2.405	5.2	1.7	13.5	5
23	1012	5	2.397	2.39	6.5	0.2	15.3	6.4
23	1026	4	2.545	2.48	5	2.7	14.9	
23	1026	5	2.515	2.455	5	5	16.6	
23	1028	4	2.52	2.36	5.1	6.5	18	
23	1028	5	2.5	2.34	5.1	6.5	17.7	
42	1599	3	2.637		3.4			
42	1599	4	2.571	2.486	4.6	3.3	14	4.3
42	1599	5	2.522	2.425	6	3.9	16.3	5.3
9	1803	3	2.546		4.3	7.6		
9	1803	4	2.526	2.449	5.2	3.1	15.7	
34	1003	3			4.4			
34	1003	4			5.8			
34	1011	3			5			
34	1011	4			5.8			
34	1030	5			4.2			
34	1030	6			5.4			
34	1031	3			4.6			
34	1031	4			5.6			

Table B-3: Continued

STATE CODE	SHRP ID	LAYER NO	MAX SP.GRAVITY	BULK SP.GRAVITY MEAN	ASPHALT CONTENT MEAN	PERCENT AIR VOIDS MEAN	VOIDS MINERAL AGGREGATE	EFFECTIVE ASPHALT CONTENT
34	1033	3			4.7		16.4	
34	1033	4			4.7		16.6	
34	1033	5			5.9		19.5	
34	1034	2			4.9			
34	1034	3			4.4		13.9	
34	1638	3			4.4			
34	1638	4			4.9			

Table B-4: Binder Gradation

STATE CODE	SHRP ID	Layer No	AC Grade	AC SG	AC viscosit y 140 f	AC viscosit y 275 f	AC Penetration 77 F	Lab viscosity 140 f	Lab viscosity 275 f	Lab Duct. 77 f	Lab penetration 77 f
23	1001	4	AC-10	1.031	1058	350	114	1120	323.8	150	56
23	1001	5	AC-10	1.031	1058	350	114	1120	323.8	150	56
23	1001	6	AC-10	1.031	1058	350	114	1120	323.8	150	56
23	1001	7	AC-20	1.04	1810	418.33	83	1800	425	150	48
50	1002	3	85-100 pen	1.022	1144	308	92				
50	1002	4	85-100 pen	1.022	1144	308	92				
50	1002	5	85-100 pen	1.022	1144	308	92				
25	1003	3	AC-20	1.026	2064	401	73	4042			
25	1003	4	AC-20	1.026	1772	377	82	3976			
34	1003	3	AC-20	1.025	2021		72				
34	1003	4	AC-20	1.025	2021		72				
50	1004	3	85-100 pen	1.022	1159	311	90				58
50	1004	4	85-100 pen	1.023	1159	311	90				60
50	1004	5	85-100 pen	1.023	1159	311	59				60
23	1009	4	85-100 pen	1.023	1778	400	89			150	58
23	1009	5	85-100 pen	1.023	1765	390.5	90			150	60
34	1011	3	85-100 pen	1.025			91				
34	1011	4	85-100 pen	1.029			91				

Table B-4: Continued

STATE CODE	SHRP ID	Layer No	AC Grade	AC SG	AC viscosity 140 f	AC viscosity 275 f	AC Penetration 77 F	Lab viscosity 140 f	Lab viscosity 275 f	Lab Duct. 77 f	Lab penetration 77 f
36	1011	4	AC-20	1.024							
23	1028	4	AC-10	1.014	1125	311	120	2420		150	74
23	1028	5	AC-10	1.014	1125	311	120	2420		150	74
34	1030	5	AC-20	1.025							
34	1030	6	AC-20	1.025							
34	1031	3	AC-20	1.025	1793	465	74				
34	1031	4	AC-20	1.025	1968	412	70				
34	1033	3	AC-20	1.025	2124	446	67				
34	1033	4	AC-20	1.025	2124	446	67				
34	1033	5	AC-20	1.025	2124	446	67				
34	1034	2	AC-20	1.02	2108	406	77				
34	1034	3	AC-20	1.02	2108	406	77				
42	1597	3	AC-20		2000						
42	1597	4	AC-20	1.01	2000						
1599	42	3	AC-20	1.024	2037	452	79				
1599	42	4	AC-20	1.024	2037	452	79				
1599	42	5	AC-20	1.024	2037	452	79				

Table B-4: Continued

STATE CODE	SHRP ID	Layer No	AC Grade	AC SG	AC viscosity 140 f	AC viscosity 275 f	AC Penetration 77 F	Lab viscosity 140 f	Lab viscosity 275 f	Lab Duct. 77 f	Lab penetration 77 f
34	1638	3	AC-20	1.02	2108	406	77				
34	1638	4	AC-20	1.02	2108	406	77				
50	1681	5	85-100 pen	1.01							
50	1683	5	85-100 pen	1.01							
9	1803	3	AC-20	1.01	2052		69	4462			54
9	1803	4	AC-20	1.01	2052		69	4462			54

Table B-5: Subgrade Soil Data

STATE CODE	SHRP ID	CONSTRUCTION NO	LAYER NO	AASHTO SOIL CLASS	CBR	PLASTICITY INDEX	LIQUID LIMIT	MAXIMUM LAB DRY DENSITY	OPTIMUM LAB MOISTURE CONTENT	IN SITU DRY DENSITY MEAN	IN SITU MOISTURE OPTIMUM MEAN
23	1001	1	1	A-4				135	6.7		
50	1002	1	1	A-7-6							
25	1003	1	1	A-2-4	10			114	12	106	
50	1004	1	1	A-6		0	0	112	12.6	102	82.1
23	1009	1	1	A-4							
23	1028	1	1	A-1a		0	0	128	8.5		
50	1681	1	1	A-1a		3	18				
50	1683	1	1	A-1a		11	26				
9	1803	1	1					122	12.4		118.2
34	1003	1	1	A-7-6							
34	1011	1	1	A-7-6							
34	1030	1	1	A-4							
34	1031	1	1	A-7-6							
34	1033	1	1	A-2-4							
34	1034	1	1	A-1-a							
34	1638	1	1	A-1-b							
42	1597	1	1	A-7-5							
42	1599	1	1	A-7-5							

Table B-6: Base Layer Data

STATE CODE	SHRP ID	CONSTRUCTION NO	LAYER NO	AASHTO SOIL CLASS	PLASTICITY INDEX	MAX LAB DRY DENSITY	OPTIMUM LAB MOISTURE	IN SITU DRY DENSITY MEAN	IN SITU MOISTURE MEAN
23	1001	1	2	A-1-b	1	131	6.5	129	7
23	1001	1	3	A-1-a		139	6.1		
25	1003	1	2	A-1-a		125	8.4		
23	1009	1	2	A-1-b	1	133	10	126	3
23	1009	1	3	A-1-a		139	7.9	139	3
23	1028	1	2	A-1-a		142	6.2	141	4
23	1028	1	3	A-1-a		143	7.4	137	3
34	1031	1	2	A-1-a					7
34	1033	1	2	A-1-a					5
9	1803	1	2	A-1-a		137	7.6	138	5

Table B-7: Layer Thickness

State Code	SHRP ID	Layer #	Description	Material Type	Mean Thickness
23	1001	1	Subgrade	Poorly Graded Sand	
23	1001	2	Subbase Layer	Sand	42
23	1001	3	Base Layer	Crushed Stone, Gravel or Slag	4
23	1001	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
23	1001	5	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
23	1001	6	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	2.2
23	1001	7	Friction Course	Hot Mixed, Hot Laid Asphalt Concrete, Open Graded (Porous Friction Course)	0.8
50	1002	1	Subgrade	Gravel	
50	1002	2	Base Layer	Crushed Stone, Gravel or Slag	24
50	1002	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	5
50	1002	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	1.8
50	1002	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.3
25	1003	1	Subgrade	Poorly Graded Sand	
25	1003	2	Base Layer	Gravel (Uncrushed)	12
25	1003	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	4.7
25	1003	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.2
34	1003	1	Subgrade	Sandy Silt	
34	1003	2	Subbase Layer	Soil-Aggregate Mixture (Predominantly Coarse-Grained Soil)	24

Table B-7: Continued

State Code	SHRP ID	Layer No	Description	Material Type	Mean Thickness
50	1004	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	5
50	1004	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	1.8
50	1004	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.3
23	1009	1	Subgrade	Poorly Graded Sand	
23	1009	2	Subbase Layer	Soil-Aggregate Mixture (Predominantly Coarse-Grained Soil)	20
23	1009	3	Base Layer	Crushed Stone, Gravel or Slag	4
23	1009	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
23	1009	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	3
34	1011	1	Subgrade	Silty Sand	
34	1011	2	Subbase Layer	Soil-Aggregate Mixture (Predominantly Coarse-Grained Soil)	10
34	1011	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	7.5
34	1011	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.5

Table B-7: Continued

State Code	SHRP ID	Layer #	Description	Material Type	Mean Thickness
34	1031	1	Subgrade	Silty Sand	
34	1031	2	Base Layer	Crushed Stone, Gravel or Slag	16
34	1031	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	6.5
34	1031	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.5
34	1033	1	Subgrade	Clayey Gravel	
34	1033	2	Subbase Layer	Crushed Stone, Gravel or Slag	14
34	1033	3	Base Layer	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	4
34	1033	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	1.5
34	1033	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.5
34	1034	1	Subgrade	Poorly Graded Sand	
34	1034	2	Base Layer	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	10
34	1034	3	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	2
42	1597	1	Subgrade	Silty Clay	
42	1597	2	Base Layer	Gravel (Uncrushed)	17
42	1597	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	5
42	1597	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.5
42	1599	1	Subgrade	Silty Clay	
42	1599	2	Base Layer	Gravel (Uncrushed)	12
42	1599	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	5
42	1599	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	4

Table B-7: Continued

State Code	SHRP ID	Layer #	Description	Material Type	Mean Thickness
34	1031	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	2
50	1681	1	Subgrade	Gravel	
50	1681	2	Subbase Layer	Sand	12
50	1681	3	Subbase Layer	Gravel (Uncrushed)	20
50	1681	4	Base Layer	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
50	1681	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	3
50	1683	1	Subgrade	Silty Sand	
50	1683	2	Subbase Layer	Sand	12
50	1683	3	Subbase Layer	Gravel (Uncrushed)	20
50	1683	4	Base Layer	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
50	1683	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	3
9	1803	1	Subgrade	Silty Sand	
9	1803	2	Base Layer	Gravel (Uncrushed)	10
9	1803	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	4
9	1803	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	3

APPENDIX C

Extracted Long Term Pavement Performance (LTPP) Performance Data from Momin (2011)

Table C-1: Rutting

Site	Year	Month	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
231001	1989	August	0.204	0.147	0.164	0.515
	1990	August	0.199	0.129	0.145	0.473
	1991	August	0.182	0.118	0.134	0.434
	1992	April	-	-	-	-
	1993	April	0.235	0.129	0.149	0.513
	1994	August	-	-	-	-
231009	1989	August	0.037	0.092	0.127	0.257
	1990	August	0.044	0.095	0.137	0.276
	1991	August	0.045	0.093	0.138	0.276
	1992	April	-	-	-	-
231028	1989	August	0.104	0.156	0.152	0.413
	1990	August	0.137	0.159	0.158	0.453
	1991	August	0.144	0.164	0.164	0.471
251003	1989	August	0.022	0.045	0.090	0.157
341003	1989	July	0.155	0.284	0.290	0.728
	1990	September	0.230	0.278	0.298	0.861
	1991	August	0.208	0.237	0.264	0.799
	1992	September	0.263	0.264	0.300	0.827
	1993	June	-	-	-	-
341011	1989	October	0.100	0.042	0.154	0.295
	1990	September	0.140	0.049	0.184	0.374
	1991	September	-	-	-	-
	1992	April	0.113	0.037	0.145	0.295
	1993	February	0.153	0.045	0.177	0.375
	1994	June	-	-	-	-
	1995	November	0.176	0.043	0.175	0.394
	1997	July	0.154	0.035	0.146	0.335
341030	1989	July	0.098	0.215	0.377	0.692
	1990	September	0.121	0.244	0.443	0.886
341031	1989	October	0.146	0.101	0.246	0.493
	1990	September	0.157	0.090	0.226	0.472
	1991	September	-	-	-	-
	1992	April	0.169	0.084	0.220	0.473
	1993	February	0.169	0.078	0.206	0.453
	1994	June	-	-	-	-
	1995	November	0.239	0.085	0.229	0.552
341033	1989	October	0.064	0.075	0.135	0.274

Table C-1: Continued

Site	Year	Month	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
	1990	September	0.097	0.093	0.166	0.356
	1991	September	-	-	-	-
	1992	April	0.082	0.068	0.126	0.276
	1993	February	0.110	0.079	0.146	0.336
	1994	June	-	-	-	-
341033	1995	November	0.130	0.078	0.145	0.354
	1989	October	0.046	0.000	0.092	0.138
	1990	September	0.103	0.000	0.173	0.276
	1991	September	-	-	-	-
	1992	April	0.070	0.000	0.107	0.178
	1993	February	0.097	0.000	0.139	0.237
	1994	June	-	-	-	-
	1995	November	0.112	0.000	0.144	0.256
	1997	July	0.080	0.000	0.097	0.178
	1998	August	-	-	-	-
	1999	September	-	-	-	-
	2000	July	0.105	0.000	0.112	0.217
	2001	December	-	-	-	-
	2002	June	0.104	0.000	0.103	0.275
	2004	May	-	-	-	-
	2005	November	0.135	0.000	0.121	0.256
341034	2007	June	0.146	0.000	0.131	0.276
	1989	October	0.071	0.050	0.076	0.197
	1990	September	0.124	0.075	0.116	0.315
	1991	August	0.079	0.045	0.073	0.197
	1992	April	-	-	-	-
	1993	February	0.067	0.035	0.056	0.158
	1994	June	-	-	-	-
	1995	November	0.081	0.037	0.059	0.177
	1997	July	0.092	0.040	0.064	0.197
	1998	August	-	-	-	-
	1999	September	-	-	-	-
	2000	July	0.115	0.043	0.069	0.227
	2001	December	-	-	-	-
	2002	June	0.102	0.036	0.059	0.197
	2003	May	0.113	0.040	0.064	0.217
341638	2004	May	-	-	-	-

Table C-1: Continued

Site	Year	Month	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
341638	2005	November	0.129	0.042	0.066	0.236
421597	1989	August	0.026	0.055	0.078	0.158
	1989	August	0.035	0.037	0.105	0.177
	1990	September	0.054	0.040	0.122	0.216
	1991	August	0.052	0.035	0.109	0.197
	1992	October	-	-	-	-
	1993	March	0.089	0.053	0.173	0.315
	1995	June	0.080	0.044	0.151	0.275
	1996	July	0.084	0.043	0.148	0.275
	1997	November	-	-	-	-
421599	1998	March	0.087	0.042	0.147	0.275
	1989	August	0.088	0.095	0.113	0.295
	1990	August	0.110	0.115	0.148	0.373
	1991	September	0.095	0.094	0.125	0.314
	1992	July	-	-	-	-
	1993	April	0.124	0.106	0.144	0.374
	1994	August	0.130	0.100	0.135	0.365
	1995	October	0.155	0.113	0.156	0.424
	1996	October	0.133	0.093	0.129	0.355
	1997	October	0.168	0.115	0.162	0.445
	1998	June	0.167	0.111	0.156	0.434
	1999	November	0.194	0.120	0.170	0.483
	2000	June	0.225	0.130	0.185	0.540
	2001	September	0.235	0.135	0.193	0.563
	2002	May	0.243	0.138	0.199	0.590
	2003	November	0.267	0.144	0.209	0.620
501002	2004	April	-	-	-	-
	1989	August	0.024	0.047	0.088	0.158
	1990	August	0.042	0.073	0.140	0.255
	1991	September	0.036	0.054	0.106	0.196
	1992	July	-	-	-	-
	1993	April	0.053	0.068	0.135	0.256
	1994	July	-	-	-	-
	1995	October	0.057	0.059	0.120	0.237
501004	1997	November	0.067	0.062	0.127	0.256
501681	1989	August	0.078	0.202	0.134	0.413
	1990	August	0.115	0.228	0.149	0.492

Table C-1: Continued

501683	1989	August	0.133	0.374	0.184	0.692
	1990	August	0.210	0.442	0.214	0.866

Table C-2: Cracking and IRI

Site	Year	Month	Longitudinal Cracking (ft/mi)	Alligator Cracking (%)	Transverse Cracking (ft/mi)	IRI (in/mi)
231001	1989	August	4814.515	0.000	779.328	118.407
	1990	August	5008.481	0.000	2286.029	138.492
	1991	August	5060.436	0.771	1233.070	115.695
	1992	April	-	-	-	109.651
	1993	April	668.490	0.000	4010.941	-
	1994	August	-	-	-	125.275
231009	1989	August	4727.923	1.597	824.356	61.231
	1990	August	5753.172	0.000	973.294	67.238
	1991	August	5625.016	1.292	1780.332	61.485
	1992	April	-	-	-	62.258
231028	1989	August	8628.027	0.000	803.574	85.523
	1990	August	9812.605	0.000	1423.572	86.056
	1991	August	5271.721	0.000	1001.004	91.707
251003	1989	August	17245.663	0.000	3082.675	122.564
341003	1989	July	1246.925	22.335	3262.787	124.471
	1990	September	1818.432	22.604	3245.468	-
	1991	August	1378.545	22.407	2279.101	102.998
	1992	September	5649.262	18.675	2549.268	95.750
	1993	June	-	-	-	103.442
341011	1989	October	5971.384	0.000	1364.690	101.972
	1990	September	6033.731	0.000	2036.644	102.529
	1991	September	-	-	-	108.548
	1992	April	5472.614	0.000	1728.376	102.136
	1993	February	5933.284	0.000	1804.577	109.220
	1994	June	-	-	-	115.645
	1995	November	10474.168	0.036	6383.562	115.746
	1997	July	-	-	-	117.951
341030	1989	July	2590.833	10.602	744.691	225.004
	1990	September	4208.371	20.469	2895.636	252.857
341031	1989	October	9750.259	3.534	3532.954	111.247
	1990	September	7155.963	2.834	1977.761	114.720
	1991	September	-	-	-	121.791
	1992	April	6082.222	4.862	1887.706	115.100
	1993	February	6549.819	10.154	6179.205	126.593
	1994	June	-	-	-	155.409
	1995	November	10692.380	9.688	5898.647	144.702
341033	1989	October	1271.171	0.000	1967.370	201.726

Table C-2: Continued

Site	Year	Month	Longitudinal Cracking (ft/mi)	Alligator Cracking (%)	Transverse Cracking (ft/mi)	IRI (in/mi)
	1990	September	1319.662	0.000	2448.822	173.796
	1991	September	-	-	-	176.610
	1992	April	1167.260	1.310	2102.454	184.010
	1993	February	710.054	0.108	2279.101	181.716
	1994	June	-	-	-	183.845
341033	1995	November	1420.109	0.251	2930.273	199.115
	1989	October	2002.007	0.000	0.000	85.245
	1990	September	2871.391	0.000	0.000	85.447
	1991	September	-	-	-	88.159
	1992	April	3484.462	0.000	0.000	87.678
	1993	February	3990.159	0.000	0.000	88.843
	1994	June	-	-	-	90.820
	1995	November	5410.268	0.000	0.000	93.279
	1997	July	-	-	-	94.153
	1998	August	-	-	-	94.964
	1999	September	-	-	-	93.545
	2000	July	13234.721	0.000	1728.376	-
	2001	December	-	-	-	98.525
	2002	June	13865.111	0.161	2885.245	96.320
	2004	May	-	-	-	96.206
	2005	November	-	-	-	97.612
341034	2007	June	-	-	-	101.655
	1989	October	516.088	0.000	0.000	56.923
	1990	September	904.020	0.000	0.000	59.685
	1991	August	-	-	-	60.762
	1992	April	910.948	0.000	0.000	56.973
	1993	February	3338.988	0.000	0.000	58.469
	1994	June	-	-	-	60.864
	1995	November	4966.917	0.000	0.000	-
	1997	July	-	-	-	65.261
	1998	August	-	-	-	63.297
	1999	September	-	-	-	65.121
	2000	July	5524.570	0.000	148.938	-
	2001	December	-	-	-	67.364
	2002	June	6601.774	0.072	443.351	65.989
	2003	May	-	-	-	64.627
341638	2004	May	-	-	-	65.311

Table C-2: Continued

Site	Year	Month	Longitudinal Cracking (ft/mi)	Alligator Cracking (%)	Transverse Cracking (ft/mi)	IRI (in/mi)
341638	2005	November	-	-	-	66.059
421597	1989	August	547.261	0.000	762.010	107.015
	1989	August	0.000	0.000	0.000	86.651
	1990	September	0.000	0.000	0.000	88.590
	1991	August	72.737	0.000	405.251	89.414
	1992	October	-	-	-	92.151
	1993	March	0.000	0.000	0.000	93.836
	1995	June	-	-	-	100.552
	1996	July	422.569	0.000	155.866	-
	1997	November	-	-	-	102.491
421599	1998	March	-	-	-	103.011
	1989	August	0.000	0.000	0.000	77.958
	1990	August	27.709	0.000	0.000	77.439
	1991	September	786.255	0.000	0.000	68.023
	1992	July	-	-	-	70.697
	1993	April	2445.358	0.000	976.758	-
	1994	August	-	-	-	80.090
	1995	October	1666.030	0.000	980.221	80.727
	1996	October	-	-	-	78.136
	1997	October	-	-	-	82.502
	1998	June	-	-	-	82.143
	1999	November	-	-	-	86.170
	2000	June	6463.227	0.413	4748.705	93.494
	2001	September	-	-	-	91.986
	2002	May	9192.607	0.072	4533.957	93.514
	2003	November	-	-	-	93.332
501002	2004	April	-	-	-	95.116
	1989	August	3480.998	0.000	45.028	104.544
	1990	August	3813.512	0.108	138.547	106.825
	1991	September	4527.030	4.108	308.268	92.379
	1992	July	-	-	-	93.329
	1993	April	5330.604	0.771	1918.879	131.459
	1994	July	-	-	-	131.789
	1995	October	5230.157	0.574	2985.692	132.600
501004	1997	November	-	-	-	129.495
	1989	August	2085.135	0.000	27.709	76.361
501681	1990	August	308.268	0.000	131.620	76.311
	1989	August	7914.509	0.771	1291.953	134.450
501683	1990	August	2251.392	1.453	1517.092	142.560

APPENDIX D

Computed Distresses of Long Term Pavement Performance (LTPP) sites by AASHTOWare Pavement ME 2.1

Table D-1: Computed Rutting by the Global Calibration Factors

Site #	Month	Year	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
091803	1989	0.9	0.03	0.03	0.08	0.14
091803	1990	2.1	0.04	0.04	0.10	0.18
091803	1991	3.0	0.05	0.04	0.10	0.19
091803	1992	4.1	0.06	0.04	0.11	0.21
091803	1994	6.1	0.07	0.05	0.12	0.23
091803	1995	7.2	0.08	0.05	0.12	0.24
091803	1996	8.2	0.08	0.05	0.12	0.25
091803	1997	9.2	0.09	0.05	0.13	0.26
091803	1998	9.8	0.09	0.05	0.13	0.27
091803	2000	11.9	0.10	0.05	0.13	0.28
091803	2002	13.8	0.11	0.05	0.13	0.29
091803	2003	14.8	0.11	0.05	0.14	0.30
091803	2004	15.7	0.12	0.05	0.14	0.31
091803	2007	18.8	0.13	0.05	0.14	0.32
231001	1989	1.0	0.01	0.04	0.05	0.10
231001	1990	2.0	0.06	0.06	0.08	0.20
231001	1991	3.0	0.06	0.06	0.08	0.20
231001	1993	4.7	0.08	0.07	0.10	0.25
231001	1995	7.2	0.10	0.08	0.11	0.29
231001	1999	11.1	0.13	0.08	0.11	0.32
231001	2000	11.8	0.13	0.08	0.11	0.33
231001	2002	13.8	0.14	0.08	0.11	0.33
231009	1989	1.0	0.01	0.06	0.10	0.18
231009	1990	2.0	0.06	0.11	0.17	0.34
231009	1991	3.0	0.07	0.12	0.19	0.38
231009	1993	4.8	0.08	0.13	0.21	0.41
231009	1995	7.2	0.10	0.14	0.22	0.46
231009	1997	9.0	0.12	0.14	0.23	0.49
231009	1998	10.1	0.12	0.14	0.24	0.50
231009	1999	10.8	0.12	0.14	0.24	0.51
231009	2001	13.0	0.14	0.15	0.25	0.53
231009	2003	14.8	0.14	0.15	0.26	0.55
231009	2004	15.7	0.15	0.15	0.26	0.56
231028	1989	1.0	0.06	0.06	0.10	0.22
231028	1990	2.0	0.08	0.06	0.12	0.26
231028	1991	3.0	0.09	0.06	0.13	0.28
231028	1993	4.7	0.10	0.07	0.13	0.31

Table D-1: Continued

231028	1995	7.2	0.14	0.07	0.14	0.35
231028	1998	9.8	0.15	0.08	0.15	0.38
231028	1999	10.8	0.17	0.08	0.15	0.40
231028	2001	13.0	0.20	0.08	0.16	0.43
231028	2003	14.8	0.21	0.08	0.16	0.45
231028	2004	15.8	0.22	0.08	0.16	0.46
251003	1989	1.1	0.03	0.05	0.11	0.19
251003	1990	2.2	0.04	0.05	0.13	0.22
251003	1991	3.1	0.05	0.05	0.13	0.24
251003	1992	4.2	0.06	0.06	0.14	0.26
251003	1995	7.3	0.07	0.07	0.15	0.30
251003	1996	8.3	0.08	0.07	0.16	0.30
251003	1998	9.9	0.08	0.07	0.16	0.32
341003	1989	0.8	0.06	0.06	0.15	0.27
341003	1990	2.0	0.11	0.06	0.19	0.36
341003	1991	2.9	0.12	0.07	0.20	0.38
341003	1992	4.0	0.14	0.07	0.22	0.42
341003	1994	6.2	0.17	0.07	0.23	0.47
341003	1995	7.1	0.18	0.07	0.24	0.49
341003	1999	10.5	0.22	0.08	0.26	0.56
341003	2000	11.8	0.25	0.08	0.27	0.59
341003	2002	14.0	0.27	0.08	0.28	0.63
341003	2005	17.2	0.31	0.08	0.29	0.68
341011	1989	1.2	0.08	0.04	0.14	0.26
341011	1990	2.1	0.11	0.04	0.16	0.31
341011	1992	3.7	0.12	0.05	0.17	0.34
341011	1993	4.5	0.14	0.05	0.18	0.38
341011	1995	7.3	0.19	0.05	0.20	0.45
341011	1997	8.9	0.21	0.05	0.21	0.48
341011	1999	11.2	0.24	0.06	0.22	0.52
341011	2000	11.9	0.25	0.06	0.23	0.54
341011	2002	14.1	0.29	0.06	0.24	0.58
341011	2007	19.2	0.36	0.06	0.26	0.68
341030	1989	1.5	0.03	0.08	0.10	0.22
341030	1990	2.7	0.06	0.10	0.13	0.29
341030	1991	3.6	0.06	0.10	0.14	0.30
341030	1992	4.7	0.07	0.11	0.15	0.33
341030	1995	7.8	0.09	0.12	0.17	0.38
341030	1997	9.5	0.10	0.13	0.18	0.40
341030	1999	11.3	0.11	0.13	0.18	0.42

Table D-1: Continued

341030	2000	12.5	0.12	0.13	0.18	0.43
341030	2001	13.7	0.12	0.13	0.19	0.45
341030	2005	17.8	0.14	0.14	0.20	0.47
341030	2007	19.4	0.14	0.14	0.20	0.48
341031	1989	1.2	0.08	0.04	0.15	0.26
341031	1990	2.1	0.10	0.04	0.17	0.31
341031	1992	3.7	0.13	0.05	0.18	0.36
341031	1993	4.5	0.15	0.05	0.19	0.39
341031	1995	7.3	0.21	0.05	0.22	0.48
341031	1996	8.0	0.22	0.05	0.22	0.50
341031	1999	11.1	0.27	0.06	0.24	0.57
341031	2000	11.9	0.29	0.06	0.25	0.59
341031	2002	14.1	0.33	0.06	0.26	0.65
341031	2005	17.3	0.39	0.06	0.28	0.73
341033	1989	1.2	0.04	0.04	0.08	0.16
341033	1990	2.1	0.06	0.05	0.09	0.19
341033	1992	3.7	0.07	0.05	0.09	0.21
341033	1993	4.5	0.08	0.05	0.10	0.23
341033	1995	7.3	0.11	0.06	0.11	0.27
341033	1997	9.2	0.13	0.06	0.11	0.30
341033	2000	12.2	0.16	0.06	0.12	0.34
341033	2002	13.8	0.17	0.06	0.12	0.35
341033	2003	14.9	0.18	0.07	0.12	0.37
341033	2004	15.7	0.18	0.07	0.12	0.37
341033	2007	18.8	0.21	0.07	0.13	0.41
341034	1989	1.8	0.04	0.03	0.08	0.15
341034	1990	2.7	0.06	0.03	0.09	0.17
341034	1992	4.3	0.07	0.04	0.10	0.21
341034	1993	5.1	0.08	0.04	0.10	0.22
341034	1995	7.8	0.11	0.04	0.11	0.26
341034	1997	9.5	0.11	0.04	0.11	0.27
341034	2000	12.8	0.14	0.04	0.12	0.29
341034	2002	14.4	0.15	0.05	0.12	0.31
341034	2005	17.8	0.17	0.05	0.12	0.34
341034	2007	19.4	0.18	0.05	0.12	0.35
341638	1989	1.8	0.10	0.10	0.12	0.32
341638	1990	2.7	0.14	0.11	0.14	0.39
341638	1991	3.6	0.17	0.11	0.15	0.43
341638	1993	5.1	0.19	0.12	0.16	0.47
341638	1995	7.8	0.25	0.13	0.17	0.56
341638	1997	9.5	0.27	0.14	0.18	0.58

Table D-1: Continued

341638	2000	12.8	0.33	0.14	0.18	0.65
341638	2002	14.4	0.36	0.14	0.19	0.69
341638	2003	15.3	0.37	0.14	0.19	0.70
341638	2005	17.8	0.41	0.15	0.19	0.75
421597	1989	0.9	0.02	0.04	0.05	0.11
421597	1990	1.8	0.02	0.04	0.06	0.13
421597	1991	2.9	0.03	0.05	0.07	0.15
421597	1993	4.5	0.04	0.05	0.08	0.17
421597	1994	5.8	0.04	0.05	0.09	0.19
421597	1995	7.0	0.05	0.05	0.09	0.20
421597	1996	7.8	0.05	0.06	0.09	0.20
421597	1997	9.0	0.06	0.06	0.10	0.21
421597	2000	12.2	0.07	0.06	0.10	0.23
421597	2002	13.7	0.07	0.06	0.11	0.24
421597	2003	14.8	0.08	0.06	0.11	0.25
421597	2007	18.9	0.09	0.06	0.12	0.28
421599	1989	0.8	0.03	0.03	0.08	0.14
421599	1990	1.9	0.05	0.03	0.10	0.18
421599	1991	3.0	0.06	0.03	0.11	0.20
421599	1993	4.5	0.07	0.04	0.12	0.22
421599	1995	6.8	0.08	0.04	0.14	0.25
421599	1996	7.8	0.09	0.04	0.14	0.27
421599	1998	9.5	0.10	0.04	0.15	0.28
421599	2000	11.9	0.11	0.04	0.15	0.31
421599	2001	13.0	0.12	0.04	0.16	0.32
421599	2002	13.8	0.12	0.04	0.16	0.32
421599	2003	14.6	0.12	0.04	0.16	0.33
421599	2005	16.9	0.14	0.04	0.17	0.35
501002	1989	0.9	0.04	0.05	0.06	0.15
501002	1990	1.9	0.05	0.05	0.07	0.17
501002	1991	3.0	0.06	0.06	0.08	0.19
501002	1993	4.6	0.07	0.06	0.08	0.22
501002	1994	5.9	0.09	0.07	0.09	0.24
501002	1995	7.1	0.10	0.07	0.09	0.26
501002	1996	8.1	0.10	0.07	0.09	0.27
501002	1997	9.1	0.11	0.07	0.10	0.27
501002	1998	9.8	0.11	0.07	0.10	0.28
501002	1999	11.2	0.12	0.07	0.10	0.30
501002	2000	12.0	0.13	0.07	0.10	0.31

Table D-1: Continued

501002	2001	13.0	0.14	0.07	0.10	0.31
501002	2002	14.1	0.14	0.07	0.11	0.32
501002	2003	15.2	0.15	0.08	0.11	0.33
501004	1989	0.9	0.02	0.04	0.08	0.14
501004	1990	1.9	0.03	0.05	0.09	0.16
501004	1991	3.0	0.04	0.05	0.10	0.18
501004	1993	4.6	0.04	0.05	0.10	0.20
501004	1995	7.1	0.06	0.06	0.11	0.23
501004	1997	9.2	0.07	0.06	0.12	0.25
501004	1999	10.8	0.08	0.06	0.12	0.26
501004	2000	11.8	0.08	0.06	0.12	0.26
501004	2001	12.9	0.08	0.06	0.13	0.27
501004	2002	13.7	0.08	0.06	0.13	0.27
501004	2004	15.7	0.09	0.06	0.13	0.28
501004	2007	18.9	0.10	0.07	0.14	0.30
501681	1989	1.1	0.02	0.06	0.04	0.11
501681	1990	2.1	0.04	0.09	0.06	0.19
501681	1991	3.2	0.06	0.11	0.07	0.24
501681	1993	4.8	0.07	0.11	0.07	0.26
501681	1995	7.3	0.10	0.13	0.08	0.30
501681	1998	9.9	0.13	0.13	0.08	0.34
501681	1999	10.9	0.13	0.13	0.08	0.35
501681	2001	13.2	0.15	0.14	0.09	0.37
501681	2003	14.9	0.16	0.14	0.09	0.39
501681	2004	16.1	0.17	0.14	0.09	0.40
501683	1989	1.1	0.02	0.05	0.03	0.10
501683	1990	2.1	0.04	0.09	0.04	0.18
501683	1991	3.2	0.06	0.10	0.05	0.21
501683	1993	4.8	0.07	0.11	0.05	0.24
501683	1995	7.3	0.10	0.12	0.06	0.28
501683	1998	9.9	0.12	0.13	0.06	0.31
501683	1999	10.9	0.13	0.13	0.06	0.32
501683	2001	13.2	0.15	0.13	0.06	0.35
501683	2003	14.9	0.16	0.14	0.07	0.36
501683	2004	16.1	0.17	0.14	0.07	0.38
501683	2007	19.1	0.19	0.15	0.07	0.40

Table D-2: Computed Cracking and IRI Distresses by the Global Calibration Factors

Site #	Month	Year	Alligator Cracking (%)	Transverse Cracking (ft/mi)	IRI (in/mi)
231001	1989	1	0.0065	1,556.30	113.9
231001	1990	2	0.0123	1,590.53	116.4
231001	1991	3	0.0131	1,596.72	117.7
231001	1993	4.67	0.0224	2,112.00	128.5
231009	1989	1	0.0210	2,112.00	126.3
231009	1990	2	0.0462	2,112.00	130.2
231009	1991	3	0.0709	2,112.00	133.3
231028	1989	1	0.0169	0.02	97.9
231028	1990	2	0.0347	1.14	101
231028	1991	3	0.0526	1,535.28	120.9
251003	1989	1.08	0.0072	1,594.43	114.9
341003	1989	0.83	0.0415	1,461.87	117.9
341003	1990	2	0.1200	1,498.01	124.3
341003	1991	2.92	0.1610	1,553.44	127.4
341003	1992	4	0.2280	1,767.17	133.6
341011	1989	1.17	0.0197	0.02	100.7
341011	1990	2.08	0.0378	6.91	104.4
341011	1992	3.67	0.0587	826.01	117.6
341011	1993	4.5	0.0829	1,518.83	128.8
341011	1995	7.25	0.1610	1,552.86	138.4
341030	1989	1.5	0.0143	0.02	98.1
341030	1990	2.67	0.0411	0.54	103
341031	1989	1.17	0.0121	0.00	100.7
341031	1990	2.08	0.0225	0.01	104.3
341031	1992	3.67	0.0412	24.97	109.5
341031	1993	4.5	0.0551	24.99	112.5
341031	1995	7.25	0.1130	38.17	123.0
341033	1989	1.17	0.0045	12.88	95.2
341033	1990	2.08	0.0083	27.60	97.7
341033	1992	3.67	0.0134	1,075.56	113.0
341033	1993	4.5	0.0182	1,812.88	123.9
341033	1995	7.25	0.0342	1,908.68	131.8
341034	1989	1.75	0.0011	0.00	93.9
341034	1990	2.67	0.0021	0.00	96.0
341034	1992	4.25	0.0041	153.56	101.2
341034	1993	5.08	0.0054	153.58	103.0
341034	1995	7.83	0.0096	212.00	109.9

Table D-2: Continued

341034	2002	14.42	0.0214	965.12	134.7
341638	1989	1.75	0.1360	0.00	104.0
341638	1990	2.67	0.2870	0.00	108.4
341638	1993	5.08	0.6510	325.74	120.6
341638	2002	14.42	2.6600	1,350.81	165.3
421597	1989	0.92	0.0010	0.02	92.1
421599	1989	0.83	0.0007	0.11	93.3
421599	1990	1.92	0.0020	5.44	96.8
421599	1991	3	0.0033	86.22	100.6
421599	1993	4.5	0.0048	1,890.96	125.0
501002	1989	0.92	0.0010	2,112.00	119.1
501002	1990	1.92	0.0021	2,112.00	121.2
501002	1991	3	0.0036	2,112.00	123.8
501002	1993	4.58	0.0054	2,112.00	127.5
501002	1995	7.08	0.0098	2,112.00	134.3
501002	2000	12	0.0186	2,112.00	149.1
501002	2002	14.08	0.0225	2,112.00	155.6
501004	1989	0.92	0.0005	2,112.00	118.4
501004	1990	1.92	0.0010	2,112.00	120.5
501004	1991	3	0.0017	2,112.00	122.8
501004	1995	7.08	0.0042	2,112.00	132.0
501681	1989	1.08	0.0007	0.00	91.4
501681	1990	2.08	0.0098	2,112.00	121.3
501683	1989	1.08	0.0008	0.00	90.7
501683	1990	2.08	0.0102	2,112.00	120.5

Table D-3: Computed Rutting by the Local Calibration Factors

Site #	Month	Year	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
91803	1989	0.92	0.02	0.03	0.06	0.10
91803	1990	2.08	0.02	0.03	0.07	0.13
91803	1991	3	0.03	0.03	0.08	0.14
91803	1992	4.08	0.03	0.03	0.08	0.15
91803	1994	6.08	0.04	0.04	0.09	0.17
91803	1995	7.17	0.05	0.04	0.09	0.17
91803	1996	8.17	0.05	0.04	0.09	0.18
91803	1997	9.17	0.05	0.04	0.09	0.18
91803	1998	9.83	0.05	0.04	0.09	0.19
91803	2000	11.92	0.06	0.04	0.10	0.20
91803	2002	13.75	0.06	0.04	0.10	0.21
91803	2003	14.83	0.07	0.04	0.10	0.21
91803	2004	15.67	0.07	0.04	0.10	0.21
91803	2007	18.83	0.08	0.04	0.11	0.23
231001	1989	1	0.01	0.03	0.04	0.08
231001	1990	2	0.04	0.05	0.06	0.14
231001	1991	3	0.04	0.05	0.06	0.15
231001	1993	4.67	0.04	0.06	0.08	0.18
231001	1995	7.17	0.06	0.06	0.08	0.20
231001	1999	11.08	0.08	0.07	0.08	0.23
231001	2000	11.83	0.08	0.07	0.08	0.23
231001	2002	13.83	0.08	0.07	0.08	0.23
231009	1989	1	0.01	0.05	0.07	0.13
231009	1990	2	0.03	0.09	0.13	0.26
231009	1991	3	0.04	0.10	0.14	0.28
231009	1993	4.75	0.05	0.10	0.15	0.30
231009	1995	7.17	0.06	0.11	0.17	0.34
231009	1997	9	0.07	0.11	0.17	0.35
231009	1998	10.08	0.07	0.12	0.18	0.36
231009	1999	10.83	0.07	0.12	0.18	0.37
231009	2001	13	0.08	0.12	0.18	0.39
231009	2003	14.83	0.09	0.12	0.19	0.40
231009	2004	15.67	0.09	0.12	0.19	0.40
231028	1989	1	0.03	0.05	0.08	0.16
231028	1990	2	0.05	0.05	0.09	0.18
231028	1991	3	0.05	0.05	0.09	0.20
231028	1993	4.67	0.06	0.06	0.10	0.22

Table D-3: Continued

231028	1995	7.17	0.08	0.06	0.11	0.25
231028	1998	9.83	0.09	0.06	0.11	0.27
231028	1999	10.83	0.10	0.06	0.11	0.28
231028	2001	13	0.12	0.07	0.12	0.30
231028	2003	14.83	0.12	0.07	0.12	0.31
231028	2004	15.75	0.13	0.07	0.12	0.32
251003	1989	1.08	0.02	0.04	0.08	0.14
251003	1990	2.17	0.02	0.04	0.09	0.16
251003	1991	3.08	0.03	0.04	0.10	0.17
251003	1992	4.17	0.03	0.05	0.10	0.19
251003	1995	7.25	0.04	0.05	0.11	0.21
251003	1996	8.25	0.05	0.05	0.12	0.22
251003	1998	9.92	0.05	0.06	0.12	0.23
341003	1989	0.83	0.04	0.05	0.11	0.20
341003	1990	2	0.06	0.05	0.14	0.26
341003	1991	2.92	0.07	0.05	0.15	0.27
341003	1992	4	0.08	0.06	0.16	0.30
341003	1994	6.17	0.10	0.06	0.17	0.33
341003	1995	7.08	0.11	0.06	0.18	0.35
341003	1999	10.5	0.13	0.06	0.19	0.39
341003	2000	11.83	0.15	0.07	0.20	0.41
341003	2002	14	0.16	0.07	0.21	0.44
341003	2005	17.17	0.18	0.07	0.22	0.47
341011	1989	1.17	0.05	0.03	0.10	0.19
341011	1990	2.08	0.06	0.04	0.12	0.22
341011	1992	3.67	0.07	0.04	0.13	0.24
341011	1993	4.5	0.09	0.04	0.14	0.26
341011	1995	7.25	0.11	0.04	0.15	0.31
341011	1997	8.92	0.13	0.04	0.16	0.33
341011	1999	11.17	0.14	0.05	0.17	0.36
341011	2000	11.92	0.15	0.05	0.17	0.37
341011	2002	14.08	0.17	0.05	0.18	0.39
341011	2007	19.17	0.22	0.05	0.19	0.46
341030	1989	1.5	0.02	0.07	0.08	0.17
341030	1990	2.67	0.03	0.08	0.10	0.21
341030	1991	3.58	0.04	0.09	0.10	0.23
341030	1992	4.67	0.04	0.09	0.11	0.25
341030	1995	7.75	0.05	0.10	0.12	0.28
341030	1997	9.5	0.06	0.10	0.13	0.29
341030	1999	11.33	0.06	0.11	0.13	0.31

Table D-3: Continued

341030	2000	12.5	0.07	0.11	0.14	0.31
341030	2001	13.67	0.07	0.11	0.14	0.32
341030	2005	17.83	0.08	0.11	0.15	0.34
341030	2007	19.42	0.08	0.12	0.15	0.35
341031	1989	1.17	0.05	0.03	0.11	0.19
341031	1990	2.08	0.06	0.03	0.12	0.22
341031	1992	3.67	0.08	0.04	0.14	0.25
341031	1993	4.5	0.09	0.04	0.14	0.27
341031	1995	7.25	0.12	0.04	0.16	0.33
341031	1996	8	0.13	0.04	0.17	0.34
341031	1999	11.08	0.16	0.05	0.18	0.39
341031	2000	11.92	0.17	0.05	0.18	0.40
341031	2002	14.08	0.20	0.05	0.19	0.44
341031	2005	17.25	0.23	0.05	0.21	0.49
341033	1989	1.17	0.02	0.04	0.06	0.12
341033	1990	2.08	0.03	0.04	0.06	0.14
341033	1992	3.67	0.04	0.04	0.07	0.15
341033	1993	4.5	0.05	0.04	0.07	0.16
341033	1995	7.25	0.06	0.05	0.08	0.19
341033	1997	9.17	0.08	0.05	0.08	0.21
341033	2000	12.17	0.10	0.05	0.09	0.23
341033	2002	13.83	0.10	0.05	0.09	0.24
341033	2003	14.92	0.11	0.05	0.09	0.25
341033	2004	15.67	0.11	0.05	0.09	0.25
341033	2007	18.83	0.13	0.06	0.09	0.28
341034	1989	1.75	0.02	0.03	0.06	0.11
341034	1990	2.67	0.03	0.03	0.06	0.12
341034	1992	4.25	0.04	0.03	0.07	0.15
341034	1993	5.08	0.05	0.03	0.07	0.15
341034	1995	7.83	0.06	0.03	0.08	0.18
341034	1997	9.5	0.07	0.03	0.08	0.18
341034	2000	12.75	0.08	0.04	0.09	0.20
341034	2002	14.42	0.09	0.04	0.09	0.21
341034	2005	17.83	0.10	0.04	0.09	0.23
341034	2007	19.42	0.11	0.04	0.09	0.24
341638	1989	1.75	0.06	0.08	0.09	0.23
341638	1990	2.67	0.08	0.09	0.10	0.27
341638	1991	3.58	0.10	0.09	0.11	0.30
341638	1993	5.08	0.11	0.10	0.12	0.33
341638	1995	7.83	0.15	0.11	0.13	0.39

Table D-3: Continued

341638	1997	9.5	0.16	0.11	0.13	0.40
341638	2000	12.75	0.20	0.12	0.14	0.45
341638	2002	14.42	0.21	0.12	0.14	0.47
341638	2003	15.33	0.22	0.12	0.14	0.48
341638	2005	17.83	0.25	0.12	0.14	0.51
421597	1989	0.92	0.01	0.03	0.04	0.08
421597	1990	1.83	0.01	0.04	0.05	0.10
421597	1991	2.92	0.02	0.04	0.05	0.11
421597	1993	4.5	0.02	0.04	0.06	0.12
421597	1994	5.75	0.03	0.04	0.06	0.14
421597	1995	7	0.03	0.04	0.07	0.14
421597	1996	7.83	0.03	0.05	0.07	0.15
421597	1997	9	0.04	0.05	0.07	0.15
421597	2000	12.17	0.04	0.05	0.08	0.17
421597	2002	13.67	0.04	0.05	0.08	0.17
421597	2003	14.83	0.05	0.05	0.08	0.18
421597	2007	18.92	0.06	0.05	0.09	0.20
421599	1989	0.83	0.02	0.02	0.06	0.10
421599	1990	1.92	0.03	0.03	0.07	0.13
421599	1991	3	0.03	0.03	0.08	0.14
421599	1993	4.5	0.04	0.03	0.09	0.16
421599	1995	6.75	0.05	0.03	0.10	0.18
421599	1996	7.83	0.05	0.03	0.10	0.19
421599	1998	9.5	0.06	0.03	0.11	0.20
421599	2000	11.92	0.07	0.03	0.11	0.21
421599	2001	13	0.07	0.03	0.12	0.22
421599	2002	13.75	0.07	0.04	0.12	0.22
421599	2003	14.58	0.07	0.04	0.12	0.23
421599	2005	16.92	0.08	0.04	0.12	0.24
501002	1989	0.92	0.03	0.04	0.04	0.11
501002	1990	1.92	0.03	0.04	0.05	0.12
501002	1991	3	0.04	0.05	0.06	0.14
501002	1993	4.58	0.04	0.05	0.06	0.16
501002	1994	5.92	0.05	0.05	0.07	0.17
501002	1995	7.08	0.06	0.05	0.07	0.18
501002	1996	8.08	0.06	0.06	0.07	0.19
501002	1997	9.08	0.06	0.06	0.07	0.19
501002	1998	9.75	0.07	0.06	0.07	0.20
501002	1999	11.17	0.07	0.06	0.07	0.21
501002	2000	12	0.08	0.06	0.08	0.22
501002	2001	13	0.08	0.06	0.08	0.22

Table D-3: Continued

501002	2002	14.08	0.08	0.06	0.08	0.22
501002	2003	15.17	0.09	0.06	0.08	0.23
501004	1989	0.92	0.01	0.03	0.06	0.10
501004	1990	1.92	0.02	0.04	0.06	0.12
501004	1991	3	0.02	0.04	0.07	0.13
501004	1993	4.58	0.03	0.04	0.08	0.14
501004	1995	7.08	0.04	0.05	0.08	0.16
501004	1997	9.17	0.04	0.05	0.09	0.18
501004	1999	10.83	0.05	0.05	0.09	0.18
501004	2000	11.75	0.05	0.05	0.09	0.19
501004	2001	12.92	0.05	0.05	0.09	0.19
501004	2002	13.67	0.05	0.05	0.09	0.19
501004	2004	15.67	0.05	0.05	0.10	0.20
501004	2007	18.92	0.06	0.05	0.10	0.21
501681	1989	1.08	0.01	0.05	0.03	0.08
501681	1990	2.08	0.02	0.08	0.04	0.14
501681	1991	3.17	0.04	0.09	0.05	0.18
501681	1993	4.75	0.04	0.09	0.05	0.19
501681	1995	7.25	0.06	0.10	0.06	0.22
501681	1998	9.92	0.08	0.11	0.06	0.24
501681	1999	10.92	0.08	0.11	0.06	0.25
501681	2001	13.17	0.09	0.11	0.06	0.27
501681	2003	14.92	0.10	0.11	0.06	0.28
501681	2004	16.08	0.10	0.12	0.07	0.28
501683	1989	1.08	0.01	0.04	0.02	0.07
501683	1990	2.08	0.02	0.08	0.03	0.13
501683	1991	3.17	0.04	0.08	0.04	0.16
501683	1993	4.75	0.04	0.09	0.04	0.17
501683	1995	7.25	0.06	0.10	0.04	0.20
501683	1998	9.92	0.07	0.11	0.05	0.22
501683	1999	10.92	0.08	0.11	0.05	0.23
501683	2001	13.17	0.09	0.11	0.05	0.25
501683	2003	14.92	0.10	0.11	0.05	0.26
501683	2004	16.08	0.10	0.11	0.05	0.27
501683	2007	19.08	0.11	0.12	0.05	0.28

Table D-4: Computed Cracking and IRI Distresses by the Local Calibration Factors

Site #	Month	Year	Alligator Cracking (%)	IRI (in/mi)
231001	1989	1	2.8091	80.3
231001	1990	2	3.2349	96.9
231001	1991	3	3.2818	97.2
231001	1993	4.67	3.6901	105.2
231009	1989	1	3.4533	93.1
231009	1990	2	4.1074	121.1
231009	1991	3	4.5052	126.8
231028	1989	1	3.2940	99.6507
231028	1990	2	3.8541	106.285
231028	1991	3	4.2207	109.947
251003	1989	1.08	2.7144	94.8
341003	1989	0.83	4.1116	109.1
341003	1990	2	5.1821	128.1
341003	1991	2.92	5.5131	134.9
341003	1992	4	5.9447	107.2
341011	1989	1.17	3.5878	115.22
341011	1990	2.08	4.1362	120.485
341011	1992	3.67	4.5519	126.572
341011	1993	4.5	4.9039	138.952
341011	1995	7.25	5.6651	100.172
341030	1989	1.5	3.1728	111.082
341030	1990	2.67	3.9995	107.411
341031	1989	1.17	3.1884	115.4
341031	1990	2.08	3.6552	123.4
341031	1992	3.67	4.1710	128.3
341031	1993	4.5	4.4432	143.7
341031	1995	7.25	5.1880	90.6
341033	1989	1.17	2.5165	94.9
341033	1990	2.08	2.8909	98.8
341033	1992	3.67	3.2131	102.3
341033	1993	4.5	3.4345	109.2
341033	1995	7.25	3.9438	88.2
341034	1989	1.75	1.9147	92.4
341034	1990	2.67	2.2323	97.9
341034	1992	4.25	2.5843	100.1
341034	1993	5.08	2.7353	106.3
341034	1995	7.83	3.1123	116.1

Table D-4: Continued

341034	2002	14.42	3.7101	113.7
341638	1989	1.75	5.3196	116.5
341638	1990	2.67	6.2426	108.1
341638	1993	5.08	7.4308	40.7
341638	2002	14.42	9.9965	82.2
421597	1989	0.92	1.7980	85.9
421599	1989	0.83	1.7465	93.0
421599	1990	1.92	2.1805	97.2
421599	1991	3	2.4428	101.1
421599	1993	4.5	2.6487	88.8
501002	1989	0.92	1.8494	92.5
501002	1990	1.92	2.1761	96.2
501002	1991	3	2.4391	100.3
501002	1993	4.58	2.6758	-
501002	1995	7.08	3.0472	106.5
501002	2000	12	3.5096	115.5
501002	2002	14.08	3.6576	117.6
501004	1989	0.92	1.5572	86.6
501004	1990	1.92	1.8475	90.7
501004	1991	3	2.0623	93.9
501004	1995	7.08	2.5268	101.9
501681	1989	1.08	1.6488	81.6
501681	1990	2.08	2.9187	95.9
501683	1989	1.08	1.6635	79.5
501683	1990	2.08	2.9471	92.9849

APPENDIX E

AASHTO ME Developed Design Tables for New York State

Table E-1: Developed Design Tables for Region 1-Albany

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	4	0
8	250	6	0
16	500	8.5	6
32	1000	10.5	6
64	2000	12.5	6
129	4000	14	6
161	5000	15	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	5	0
16	500	7	0
32	1000	9.5	6
64	2000	12	6
129	4000	13.5	6
161	5000	14	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	6	0
32	1000	8	0
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5.5	0
32	1000	7	0
64	2000	10.5	6
129	4000	12.5	6
161	5000	13	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6.5	0
64	2000	9	0
129	4000	12	6
161	5000	13	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	6	0
64	2000	9	0
129	4000	12	0
161	5000	12.5	6

Table E-2: Developed Design Tables for Region 1-Glens Falls

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	4.5	0
8	250	6.5	0
16	500	8.5	6
32	1000	10.5	6
64	2000	12.5	6
129	4000	14.5	6
161	5000	15	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	5	0
16	500	7	0
32	1000	9.5	6
64	2000	12	6
129	4000	13.5	6
161	5000	14	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	0
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5.5	0
32	1000	7.5	0
64	2000	10	0
129	4000	12.5	6
161	5000	13	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6.5	0
64	2000	9	0
129	4000	12	6
161	5000	13	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	6	0
64	2000	8.5	0
129	4000	12	0
161	5000	12.5	6

Table E-3: Developed Design Tables for Region 2-Utica

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	4.5	0
8	250	6.5	0
16	500	8.5	6
32	1000	10	6
64	2000	12.5	6
129	4000	14.5	6
161	5000	15	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5	0
16	500	7	0
32	1000	9	0
64	2000	12	6
129	4000	13.5	6
161	5000	14	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6.5	0
32	1000	8	0
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5.5	0
32	1000	7	0
64	2000	10.5	6
129	4000	12.5	6
161	5000	13	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	4.5	0
32	1000	6.5	0
64	2000	9	0
129	4000	11.5	6
161	5000	13	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	6	0
64	2000	8.5	0
129	4000	11	0
161	5000	12.5	6

Table E-4: Developed Design Tables for Region 3-Syracuse

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5.5	0
16	500	7.5	0
32	1000	9.5	6
64	2000	11.5	6
129	4000	13.5	6
161	5000	13.5	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	0
64	2000	10.5	6
129	4000	12.5	6
161	5000	13.5	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5	0
32	1000	7	0
64	2000	9.5	0
129	4000	11	6
161	5000	11.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6	0
64	2000	8.5	0
129	4000	11.5	6
161	5000	11	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5.5	0
64	2000	8	0
129	4000	10.5	6
161	5000	11	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	4.5	0
64	2000	7.5	0
129	4000	10	0
161	5000	10.5	6

Table E-5: Developed Design Tables for Region 4-Rochester

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5	0
16	500	7	0
32	1000	9	0
64	2000	11.5	6
129	4000	13.5	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	0
64	2000	10.5	6
129	4000	12.5	6
161	5000	13	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	4.5	0
32	1000	7	0
64	2000	9	0
129	4000	11	6
161	5000	11.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6	0
64	2000	8	0
129	4000	11	6
161	5000	11.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5.5	0
64	2000	7.5	0
129	4000	10	6
161	5000	10.5	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	4.5	0
64	2000	7	0
129	4000	10	0
161	5000	10	6

Table E-6: Developed Design Tables for Region 5-Buffalo

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5.5	0
16	500	7	0
32	1000	8.5	6
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	0
64	2000	10	0
129	4000	12.5	6
161	5000	13	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5	0
32	1000	7	0
64	2000	9	0
129	4000	11.5	6
161	5000	12.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	6	0
64	2000	8	0
129	4000	11	6
161	5000	11.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5	0
64	2000	7.5	0
129	4000	10	6
161	5000	11	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	4.5	0
64	2000	6.5	0
129	4000	10	0
161	5000	10	6

Table E-7: Developed Design Tables for Region 5-Dunkirk

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5.5	0
16	500	7.5	0
32	1000	9.5	6
64	2000	12	6
129	4000	13.5	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6.5	0
32	1000	8.5	0
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5	0
32	1000	7.5	0
64	2000	10	0
129	4000	12	6
161	5000	13	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6.5	0
64	2000	9	0
129	4000	11.5	6
161	5000	12.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5.5	0
64	2000	8.5	0
129	4000	11	6
161	5000	12	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5	0
64	2000	8	0
129	4000	11	0
161	5000	11.5	6

Table E-8: Developed Design Tables for Region 5-Niagara Falls

Mr 4 KSI				Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)	ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0	2	50	3	0
3	100	3.5	0	3	100	3	0
8	250	5	0	8	250	4.5	0
16	500	7	0	16	500	6.5	0
32	1000	9	6	32	1000	8	0
64	2000	11	6	64	2000	10	6
129	4000	12.5	6	129	4000	12	6
161	5000	13	6	161	5000	12.5	6
Mr 6 KSI				Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)	ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0	2	50	3	0
3	100	3	0	3	100	3	0
8	250	3.5	0	8	250	3	0
16	500	4.5	0	16	500	5	0
32	1000	6.5	0	32	1000	6	0
64	2000	9	0	64	2000	8	0
129	4000	11	6	129	4000	11	6
161	5000	11.5	6	161	5000	11.5	6
Mr 8 KSI				Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)	ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0	2	50	3	0
3	100	3	0	3	100	3	0
8	250	3	0	8	250	3	0
16	500	3.5	0	16	500	3.5	0
32	1000	5	0	32	1000	4.5	0
64	2000	7	0	64	2000	6.5	0
129	4000	9.5	6	129	4000	9.5	0
161	5000	10.5	6	161	5000	10	6

Table E-9: Developed Design Tables for Region 6-Dansville

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5.5	0
16	500	8	0
32	1000	9.5	6
64	2000	12	6
129	4000	13.5	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6.5	0
32	1000	9	0
64	2000	11	6
129	4000	12	6
161	5000	13.5	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5	0
32	1000	8	0
64	2000	10	6
129	4000	12.5	6
161	5000	13	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	7	0
64	2000	9.5	0
129	4000	12	6
161	5000	12.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	6	0
64	2000	9	0
129	4000	11.5	6
161	5000	12	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5	0
64	2000	8	0
129	4000	11	0
161	5000	12	6

Table E-10: Developed Design Tables for Region 6-Elmira

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5.5	0
16	500	7.5	0
32	1000	9.5	6
64	2000	11.5	6
129	4000	13.5	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6.5	0
32	1000	8	6
64	2000	11	6
129	4000	12.5	6
161	5000	13	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5	0
32	1000	8	0
64	2000	10	0
129	4000	12	6
161	5000	12.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	4.5	0
32	1000	6.5	0
64	2000	9	0
129	4000	11.5	6
161	5000	12	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	6	0
64	2000	8	0
129	4000	11	6
161	5000	11.5	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5	0
64	2000	7.5	0
129	4000	10.5	0
161	5000	11.5	6

Table E-11: Developed Design Tables for Region 6- Wellsville

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	4	0
8	250	6	0
16	500	7.5	0
32	1000	9	6
64	2000	11.5	6
129	4000	13	6
161	5000	13.5	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	6
64	2000	10.5	6
129	4000	12.5	6
161	5000	13	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5	0
32	1000	7	0
64	2000	9.5	0
129	4000	11.5	6
161	5000	12	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	4.5	0
32	1000	6	0
64	2000	8.5	0
129	4000	11	6
161	5000	11.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5.5	0
64	2000	8	0
129	4000	10.5	6
161	5000	11	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5	0
64	2000	7	0
129	4000	10	0
161	5000	11	6

Table E-12: Developed Design Tables for Region 7- Massena

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	4	0
8	250	6	0
16	500	8	0
32	1000	10	6
64	2000	12	6
129	4000	14	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5	0
16	500	7	0
32	1000	9	0
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	0
64	2000	10	6
129	4000	12.5	6
161	5000	13	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5	0
32	1000	7	0
64	2000	9.5	0
129	4000	12	6
161	5000	12.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6	0
64	2000	9	0
129	4000	11.5	6
161	5000	12	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	6	0
64	2000	8	0
129	4000	11.5	0
161	5000	12	6

Table E-13: Developed Design Tables for Region 7- Plattsburgh

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	4	0
8	250	6	0
16	500	8	0
32	1000	10	6
64	2000	12	6
129	4000	14	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5	0
16	500	7	0
32	1000	9	0
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	0
64	2000	10	6
129	4000	12.5	6
161	5000	13	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5	0
32	1000	7	0
64	2000	9.5	0
129	4000	12	6
161	5000	12.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6	0
64	2000	9	0
129	4000	11.5	6
161	5000	12	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	6	0
64	2000	8	0
129	4000	11.5	0
161	5000	12	6

Table E-14: Developed Design Tables for Region 7- Watertown

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	4	0
8	250	6	0
16	500	8	0
32	1000	9.5	6
64	2000	12	6
129	4000	13.5	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5	0
16	500	6.5	0
32	1000	9	0
64	2000	11	6
129	4000	13	6
161	5000	13	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5.5	0
32	1000	8	0
64	2000	10	0
129	4000	12	6
161	5000	12.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	4.5	0
32	1000	7	0
64	2000	9.5	0
129	4000	11.5	6
161	5000	12.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6	0
64	2000	8.5	0
129	4000	11	6
161	5000	12	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5	0
64	2000	8	0
129	4000	11	0
161	5000	11.5	6

Table E-15: Developed Design Tables for Region 8- Montgomery

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	6	0
16	500	7.5	0
32	1000	9.5	6
64	2000	12	6
129	4000	13.5	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6.5	0
32	1000	9	0
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5	0
32	1000	7.5	0
64	2000	10.5	6
129	4000	12.5	6
161	5000	13	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	4.5	0
32	1000	7	0
64	2000	9.5	0
129	4000	12	6
161	5000	12.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	5	0
32	1000	6	0
64	2000	8.5	0
129	4000	11.5	6
161	5000	12	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	5	0
32	1000	5.5	0
64	2000	9.5	0
129	4000	11	0
161	5000	11.5	6

Table E-16: Developed Design Tables for Region 8- Poughkeepsie

Mr 4 KSI				Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)	ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0	2	50	3	0
3	100	4	0	3	100	3	0
8	250	6	0	8	250	5	0
16	500	8	0	16	500	7	0
32	1000	10	6	32	1000	9	6
64	2000	12	6	64	2000	11	6
129	4000	14	6	129	4000	12.5	6
161	5000	14.5	6	161	5000	13	6
Mr 6 KSI				**Mr 7 KSI**			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)	ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0	2	50	3	0
3	100	3	0	3	100	3	0
8	250	4	0	8	250	3.5	0
16	500	5.5	0	16	500	5	0
32	1000	8	0	32	1000	7	0
64	2000	10	6	64	2000	9.5	6
129	4000	12.5	6	129	4000	12	6
161	5000	13	6	161	5000	12.5	6
Mr 8 KSI				**Mr 9 KSI**			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)	ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0	2	50	3	0
3	100	3	0	3	100	3	0
8	250	3	0	8	250	3	0
16	500	4.5	0	16	500	4	0
32	1000	6	0	32	1000	6	0
64	2000	9	0	64	2000	8.5	0
129	4000	11.5	6	129	4000	11	0
161	5000	12	6	161	5000	12	6

Table E-17: Developed Design Tables for Region 8- White Plains

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5.5	0
16	500	7	0
32	1000	9	6
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	6
64	2000	10	6
129	4000	12	6
161	5000	12.5	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5	0
32	1000	7	0
64	2000	9	6
129	4000	11.5	6
161	5000	12	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6	0
64	2000	8	6
129	4000	11.5	6
161	5000	12	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5	0
64	2000	7.5	0
129	4000	10	6
161	5000	11	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5	0
64	2000	7	0
129	4000	9.5	0
161	5000	10.5	6

Table E-18: Developed Design Tables for Region 9- Virtual Climatic Stations

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5.5	0
16	500	7.5	0
32	1000	9	6
64	2000	11.5	6
129	4000	13.5	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	6
64	2000	10.5	6
129	4000	12.5	6
161	5000	13	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5	0
32	1000	7.5	0
64	2000	9.5	6
129	4000	12	6
161	5000	12.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	4.5	0
32	1000	6.5	0
64	2000	9	0
129	4000	11.5	6
161	5000	12	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5.5	0
64	2000	8	0
129	4000	11	6
161	5000	11.5	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5	0
64	2000	7.5	0
129	4000	11	0
161	5000	11	6

Table E-19: Developed Design Tables for Region 10- Farmingdale

Mr 4 KSI				Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)	ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0	2	50	3	0
3	100	3.5	0	3	100	3	0
8	250	5	0	8	250	4	0
16	500	7	0	16	500	5.5	0
32	1000	8.5	6	32	1000	8.5	0
64	2000	11	6	64	2000	10	6
129	4000	13	6	129	4000	12.5	6
161	5000	13.5	6	161	5000	13	6
Mr 6 KSI				**Mr 7 KSI**			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)	ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0	2	50	3	0
3	100	3	0	3	100	3	0
8	250	3.5	0	8	250	3	0
16	500	4.5	0	16	500	4.5	0
32	1000	7	0	32	1000	6	0
64	2000	9.5	6	64	2000	8	0
129	4000	11.5	6	129	4000	11	6
161	5000	12	6	161	5000	11.5	6
Mr 8 KSI				**Mr 9 KSI**			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)	ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0	2	50	3	0
3	100	3	0	3	100	3	0
8	250	3	0	8	250	3	0
16	500	3.5	0	16	500	3.5	0
32	1000	5	0	32	1000	4.5	0
64	2000	7.5	0	64	2000	6.5	0
129	4000	10.5	6	129	4000	9.5	0
161	5000	11	6	161	5000	10.5	6

Table E-20: Developed Design Tables for Region 10- Islip

Mr 4 KSI

ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5	0
16	500	7	0
32	1000	8.5	6
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 5 KSI

ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5.5	0
32	1000	8	0
64	2000	10	6
129	4000	12.5	6
161	5000	13	6

Mr 6 KSI

ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	4.5	0
32	1000	7	0
64	2000	9	6
129	4000	11.5	6
161	5000	12	6

Mr 7 KSI

ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6	0
64	2000	8	0
129	4000	11	6
161	5000	11.5	6

Mr 8 KSI

ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5	0
64	2000	7.5	0
129	4000	10.5	6
161	5000	11	6

Mr 9 KSI

ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	4.5	0
64	2000	6.5	0
129	4000	9.5	0
161	5000	10.5	6

Table E-21: Developed Design Tables for Region 10- Shirley

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5	0
16	500	7.5	0
32	1000	9.5	6
64	2000	11.5	6
129	4000	13.5	6
161	5000	14	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	0
64	2000	10.5	6
129	4000	12.5	6
161	5000	13.5	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6	0
32	1000	8	0
64	2000	10.5	6
129	4000	12.5	6
161	5000	13.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	5	0
32	1000	6	0
64	2000	9	6
129	4000	11.5	6
161	5000	12	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5	0
64	2000	8	0
129	4000	11	6
161	5000	11.5	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	4.5	0
64	2000	7	0
129	4000	10	0
161	5000	11.5	6

Table E-22: Developed Design Tables for Region 11-NY 94728

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3.5	0
8	250	5.5	0
16	500	8	0
32	1000	9.5	6
64	2000	12.5	6
129	4000	14	6
161	5000	14	12

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4.5	0
16	500	6.5	0
32	1000	9	0
64	2000	11.5	6
129	4000	13.5	6
161	5000	14	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5	0
32	1000	8	0
64	2000	10.5	6
129	4000	13	6
161	5000	13.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	7	0
64	2000	10	6
129	4000	12.5	6
161	5000	13	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6	0
64	2000	9	0
129	4000	12	6
161	5000	12.5	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5.5	0
64	2000	8.5	0
129	4000	11.5	0
161	5000	12	6

Table E-23: Developed Design Tables for Region 11-NY 94789

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	5	0
16	500	6.5	0
32	1000	8.5	6
64	2000	11	6
129	4000	12.5	6
161	5000	13.5	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5	0
32	1000	7.5	0
64	2000	9.5	6
129	4000	12	6
161	5000	12.5	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6.5	0
64	2000	8.5	6
129	4000	11	6
161	5000	11.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5.5	0
64	2000	7.5	6
129	4000	10	6
161	5000	11	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	4.5	0
64	2000	7	0
129	4000	9.5	6
161	5000	10	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	4.5	0
64	2000	6	0
129	4000	9	0
161	5000	10	6

Table E-24: Developed Design Tables for Region 11-NY 14732

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	5	0
16	500	7	0
32	1000	8.5	6
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	5.5	0
32	1000	8	0
64	2000	10	6
129	4000	12	6
161	5000	13	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	4.5	0
32	1000	6.5	0
64	2000	9	6
129	4000	11.5	6
161	5000	12	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	5.5	0
64	2000	8	6
129	4000	10.5	6
161	5000	11.5	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	5	0
64	2000	7.5	0
129	4000	10	6
161	5000	11	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	3.5	0
32	1000	4.5	0
64	2000	6.5	0
129	4000	9.5	0
161	5000	10.5	6

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