New England Verification of National Cooperative Highway Research Program (NCHRP) 1-37A Mechanistic-Empirical Pavement Design Guide (MEPDG)

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16. Abstract

In 1996, the National Cooperative Highway Research Program (NCHRP) launched Project 1-37A to develop a new design guide for pavement structures. The design guide recommended by the project team in 2004 is based on mechanistic-empirical (M-E) principles and is accompanied by software that handles the execution of the design and performance prediction. The mechanistic empirical pavement design guide (MEPDG) software has gone through various version upgrades and improvements to the incorporated models and user interface, the latest being DarwinME. The design inputs needed for the MEPDG software are classified according to a hierarchy system where the designer can select the level of data accuracy and sophistication based on the economic impact of the project. The selection is also a function of the state-of-knowledge and availability of the data. The levels vary from Level 1, for which design inputs are generally site specific and are determined from material testing and/or in-situ measurement to Level 2 and 3, where default or user-selected values obtained from national and regional experiences such as LTTP sites are used. The performance prediction models incorporated in the MEPDG were validated and calibrated using field performance of selected pavement sections throughout the United States. Coefficients incorporated in the models can thus be regarded as national averages derived from the performance measured from the sites selected for the calibration. While the State Highway Agencies (SHAs) can use those models with the "default" coefficients, a higher level of reliability can be achieved in predicting the distresses if the agencies adjust the coefficients to better suit the conditions prevalent in their states. It is widely recognized that local calibration of the models should thus be performed to take full advantage of the MEPDG.

The main goal of this research was to offer the New England and New York state highway agencies guidelines for the implementation of the MEPDG for designing flexible pavements and AC overlays. This report documents the current design practices of the six New England States and New York as well as progress of MEPDG implementation initiatives undertaken by other states. A comprehensive sensitivity analysis of the MEPDG Level 2 and 3 inputs for each of the seven states involved in this study was conducted. The extensive software runs conducted allow for an evaluation of the MEPDG functionality and accuracy for the level of inputs used by comparing predicted distresses with field-measured distresses, and provide individual states with an idea on adequacy of their input database and accuracy that the embedded distress models with nationally calibrated coefficients provide. The findings can be used by the state agencies in their decision on whether to start implementing the MEPDG with current models and coefficients and for what level of analysis, and in prioritizing implementation activities.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	Litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

* SI is the symbol for the International System of Measurement

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
		AREA		
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
		VOLUME		
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C+32	Fahrenheit temperature	°F

°F 32 98.6 212 °F

-40 0 40 80 120 180 200

-40 -20 0 20 40 60 80 100

°C 37 °C

ABSTRACT

In 1996, the National Cooperative Highway Research Program (NCHRP) launched Project 1-37A to develop a new design guide for pavement structures. The design guide recommended by the project team in 2004 is based on mechanistic-empirical (M-E) principles and is accompanied by software that handles the execution of the design and performance prediction. The mechanistic empirical pavement design guide (MEPDG) software has gone through various version upgrades and improvements to the incorporated models and user interface, the latest being DarwinME. The design inputs needed for the MEPDG software are classified according to a hierarchy system where the designer can select the level of data accuracy and sophistication based on the economic impact of the project. The selection is also a function of the state-of-knowledge and availability of the data. The levels vary from Level 1, for which design inputs are generally site specific and are determined from material testing and/or in-situ measurement to Level 2 and 3, where default or user-selected values obtained from national and regional experiences such as LTPP sites are used. The performance prediction models incorporated in the MEPDG were validated and calibrated using field performance of selected pavement sections throughout the United States. Coefficients incorporated in the models can thus be regarded as national averages derived from the performance measured from the sites selected for the calibration. While the State Highway Agencies (SHAs) can use those models with the "default" coefficients, a higher level of reliability can be achieved in predicting the distresses if the agencies adjust the coefficients to better suit the conditions prevalent in their states. It is widely recognized that local calibration of the models should thus be performed to take full advantage of the MEPDG.

The main goal of this research was to offer the New England and New York state highway agencies guidelines for the implementation of the MEPDG for designing flexible pavements and AC overlays. This report documents the current design practices of the six New England States and New York as well as progress of MEPDG implementation initiatives undertaken by other states. A comprehensive sensitivity analysis of the MEPDG Level 2 and 3 inputs for each of the seven states involved in this study was conducted. The extensive software runs conducted allow for an evaluation of the MEPDG functionality and accuracy for the level of inputs used by comparing predicted distresses with field-measured distresses, and provide individual states with an idea on adequacy of their input database and accuracy that the embedded distress models with nationally calibrated coefficients provide. The findings can be used by the state agencies in their decision on whether to start implementing the MEPDG with current models and coefficients and for what level of analysis, and in prioritizing implementation activities.

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LIST OF ABBREVIATIONS

MEPDG	Mechanistic Empirical Pavement Design Guide
NCHRP	National Cooperative Highway Research Program
AASHTO	American Association of State Highway and Transportation Officials
DOT	Department of Transportation
SHA	State Highway Agency
AC	Asphalt Concrete
AADT	Average Annual Daily Traffic
AADTT	Average Annual Daily Truck Traffic
SP	Superpave Specifications – Special Publication Documents
USGS	United States Geological Survey
LTPP	Long Term Pavement Performance
FHWA	Federal Highway Administration
NETC	New England Transportation Consortium
DGIT	Design Guide Implementation Team
PG	Performance Grade Asphalt Binder Specification
NH	New Hampshire
CT	Connecticut
ME	Maine
NY	New York
RI	Rhode Island
VT	Vermont
CTC	Coefficient of Thermal Contraction of the Asphalt Concrete Mix

1. INTRODUCTION

In 1996, the National Cooperative Highway Research Program (NCHRP) launched Project 1-37A to develop a new design guide for pavement structures. The design guide recommended by the project team in 2004 is based on mechanistic-empirical (M-E) principles. Pavement performance is determined by the existing traffic and environmental conditions, pavement structure and material properties. With increasing traffic volume on highways and intensive material specification, a correspondingly efficient pavement design methodology is required.

The MEPDG is therefore considered by the FHWA (1) as an important factor in improving the national highway system. MEPDG uses performance prediction models to predict pavement performance over a specified design life period. The design guide can therefore be used to design pavements such that their performance is maximized over the design life. The FHWA has developed a Design Guide Implementation Team (DGIT) whose mission is “To raise awareness, assist, and support State Highway Agencies and their industry partners in the development and implementation of the new mechanistic-empirical Design Guide”. A Lead States Group has also been established and includes: Arizona, California, Florida, Kentucky, Maryland, Minnesota, Mississippi, Missouri, Montana, New Jersey, New Mexico, Pennsylvania, Texas, Utah, Virginia, Washington, and Wisconsin. These states are actively pursuing implementation of the MEPDG, have obtained upper management support of the process, and are willing to act as champions for implementation (2).

The New England states and New York need to gather more information on what will be involved and what the advantages will be before the decision to implement the MEPDG can be made. The Level 2 and Level 3 input variables which will require state specific information and for which variables the national default values are acceptable were determined by conducting design runs on the MEPDG software. The report explains the research conducted to gather this information for each individual state and provide recommendations on steps that need to be taken to successfully implement the MEPDG. Regional and/or local calibrations can be performed for the states as a future activity if appropriate data is available for that purpose.

2. THE M-E PAVEMENT DESIGN GUIDE

The Mechanistic Empirical Pavement Design Guide (MEPDG) is a significant improvement over the previous AASHTO design guides for pavement design. It was developed under NCHRP Project 1-37A based on mechanistic-empirical principles. The M-E design guide is built into the form of software that is capable of using various parameters involved in pavement design as input to predict the performance of the pavement over a specified design life.

Briefly stated, the M-E performance prediction model consists of four sub-models: the environmental effects model, pavement response model, material characterization model, and performance prediction model (Figure 1). The model is termed mechanistic due to the mechanistic calculation of stresses, strains, and deflections of a pavement structure, which are the fundamental pavement responses under repeated traffic loadings. The empirical component comes into play by relating the pavement responses to field distresses and performance using existing empirical relationships, widely known as transfer functions (3). The design process is an iterative procedure that starts with a trial design and ends when predicted distresses meet the acceptable limits based on the level of statistical reliability desired.

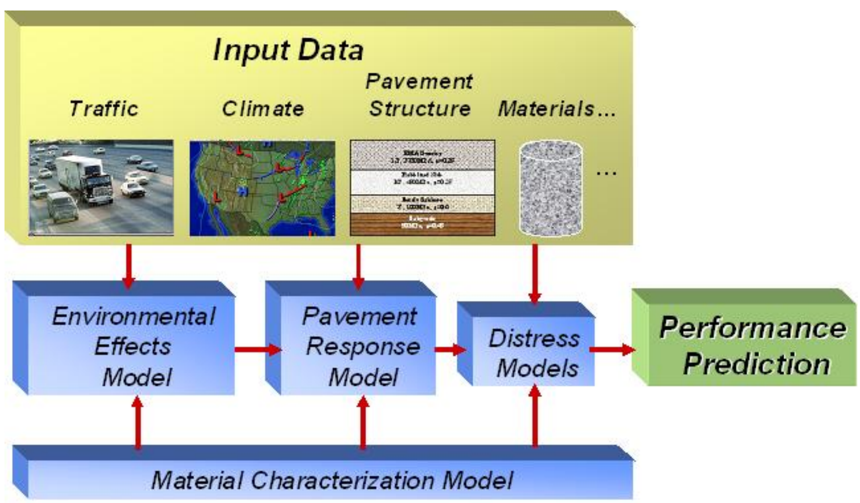

Figure 1 Mechanistic Empirical Design Procedure incorporated in the MEPDG

In addition, design inputs are classified according to a hierarchy system where the designer can select the Level of data accuracy and sophistication based on the economic impact of the project. The selection is also a function of the state-of-knowledge and availability of the data. A summary of the hierarchical Design Levels follows below:

- Level 3 represents the lowest level of the hierarchy system and provides the lowest level of reliability; the inputs consist of default or user-selected values obtained from national and regional experiences such as LTPP sites.

- Level 2 represents a higher level in the hierarchy system and provides more reliability than Level 3. Design inputs are based on laboratory test data and/or default predictive equations. This level is expected to be used on pavement design projects of higher significance.
- Level 1 represents the highest level in the hierarchy system and provides the highest degree of reliability. Design inputs are generally site specific and are determined from material testing and/or in-situ measurement.

The engineers select the inputs and determine the types and quantities of data needed for a reliable design. This process requires a thorough evaluation of all of design parameters and a detailed analysis of how the input values will affect the predicted performance. The MEPDG design process therefore demands a huge amount of information from the engineers concerning pavement inputs and pavement performance.

The current distress prediction models incorporated in the M-E Pavement Design Guide (MEPDG) were validated and calibrated using field performance of selected pavement sections throughout the United States, among which are numerous Long Term Pavement Performance (LTPP) test sites. Thus, the constants incorporated in the models can be regarded as national averages derived from the performance measured from the sites selected for the calibration. While the State Highway Agencies (SHAs) can use those models with the "default" coefficients, a higher level of reliability can be achieved in predicting the distresses if the agencies adjust the coefficients to better suit the conditions prevalent in their states. For more accuracy, the SHA's can go as far as adopting coefficients calibrated for different regions in their states that exhibit different design conditions such as climate, traffic, and subgrade type. For that purpose, the distress models incorporated in the new MEPDG include calibration constants that can be determined by each SHA for its state or regions within the state. It is widely recognized that local calibration of the models should be performed to take full advantage of the MEPDG.

Figure 2 provides a flow chart for the mechanistic-empirical design approach as implemented in the MEPDG procedures (4).

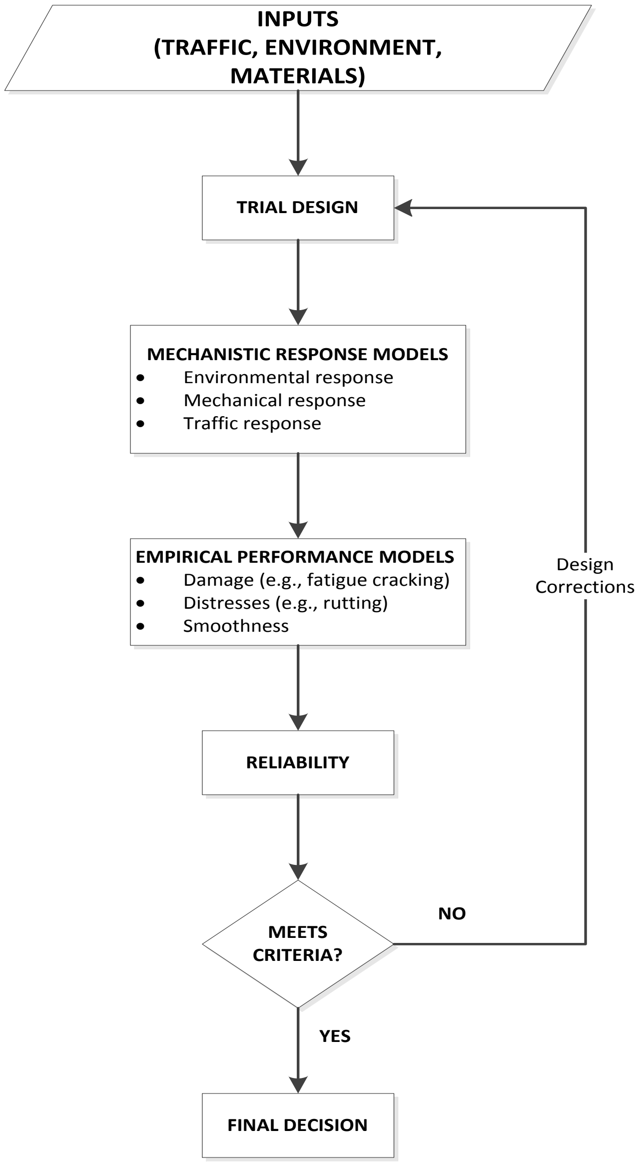

Figure 2 Flow Chart for Mechanistic-Empirical Design Methodology (4)

The following lists the major steps in this design methodology for a new flexible pavement:

1. Specify and define the required inputs including traffic, environmental, materials, etc.
2. Select a trial pavement section for analysis.
3. Define the properties of materials in the various pavement layers.
4. Analyze the pavement response due to traffic loading and environmental influences.
5. Empirically relate critical pavement responses to damage and distress for the pavement distresses of interest.

6. Adjust the predicted distresses for the specified design reliability.
7. Compare the predicted distresses at the end of pavement design life against design limits.
8. If necessary, adjust the trial pavement section and repeat steps 3-7 until all predicted distresses are within design limits.

To implement the above mechanistic-empirical methodology, the following corresponding major components are needed:

- Inputs – traffic, materials, climate and other general values (e.g. design life, latitude, longitude and elevation)
- Pavement response model
- Environmental response model
- Material characterization model
- Performance prediction model
- Design reliability – to increase the safety of the design
- Software – to implement the mechanistic-empirical models and calculation in a usable form.

Pavement response is a function of three primary influences: environmental (climate), traffic, and pavement (materials and thicknesses). The mechanistic-empirical process is outlined in Figure 3 (5).

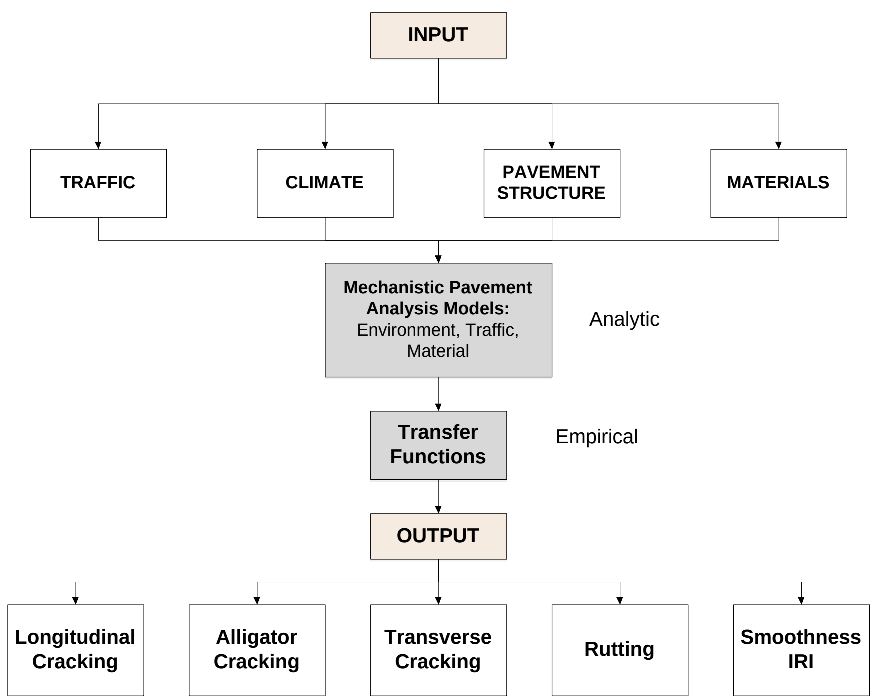

Figure 3 MEPDG Outline Process (5)

The environmental model plays a significant role in the performance of pavement. The MEPDG software provides environmental data sets for specific locations from over 800 weather stations throughout the U.S., as well as historical records for up to 10 years. This model recognizes not only external factors such as temperature, precipitation, freeze-thaw cycles and depth to water table, but also internal factors such as the susceptibility of the pavements materials to moisture and frost heaving, drainage ability of the paving layers and potential infiltration of the pavements. Temperature and moisture variations within the pavement structures and subgrade over the design life of pavement are simulated by the Enhanced Integrated Climatic Model (EICM).

The traffic model inputs are also significant for the analysis and design of pavement structures. The mechanistic response model in the MEPDG requires the magnitudes and frequencies of the actual wheel load that the pavement is expected to experience over its design life. Typically, state highway agencies collect two categories of traffic data: weight-in-motion (WIM) and Automatic Vehicle Classification (AVC). WIM data provides information about truck axle weights and gross vehicle weights as they drive over a sensor. AVC data provides information about the number and types of vehicles that use a given roadway over some period of time.

The material characterization model is used in the MEPDG to calculate the stresses, strains and deflections in the pavement. Pavement performance is evaluated in the MEPDG by individual empirical distress models, also termed as transfer functions.

“The transfer function is the empirical part of the distress prediction model that relates the critical pavement response parameter, either directly or through the damage concept, to pavement distress" (6).

Empirical models are incorporated in the MEPDG for the major structural distresses and smoothness estimation in flexible pavements.

2.1 The MEPDG Software User Interface

The MEPDG software consists of a user-friendly interface which allows users to input data for designing a new or rehabilitated, flexible or rigid pavement or overlays. Figures 4 and 5 show the MEPDG software user interface. The interface is divided into various panels for entering the inputs in a systematic way. The various panels of the MEPDG interface are the project information panel, input parameters panel, results (predicted performance) panel, and user information panels – analysis status, general project information summary and the output properties information panels.

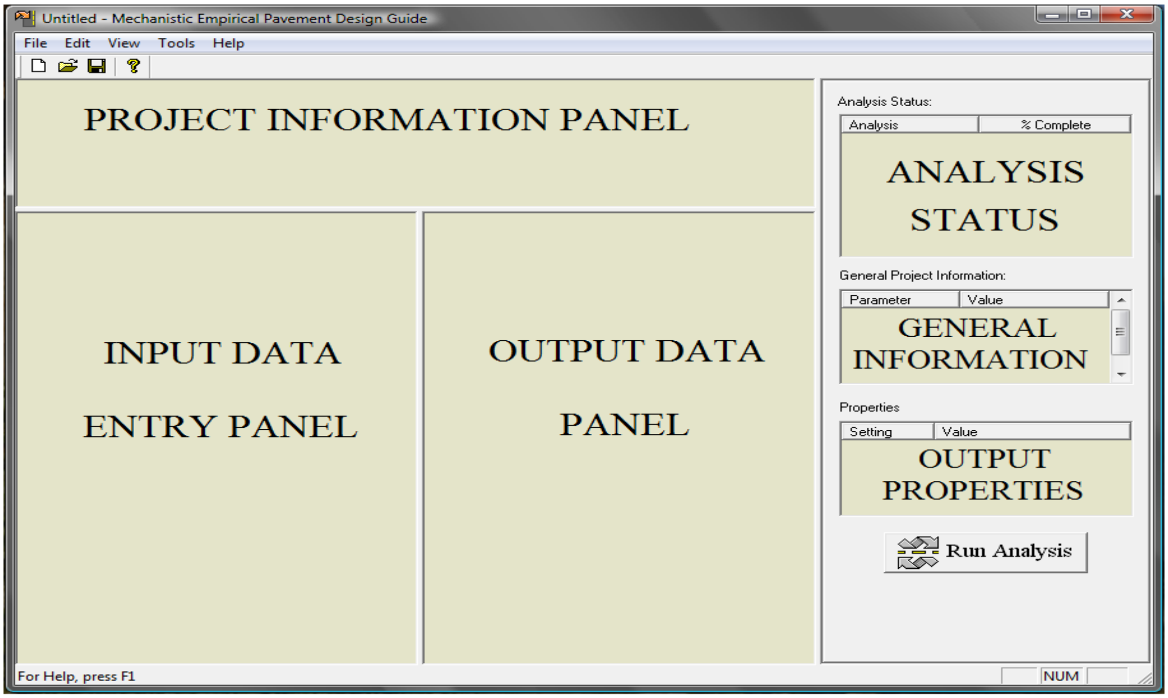

Figure 4 MEPDG Interface Explanation – Description of Interface Panels

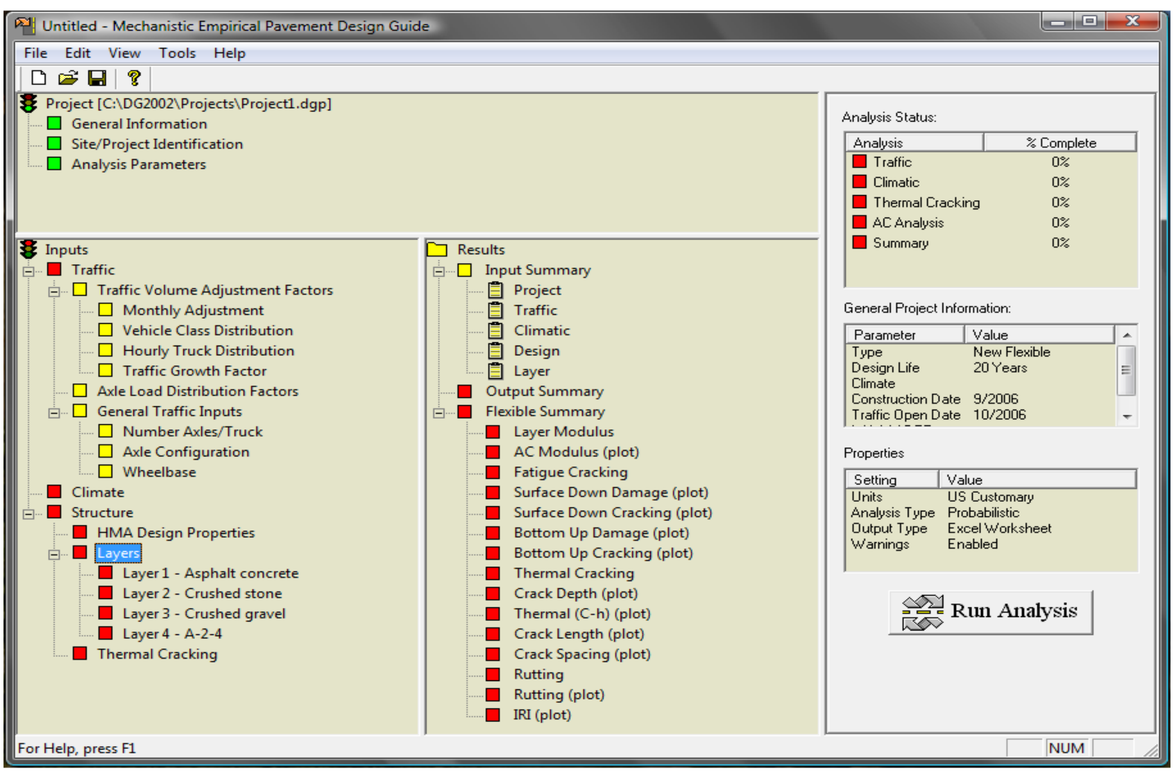

Figure 5 MEPDG Software User Interface

A summary of the design information including the type of design is required by the MEPDG as general information. The inputs required by the MEPDG can be broadly divided into four categories:

2.1.1. Traffic Inputs

The traffic inputs required (Figure 6) by the MEPDG are explained in this section. Traffic input parameters are further divided into three types – traffic volume adjustment factors, axle load distribution factors and general traffic inputs. Axle load distribution factors were not used in the study due to unavailability of data for the selected sections. Monthly and hourly volume distribution factors were kept at the default level as their effect on predicted performance is not significant. This was identified in the literature review conducted on previous studied using the MEPDG. Values for various other parameters were obtained from LTPP database and Department of Transportation websites.

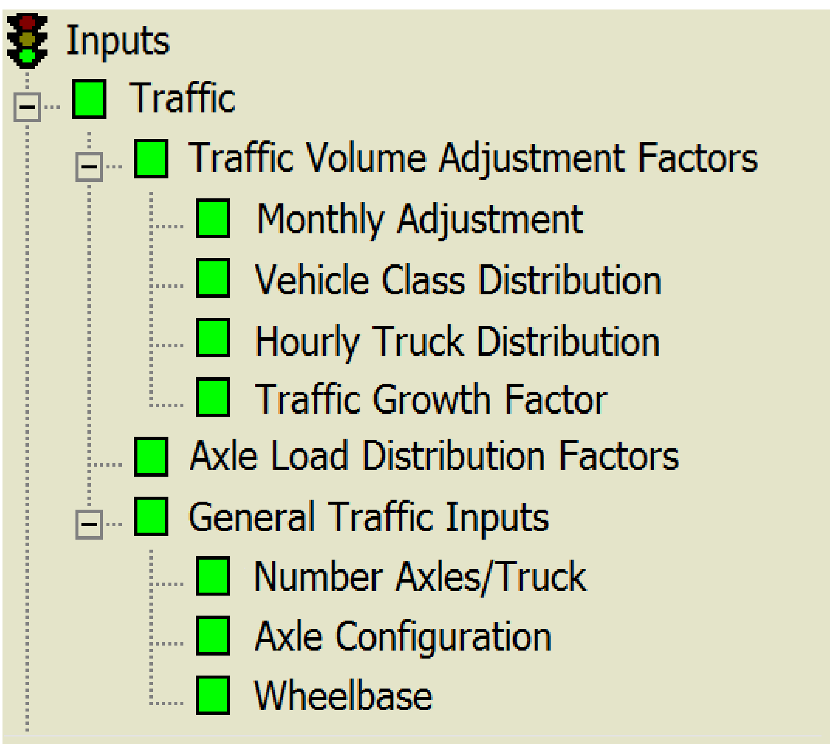

Figure 6 MEPDG Traffic Inputs

The important variables which require user-defined inputs for traffic are:
Traffic:

- Initial 2 – way AADTT
- Number of lanes in the design direction

- Percentage of trucks in design direction
- Percentage of trucks in design lane
- Operational Speed on the highway (mph)

Traffic Volume Adjustment Factors:

- Monthly and hourly traffic volume distributions can be retained at default level
- Truck Class Distribution factors must be obtained for the highway (Figure 7)
- Traffic growth factors – a linear traffic growth rate was used for the study, and is obtained from linear regression of traffic volume count history

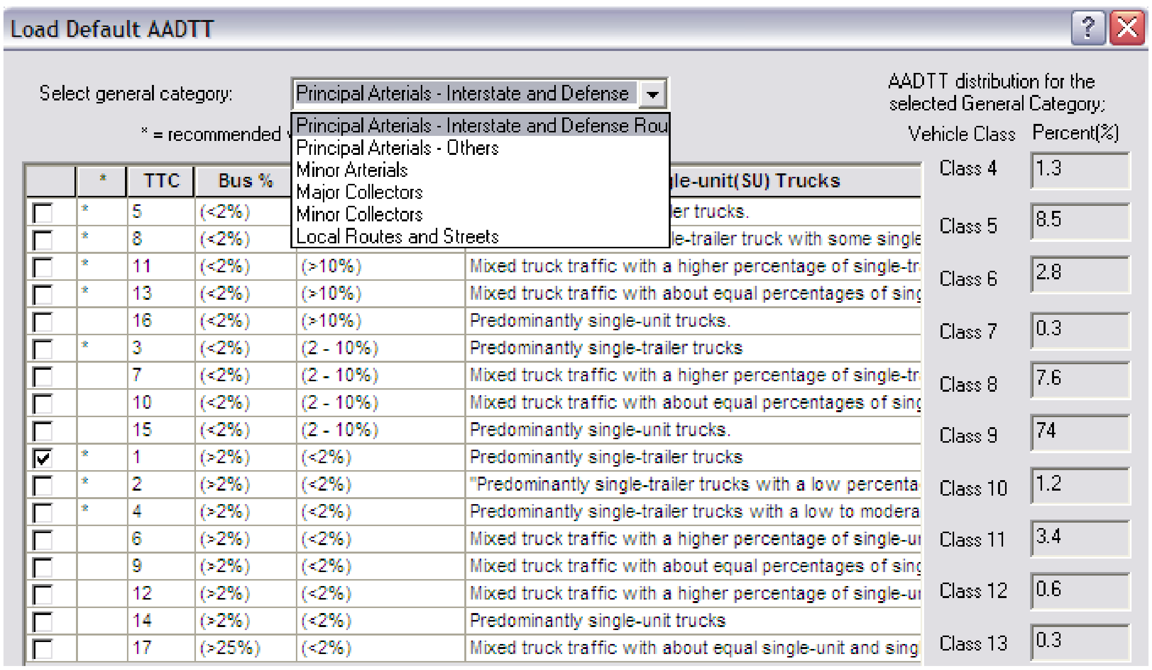

Figure 7 AADTT Distribution Default Values for the Selected General Category

General Traffic Inputs:

- Traffic wander and mean wheel location from lane marking was used as default
- Design lane width was obtained from LTPP data
- Number of axles per truck, axle configuration and wheelbase were assigned default values given in the MEPDG

2.1.2 Climate Inputs

Climate data is incorporated into the MEPDG through the HCD climate database. The database consists of all climate-related variables such as daily and monthly maximum and minimum temperature, precipitation, wind speed, etc for a large number of stations all over the US. The climate data for the pavement construction location can either be directly used if present in the database, or interpolated using the latitude, longitude and elevation of the location from any number of the surrounding six climate stations identified by the MEPDG (Figure 8).

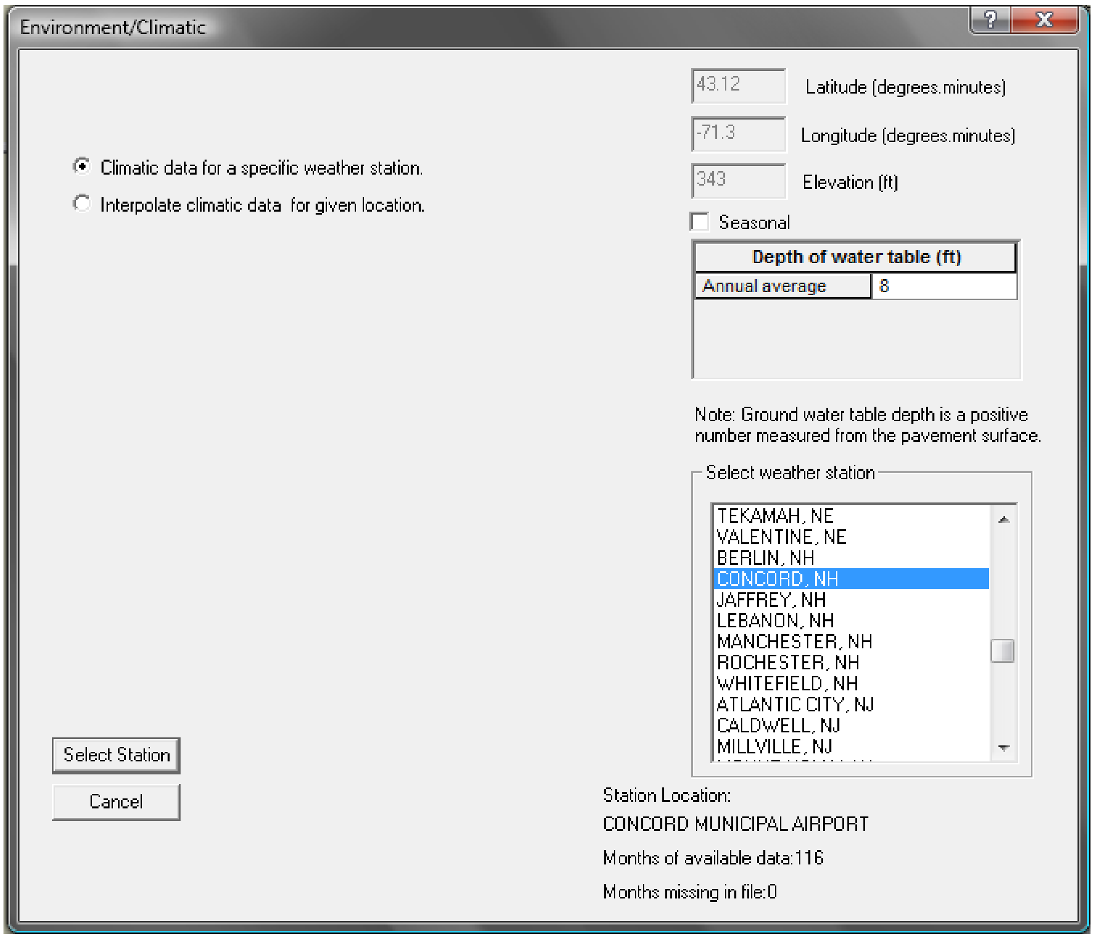

Figure 8 MEPDG Climate Inputs

The climate data for the stations present in the database also displays the total number of months for which data has been collected and stored in the database for computation. The higher the number of months of collected data, the greater the reliability of the predicted distresses. MEPDG design usually consists of performance prediction for pavements for a design life greater than the maximum months of available data (116 for the MEPDG) (6). Therefore, the data is reactivated from month 1 and is appended as the data for month 117. A larger collection of monthly climate data leads to reduction in error in predicting temperature-dependent material properties and provides a more realistic distribution of temperatures over the design life.

Water table depth has to be entered by the user for the location. The water table depths can be obtained from field tests at the construction site or interpolated from data for various groundwater monitoring stations present in the US Geological Survey website (7).

2.1.3 Pavement Layer Structure

The pavement layer structure must be determined in the structure input module (Figure 9). The surface shortwave absorptivity is assigned a default value of 0.85 for all levels of design in the MEPDG. It can be assigned values by the user based on the type of pavement, with a value of 0.90 to 0.95 for new asphalt pavement design. The design guide recommends a surface shortwave absorptivity value of 0.70 – 0.90 for aged PCC layer, 0.80 to 0.90 for weathered asphalt layer and 0.90 to 0.98 for new asphalt layer.

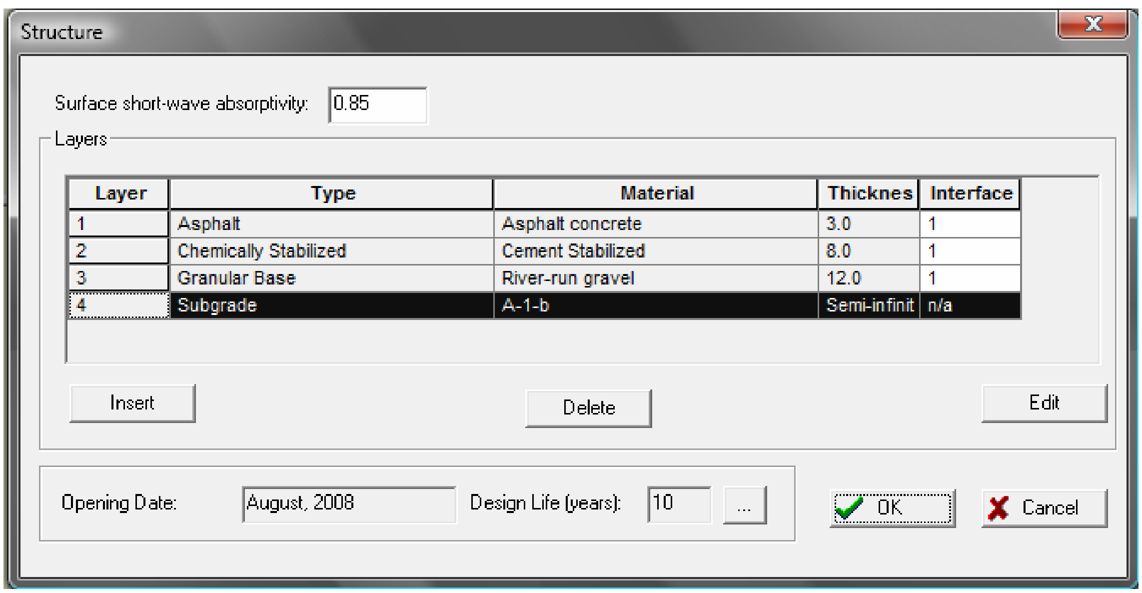

Figure 9 MEPDG Structure Inputs

The various layers that the pavement is designed to consist of must be entered at this screen. The type of layer and the material the layer is made of are the two main inputs that the user would be concerned with. The thickness of the layers can be entered along with the layer properties. It is important to note that the material used for each layer can be changed at the interface for the layer properties, but the type of layer cannot be changed. To change a layer type, the layer has to be deleted in the structure module and a new layer with the desired layer type has to be added.

2.1.4 Layer Properties / Materials Input

The properties for each layer must be entered in this module (Figure 10). The various input parameters that must be entered for different types of layers are listed below.

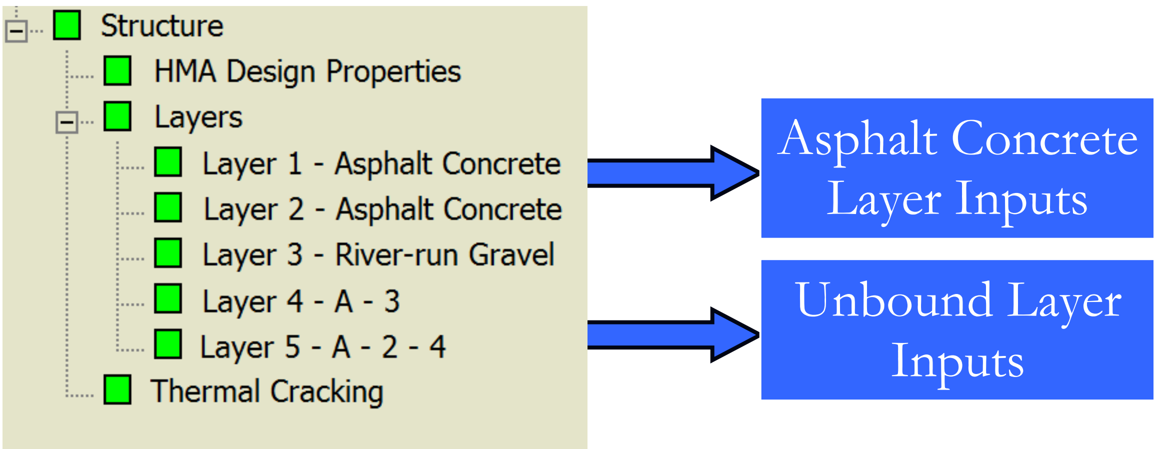

Figure 10 Layer Types

Asphalt Concrete Layer

Level 3

Asphalt Mix Parameters

- Asphalt concrete layer thickness
- Aggregate gradation for the asphalt concrete mix

Asphalt Binder Parameters

- Binder grade – Three different methods of binder grading are available in the MEPDG, namely Superpave PG binder grading system, conventional viscosity grading system and the conventional penetration grading system

Asphalt General Parameters

- Reference temperature can be assigned a default value of 70°F
- Volumetric properties of the asphalt concrete mix – air void content, effective binder content and total unit weight of the mix
- Poisson's ratio for asphalt concrete layer
- Thermal properties

The thermal properties in the asphalt layer properties can be left as default values as they are not found to significantly affect performance prediction of the pavement (identified from literature review). Figure 11 shows the asphalt material properties input screen.

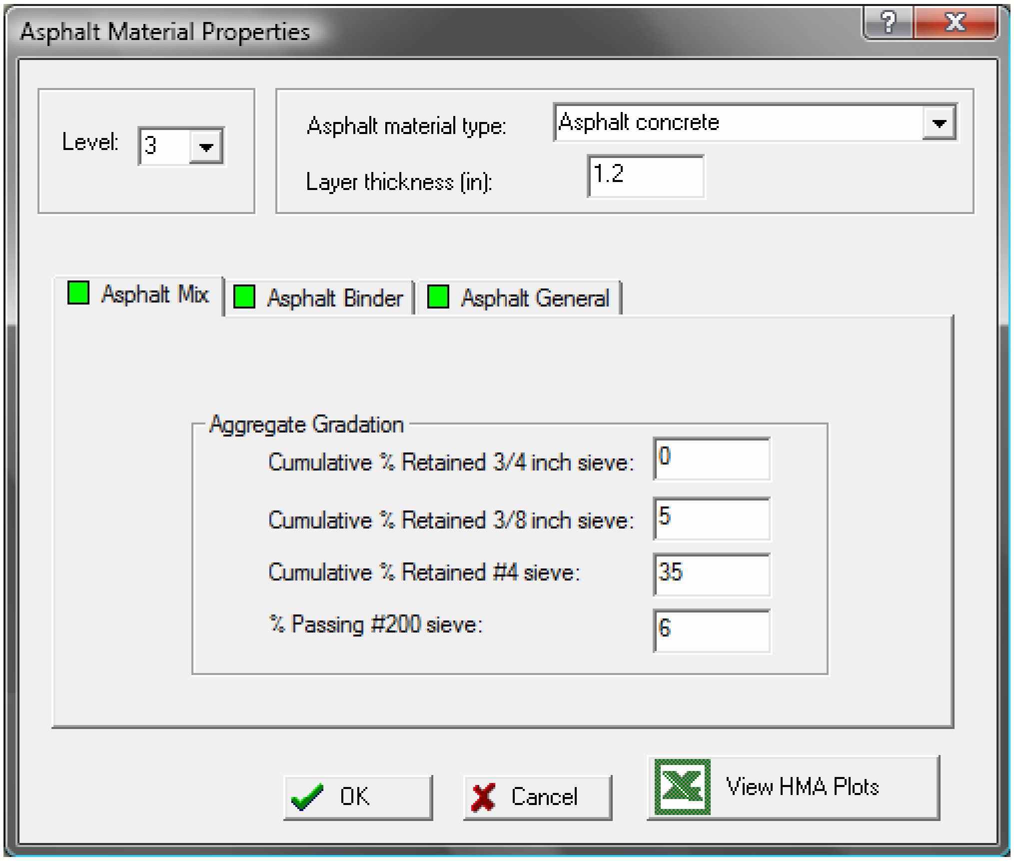

Figure 11 Asphalt Mix Properties Input Screen

Level 2

Asphalt Mix

- Level 2 also uses the mix aggregate gradation for computation of the modulus of the layer; hence the same inputs are required for Level 2 asphalt mix properties

Asphalt Binder Parameters

- Superpave PG grading system requires the G* and sin δ parameters for the asphalt binder, whereas conventional binder test data requires the parameters shown in Figure 12.

Number of penetrations: 3 Number of Brookfield viscosities: 3

Test	Temperature (°F)	Binder property
Softening point (P)	0	13000
Absolute viscosity (P)	140	0
Kinematic viscosity (CS)	275	0
Specific gravity	77	0
Penetration		
Brookfield viscosity		

Figure 12 Conventional Binder Grading – Level 2 Properties

Asphalt General Properties

- The input parameters for asphalt general properties are the same for Levels 2 & 3

Unbound Layers – Base Course / Subgrade

The base course and subgrade input parameters that are required for Levels 2 and 3 of design are approximately the same. MEPDG contains a table of average resilient modulus values for all types of base course and subgrade materials obtained from national averages, which can be used for Level 3 design (Figure 13). Level 2 requires a resilient modulus value from laboratory test data or state-specific values from a modulus database. The various input parameters required for unbound layers are:

- Layer type – Layer type can be identified from the list of different types provided in the MEPDG. Subgrade soils can be identified according to AASHTO or USCS classifications systems.
- Layer thickness
- Poisson's ratio
- Coefficient of lateral pressure can be assigned a default value of 0.50
- Material Property – This can be entered either as the resilient modulus (in psi), CBR value, R-value, layer coefficient, dynamic cone penetration value or can be calculated from the plasticity index and gradation entered on the ICM screen

The subgrade must be indicated as the last layer. This study does not include pavement structures on bedrock.

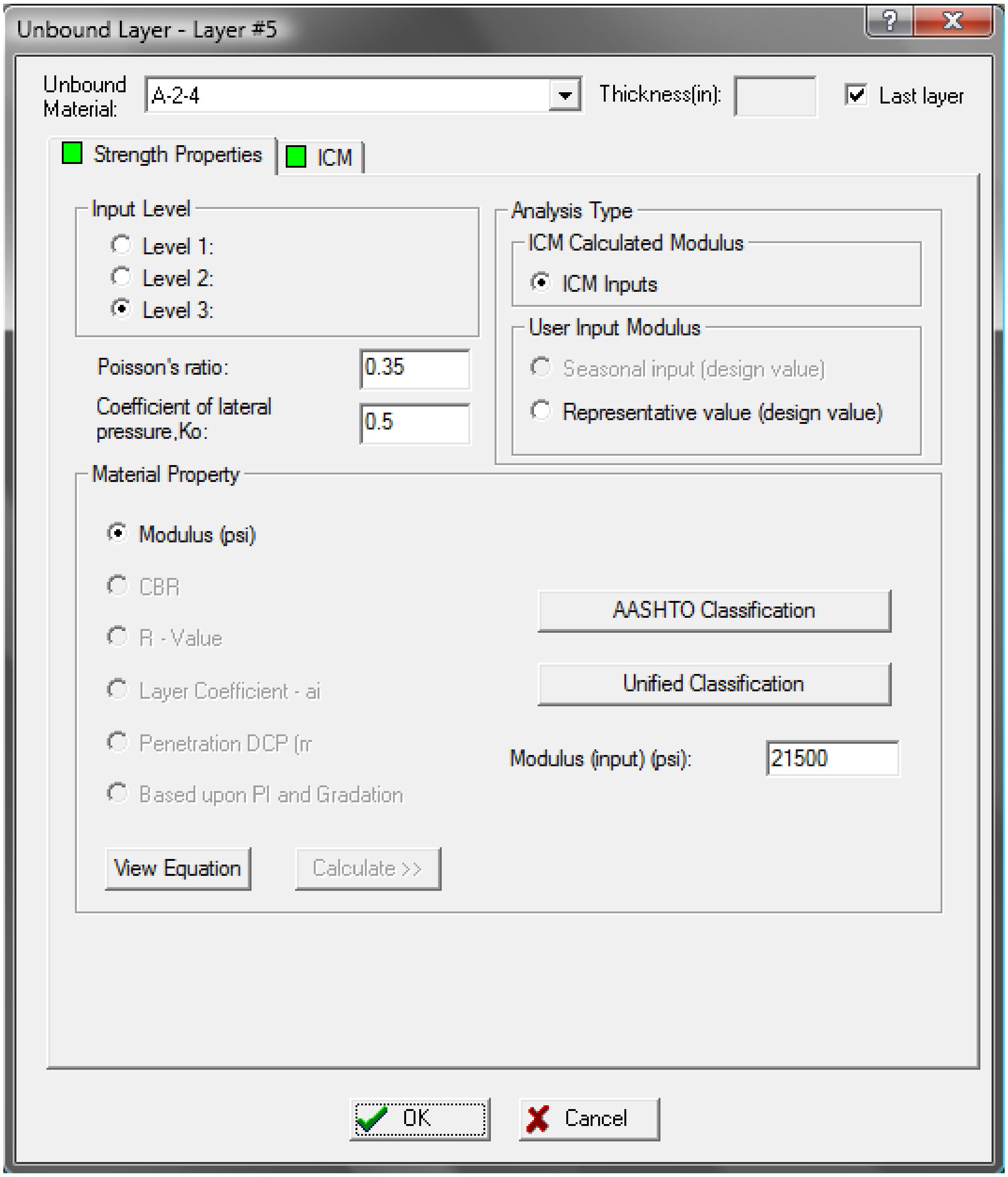

Figure 13 Unbound Layer Inputs

2.2 MEPDG Output Parameters – Performance Criteria for Flexible Pavements

The ME Pavement Design Guide utilizes transfer functions built into the form of software and the input data provided as specified above to provide performance criteria as the output to the user. The output is obtained in the form of an EXCEL file and contains predicted monthly values of the following pavement performance criteria:

- Bottom-Up (Fatigue) Cracking – measured in % area of the lane
- Top-Down (Longitudinal) Cracking – measured in feet per mile length of lane
- Rutting in asphalt concrete layer – measured in inches
- Total rutting of the pavement – measured in inches
- Thermal crack length – measured in feet per mile length of lane
- International Roughness Index – measured in inches per mile length

The predicted values of concern are the distress values at the end of the design life, as well as the time required by the pavement after it becomes functional to reach the failure limit in a particular type of distress.

This section contains an explanation of the various types of pavement distresses, their causes and factors that affect the distress *(*8*)*.

2.2.1 Bottom-Up or Fatigue Cracking

This type of fatigue cracking first shows up as short longitudinal cracks in the wheel path that quickly spread and become interconnected to form a chicken wire/alligator cracking pattern. These cracks initiate at the bottom of the HMA layer and propagate to the surface under repeated load applications.

This type of fatigue cracking is a result of the repeated bending of the HMA layer under traffic. Basically, the pavement and HMA layer deflects under wheel loads that results in tensile strains and stresses at the bottom of the layer. With continued bending, the tensile stresses and strains cause cracks to initiate at the bottom of the layer and then propagate to the surface. The following briefly lists some of the reasons for higher tensile strains and stresses to occur at the bottom of the HMA layer:

- Relatively thin or weak HMA layers for the magnitude and repetitions of the wheel loads.
- Higher wheel loads and higher tire pressures.
- Soft spots or areas in unbound aggregate base materials or in the subgrade soil.
- Weak aggregate base/subbase layers caused by inadequate compaction or increases in moisture contents and/or extremely high ground water table (GWT).

2.2.2 Top-Down Fatigue Cracking or Longitudinal Cracking

Most fatigue cracks initiate at the bottom of the HMA layer and propagate upward to the surface of the pavement. However, there is increasing evidence that suggests load-related cracks do initiate at the surface and propagate downward. There are various opinions on the mechanisms that cause these types of cracks, but there are no conclusive data to suggest that one is more applicable than the other. Some of the suggested mechanisms are:

- Wheel load induced tensile stresses and strains that occur at the surface and cause cracks to initiate and propagate in tension. Aging of the HMA surface mixture accelerates this crack initiation-propagation process.
- Shearing of the HMA surface mixture caused from radial tires with high contact pressures near the edge of the tire. This leads to cracks to initiate and propagate both in shear and tension.
- Severe aging of the HMA mixture near the surface resulting in high stiffness and when combined with high contact pressures, adjacent to the tire loads, cause the cracks to initiate and propagate.

In the approach described in the design guide a preliminary surface-down cracking model has been incorporated that considers high tensile strains due to load-related effects and the effects of age-hardening of asphalt materials. This theoretical methodology has been calibrated to field longitudinal cracking data.

2.2.3 Permanent Deformation or Rutting

Rutting is a surface depression in the wheel paths caused by inelastic or plastic deformations in any or all of the pavement layers and subgrade. These plastic deformations are typically the result of:

- Densification or one-dimensional compression and consolidation and
- Lateral movements or plastic flow of materials (HMA, aggregate base, and subgrade soils) from wheel loads.

The more severe premature distortion and rutting failures are related to lateral flow and/or inadequate shear strength any pavement layer, rather than one-dimensional densification. Rutting is categorized into two types as defined below.

- *One-dimensional densification or vertical compression:* A rut depth caused by material densification is a depression near the center of the wheel path without an accompanying hump on either side of the depression. Densification of materials is generally caused by excessive air voids or inadequate compaction for any of the bound or unbound pavement layers. This allows the mat or underlying layers to compact when subjected to traffic loads. This type of rut depth usually results in a low to moderate severity level of rutting.

- *Lateral flow or plastic movement.* A rut depth caused by the lateral flow (downward and upward) of material is a depression near the center of the wheel path with shear upheavals on either side of the depression. This type of rut depth usually results in a moderate to high severity level of rutting. Lateral flow or the plastic movement of materials will occur in those mixtures with inadequate shear strength and/or large shear stress states due to the traffic loads on the specific pavement cross-section used. Over-densification of the HMA layer by heavy wheel loads can also result in bleeding or flushing in the pavement surface. This type of rutting is the most difficult to predict and measure in the laboratory.

2.2.4 Thermal Cracking

Cracking in flexible pavements due to cold temperatures or temperature cycling is commonly referred to as thermal cracks. Thermal cracks typically appear as transverse cracks on the pavement surface roughly perpendicular to the pavement centerline. These cracks can be caused by shrinkage of the HMA surface due to low temperatures, hardening of the asphalt, and/or daily temperature cycles. Thermal crack initiate at the pavement surface and propogate down.

Cracks that result from the coldest in temperature are referred to as low temperature cracking. Cracking that result from thermal cycling is generally referred to as thermal fatigue cracking. Low temperature cracking is associated with regions of extreme cold whereas thermal fatigue cracking is associated with regions that experience large extremes in daily and seasonal temperatures.

There are two types of non-load related thermal cracks: transverse cracking and block cracking. Transverse cracks usually occur first and are followed by the occurrence of block cracking as the asphalt ages and becomes more brittle with time. Transverse cracking is the type that is predicted by models in this design guide, while block cracking is handled by material and construction variables.

2.2.5 Roughness

The IRI over the design period depends upon the initial as-constructed profile of the pavement from which the initial IRI is computed and upon the subsequent development of distresses over time. These distresses include rutting, bottom-up/top-down fatigue cracking, and thermal cracking for flexible pavements. The IRI model uses the distresses predicted using the models included in this Guide, initial IRI, and site factors to predict smoothness over time. The site factors include subgrade and climatic factors to account for the roughness caused by shrinking or swelling soils and frost heave conditions. IRI is estimated incrementally over the entire design period.

2.3 Performance Prediction Curves

The MEPDG output file contains performance prediction curves with the predicted distress plotted on the Y-axis (dependent variable) versus time on the X-axis (independent variable). The analysis of the performance prediction for a particular design run can be made using the plots generated by the MEPDG software. However, for a sensitivity analysis using graphical methods, these plots do not provide much information. Hence, the predicted monthly distress values are tabulated in spreadsheet software (MS EXCEL) and separate performance prediction trends were generated for the purpose of conducting a sensitivity analysis.

The user-defined performance prediction curves were based on plots prepared for earlier research studies that were studied during review and consists of the same variables on the axes. The predicted distress values at the end of each year were used to generate the plots and a graphical analysis is made using the behavior of trends. A sample plot is shown in Figure 14 and the information contained in the plot is explained.

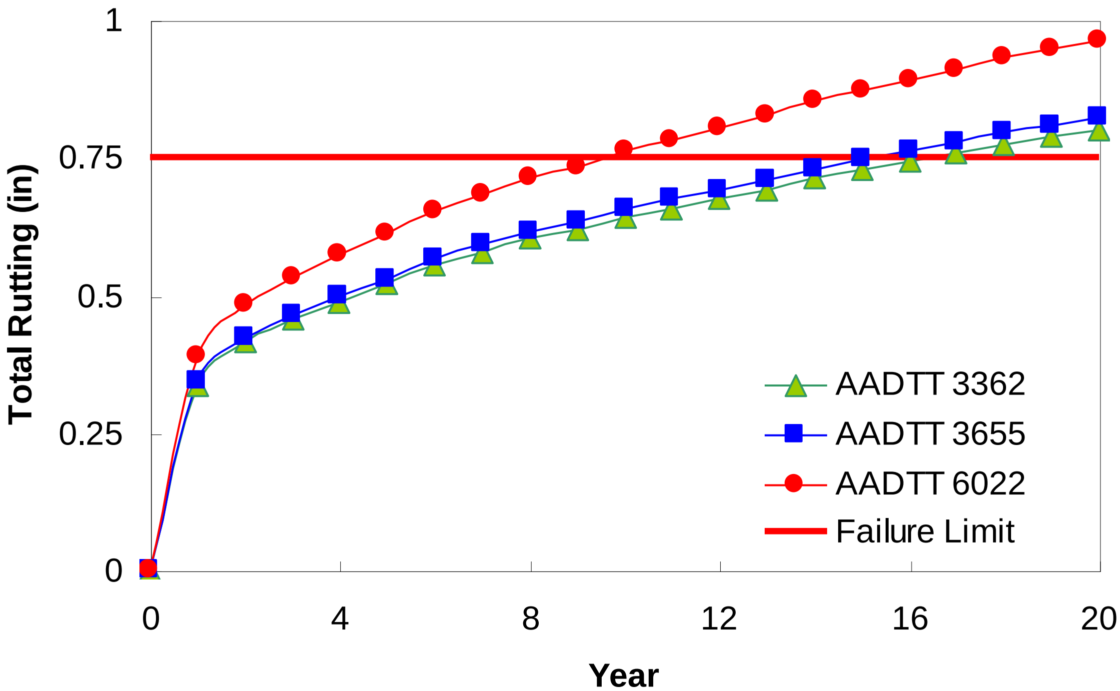

Figure 14 Performance Prediction Curve – Variation of Total Rutting with AADTT

The plot above shows the variation of total rutting of the pavement for different truck traffic volumes. The points on the curve represent the value of the predicted total rutting at the end of each year, and the red horizontal line represents the failure limit. The pavement is considered to reach the failure criterion for total rutting if the performance curve crosses the failure limit line in the positive Y- direction. The year corresponding to the point on the performance prediction curve just before it crosses the failure limit line is the number of years (n) that the pavement performs satisfactorily before failing in that type of distress. Such a pavement is considered to be failed in the $(n+1)^{th}$ year. This year is reported as the failure year of the pavement in a particular type of distress, and this data is also reported along with the predicted distress values. If the performance curve does not cross the failure limit line, or if the failure limit line does not appear on the graph, then the predicted distress value at the end of design life is lower than the failure limit, and the design can be considered successful in terms of performance.

3. RESEARCH OBJECTIVE AND APPROACH

Objective

The main goal of this research was to offer the New England and New York state highway agencies guidelines for the implementation of the MEPDG, with focus on flexible pavements and AC overlays. The research team in this report addressed some of the issues and concerns that arise in the transition from current AASHTO empirical design methodologies, such as those in the 1972, 1986 and 1993 guides, to the new mechanistic-empirical design methodologies incorporated in the MEPDG. Within the scope of this project, the proposal team answered some questions that highway agencies have or will encounter with regard to the MEPDG implementation, as shown in Figure 15.

Specifically, the objectives of this research project were as follows:

- Determine the design and data collection methods, material tests, and testing equipment currently in use by each state.
- Identify the Level 2 and Level 3 design guide inputs for which regional or local values are required.
- Provide state specific recommendations on implementation of the MEPDG including changes in data collection & measurement, equipment needs, training, and anticipated benefits.
- Provide specific recommendations for regional and local calibration of the MEPDG by identifying appropriate field test & monitoring sites, data to be collected, and perform local calibrations if appropriate field data is available.

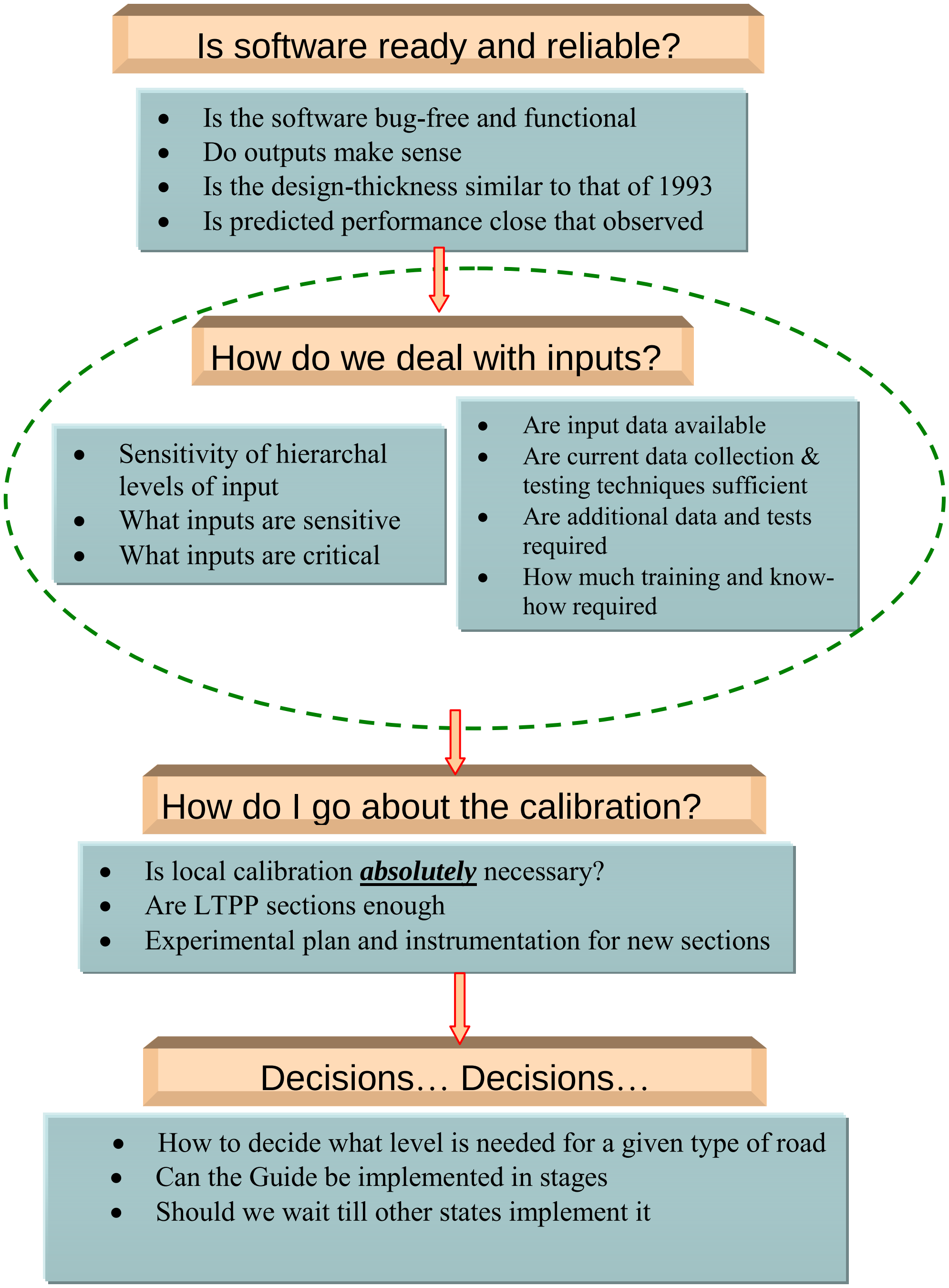

Figure 15 Possible Concerns Regarding the MEPDG Implementation

Approach

To obtain the project goals, valuable information can be obtained from on-going research reports and experiences of select states in implementing the MEPDG. A literature review was conducted to obtain information on research and practical work conducted on evaluating the functionality and suitability of adopting the MEPDG in other states. In that regard, knowledge of input variable selection methodology for evaluation and implementation studies and approaches to sensitivity study is essential for formulation of recommendations well-founded on engineering and research experience.

The status of implementation in other states was used to evaluate the current standing of the New England states and New York in terms of efforts currently expended and required in the future for successfully implementing the MEPDG. The results of this effort can also be used for reference to successfully completed research activities in the states reviewed.

Currently, states have their own design practices in use, their own types of equipment to measure material properties, and their own default values that are used in pavement design. Therefore, there will likely be varying levels of effort needed to implement the new MEPDG. The objective was to identify and document the current design practices of the states involved in this study.

Several people in various departments/ bureaus/ sections in each state were contacted. The first step was to identify the most likely points of initial contact within each state to obtain the required information. These individuals were identified with help from the technical committee, personal contacts, and research on state websites.

Surveys were developed and sent out to the various state personnel to obtain the required information on current design practices, equipment, etc.

Sensitivity Analysis with MEPDG versions 0.91, 1.0 and 1.1

The objective of the sensitivity analysis was to identify which inputs required for Level 2 and Level 3 analysis will require state specific data, and for which variables regional default values will suffice. This was achieved by conducting a sensitivity analysis in which critical inputs were varied for a range of values typical of the New England states for both Level 2 and Level 3 analysis, or values extracted from LTPP sites. Different sets of input variables which have been found to affect the different pavement distresses were identified from literature review and elaborated on in the following sections.

A first step in conducting the sensitivity study was to create an input file which served as a reference file, the output of which served as a benchmark or baseline for the sensitivity analysis. Data and values for the reference input file were extracted from a database for an existing pavement structure with construction data, material properties, and monitored performance. A reference input (control) file for each state is designed based on an LTPP flexible pavement section (GPS-1: Asphalt Concrete on Granular Base and GPS-7: Asphalt Concrete Overlay on PCC Pavement). The control file is constructed from input values based on design and testing values for the LTPP section, in addition to pavement design methodologies and specifications currently used by the states.

The sensitivity of a certain input is assessed by changing its value over its typical range while holding the values of all other inputs constant. The resulting change in predicted distresses serves as indicator of the sensitivity of the various distresses to that input. This process was repeated for all other critical inputs for the design, including material properties, structural design parameters, as well as climatic and traffic design inputs existent at the time of construction and during the service life of the pavement.

4. LITERATURE REVIEW

Reports from various studies conducted on the evaluation of MEPDG were reviewed to obtain information helpful to conduct the research. The following section presents an extensive literature review from the reports studied, and significant findings from the studies have been applied to optimize research efforts and devise a research strategy to approach the project objectives in a systematic manner.

4.1 Findings from completed research activities on MEPDG Implementation

Reports of completed research activities related to verification and implementation of MEPDG in the lead states were studied. Statewide traffic volume adjustment factors for MEPDG such as truck class distribution, monthly and hourly distribution factors were developed for the state of Arkansas by (9). 23 out of the 55 weigh-in-motion (WIM) sites which provided data suitable for the study were used to develop traffic adjustment factors. The study concluded that the state-specific truck class distribution factors have a significant effect on predicted pavement performance compared to the default values. The effect of monthly and hourly distribution factors was found to be insignificant. Therefore, state-specific class distribution and default monthly and hourly factors are recommended for use as traffic inputs for MEPDG. It is also recommended to update the truck class distribution factors periodically.

An implementation plan for the MEPDG in Indiana was developed with emphasis on flexible and rigid pavement design (10). Implementation of the design guide is accelerated in agencies which have integrated pavement design, materials and research departments in their organizational structure. The implementation plan followed is as follows:

1. Review of existing pavement design and management procedures
2. Review and documentation of design input parameters for the three levels of design – the objective is to document input parameters to which pavement distresses and smoothness is sensitive
3. Review and documentation of data from pavement design department and LTPP database which can be used as inputs in the MEPDG
4. Review of laboratory and field equipment for data collection and testing for higher level design inputs, and acquire additional equipment for obtaining sensitive design inputs
5. Strategic plan for establishment of mini-LTPP sites designed using MEPDG for local calibration and validation of distress models. Environmental and climate database must also be expanded with establishment of additional weather stations to avoid inappropriate interpolation for stations with large temperature gradients with respect to latitude
6. Dissemination of knowledge and necessary training on M-E design guide to all pavement design divisions, districts, local agencies, contractors and consultants

Initiatives were taken to integrate traffic data from WIM and AVC with GIS and GPS technologies, and analyze this data to generate axle load spectra. Flexible pavement design implementation was initiated by analyzing the effects of various HMA input parameters on pavement distresses, holding the other parameters constant. This activity was performed to determine further efforts needed to evaluate HMA inputs in the local calibration and implementation plan.

For unbound materials, the most important input parameter in the design guide is the resilient modulus M_R. Resilient modulus is determined from repeated triaxial tests, and is a required input for AASHTO design guide as well. Therefore, the objectives of unbound materials implementation plan was to generate a database of M_R values for Indiana subgrades, simplify resilient modulus testing procedure and develop a model to calculate M_R from measurable soil properties.

A research study for the state of Iowa (11) consisted of sensitivity analysis for flexible pavement systems performed by using typical values suggested by the M-E design guide software. Field data pertaining to two pavement systems from I-20 in Buchanan County and I-80 in Cedar County was used to study relative sensitivity of pavement distresses to AC material inputs, traffic and climate. Importance was given to pavement construction activity dates due to the following two reasons:

1. Environmental module should generate climate data in accordance with activity dates
2. Climate module should be correctly synchronized with the opening of traffic on the pavement, which affects the prediction of pavement distresses

Activity dates are difficult to predict much ahead of the actual construction schedule; therefore they should be approximately determined from typical construction histories.

The reliability input (default value in the guide is 90%) should be ignored during initial implementation of the MEPDG.

Montana DOT (12) recommends the use of the following values of input parameters for HMA mixtures for implementation of the M-E design guide:

i. Aggregate Gradation: Values near the mid-range of project or design specifications or average values from previous construction records for a particular mix
ii. Air Voids, effective asphalt content, mix density: Average values from previous construction records for a particular mix
iii. Poisson's ratio: Temperature-calculated values within the MEPDG, by checking the box to use the predictive model to calculate Poisson's ratio from pavement temperatures
iv. Dynamic modulus, creep compliance, indirect tensile strength: Level 3 or Level 2 inputs, which include aggregate mix gradation or G* and sinδ values from DSR
v. Surface shortwave absorptivity: Default value of 0.85 given in design guide
vi. Coefficient of thermal contraction of mix: Use default values as given in the guide for different mixtures and aggregates
vii. Reference temperature: 70^0 F
viii. Thermal conductivity and heat capacity of asphalt: Default values

Ongoing research activities in various states reflect the efforts being expended towards implementation of the M-E pavement design guide. Texas DOT is working on a project (14) to develop an integrated database that includes material properties, pavement structural characteristics, highway traffic information, environmental conditions and performance data such as distress values, etc. Data on these parameters have been already collected for other purposes, but needs to be integrated for validating and calibrating M-E flexible pavement design models at project level.

Arkansas State Highway and Transportation Department (AHTD) is involved in the development of a master plan for calibration and implementation of the M-E design guide (15). A similar research activity is being conducted by North Carolina DOT for flexible pavement design (16) by developing a database of typical layer materials – HMA and unbound materials. The scope of this research includes fatigue cracking and rutting. The aim of the research project is to develop local HMA performance model coefficients and thereby use the modified coefficients to improve the accuracy of the M-E PDG performance prediction models.

4.1.1 Background of Flexible Pavement Design

Existing AASHTO Methodology

Starting in the 1920s the State Highway Agencies and the Bureau of Public Roads started a series of road tests to determine the relationship between axle loading and pavement structure on pavement performance (17). This knowledge was needed to assist in the

design of pavements to establish maximum load limits, and to provide a basis for the allocation of highway user taxation. The AASHO Road Test (1958-1960) was the last of the series. It was conducted with limited structural sections at one location in Ottawa, Illinois. The test studied the performance of known thickness pavement structures under moving loads of known magnitude and frequency. These tests were conducted for both pavement types: asphaltic concrete and portland cement concrete. The test facilities had six loops of 7 mile two-lane pavements (Figure 16), which contained 836 test sections with a wide range of surface, base and subbase thicknesses. Test traffic was inaugurated on October 15, 1958 and ended November 30, 1960. Five of the loops were exposed to traffic loading shown in Figure 17, and one was used to test environmental effects.
The test data established the relationships for pavement structural designs based on expected loadings over the life of a pavement.

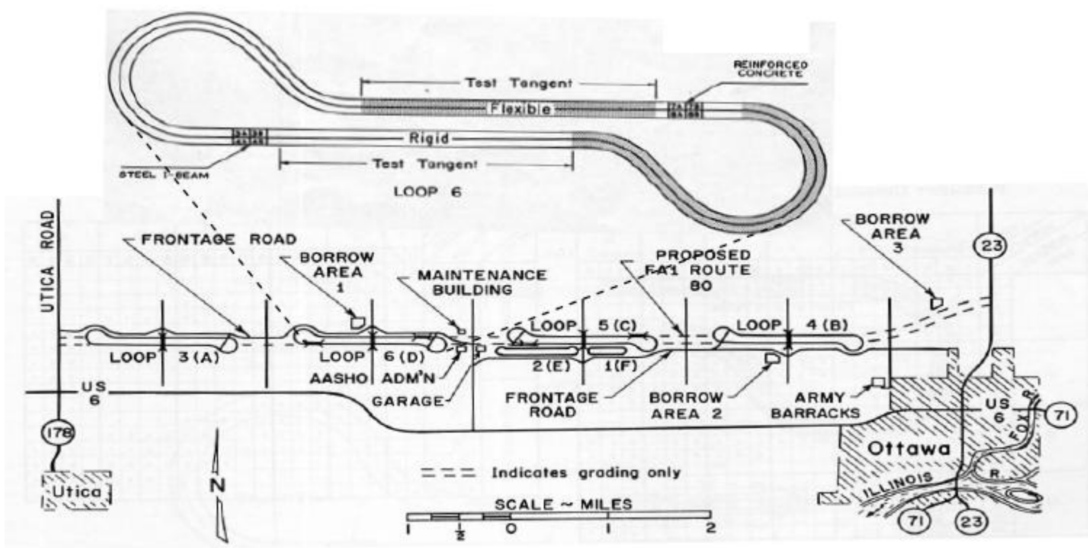

Figure 16 AASHO Road Test Layout (18)

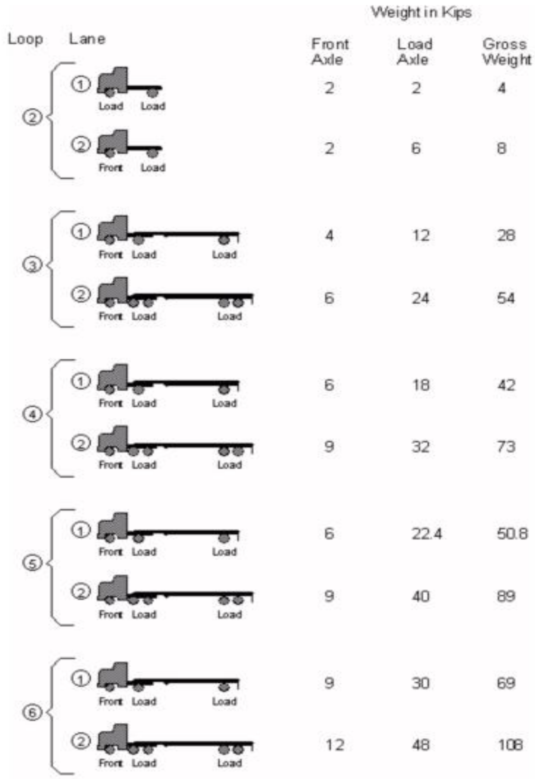

Figure 17 Axle Weights and Distributions Used on Various Loops of the AASHO Road Test (19)

Following completion of the Road Test, in May 1962 the AASHO Design Committee reported the development of the AASHO Interim Design Guides (1^{st} – Flexible, and 2^{nd} – Rigid Pavement Structures). All the pavement design procedures within these Interim Design Guides were based on the results from the AASHO Road Test and were supported by existing design procedures and available theory. Although the AASHO Road Test represented the most comprehensive development of the relationship between traffic loadings, material characteristics, structural thicknesses and performance, the results were limited by the scope of the test and conditions under which it was conducted. The performance equations from the AASHO Road Test were developed based on (17):

- Specific set of paving materials
- One subgrade material type
- A single environment
- An accelerated procedure for accumulating traffic
- Accumulation of traffic on each test section by operating vehicles with identical loads and axle configuration, rather than by mixed traffic.

To develop a new design procedure for a different location it was necessary to make certain assumptions, which adjusted the different traffic conditions, specific climate and material types. The assumptions and limitations associated with each design procedure were enumerated in the guides, and each emphasized that:

"The Guide is interim in nature and it is subject to adjustment based on experience and additional research" (17).

The 1962 Interim Guide was first revised in 1972 (17). The design methods and procedures contained in 1962 version of the guide were not changed in the 1972 revision, but both the flexible and rigid design guides were incorporated into one document.
A more significant revision to the Interim Guide was made in 1986, however the procedures were still based on the performance equations developed in the 1960s (20). At this revision several important items were considered:

- Resilient modulus for roadbed soils was recommended for characterizing soil support
- Design reliability for adding safety to the pavement structure
- The resilient modulus test (AASHTO Test T-247) was recommended for determining layer coefficient in flexible pavement design
- Subsurface drainage
- Environmental factors such as frost heave, thaw weakening and swelling soils
- Rehabilitation of pavements
- Discussion on the mechanistic-empirical design.

The 1986 Guide for Design of Pavement Structures was, for the first time, not labeled as interim. The most recent revision of the Guide for Design of Pavement Structures, which guide included the consideration of the flexible pavements was introduced in 1993 (21). The main differences between 1986 and 1993 Design Guide are: 1) refined material characterization; 2) more topics on rehabilitation of pavements; 3) more consistency between flexible and rigid design; 4) modifications to the overlay design procedure.

MEPDG Methodology

In December 1996, the National Cooperative Program (NCHRP) started Project 01-37A: "Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures," which was the initial step for developing a new pavement design process. The design procedure developed under this project was a large leap forward from existing practice. Project 1-37A was completed in 2004 and has entered the implementation process. As of December, 2010 forty states in the US (22) are planning to adopt this design procedure (a few states are already using it), now known as the Mechanistic-Empirical Pavement Design Guide (MEPDG).

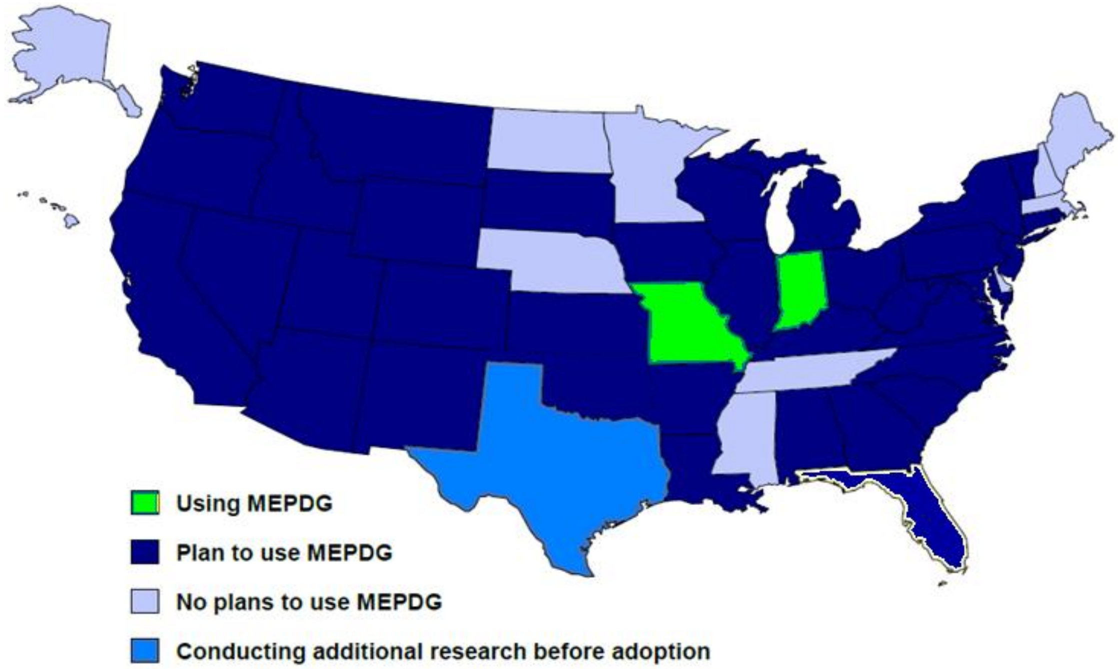

Figure 18 MEPDG Implementation Status as of December, 2010 (22)

AASHTO (Empirical) vs. AASHTO (M-E)

Table 1 shows some major differences between the early empirical AASHTO pavement design guides (e.g., 1972, 1986, and 1993) and the newer mechanistic-empirical design: AASHTO (M-E).

Table 1 AASHTO (Empirical) versus AASHTO (M-E)*

AASHTO (Empirical)	AASHTO (M-E)
Predicts AC thickness	Predicts pavement performance
Northern Illinois (wet-freeze climate) based	Uses more than 800 weather stations
One subgrade type (A-6 silty sand)	Project specific subgrade type
Uses equivalent single axle load (ESAL)	Individual Axle type and actual loading per axle
Uses Structural Number (SN) for flexible pavements	HMA specific characteristics
AASHO Road Test database	LTPP and NCDC databases

* - The comparison is based on the original guide in the 60’s, but later designs allow for change in subgrade, climate zone, etc.

4.1.2 MEPDG Implementation – Indiana Study

A study on HMA overlays over fractured slabs (10) has been conducted for the Indiana Department of Transportation to determine the extent to which various design inputs need to be incorporated and further evaluated in the local calibration of distress prediction models for Indiana State. Trial runs were performed using data obtained from a test section on I-65 in Rensselaer and the predicted response values were analyzed with respect to the inputs to determine the sensitivity of response to these variables. The input levels were not predetermined but their incorporation into the design procedure at different levels was based on the availability of data. A base input data or control file was prepared based on the values used for the original design of the test section using the 1993 AASHTO design procedure. The design methodology was examined thoroughly to devise a method of generating and analyzing performance prediction data.

Climate, traffic, materials and structural inputs were varied one at a time and the predicted responses were compared to that obtained from the base run to measure sensitivity of output to these inputs, which was classified as very high, high, medium or low. The following are the inferences drawn from the study:

- None of the input variables had a significant effect on the roughness index of the pavement (i.e. low sensitivity of all input parameters on the IRI prediction model). Therefore, intensive data collection techniques are not necessary for designing pavements for which ride quality is the major cause of concern.
- The state of Indiana was divided into three different climatic regions and the effect of climate on various types of distresses was studied. The climate data for three places namely Rensselaer, Indianapolis and Evansville selected by interpolation of latitude and longitude of the site where the section is located. The effect of variation in climate on various parameters is given in
- Table 2. The sensitivity of various pavement distresses was similar for the three locations. The fields marked in yellow indicate the values used for the base input parameter (control) file.

Table 2 Effect of climate on predicted pavement response

	Sensitivity				
Variables	Longitudinal Cracking	Fatigue Cracking	Rutting	Thermal Cracking	IRI
Rensselaer	High	Low	Medium	Medium	Low
Indianapolis					
Evansville					

- Traffic data obtained from WIM measurement was used for level 2 of input. Traffic volume was varied between -30% and +30% of traffic measured from WIM whose values moderately affected longitudinal and fatigue cracking and rutting (Table 3).

Vehicle class distribution had minimal effect on pavement response. Hourly and monthly axle distribution only affected the fatigue distress in the pavement.

Table 3 Effect of Traffic on Predicted Pavement Response

		Sensitivity				
Variables		Longitudinal Cracking	Fatigue Cracking	Rutting	Thermal Cracking	IRI
Volume	Low (-30%) WIM (1993) High (+30%)	Medium	Medium	Medium		Low
Class Distribution	Default WIM (1993)	Low	Low	Low		Low
Axle Dist. (Hourly, Monthly)	Default	Low	High	Low		Low

Structural inputs were studied in three stages: layer structure inputs, layer material properties and thermal cracking inputs. The study concluded that the M-E Pavement Design Guide software results are in complete accordance with traditionally expected trends and validated the functionality of the software. Accurate knowledge of in-situ conditions and material properties are extremely important for correctly predicting rehabilitated pavement distresses. The sensitivity of predicted response to pavement structure and materials is given in Table 4.

Table 4 Sensitivity of Predicted Response to Pavement Structure and Materials

Variables		Sensitivity: Longitudinal Cracking	Fatigue Cracking	Thermal Cracking	Rutting	IRI
HMA Layer Thickness	1 – 4 – 8 1.5 – 3.5 – 8 2 – 4 – 7 2 – 4 – 8	Medium	Medium	Medium	Medium	Medium
Binder Type	AC 20 PG 64-22 PG 76-28	Medium	Low	High	Medium	Low
HMA Design Level	Level 3 AC 20 Level 2 G*, sin δ	Medium	Low	High	Low	Low
Air Voids (%)	4% 6% 7% 9% 10%	Very High	High	Medium	High	Low
Rubblized Modulus	100,000 PSI 200,000 PSI 300,000 PSI In-Situ FWD	High	High	Low	Low	Low
Unbound Layer Modulus	Typical: 35000 Level 3 Default In-Situ FWD	High	Low	Low	Low	Low
Subgrade Type	A – 2 – 4 A – 7 – 6 A – 1 – a	High	Medium	Low	Low	Low
EICM	YES	Medium	Low	Low	Low	Low

Thermal cracking of AC pavements and its sensitivity to the coefficient of thermal contraction (CTC or α) was studied (12). The study was conducted on a pavement section on Interstate I-65 North-bound near Rensselaer, Indiana which consists of a 13” HMA layer over a rubblized 10” concrete layer. The input values were those used for the construction of the section and thermal cracking inputs (creep compliance and indirect tensile strength) were obtained. Climate data was generated by using data from surrounding weather stations. Thermal cracking inputs were Level 1 inputs, providing highest degree of reliability.

Three AC mixtures were used for studying the sensitivity of (α) on thermal cracking. All design parameters were kept constant for all three mixtures except: CTC, binder grade, creep compliance and indirect tensile strength. The CTC values used were 1.0 E-05 (low), 1.5 E-05 (medium) and 2 E-05 (high).

The three selected mixes and the effect of CTC on thermal cracking for these mixes are listed in Table 5.

Table 5 Effect of CTC on Thermal Cracking

Mix Number	Strength	Ductility	Effect of CTC
1	Low	High	Insignificant (quick failure)
2	Medium	Medium	Increases with increase in CTC
3	High	Low	Insignificant (no failure)

Further study was done to analyze the effect of CTC on thermal cracking in mix 2 (whose predicted values showed sensitivity to CTC values). The study showed that for after each year after construction of the pavement, the thermal cracking showed an increasing trend (with values remaining constant during few periods) and the crack percentage was higher for a higher value of CTC.

The pavement distresses are sensitive to various design parameters (10). The properties to which the predicted performance of the pavement is sensitive are listed in Table 6.

Table 6 List of Most Critical MEPDG Material Input Parameters - Levels 2 & 3

Pavement Type	**Distress Type**	**Critical Input Variables**
New HMA	Longitudinal (top-down) cracking in the wheelpath	• HMA mix stiffness • Foundation support (base/subgrade resilient modulus • HMA thickness
	Transverse (thermal) cracking	• Binder type • HMA thickness • HMA strength • HMA creep compliance • Coefficient of thermal contraction
	Fatigue (bottom-up) cracking	• HMA thickness • HMA mix stiffness • Binder content • Percent air voids
	Rutting	• HMA gradation • HMA mix stiffness • HMA thickness • Base/subgrade resilient modulus

South Dakota state MEPDG implementation plan, developed by Applied Pavement Technology, focused on sensitivity analysis for AC roads for rural highways, apart from other designs (23). The input values were provided for the study by the South Dakota DOT based on 'standard design practices' followed in the state. A noteworthy finding from this report is an analysis of variance was conducted on the obtained pavement performance prediction data. Earlier studies conducted on MEPDG implementation did not reinforce the degree of sensitivity on a valid statistical basis.

In analysis of variance, the significance of a variable as a predictor is determined by its associated F-value. The study ranked the variables on the basis of the F-ratio, computed as

$$F - ratio = \frac{MSE_{MEPDG\ Input}}{MSE_{Total}}$$

$MSE_{MEPDG\ Input}$ is the mean square error of the predicted distress data with the individual MEPDG input being investigated. MSE_{Total} is the mean square error of the predicted distress data with all investigated MEPDG inputs.

Sensitivity studies conducted so far have not included interaction effect of input variables on the predicted distresses. One-factor-at-a-time studies are sufficient to study the effect of input variables only when there is no interaction between the independent variables.

4.1.3 Implementing the MEPDG for Cost Savings in Indiana

The implementation of the new pavement design methodology is a huge task for the state Departments of Transportation (DOT). Indiana DOT's experience is a good example of how to handle this difficult and time consuming task (24). Implementation of the MEPDG design process demands knowledge about pavement design inputs and pavement performance. This task was completed by interactions among the highway agency personnel who work in traffic, material, geotechnical areas and pavement structures to identify the proper parameters for the design. To ensure successful outcome of the analysis and design process, the team of engineers had sufficient knowledge in pavement engineering. The implementation process was coordinated with other agencies such as Federal Highway of Administration (FHWA), state pavement associations and contractor associations. FHWA must approve all projects supported by government funds and the contractor association members actually build the pavements.

The full MEPDG implementation in Indiana began on January 1, 2009, although initial implementation efforts started seven years earlier, in 2002. Indiana DOT coordinates all implementation activities with agency pavement design engineers, FHWA, pavement association and contractor associations. There were regular monthly meetings, where implementation issues were discussed and approved for the next steps in the process. Training sessions were initiated throughout the entire implementation process for all involved parties.

In 2009, Indiana DOT's engineers and consultants designed over 100 pavement sections using the MEPDG procedure. All the new MEPDG design pavement thicknesses were documented and compared to the thicknesses estimated according to the 1993 AASHTO design. They provided profit calculations based on the material, labor cost and time savings. Savings resulted from more efficient MEPDG design which also reduced thickness of the pavement; most pavements were reduced by 2 inches. Significant savings of material, labor cost and time were realized.

Summarizing Indiana DOT's experience, the implementation of the MEPDG results in more efficient pavement designs, that can be built at a lower cost as shown in Table 7 (24).

Table 7 Cost Savings Attributed to the MEPDG Implementation in Indiana

Road	AASHTO 1993 HMA Thickness	MEPDG HMA Thickness	Estimated Contract Saving	Actual Contract Saving
SR 14	15"	13.5"	$333,000	$155,440
US 231	15.5"	13"	$557,000	$673,796
SR 62	16"	13"	$403,000	$420,548

4.1.4 MEPDG Sensitivity Analysis Results for New HMA in Ohio

In Ohio, MEPDG research mainly focused on the characterization of paving materials utilized in that state. In this study (25), the basic HMA properties such as air voids %, effective binder content and total unit weight were obtained from job mix formulas (JMF) for level 3 design. A very limited amount of effort has been expended on traffic related studies under Ohio Department of Transportation's (ODOT) research program. ODOT typically collects three categories of traffic data: weight-in-motion (WIM), automatic vehicle classification (AVC) and traffic volume, however most of this information has not been analyzed for MEPDG purposes. The following observations were obtained from the research and from sensitivity analysis:

- Longitudinal cracking was mostly affected by thickness of the HMA layer alone, and was caused mostly by poor construction methods. The subgrade and base stiffness did not influence the longitudinal cracking.
- Transverse (thermal) cracking was highly affected by climate, volumetric binder content and base type. HMA thickness had a moderate influence with thicker asphalt pavements showing lower thermal cracking predictions.
- Alligator cracking was significantly affected by HMA thickness and asphalt binder content. Higher thicknesses and higher asphalt contents lead to lower predicted alligator cracking. Also the base type had a major impact on the alligator cracking. Percentage of heavy trucks (class 9 or greater), subgrade type and climate affected alligator cracking moderately.

- Total rutting (includes HMA layers, base and subgrade) as expected, was affected mostly by the percentage of heavy trucks. Other significant factors affecting total rutting were HMA thicknesses (the higher the pavement thickness, lower the rutting), binder content (the higher the content, higher the rutting), and base type (asphalt treated based showing lesser rutting). Moderate impacts on the predicted rutting were observed with the air voids content (higher air voids leading to increasing rutting), climate and subgrade type.
- Smoothness IRI (ride quality) was mostly affected by pavement thickness (thicker pavements exhibited lower IRI). Base and subgrade stiffnesses had a moderate effect on IRI (sections with stiffer layers having more beneficial IRI).

4.1.5 MEPDG Sensitivity Analysis Results in South Dakota

The pavement performance for the sensitivity analysis in South Dakota (26) was expressed using the following performance indicators:

- Top-down fatigue (longitudinal) cracking,
- Bottom-up fatigue (alligator) cracking,
- AC rutting,
- Total rutting,
- Smoothness (IRI).

The transverse cracking performance predictions were omitted due to the MEPDG version 1.1 software having specific shortcomings (transverse cracking values equal to “0”). Before conducting any runs for MEPDG sensitivity analysis the South Dakota DOT Technical Panel (26) needed to determine:

- Fixed variables and their levels,
- Determine which inputs needed to be investigated,
- Input value ranges were to represent typical South Dakota conditions.

The newly designed rural AC pavement was evaluated based on 56 MEPDG software simulations. The parameters in Table 8 are placed in decreasing order of their significance for each investigated performance indicator.

Table 8 Summary of Significance for New AC (Rural Design)

Pavement Type	Distress Type	Critical input Variables
New HMA	Top-down (longitudinal cracking)	• AC layer thickness • Initial 2-way AADTT • Base resilient modulus • AC binder grade
	Bottom-up fatigue (alligator cracking)	• Initial 2-way AADTT • AC binder grade • AC layer thickness • Base resilient modulus
	AC rutting	• Initial 2-way AADTT • AC layer thickness • AC binder grade • Location (climate)
New HMA Continued	Total rutting	• Initial 2-way AADTT • AC layer thickness • Subgrade resilient modulus • Depth of water table • AC binder grade • Base resilient modulus
	Smoothness (IRI)	• Bottom-up fatigue (alligator) cracking • Total permanent deformation (rutting)

In the overall ranking, it was observed that the initial 2-way AADTT variable had the largest performance affect on all of the pavement distress types for the new HMA design, followed by: AC layer thickness, AC binder grade, base resilient modulus, and subgrade resilient modulus.

The smoothness indicator (IRI) was predicted as a function of the initial (as-constructed) IRI and the predicted longitudinal cracking, alligator cracking and total rutting. Based on these correlations the bottom-up fatigue cracking has the largest affect on the pavement smoothness in South Dakota.

4.2 New England and New York State Specific Review

A review of the design specifications of the New England state agencies (Table 9) was conducted for a complete understanding of the current design practices followed by the states. This was done in order to develop a methodology to collect and generate data which was used as input to the design guide in this study.

Table 9 Online Resources for State Specifications for Pavement Design, Materials & Construction

State	Access Locations
Connecticut	www.conndot.ct.gov\specpro\provisions.aspx Section M.04 which are the material specifications for granular materials and HMA
Maine	http://www.maine.gov/mdot/contractor-consultant-information/ss_standard_specification_2002.php
Rhode Island	http://www.dot.state.ri.us/engineering/proj/bluebook/CD-Bluebook.pdf
New Hampshire	The NH DOT standard specifications can be found at http://www.nh.gov/dot/bureaus/highwaydesign/specifications/index.htm The NH DOT supplemental specifications can be found at http://www.nh.gov/dot/bureaus/highwaydesign/specifications/supplementals/index.htm Also see attached table for corrected values
Massachusetts	http://www.mhd.state.ma.us/default.asp?pgid=content/publicationmanuals&sid=about
Vermont	Vermont Agency of Transportation Flexible Pavement Design Procedures for use with the 1993 AASHTO Guide for Design of Pavement Structures; March 1, 2002 http://www.aot.state.vt.us/Planning/Documents/TrafResearch/Publications/pub.htm
New York	NYS DOT Comprehensive Pavement Design Manual (July, 2002) and Revision (January, 2009) https://www.nysdot.gov/divisions/engineering/design/dqab/cpdm https://www.nysdot.gov/divisions/engineering/design/dqab/cpdm/cpdm-revision-log

The New England states were contacted for information on the current design practices being used and the major pavement performance-related issues in the state. Table 10 and Table 11 show the results of the survey conducted in four of the New England states. The following questions were included in the survey:

- Who performs pavement designs: in-house, contractors, division/main offices?
- What is the current design methodology: AASHTO 1972, 1986, 1993, M-E?
- What information and data is used in current pavement design?
- What are the major distresses and issues of concern: skid resistance, smoothness?
- What are the failure criteria: % cracking, rut depth, IRI?
- Reliability level in design (error tolerance) – not considered in present design

- Classification of roads: low volume vs. high volume roads?
- Where are material properties measured (main lab, division labs, research labs)
- Are materials, design, and construction specifications detailed and appropriate for use in developing default input data?

The information in the specifications was mostly used to generate ranges within which the input variables were varied based on the tolerances for each state.

Table 10 Initial survey sent to technical committee (CT, ME, RI)

STATE	CONNECTICUT	MAINE	RHODE ISLAND
Pavement designs performed by	In-house engineers, main office	Mix design – handled by paving contractors Road design handled by in-house engineers	In-house engineers – Main office Consultants
Currently used methodology	AASHTO 1993	AASHTO 1993	AASHTO 1993
Does agency have in-house initiatives to implement the new MEPDG	Yes, but start date not scheduled	Limited Done E* field and lab testing, transfer functions for Maine HMA, modulus lab and field testing	Not at this time
Person to contact in regard to above	Dean Dickinson dean.dickinson@po.state.ct.us	Timothy Soucie Timothy.Soucie@maine.gov	Kathy Wilson-Hofman kwhofman@dot.state.ri.us
Have personnel attended workshops and training on MEPDG	CT-DOT personnel attended FHWA training in Rocky Hill, CT, Sep 18-20, 2006	Yes Maine DOT personnel have attended one or more online FHWA DGIT workshops [a]	Materials, road design and pavement management personnel – July 2004, April 2005 and September 2006
Existing or planned instrumented pavement sites for local calibration	No	Yes, Rte.6/15/16 WIMPI - moisture gages, thermocouples, pressure cells & strain gages in subbase	No
Major distresses and issues of concern	Fatigue (bottom-up) cracking Longitudinal cracking Transverse (thermal) cracking Smoothness	Rutting, Fatigue (bottom-up) cracking Top-down cracking Longitudinal cracking Transverse (thermal) cracking Ride Quality	Fatigue (bottom-up) cracking Top-down cracking Longitudinal cracking
Where are material properties measured	Main Lab	Division labs Research labs	Main Lab

Table 11 Initial survey sent to technical committee (NH, MA, VT)

STATE	NEW HAMPSHIRE	MASSACHUSETTS	VERMONT
Pavement designs performed by	In-house engineers – Main Office	In-house engineers – Main Office, Division Office, Consultants (other)	
Currently used methodology	AASHO 1972 Interim Design Guide	AASHTO 1972 Interim Guide AASHTO 1993 Design Guide	AASHTO 1993 Design Guide
Does agency have in-house initiatives to implement the MEPDG	MEPDG Version 1.0 used to evaluate comparative designs using AASHTO 1972 and 1993, and AI Perpetual pavement model for upcoming I-93 project – Level 2/3 inputs used. Project with UNH for instrumentation of I-93	Yes, informal initiatives	Yes
Person to contact in regard to above	Eric Thibodeau	Edmund Naras Kevin Fitzgerald	Nicholas Meltzer
Have personnel attended workshops and training on MEPDG	Eric Thibodeau – NCHRP Project Panel 01-40 Seminar April 10-11, 2007, Irvine, CA	Edmund Naras & Kevin Fitzgerald - Online webcast and kickoff meeting in CT	Yes
Existing or planned instrumented pavement sites for local calibration	Planned: Section of I-93 as part of Salem – Manchester widening project	Yes, WIM only. Interstate highways, WIM present various locations statewide	Yes
Major distresses and issues of concern	Fatigue (bottom-up) cracking Top-down cracking Transverse (thermal) cracking	Rutting, fatigue (bottom-up) cracking, Transverse (thermal) cracking	
Where are material properties measured	Main lab and contractor labs	Main lab and division labs	Main lab and division labs

5. RESEARCH METHODOLOGY

5.1 Experimental Plan

The experimental plan consisted of performing runs on the MEPDG software using state-specific input parameters. State-specific values were chosen for the study so that recommendations could be made accordingly to the state highway design agencies to modify their existing design practices and transition smoothly to the MEPDG.

Various input parameters that are identified to critically affect the pavement performance have been identified in Table 6. These parameters were translated into the corresponding variables that the MEPDG accepts as input and the values of these variables were obtained from various sources and databases. The experimental plan was developed by varying these translated MEPDG input variables between a high and low value against a pre-determined mean value, which is the property of a selected pavement section for a state.

Data Collection

The data collection methodology applied to the research was developed to incorporate state-specific values for input parameters and analyze the predicted performance. Design specifications for each state were studied and information pertaining to values of input parameters and their tolerances were obtained. The mean values to be used for the control MEPDG design input file and the range within which they were varied were derived from this information.

The information was collected for each state for the following inputs:

- Construction period
- Locations of weather stations
- HMA mix design specifications – pavement layer structure specifications and material property values and tolerances
- Unbound layer (base course and subgrade) material properties if available

Information from literature review and extracted data from LTPP general pavement section (GPS) sites (27) for each state were organized into input parameter selection documents which served as the primary reference for design input values used for the sensitivity study. Traffic data was obtained from the Department of Transportation (DOT) websites of the states for specific sections selected for the study and correlated to data from LTPP traffic monitoring for validation.

Figure 19 and Figure 20 show the pavement structures from LTPP sites used in the study.

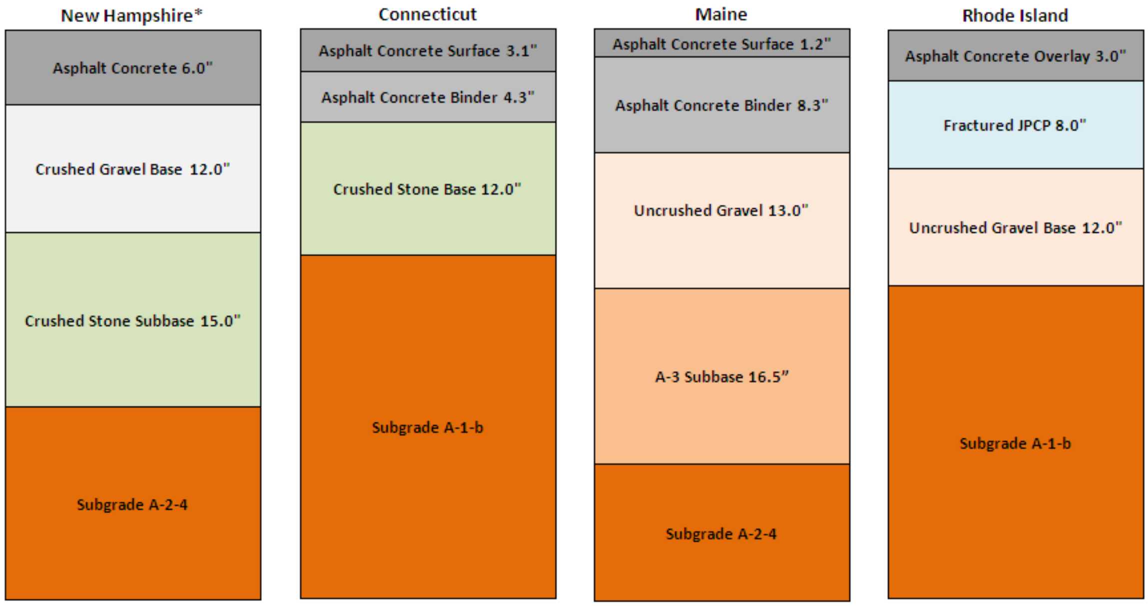

Figure 19 Control Pavement Structures for New England States NH, CT, ME, RI

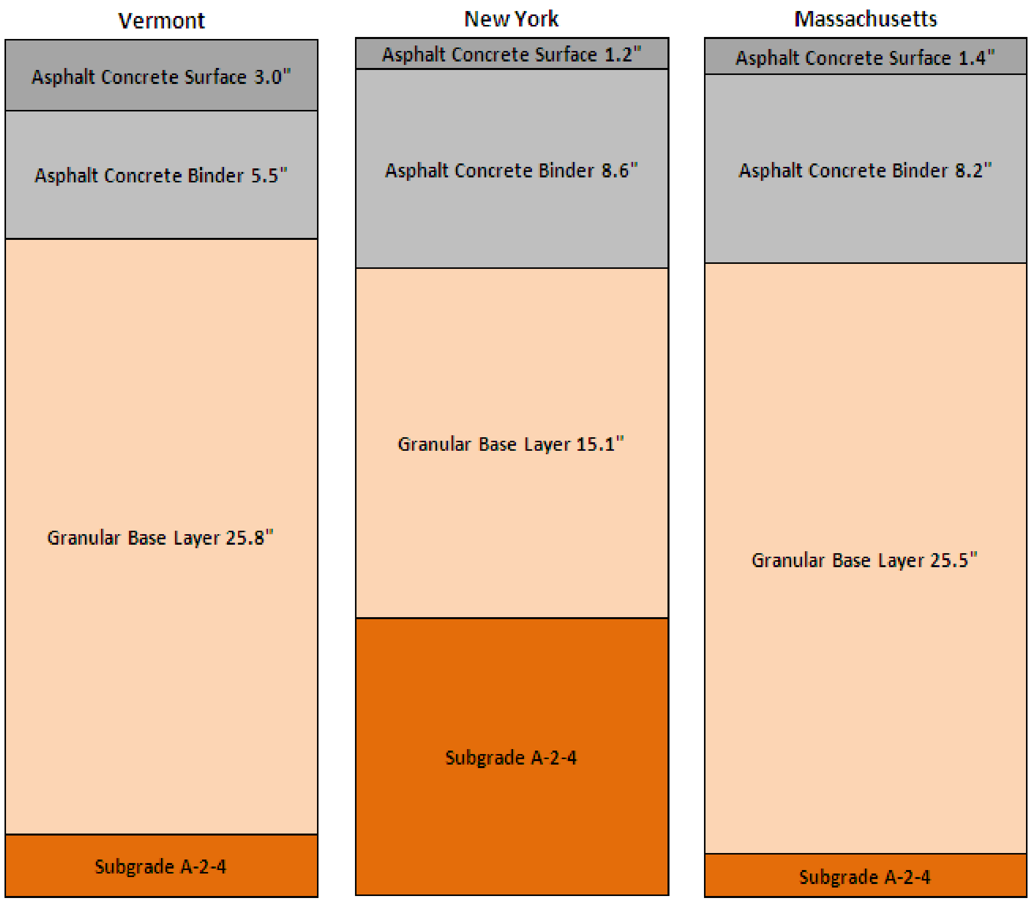

Figure 20 Control Pavement Structures for VT, NY and MA

5.2 Tolerances from State Design Specifications Documents

The tolerances for material properties used by each state agency and the allowable values of the parameters selected for the study are documented in this section.

5.2.1 HMA Gradation

HMA mix gradation for New Hampshire, Connecticut and Maine conform to Superpave specifications (Table 12).

Table 12 Range of values of HMA mix gradation – Superpave Specifications

NMAS of Mix	**4.75 mm**	**9.5 mm**	**12.5 mm**	**19.0 mm**	**25.0 mm**	**37.5 mm**
3/4" sieve	0	0	0 – 10	10 – NR		
3/8" sieve	0 – 5	0 – 10	10 – NR	NR		
# 4 sieve	0 – 10	10 – NR	NR	NR		
#200 sieve	6 – 12	2 – 10	2 – 10	2 – 8	1 – 7	0 – 6

* NR – No restriction on the value

The tolerances of percentage by weight of material retained on the sieves are given in Table 13.

Table 13 Tolerance for HMA mix gradation

NMAS of Mix	**9.5 mm**	**12.5 mm**	**19.0 mm**	**25.0 mm**	**37.5 mm**
Cum. % Ret 3/4"		+ 4%	+ 5%	+ 7%	
Cum. % Ret 3/8"		+ 4%	+ 5%	+ 7%	
Cum. % Ret # 4	+ 4%	+ 3%	+ 4%	+ 4%	+ 6%
#200 sieve	+ 0.8%	+ 0.8%	+ 0.8%	+ 0.8%	+ 0.8%

Rhode Island

HMA mix design for Rhode Island conforms to Marshall Method of mix design. The aggregate gradations for this study are selected to conform with the Superpave specifications. The DOT may also continue to use the currently followed mix design procedure to measure the mix aggregate gradation, subject to their acceptability of the results of analysis explained later in the report.

5.2.2 HMA Mix Stiffness

The dynamic modulus values for Levels 2 and 3 need not be entered by the user. Instead they are directly calculated by the software from the HMA mix component properties.

5.2.3 Subgrade / Base Resilient Modulus

The DOT specifications do not contain information on the acceptable range of values for resilient modulus (M_R) of subgrade/base course. The values have to be entered by the user for all design levels. Base and subgrade modulus values have been documented in the MEPDG for different types of material. They represent national averages of M_R for a given type of soil / base material. The provided values can be directly used for Level 3 design using MEPDG, or can be entered by the user from results of laboratory material testing for Level 2 design.

Appendix A contains the recommended subgrade modulus values for Level 3 design. MEPDG also provides the user flexibility in selecting the strength of the unbound material or subgrade based on other parameters such as CBR, R-value, layer coefficient, and dynamic cone penetration value or can be calculated from the plasticity index and gradation entered on the ICM screen.

5.2.4 HMA Thickness

New Hampshire

Each pavement layer should be of uniform thickness greater than 3/4” (19.0 mm). DOT does not specify a maximum pavement layer thickness. The allowable tolerance per each layer of HMA is + 1/4” from the design thickness.

Connecticut

DOT does not specify a minimum or maximum pavement layer thickness. The allowable tolerance per each layer of HMA is given in Table 14.

Table 14 Pavement Lift Thickness Tolerance

Class of Material	Tolerance
Class 4 and Superpave 25.0 & 37.5 mm	+/- 3/4 inch
Class 1, 2 and 12 and Superpave 4.75, 9.5, 12.5 & 19.0 mm	+/- 1/2 inch

Maine

Each pavement layer should be of uniform thickness greater than 3/4” (19.0 mm). DOT does not specify a maximum pavement layer thickness. The allowable tolerance per each layer of HMA is + 1/4” from the design thickness.

Rhode Island

The specifications do not mention maximum or minimum thickness for pavement layers. The allowable tolerance per each layer of HMA is + 1/4” from the design thickness. For an AC overlay, the minimum thickness is 1” and maximum thickness is 1.75” such that it can allow a maximum permanent deformation of 0.75”.

5.2.5 Binder Content

New Hampshire

Wearing course for ESAL designs of < 10 million shall have a minimum binder content of 5.8% utilizing the 50 gyration N_{design} mix. Wearing course for ESAL designs of > 10 million shall have a minimum binder content of 5.5% utilizing the 75 gyration N_{design} design mix.The tolerance limits on binder content are design binder content + 1% failing which the mix shall be rejected.

Connecticut

The tolerance limits on binder content are design binder content + 1% failing which the mix shall be rejected.

Maine

Maine DoT does not specify the limits on percentage of asphalt binder in an asphalt concrete mix. The tolerance on the final P_b is + 0.2%.

Rhode Island

Rhode Island DoT does not specify the limits on percentage of asphalt binder in an asphalt concrete mix. The tolerance on the final Pb is +0.2% to -0.3%.

5.2.6 Air Voids Percentage

New Hampshire

Air voids are measured by extracting 6" (150 mm) diameter cores from the pavement.
The tolerance on the lower limit of air void percentage is -2% (rejectable below 2%), provided the resultant air voids are greater than 3%. The tolerance on the upper limit of air void percentage is +2 % provided the resultant air voids are less than 9 %. Effectively, the in-place air voids can be varied between 3 % and 9 %. Materials and Research (M&R) requires 4 pre-blended aggregate specimens for gyratory and 2 pre-blended aggregate specimens to perform AASHTO T-209. The mix is rejected if the air voids fall outside the range of 3.0 – 5.5 %, or any other volumetric criteria is not met.

Connecticut

Air voids at $N_{desisgn}$ must be equal to 4%, with a tolerance limit of +1%. The specifications do not mention the maximum and minimum air void contents permissible in an AC layer.

Maine

The air voids percentage limits are not specified by the DoT. The tolerance on the percentage air voids in the asphalt mix are ± 0.9%.

Rhode Island

The air voids percentage range for base/binder AC courses is 3 – 8 %, for surface course is 3 – 5 % and for dense friction course is 8% minimum and ramp friction course is 5% minimum.

5.2.7 Binder Type

New Hampshire

The binder type for a particular project is to be specified by the contractor. LTPP Bind data available from New Hampshire stations is summarized in Table 15 for 50 % and 98 % reliability designs. For RAP mixtures, the added asphalt cement (virgin binder grade) may be PG 58-28, PG 64-28, or other asphalt cement grades as designated by the Bureau of Materials and Research. The percentage of Rap used shall not produce a total reusable binder greater than 1% of the total mix unless the composite binder meets the specified grade, rap stockpiles are covered, only produced in a drum plant and will only be allowed in base and binder courses.

Maximum RAP percentage for drum mixer is 30 %.

Maximum RAP percentage for batch plant mixes is 20 %.

Table 15 Binder Grade designation from LTPP Bind for New Hampshire

50 % Reliability		**98 % Reliability**	
Fast, < 3M ESALs	Slow, >30M ESALs	Fast, < 3M ESALs	Slow, >30M ESALs
PG 40 – 28	PG 58 – 28	PG 40 – 34	PG 58 – 34
PG 46 – 28	PG 64 – 28	PG 46 – 34	PG 64 – 34
PG 46 – 22	PG 64 – 22	PG 46 – 28	PG 64 – 28
PG 52 – 22	PG 70 – 22	PG 52 – 28	PG 70 – 28
PG 52 – 28	PG 70 – 28	PG 52 – 34	PG 70 – 34
PG 58 – 22	PG 76 – 22	PG 58 – 28	PG 76 – 28

Connecticut

For RAP mixtures, the binder grade for virgin binder to be used is not specified.

Maximum RAP percentage for drum mixer and batch plant mixes – 10 %

Table 16 Binder Grade designation from LTPP Bind for Connecticut

50 % Reliability		**98 % Reliability**	
Fast, < 3M ESALs	Slow, >30M ESALs	Fast, < 3M ESALs	Slow, >30M ESALs
PG 52-16	PG 70-16	PG 58-22	PG 76-22
PG 52-22	PG 70-22	PG 58-28	PG 76-28
PG 52-28	PG 70-28	PG 52-34	PG 70-34
PG 58-16	PG 70-16	PG 58-22	PG 76-22
PG 58-22	PG 70-22	PG 58-28	PG 76-28
PG 58-22	PG 70-28	PG 58-34	PG 70-34

The binder grade selection can be varied by selecting from among the above binders designated for Level 3 design. For level 2, the G* and sin δ values have to be entered for the selected binder grade.

Maine
The performance grade of asphalt binder to be used for hot mix construction must be PG 64-28, except for mixtures containing greater than 15% and less than 25% RAP where PG 58-34 should be used. A maximum of 15% RAP can be used in any course (wearing, binder or shim course) and up to 25% RAP is allowed in binder course, provided PG 58-34 binder is used.

For level 2, the G* and sin δ values have to be entered for the selected binder grade.
The DOT may approve one mix design for a particular nominal maximum aggregate size of the mix, and one 9.5 mm mix @ 50 gyrations for shimming.

Rhode Island
The performance grade of asphalt binder to be used for hot mix construction must be PG 64-28 for all non-recycled layers including friction courses. For RAP courses, the binder grade should be selected such that the effective binder grade is PG 64-28, and the selection is made by the contractor. Binder grades mentioned in the specifications are PG 64-28, PG 58-28, PG 58-34 and PG 52-34.

For level 2, the G* and sin δ values have to be entered for the selected binder grade.

5.3 Input Value Selection for New Hampshire for MEPDG Runs

The variables on which various types of pavement distresses depend were identified from literature review. The default values for these variables are used in preparing the control file for Level 3 and Level 2 analysis. Tolerances found from the state specifications for construction of flexible pavements are used to vary these parameters within the acceptable range of values.

5.3.1 Traffic Inputs

Annual Average Daily Truck Traffic (AADTT)
The annual average daily traffic (AADT) shown in Table 17 is obtained from NH DOT traffic volume counts. Truck traffic (AADTT) is calculated by taking 8.2 % of AADT, as shown in LTPP data.

Control AADTT for this study is therefore taken as 3362. Two other stations connected to the control section with different traffic volumes are used to see the effect of AADTT.

Table 17 Annual Average Daily Truck Traffic (AADTT)

CODE	VOLUME COUNT STATION ID	TRAFFIC VOLUME (AADTT)
Q1	099103	3362
Q2	099091	3655
Q3	099102	6092

Rate of Traffic Growth

Default truck traffic growth rate was assumed to be 2.8 % linear as calculated with base AADTT as 3362 and data in Table 18. Three different traffic growth rates were used for this study.

Table 18 Traffic Growth Rates

CODE	TRAFFIC GROWTH RATE
R1	2.0 % linear
R2	2.8 % linear
R3	4.0 % linear

Truck Class Distribution

Truck class distribution for the section 099103 was obtained from LTPP monitored traffic data. This distribution was used in Level 2 analysis and the default distribution given in the MEPDG was used for Level 3 analysis. The distributions chosen are as shown in Table 19. D2 is low-class concentrated truck class distribution and D3 is high-class concentrated truck class distribution (28).

Table 19 Truck Class Distribution selections*

TRUCK CLASS	D1(from LTPP)	D2	D3	Level 3
4	3.2	5.2	0.1	1.8
5	20.0	38.9	0.6	24.6
6	12.0	35.8	0.8	7.6
7	0.8	10.2	0.6	0.5
8	17.9	5.6	6.8	5.0
9	40.2	3.5	9.2	31.3
10	4.7	0.2	25.8	9.8
11	0.8	0.3	36.4	0.8
12	0.2	0.2	16.5	3.3
13	0.2	0.1	3.2	15.3

* - The sum of individual percentages of truck classes should be equal to 100

Traffic Operational Speed

Traffic operational speed is important in selecting the binder grade to be used for pavement design (Table 20). According to Superpave specifications SP-1, the binder grade selection for flexible pavement design can be varied with fast-moving traffic, slow-moving traffic or standing/stationary traffic. The effect of operational speed was therefore analyzed in conjunction with binder grade and the speed input values are chosen as follows (29).

Table 20 Binder Grade and Design Operational Speed

CODE	OPERATIONAL SPEED	BINDER GRADES
U1	5	G1(PG52-28), G2(PG 58-28), G3 (PG 64-28)
U2	25	G1, G2, G3
U3	65	G1, G2, G3

5.3.2 Climate Inputs

Three climate stations were selected from the seven stations for which climate data is available in the MEPDG. The stations Berlin, Lebanon and Concord were chosen as they are more geographically dispersed.

Sensitivity to Climate Data Interpolation

The three stations selected have climate data ready for use in the MEPDG software. The climate data for these stations is then interpolated from the nearest three stations to observe the difference between collected data and interpolated data (Table 21).

Table 21 Climate Station Selection and Interpolation

STATION	Nearest 3 Stations	Latitude	Longitude	Distance	#Months
Concord, NH	Manchester, NH	42.56	-71.26	18.7 mi	116
Lat 43.12	Rochester, NH	43.17	-70.55	29.6 mi	73
Long -71.30	Utica, NY	43.09	-72.23	44.6 mi	62
Lebanon, NH	Springfield, VT	43.20	-73.21	23.4 mi	116
Lat 43.38	Montpelier, VT	44.12	-72.35	41.6 mi	116
Long -72.18	Concord, NH	43.12	-71.30	50.1 mi	116
Berlin, NH	Morrisville, VT	44.32	-72.37	70.6 mi	116
Lat 44.35	Fryeburg, ME	43.59	-70.57	43.0 mi	116
Long -71.11	Augusta, ME	44.19	-69.48	70.6 mi	62

Water Table Depth Variation

Water table depth data was obtained for all counties in New Hampshire (7). Ground water table level is important for states such as New Hampshire because of the seasonal frost-heave and freeze-thaw cycles that occur, leading to subgrade debilitation. Many regions in NH have a water table depth as high as 2 ft below surface, posing a potential threat to subgrade strength. For analyzing sensitivity of pavement distresses to change in groundwater table depth, the pavement section on I-393 was assumed to have three different water table depths of 4ft, 8ft and 12 ft. Depths greater than 12ft do not have significant effect on the subgrade strength because at depths greater than that, the load becomes distributed over a fairly large area. The water table depth was also studied in combination with the weakest subgrade type A-7-5 (Table 22).

Table 22 Water Table Depth Values

CODE	Depth of Water Table	Combination with A-7-5 Subgrade
WT1	4 ft	WT1 E1
WT2	8 ft	WT2 E1
WT3	12 ft	WT3 E1

5.3.3 Material Inputs

Asphalt Inputs

7.2.3.1 HMA Thickness

The thickness of the HMA surface layer for the control file was chosen as 6” and varied as follows (Table 23):

Table 23 HMA Thickness

CODE	HMA SURFACE THICKNESS
T1	2 inches
T2	4 inches
T3	5 inches
T4	6 inches

Number of HMA Layers

A single AC layer versus two AC layers comparison was made by dividing the 6” AC layer into two layers of 2” AC wearing course with 9.5 mm mix gradation and a 4” AC layer with 19.0 mm mix gradation.

HMA Mix Gradation

HMA mix gradation for NH flexible pavement design must conform to Superpave specifications. The mean values for the percentages retained were used as the default values and the other gradations were obtained by choosing fine and coarse mix gradations from the acceptable range of values. The mix type A corresponds to 9.5 mm mix, B corresponds to 19.0 mm and C corresponds to 25.0 mm.

Suffix numbers correspond to the fineness or coarseness of the mix as follows:
1 – Mean values of the allowable range of values
2 – Coarse mix gradation
3 – Fine mix gradation
Table 24 shows detail HMA mix gradation input values.

Table 24 HMA Mix Gradation Input Values

% of Aggregate	A1	A2	A3	B1	B2	B3	C1	C2	C3
Retained on 3/4”	0	0	0	14.0	18.6	12.0	18.0	24.0	16.7
Retained on 3/8”	5.0	8.2	3.6	24.0	32.4	19.8	34.0	43.2	28.5
Retained on #4	35.0	48.3	22.1	42.0	52.0	34.5	48.0	58.6	41.4
Passing #200 sieve	6.0	2.8	8.5	5.0	2.8	7.2	4.0	1.5	6.5

PG Binder Grade

Five different binder grades were chosen from among the PG binders that are suitable for use in the state of New Hampshire. PG 58-28 was used as the binder grade for the control case. The binder grade is tested in conjunction with operational speed of vehicle (Table 20).

Table 25 PG Binder Grades

CODE	PG BINDER GRADE
G1	PG 52-28
G2	PG 58-28
G3	PG 64-28

Effective Binder Content Vbe

The effective binder content values were chosen to conform to the Superpave specifications. A Vbe of 14.0 is used for the control case, and the input values are taken as shown in Table 26.

Table 26 Effective Binder Content Input values

CODE	EFFECTIVE BINDER CONTENT
F1	13.0
F2	14.0
F3	15.0

Percent Air Voids

The percentage of air voids in the asphalt mixture was taken as 6 % for the control case (which is the targeted in-place air void content at the time of pavement construction).
Air void contents chosen for this study are shown in Table 27.

Table 27 Percentage Air Voids

CODE	PERCENT AIR VOIDS
V1	4.0
V2	6.0
V3	8.0

Coefficient of Thermal Contraction (CTC or α)

The default value of 1.5 E-05 was used for Level 3 sensitivity analysis. The values chosen for Level 2 are given inTable 28. The values for coefficient of thermal contraction for the mix have been obtained from a previous study (13).

Table 28 Coefficient of Thermal Contraction

CODE	CTC
N1	1.0 E-04
N2	1.5 E-05
N3	2.0 E-06

5.3.4 Unbound Layer and Subgrade Parameters

Base course aggregates are classified by NH DOT into sand, gravel, crushed gravel and crushed stone. Three different base course materials were chosen from the given types along with their aggregate gradations (Table 29).

Table 29 Base Course Aggregate Gradations

ITEM No.	M1	M2	M3
Type of course	Crushed Gravel	Crushed Stone (Fine)	Crushed Stone (Coarse)
Sieve Size	Percent Passing by Weight		
3 ½ in (90mm)	-	-	100
3 in (75mm)	100	-	92.5
2 in (50mm)	97.5	100	-
1 ½ in (37.5mm)	-	92.5	75.0
1 in (25mm)	70.0	-	-
¾ in (19mm)	-	60.0	55.0
#4 (4.75mm)	39.5	27.5	27.5
#200 (0.075mm)	6.0	2.5	2.5
Resilient Modulus	30000*	24370	33500

* - M-E PDG accepts values only between 20000 psi and 30000 psi

Subgrade Resilient Modulus M_R

The subgrade resilient modulus value for the control file was taken as 9000 psi for Level 2 and 32000 psi for Level 3 corresponding to NH 2 type of subgrade. The other subgrade types used in this study are shown in Table 30.

Table 30 Subgrade Types and Subgrade Resilient Modulus

CODE	SUBGRADE TYPE	Subgrade Type	RESILIENT MODULUS (psi) Level 2	Level 3
E1	NH 5 - Clayey Silt (Marine Deposit)	A-7-5	3000	12000
E2	NH 2 - Fine sand, some silt	A-2-4	9000	32000
E3	NH 3 - Coarse to fine gravelly, coarse to medium sand, some fine sand	A-1-a	38,500	40000

Figure 21 presents input parameters used for NH MEPDG Level 3 and 2 sensitivity analysis runs.

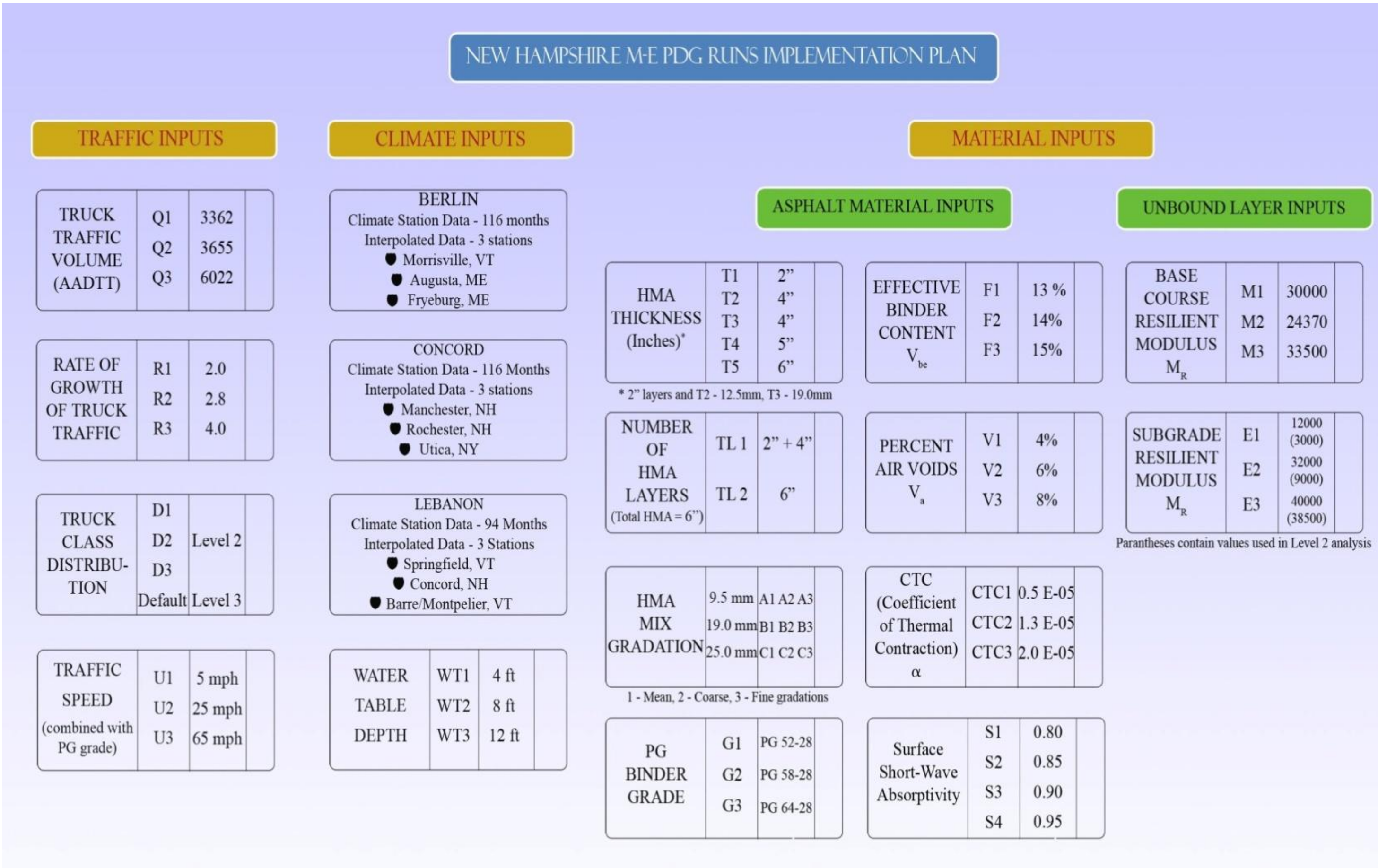

Figure 21 NH MEPDG Runs – Implementation Plan

5.4 Input Value Selection for Connecticut for MEPDG Runs

The default values for the identified critical variables were used in preparing the control file for Level 3 and Level 2 analysis. Tolerances found out from the state specifications for construction of flexible pavements were used to vary these parameters within the acceptable range of values.

The pavement layer structure was adopted from the LTPP section on Route 117 connecting US 1 (Groton) to Route 2 (Preston). The lane width entered in the input is 11 ft, which is the width of the monitored LTPP lane.

5.4.1 Traffic Inputs

Annual Average Daily Truck Traffic (AADTT)
The annual average daily traffic (AADT) shown in Table 31 below was obtained from CT DOT traffic volume counts (30). Truck traffic (AADTT) was calculated by taking 3.5 % of AADT, as given in LTPP data (27). Control AADTT for this study was taken as 376.

Table 31 AADTT

CODE	AADTT
Q1	752 (control)
Q2	1036
Q3	1400

Rate of Traffic Growth
Default truck traffic growth rate was assumed to be 1.6 % linear as calculated with base AADTT as 376. The three different traffic growth rates used for this study are shown in Table 32.

Table 32 Traffic Growth Rates

CODE	TRAFFIC GROWTH RATE
R1	1.2%
R2	1.6 % (control)
R3	2.0 %

Truck Class Distribution
Truck class distribution for the section was obtained from LTPP monitored traffic data. This distribution was used in Level 2 analysis and the default distribution given in the MEPDG is used for Level 3 analysis. The distributions chosen were as shown in Table 33. D2 is low-class concentrated truck class distribution and D3 is high-class concentrated truck class distribution (31).

Table 33 Truck Class Distribution selections*

TRUCK CLASS	D1(from LTPP)	D2	D3	Level 3
4	3.76	5.2	0.1	1.8
5	70.16	38.9	0.6	24.6
6	8.99	35.8	0.8	7.6
7	5.36	10.2	0.6	0.5
8	4.63	5.6	6.8	5.0
9	5.18	3.5	9.2	31.3
10	0.84	0.2	25.8	9.8
11	0.63	0.3	36.4	0.8
12	0.19	0.2	16.5	3.3
13	0.27	0.1	3.2	15.3

* - The sum of individual percentages of truck classes should be equal to 100

Traffic Operational Speed

Traffic operational speed is important in selecting the binder grade to be used for pavement design. According to Superpave specifications SP-1 (32), the binder grade selection for flexible pavement design can be varied with fast-moving traffic, slow-moving traffic or standing/stationary traffic. The effect of operational speed was therefore analyzed in conjunction with binder grade and the speed input values were chosen as follows in Table 34.

Table 34 Truck Class Distribution selections

CODE	OPERATIONAL SPEED	BINDER GRADES
U1	65	G1, G2, G3
U2	25	G1, G2, G3
U3	5	G1, G2, G3

5.4.2 Climate Inputs

Three climate stations were selected from the seven stations for which climate data is available in the MEPDG. The stations Groton – New London, Bridgeport – Fairfield and Bradley International Airport – Hartford were chosen as they are more geographically dispersed.

Sensitivity to Climate Data Interpolation

The three stations selected have climate data ready for use in the MEPDG software. The climate data for these stations was then interpolated from the nearest three stations to observe the difference between collected data and interpolated data (Table 35).

Table 35 Climate Station Selection and Interpolation

STATION	Nearest 3 Stations	Latitude	Longitude	Distance	#Months of data
Groton	Hartford, CT	41.44	-72.39	41.5 miles	105
	Providence, RI	41.43	-71.26	41.4 miles	116
	Westerly, RI	41.21	-71.48	13.0 miles	79
Bridgeport	Danbury, CT	41.22	-73.29	22.1 miles	94
	Tweed, CT	41.16	-72.53	15.1 miles	51
	Islip, NY	40.47	-73.06	26.6 miles	79
Bradley	Westfield, MA	42.10	-72.43	16.2 miles	91
Airport	Willimantic, CT	41.44	-72.11	29.2 miles	116
	Meriden, CT	41.31	-72.50	29.8 miles	79

Water Table Depth Variation

Water table depth data was obtained for all counties in Connecticut (7). Ground water table level is important because of the seasonal frost-heave and freeze-thaw cycles that occur, leading to subgrade debilitation. Some regions in CT have a water table depth as high as 2 ft below surface, posing a potential threat to subgrade strength.

For analyzing sensitivity of pavement distresses to change in groundwater table depth, the pavement section on Route 117 was assumed to have three different water table depths of 2 ft, 4 ft and 8 ft (Table 36). Depths greater than 8 ft do not have significant effect on the subgrade strength because at depths greater than that, the load becomes distributed over a fairly large area. This finding was confirmed by doing trial runs on New Hampshire data with different water table depths greater than 8 feet.

Table 36 Water Table Depth Values

CODE	Depth of Water Table	Combination with A-7-5 Subgrade
WT1	2 ft	WT1 E1
WT2	4 ft	WT2 E1
WT3	8 ft	WT3 E1

5.4.3 Material Inputs - Asphalt

HMA Thickness

The thickness of the HMA surface layer for the control file was chosen as 6” and varied as shown in Table 37.

Table 37 HMA Thickness

CODE	HMA SURFACE THICKNESS
T1	2” AC wearing (9.5 mm) + 4.3” AC binder (19.0 mm)
T2	3” AC wearing (9.5 mm) + 4.3” AC binder (19.0 mm)
T3	3” AC wearing (19.0 mm) + 4.3” AC binder (19.0 mm)
T4	4” AC wearing (19.0 mm) + 4.3” AC binder (19.0 mm)

Since the pavement already has two layers, effect of number of layers analysis was not possible for CT.

HMA Mix Gradation

HMA mix gradation for CT flexible pavement design must conform to Superpave specifications. The mean values for the percentages retained were used as the default values and the other gradations were obtained by choosing fine and coarse mix gradations from the acceptable range of values. The values used for HMA mix gradation are given in Table 24.

PG Binder Grade

Three different binder grades were chosen from among the PG binders that are suitable for use in the state of Connecticut (Table 38). PG 58-22 was used as the binder grade for the control case.

Table 38 PG Binder Grades

CODE	PG BINDER GRADE
G1	PG 52-22
G2	PG 58-22
G3	PG 64-22

Effective Binder Content Vbe

The effective binder content values were chosen to conform with the Superpave specifications. A Vbe of 13.0 was used for the control case, and the input values were taken as shown in Table 39.

Table 39 Effective Binder Content Input values

CODE	EFFECTIVE BINDER CONTENT
F1	12.0
F2	13.0
F3	14.0

Percent Air Voids

The percentage of air voids in the asphalt mixture was taken as 4 % for the surface AC layer. Air void contents chosen for this study are shown in Table 40. The air void content of the binder course was kept constant at 6% (27).

Table 40 Percentage Air Voids

CODE	PERCENT AIR VOIDS
V1	3.0
V2	4.0
V3	5.0

Coefficient of Thermal Contraction (CTC or α)
Coefficient of thermal contraction was assumed as 1.3 E-05 for the control case and was varied in the same manner as for New Hampshire study (13). The values are shown in Table 28.

5.4.4 Unbound Material Inputs

Base Course Resilient Modulus
Base course gradation and modulus values were chosen as default for Level 3. The base course was considered as a crushed stone layer with a resilient modulus of 30000 psi, which is the default value provided by the software for a crushed stone base. Due to unavailability of data from an agency maintained database of resilient modulus values for base course materials for CT, the value was left unchanged for Level 2 runs.

Subgrade Resilient Modulus M_R

Subgrade material for Level 3 analysis was taken as A-1-b (27). The resilient modulus of the subgrade was entered as 38000 psi. For Level 2 analysis, the subgrade resilient modulus of typical soils found in Connecticut were obtained (33). The values used for resilient modulus are shown in Table 41.

Table 41 Subgrade Resilient Modulus

CODE	SUBGRADE RESILIENT MODULUS (psi)	SUBGRADE TYPE
E1	16000	A – 1 – b (control)
E2	14000*	A – 2 – 4
E3	13000**	A – 4

* - Actual resilient modulus is 11530, MEPDG recommends a minimum value of 14000 psi for A – 2 – 4 subgrade type
** - Actual resilient modulus is 12655, MEPDG recommends a minimum value of 13000 psi for A – 4 subgrade type

Figure 22 presents input parameters used for CT MEPDG Level 3 and 2 sensitivity analysis runs.

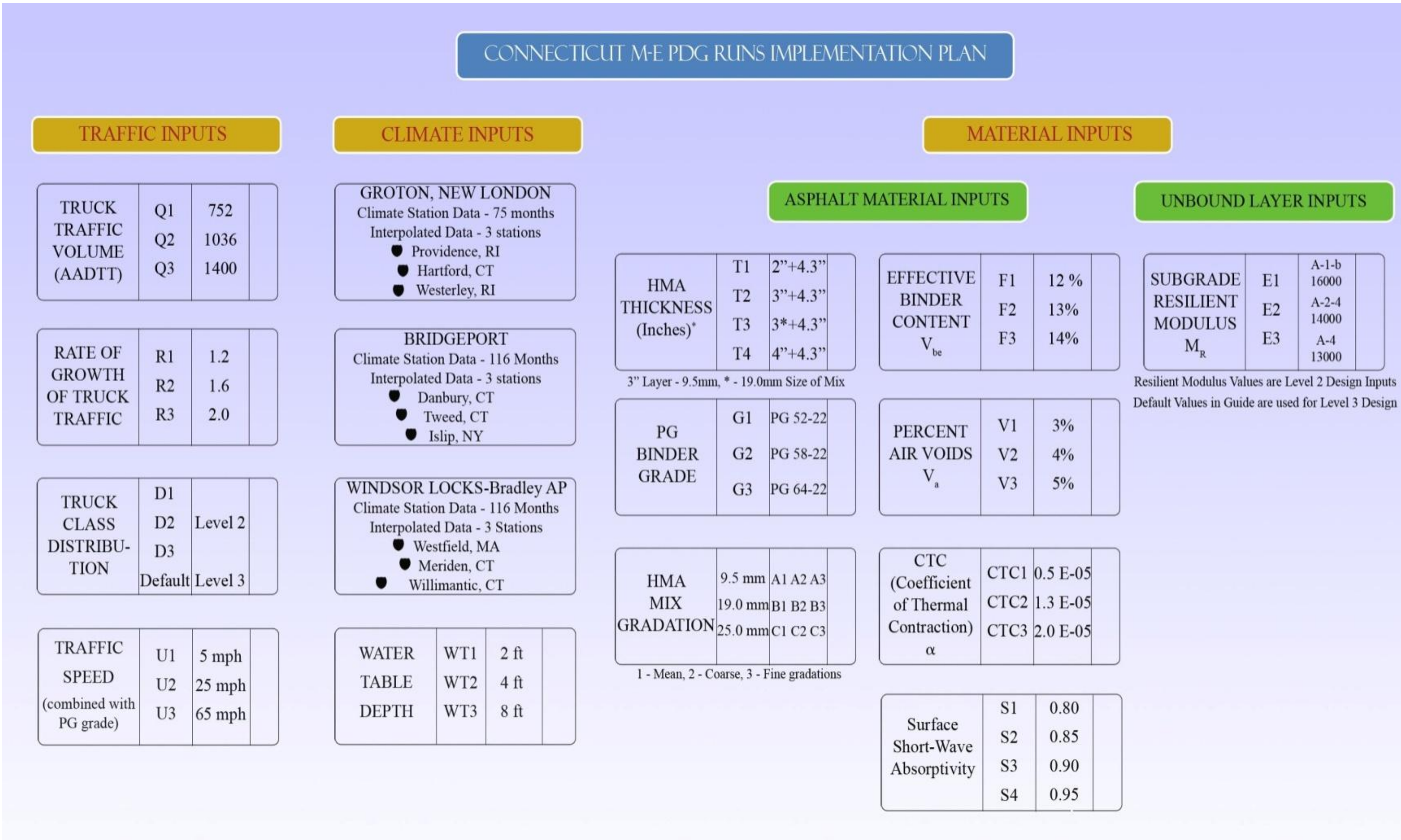

Figure 22 CT MEPDG Runs – Implementation Plan

5.5 Input Value Selection for Maine for MEPDG Runs

The variables on which various types of pavement distresses depend were identified from literature review. The default values for these variables were used in preparing the control file for Level 3 and Level 2 analysis. Tolerances found from the state specifications for construction of flexible pavements were used to vary these parameters within the acceptable range of values.

5.5.1 Traffic Inputs

Annual Average Daily Truck Traffic (AADTT)

The annual average daily traffic (AADT) was obtained from ME DOT traffic volume counts (34). Truck traffic (AADTT) shown in the table below was calculated by taking 8% of AADT, which was obtained from New Hampshire data as both roads are interstate highways having similar AADTT (LTPP database did not contain data for percentage of trucks of total AADT). The latitude and longitude of the location of LTPP section was obtained from Inventory_ID table of LTPP database, identified on the map and the traffic data on the corresponding section was extracted from the DOT traffic volume count tables. Control AADTT for this study was therefore taken as 3944.

The LTPP estimated truck count and DOT traffic counts were compared, and the LTPP count was found to be smaller than the DOT traffic count. Therefore, this provides a more conservative estimate of the truck traffic (Table 42).

Table 42 Annual Average Daily Truck Traffic (AADTT)

CODE	**VOLUME COUNT STATION ID**	**TRAFFIC VOLUME (AADTT)**
Q1	54201 (Freeport, I – 295)	3944
Q2	49402 (Nobleboro, US-1)	1796
Q3	-	6000

Rate of Traffic Growth

Default truck traffic growth rate was assumed to be 3 % linear as calculated with base AADTT as 3944. The three different traffic growth rates used for this study are shown in Table 43. The limits of $\pm$ 1% from control growth rate were chosen from the growth rates calculated for other New England states involved in this study.

Table 43 Traffic Growth Rates

CODE	TRAFFIC GROWTH RATE
R1	2.0 % linear
R2	3.0 % linear
R3	4.0 % linear

Truck Class Distribution

Truck class distribution for section 54201 was obtained from LTPP monitored traffic data (27*)*. This distribution was used in Level 2 analysis and the default distribution given in the MEPDG was used for Level 3 analysis. The distributions chosen were as shown in Table 44.

D2 is truck class distribution from the average of the LTPP sites in Maine and D3 is truck class distribution for LTPP section 23-1001 which is an interstate highway I-95. The actual section considered in the control does not have a truck class distribution obtainable from LTPP data. D3 is considered as the distribution for Level 2 control case due to similarity in function, structure and traffic on both the LTPP sections.

Table 44 Truck Class Distribution selections*

TRUCK CLASS	D1(from LTPP)	D2	D3	Level 3
4		6.72	3.3	1.8
5		25.0	18.7	24.6
6		6.86	2.1	7.6
7		1.07	0.1	0.5
8		4.64	3.8	5.0
9		35.11	57.3	31.3
10		20.19	13.8	9.8
11		0.33	0.8	0.8
12		0.05	0.1	3.3
13		0.02	0.0	15.3

* - The sum of individual percentages of truck classes should be equal to 100

Traffic Operational Speed

Traffic operational speed is important in selecting the binder grade to be used for pavement design. Binder grades indicated by the code G are reported separately for each state. Table 45 shown design operational speed and binder selection for Maine.

Table 45 Design Operational Speed and Binder Grade Selection

CODE	OPERATIONAL SPEED	BINDER GRADES
U1	5	G1, G2, G3
U2	25	G1, G2, G3
U3	65	G1, G2, G3

5.5.2 Climate Inputs

Three climate stations were selected from the nine stations for which climate data is available in the MEPDG. The stations Portland, Millinocket and Frenchville were chosen as they are more geographically dispersed (Table 46).

Sensitivity to Climate Data Interpolation

The three stations selected have climate data ready for use in the MEPDG software. The climate data for these stations was then interpolated from the nearest three stations to observe the difference between collected data and interpolated data.

Table 46 Climate Station Selection and Interpolation

STATION	Nearest 3 Stations	Latitude	Longitude	Distance	#Months
Portland, ME	Wiscasset, ME	43.58	-69.43	37.1 miles	116
Lat 43.38	Rochester, NH	43.17	-70.55	39.2 miles	73
Long -70.18	Fryeburg, ME	43.59	-70.57	40.4 miles	116
Millinocket, ME	Caribou, ME	46.52	-68.02	89.6 miles	115
Lat 45.39	Houlton, ME	46.07	-67.47	53.9 miles	66
Long -68.41	Bangor, ME	44.49	-69.49	57.9 miles	95
Frenchville, ME	Caribou, ME	46.52	-68.02	31.7 miles	115
Lat 47.17	Houlton, ME	46.07	-67.47	84.4 miles	66
Long -68.19*	Millinocket, ME	45.39	-68.41	114.1 miles	116

* - Interpolation subject to difficulty in triangulation; Frenchville does not contain enough information to be run as independent climate file, hence interpolated data is used

Water Table Depth Variation

The effect of water table depth on the prediction of pavement distresses is not significant as concluded from New Hampshire and Connecticut studies, therefore water table depth values were retained the same for Maine (Table 47).

Table 47 Water Table Depth Values

CODE	Depth of Water Table	Water Table Code
WT1	2 ft	WT1
WT2	4 ft	WT2
WT3	8 ft	WT3

5.5.3 Material Inputs - Asphalt

HMA Thickness

The thickness of the HMA surface layer for the control file was 1.2" and the thickness of the AC binder course was 8.3" (27). The thickness was varied by adjusting the total thickness between the two layers. The total thickness of asphalt concrete was kept at 9.5", and the thickness of the AC surface layer was taken as shown in Table 48.

Table 48 HMA Thickness

CODE	HMA SURFACE THICKNESS
T1	1.2 inches
T2	2 inches
T3	3 inches
T4	4 inches

HMA Mix Gradation

HMA mix gradation for ME flexible pavement design must conform to Superpave specifications (32). The mean values for the percentages retained were used as the default values and the other gradations were obtained by choosing fine and coarse mix gradations from the acceptable range of values. Gradation values given in
Table 24 are used.

PG Binder Grade

Three different binder grades were chosen from among the PG binders that are suitable for use in the state of Maine (Table 49). PG 64-28 was used as the binder grade for the control case as recommended by ME DOT specifications. The binder grade was tested in conjunction with operational speed of vehicle.

Table 49 PG Binder Grades

CODE	PG BINDER GRADE
G1	PG 58-28
G2	PG 64-28
G3	PG 70-28

Effective Binder Content V_{be}

The effective binder content values were chosen to conform with the Superpave specifications. A Vbe of 14.0 was used for the control case, and the input values were taken as shown in Table 50.

Table 50 Effective Binder Content Input values

CODE	EFFECTIVE BINDER CONTENT
F1	13.0
F2	14.0
F3	15.0

Percent Air Voids

The percentage of air voids in the asphalt mixture was taken as 5 % for the control case. Air void contents chosen for this study are shown in Table 51.

Table 51 Percentage Air Voids in Asphalt Concrete

CODE	PERCENT AIR VOIDS
V1	4.0
V2	5.0
V3	6.0

Coefficient of Thermal Contraction (CTC or α)

The default value of 1.5 E-05 is used for Level 3 sensitivity analysis. The values chosen for Level 2 are given in Table 28 (13).

5.5.4 Unbound Layer Inputs

Base Course Resilient Modulus

Base course aggregates are classified by ME DOT into three types of aggregate classes – Type A, Type B and Type C (35). Three different base course materials were chosen from the given types along with their aggregate gradations (Table 52). The specifications do not provide values for the resilient modulus of the base course materials.

Table 52 Base Course Gradations

SIEVE DESIGNATION		Percentage by weight passing square mesh sieves		
US	Customary	Type A	Type B	Type C
12.5 mm	½ inch	45 – 70	35 – 75	Not available
6.3 mm	¼ inch	30 – 55	25 – 60	25 – 70
425 um	No. 40	0 – 20	0 – 25	0 – 30
75 um	No. 200	0 – 5	0 – 5	0 – 5

Subbase materials are also classified into four different aggregate types – Type D, Type E, Type F and Type G (Table 53).

Table 53 Subbase Gradations

SIEVE DESIGNATION		**Percentage by weight passing square mesh sieves**			
US	Customary	Type D	Type E	Type F	Type G
6.3 mm	¼ inch	25 – 70	25 – 100	60 – 100	Not available
425 um	No. 40	0 – 30	0 – 50	0 – 50	0 – 70
75 um	No. 200	0 – 7	0 – 7	0 – 7	0 – 10

Subgrade Resilient Modulus M_R

The subgrade type used in the control case can be classified as A-2-4 based on its description as silty sand. Maine subgrade consists of a variety of soil types A – 1, A – 2, A – 3, A – 4, A – 5 and A – 6 (33). In this study, the subgrade types chosen were A – 1, A – 2 – 4 and A – 6. The average values of the subgrade types from the study mentioned were used for Level 2, and default values were used for Level 3.

Figure 23 presents input parameters used for Maine MEPDG Level 3 and 2 sensitivity analysis runs.

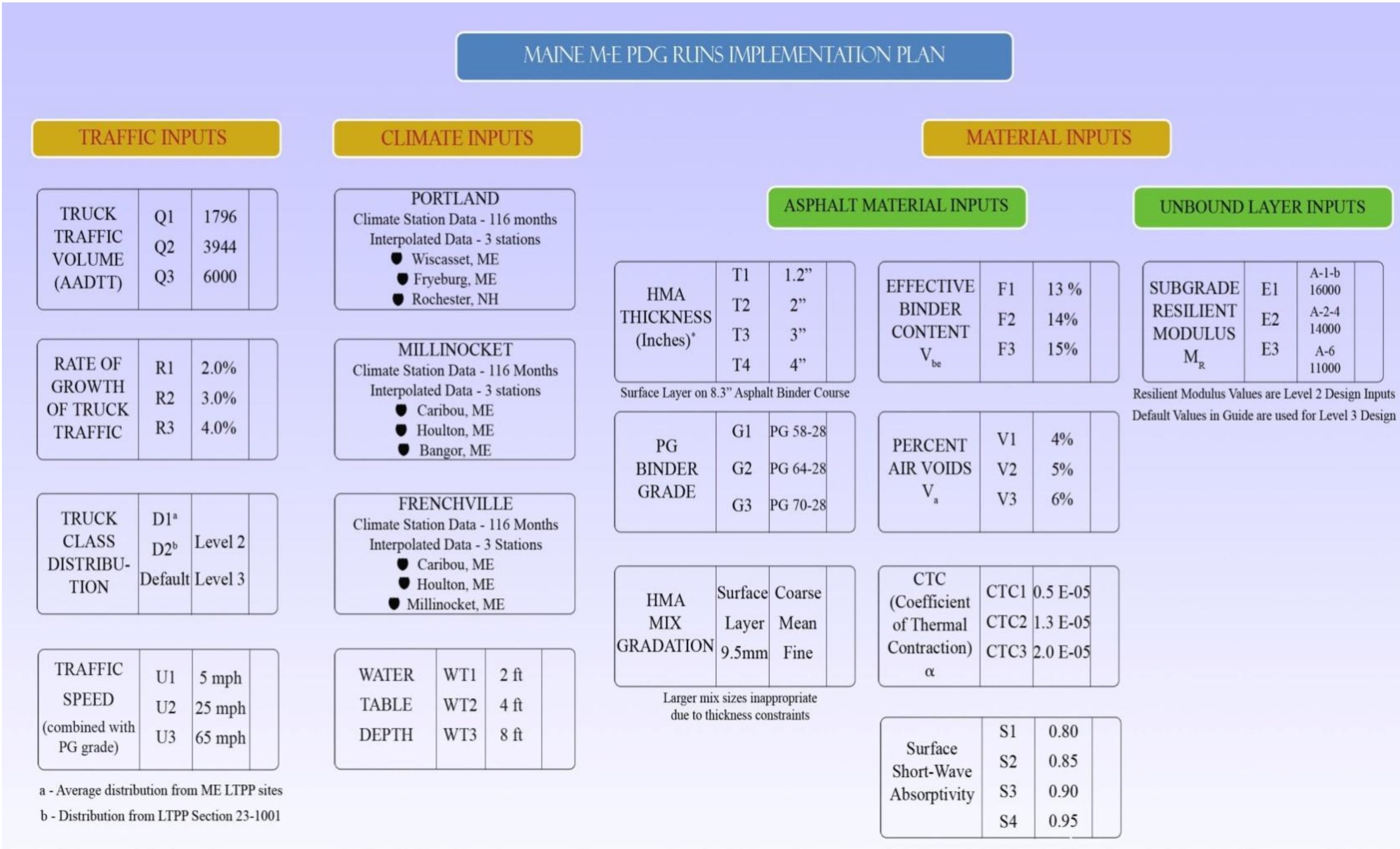

Figure 23 ME MEPDG Runs – Implementation Plan

5.6 INPUT VALUE SELECTION FOR RHODE ISLAND

The variables on which various types of pavement distresses depend were identified from literature review. The default values for these variables were used in preparing the control file for Level 3 and Level 2 analysis. Tolerances found out from the state specifications for construction of flexible pavements were used to vary these parameters within the acceptable range of values.

5.6.1 Traffic Inputs

Annual Average Daily Truck Traffic (AADTT)
The annual average daily traffic (AADT) is obtained from RI DOT traffic volume counts. Rhode Island DOT does not contain detailed information on traffic counts; hence traffic count data for LTPP section on Route 146 was obtained from DOT personnel. Truck traffic (AADTT) shown in the table below was calculated by taking 8% of AADT, which was obtained from New Hampshire data as both roads are interstate highways having similar AADTT.

Control AADTT for this study was therefore taken as 2120 (Table 54). Due to difficulty in obtaining data for traffic count stations adjacent to the one under consideration, the values were varied at ± 25%.

Table 54 Annual Average Daily Truck Traffic (AADTT)

CODE	VOLUME COUNT STATION ID	TRAFFIC VOLUME (AADTT)
Q1		1500
Q2	Route 146	2120
Q3		2500
Q4		4000

Rate of Traffic Growth
Default truck traffic growth rate was assumed to be 4 % linear as calculated with base AADTT as 2120. Therefore, the three different traffic growth rates used for this study are listed in Table 55.

Table 55 Traffic Growth Rates

CODE	TRAFFIC GROWTH RATE
R1	1.5 % linear
R2	2.5 % linear
R3	4.0 % linear

Truck Class Distribution
Truck class distribution for the section 44-7401 was obtained from LTPP monitored traffic data (27).This distribution was used in Level 2 analysis and the default distribution

given in the MEPDG was used for Level 3 analysis. D2 is low-class concentrated truck class distribution and D3 is high-class concentrated truck class distribution (31). The distributions chosen are as shown in Table 56.

Table 56 Truck Class Distribution selections*

TRUCK CLASS	D1(from LTPP)	D2	D3	Level 3
4	2.50	5.2	0.1	1.8
5	25.36	38.9	0.6	24.6
6	6.24	35.8	0.8	7.6
7	0.33	10.2	0.6	0.5
8	18.33	5.6	6.8	5.0
9	45.41	3.5	9.2	31.3
10	0.69	0.2	25.8	9.8
11	0.93	0.3	36.4	0.8
12	0.61	0.2	16.5	3.3
13	0.06	0.1	3.2	15.3

* - The sum of individual percentages of truck classes should be equal to 100

Traffic Operational Speed

Traffic operational speed is important in selecting the binder grade to be used for pavement design. According to Superpave specifications SP-1 (32), the binder grade selection for flexible pavement design can be varied with fast-moving traffic, slow-moving traffic or standing/stationary traffic. The effect of operational speed was therefore analyzed in conjunction with binder grade and the speed input values are chosen as follows. Binder grades denoted by the code G are shown in the materials section (Table 61). Table 57 shows the selections of operational speed in conjunction with binder grades.

Table 57 Design Operational Speed and Binder Grades

CODE	OPERATIONAL SPEED	BINDER GRADES
U1	65	G1, G2, G3
U2	25	G1, G2, G3
U3	5	G1, G2, G3

5.6.2 Climate Inputs

There are only three stations in Rhode Island for which climate data is available in the MEPDG. The three stations Newport, Providence and Westerly are chosen to study the effect of climate on the predicted pavement performance.

Sensitivity to Climate Data Interpolation

The three stations selected have climate data ready for use in the MEPDG software. Interpolation of climate data from the three nearest stations was carried out as a supplemental activity to study the effect of interpolation. The studies for New Hampshire, Connecticut and Maine showed that there is little to no effect of interpolation of climate data on the predicted distresses. Hence, the activity was not performed for Rhode Island.

Water Table Depth Variation

The effect of water table depth on the prediction of pavement distresses is not significant as concluded from New Hampshire, Connecticut and Maine studies, therefore water table depth values were retained the same as for Connecticut (Table 58).

Table 58 Water Table Depth Values

CODE	Depth of Water Table
WT1	2 ft
WT2	4 ft
WT3	8 ft

5.6.3 Material Inputs – Asphalt

HMA Thickness

The thickness of the HMA surface layer for the control file was 3" and the thickness of the jointed plain concrete pavement (JPCP) course was 8". The pavement section was therefore treated as an AC overlay on a fractured JPCP, as well as a cement stabilized base. The thickness was varied by adjusting only the AC layer. The thickness of the underlying concrete layer was kept constant at 8", and the thickness of the AC surface layer was taken as shown in Table 59.

Table 59 HMA Thickness

CODE	HMA SURFACE THICKNESS
T1	2 inches
T2	2.5 inches
T3	3 inches

HMA Mix Gradation

HMA mix gradation for RI flexible pavement design follows Marshall's method of mix design. Since aggregate gradation values for HMA mix design were not available as inputs for the MEPDG, Superpave recommended values were used for the study (Table 60). The mean values for the percentages retained were used as the default values and the other gradations were obtained by choosing fine and coarse mix gradations from the acceptable range of values. Only 9.5 mm mix is used for the surface AC layer to preserve the conformation of aggregate size to AC layer thickness ratio.

Table 60 HMA Mix Gradation Input Values

% of Aggregate	A1	A2	A3
Retained on 3/4” sieve	0	0	0
Retained on 3/8” sieve	5.0	8.2	3.6
Retained on #4 sieve	35.0	48.3	22.1
Passing #200 sieve	6.0	2.8	8.5

PG Binder Grade

Three different binder grades were chosen from among the PG binders that are suitable for use in the state of Rhode Island (Table 61). PG 64-28 was used as the binder grade for the control case as recommended by the specifications. The binder grade was tested in conjunction with operational speed of vehicle listed in Table 57.

Table 61 PG Binder Grades

CODE	PG BINDER GRADE
G1	PG 58-28
G2	PG 64-28
G3	PG 70-28

Effective Binder Content Vbe

The effective binder content values were chosen in Table 62 to conform with Superpave specifications. A Vbe of 13.0 was used for the control case.

Table 62 Effective Binder Content Input values

CODE	EFFECTIVE BINDER CONTENT
F1	12.0
F2	13.0
F3	14.0

Percent Air Voids

The percentage of air voids in the asphalt mixture was taken as 4 % for the control case. Air void contents chosen for this study are shown in Table 63.

Table 63 Percentage Air Void Content

CODE	PERCENT AIR VOIDS
V1	3.0
V2	4.0
V3	5.0

Coefficient of Thermal Contraction (CTC or α)
The default value of 1.5 E-05 was used for Level 3 sensitivity analysis. The values chosen for Level 2 are given in Table 64. The CTC values affect thermal cracking; hence these values were varied in runs conducted on 0.91 software for thermal cracking investigation and not in runs conducted on version 1.0 (13).

Table 64 Effective Binder Content Input values

CODE	CTC
CTC1	1.3 E-05
CTC2	1.5 E-05
CTC3	2.0 E-05

5.6.4 Unbound Layer Inputs

Base Course Resilient Modulus

The base course material gradations obtained from specifications are shown in Table 65. Resilient modulus values are not given for base course materials. Therefore, gradations were used from the table below and the resilient modulus values were taken as the default vales given in the MEPDG.

Table 65 Base and sub-base aggregate specifications

Sieve Size	I Gravel Borrow		II Crushed Stone Or Crushed Gravel	III Key stone	IV Pervious Fill	V Filler Stone	VI Cover Stone
	I(a) Bank run/ Proc Sand/ Gravel	I (b) Reclaimed Processed Material					
3”	60 – 100	100			100		
2 ½ ”			100				
2”			90 – 100				
1 ½ ”		70 – 100	30 – 55				
1 ¼ ”			0 – 25				
1”			0 – 5	100		100	
¾ ”		50 – 85		90 – 100		70 – 85	100
½ ”	50 – 85			20 – 55		10 – 40	90 – 100
3/8 ”	45 – 80			0 – 20		0 – 20	30 – 60
# 4	40 – 75	30 – 55		0 – 5	30 – 100	0 – 5	0 – 15
# 8							0 – 5
# 40	0 – 45						
# 50		8 – 25					
# 200	0 – 10	2 – 10					

Subgrade Resilient Modulus M_R

The DOT specifications do not contain information on the resilient modulus values of subgrade for Rhode Island. The subgrade resilient modulus values for commonly found subgrade soils in Rhode Island are henceforth obtained and listed in Table 66 *(33)*.

Table 66 Subgrade Type and Resilient Modulus Values – Rhode Island

CODE	Subgrade Type	Resilient Modulus value (psi)
E1	A – 1 – b	16000 (MEPDG recommended minimum)
E2	A – 1 – b	13400
E3	A – 1 – b	12000
E4	A – 3	9800

Figure 24 presents input parameters used for Rhode Island MEPDG Level 3 and 2 sensitivity analysis runs.

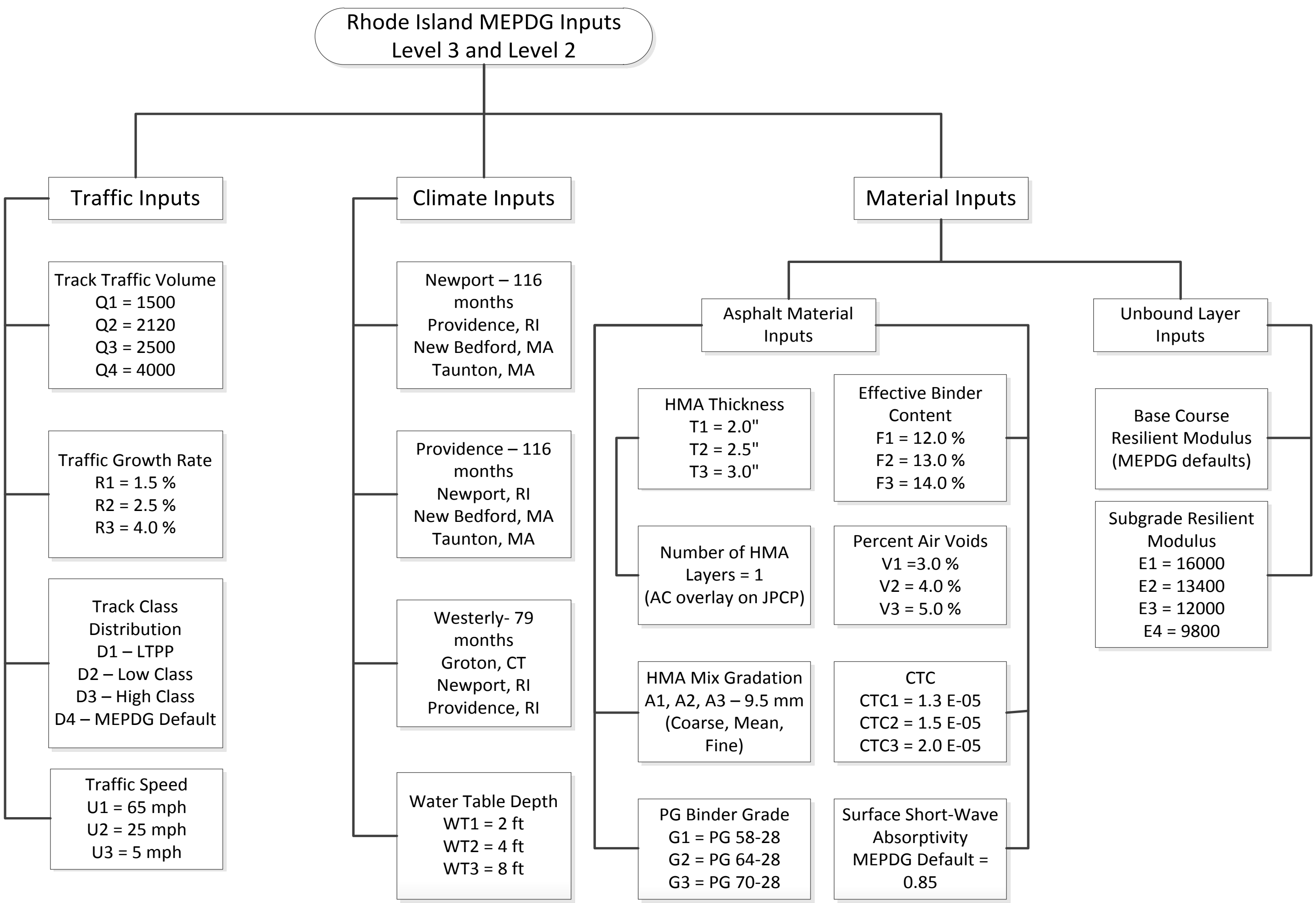

Figure 24 RI MEPDG Runs – Implementation Plan

5.7 Input Value Selection for Vermont for MEPDG Runs

The variables on which various types of pavement distresses depend were identified from literature review. The default values for these variables were used in preparing the control file for Level 3 and Level 2 analysis. Tolerances found out from the state specifications for construction of flexible pavements were used to vary these parameters within the acceptable range of values.

5.7.1 Traffic Inputs

Annual Average daily Truck Traffic (AADTT)
The AADTT was obtained from the VT AOT traffic volume counts and the 2009 VT Permanent Traffic Recorder Stations (36). Truck Traffic (AADTT) was calculated by taking 10.35% of AADT as given in 2009 Automatic Vehicle Classification Report. AADT for Rt. 7 in New Haven, VT (Addison County) was 6800. Control AADTT for this study is taken as 704 (Table 67).

Table 67 AADTT Volumes in Vermont

CODE	STATION	TRAFFIC VOLUME (AADTT)
Q1	New Haven, VT	704
Q2	Salisbury, VT	932
Q3	Burlington, VT	1576

Rate of Traffic Growth
Default track traffic growth rate was assumed to be 2.0% linear as calculated with base AADTT as 704. Therefore, the three different traffic growth rates used for this study are shown in Table 68. The limits of ± 1.0% from control growth rate were chosen from the growth rates calculated for other New England states involved in this study.

Table 68 Selected Traffic Growth Rates for Vermont

Code	Traffic Growth Rate
R1	1.0 % linear
R2 (Control)	2.0 % linear
R3	3.0 % linear

Truck Class Distribution
Table 69 presents four cases of truck class distribution investigated for the Vermont sensitivity analysis. D2 is low-class concentrated truck class distribution and D3 is high-class concentrated truck class distribution (31).

Table 69 Vermont Truck Class Distribution Summary

TRUCK CLASS	CODE			
	D1(from LTPP)	D2 (low class)	D3 (high class)	D4 (Control)
4	5.5	5.2	0.1	1.8
5	43.0	38.9	0.6	24.6
6	10.8	35.8	0.8	7.6
7	3.4	10.2	0.6	0.5
8	7.6	5.6	6.8	5.0
9	25.9	3.5	9.2	31.3
10	3.2	0.2	25.8	9.8
11	0.0	0.3	36.4	0.8
12	0.4	0.2	16.5	3.3
13	0.2	0.1	3.2	15.3

Traffic Operational Speed

Traffic operational speed for this research was analyzed in conjunction with different binder grades to observe the effects of slow and fast moving traffic. Three operational speeds were selected for the analysis: 5 mph, 25 mph and 55 mph. Traffic operational speed depends on the road functional classification and was selected to 55 mph in Vermont's Rt. 7 research (Functional Class 2).

Table 70 Traffic Operational Speeds in VT

Code	Traffic Operational Speed (mph)	Binder Grades
U1	5	G1, G2, G3
U2	25	G1, G2, G3
U3 (Control)	55	G1, G2, G3

5.7.2 Climate Inputs

Three climate stations were selected from the five stations for which climate data is available in the MEPDG. The three stations: Bennington, Barre-Montpelier and Burlington were chosen as they are more geographically dispersed.

Sensitivity to Climate Data Interpolation

The three climate stations selected have climate data ready for use in the MEPDG software (Table 71). The climate data for these stations was then interpolated from the nearest three stations to observe the difference between collected data and interpolated data.

Table 71 Climate Station Selection and Interpolation

STATION	Nearest 3 Stations	Latitude	Longitude	Distance	#Months of data
Bennington Lat. 42.53 Lon. -73.15 Elev. 803 ft	North Adams, MA	42.42	-73.1	13.3 mi	116
	Albany, NY	42.45	-73.48	29.3	116
	Pittsfield, MA	42.26	-73.17	31.1	85
Barre/ Montpelier Lat. 44.12 Lon. -72.35 Elev. 1172 ft	Morrisville, VT	44.32	-72.37	23.1	116
	Burlington, VT	44.28	-73.09	33.5	116
	Lebanon, NH	43.38	-72.18	41.6	94
Burlington Lat. 44.28 Lon. -73.09 Elev. 348 ft	Plattsburg, NY	44.41	-73.31	23.4	92
	Morrisville, VT	44.32	-72.37	26.7	116
	Barre/Montpelier	44.12	-72.35	33.5	116

Water Table Depth Variation

The water table depth is another climate input parameter that needs to be specified by the user. This input value affects pavement distresses such as fatigue cracking, total rutting and roughness of the pavement (IRI). Water table depths greater than 10 feet below the planned surface elevation have minimal affect on the pavement distress predictions. The current data for water table depths were obtained from the USGS website and are shown in Table 72 (37).

Table 72 Water Table Depth Values

CODE	Depth of Water Table	Combination with A-2-4 and A-7-6 Subgrades
WT1	2 ft	WT1 E2, WT1 E1,
WT2 (Control)	5 ft	WT2 E2, WT2 E1
WT3	8 ft	WT3 E2, WT3 E1

5.7.3 Material Inputs – Asphalt

HMA Thickness
An HMA thickness for the control file was 8.5”. To see the effect of HMA thickness on predicting distresses values two more HMA thicknesses has been selected in Table 73.

Table 73 HMA Thickness

CODE	Total HMA Thickness (in)
T1	7.0
T2 (Control)	8.5
T3	10.0

HMA Mix Gradation
HMA mix gradation for Vermont conforms to Superpave specifications (32). The mean values for the percentages retained were used as the default values and the other gradations were obtained by choosing fine and coarse mix gradations from the acceptable range of values. Gradation values given in Table 74 are used.

Table 74 VT HMA Mix Gradation Input Values

% of Aggregate	9.5 mm (3/8”)			19.0 mm (3/4”)		
	mean	**coarse**	**fine**	**mean**	**coarse**	**ne**
Retained on 3/4” sieve	0	0	0	14.0	18.6	12.0
Retained on 3/8” sieve	5.0	8.2	3.6	24.0	32.4	19.8
Retained on #4 sieve	35.0	48.3	22.1	42.0	52.0	34.5
Passing #200 sieve	6.0	2.8	8.5	5.0	2.8	7.2

The mean aggregate mix values were used as the inputs for a control file in the MEPDG sensitivity analysis.

PG Binder Grade

Three different binder grades were chosen from among the PG binders that are suitable for use in the state of Vermont (Table 75). PG 58-28 was used as the binder grade for the control case. The binder grade was tested in conjunction with operational speed of vehicle (Table 70).

Table 75 Binder Grade Selections in VT

State	Binder Grades
Vermont	PG 58-34, PG 58-28, PG 64-28

Effective Binder Content Vbe

The effective binder content values were chosen from table 490.03 B – Design Criteria in VT AOT (38). Table 76 presents calculated effective binder content input values in VT.

Table 76 Calculated Effective Binder Content Input Values in VT

VFA (%)	65			70			75		
V_a (%)	4	5	6	4	5	6	4	5	6
V_{beff} (%)	7.4	9.3	11.1	9.3	11.7	14	12	15	18

$\mathbf{V_a}$ = Air voids (%)

$\mathbf{V_{beff}}$ = Effective binder content, %

VFA = Void filled with asphalt (%)

$$\mathbf{VFA = [V_{beff} / (V_{beff} + V_a)]x100}$$

Table 77 shows selected effective binder content values in VT.

Table 77 Selected Effective Binder Content Values in VT

CODE	EFFECTIVE BINDER CONTENT
F1	9.5
F2 (Control)	11.5
F3	13.5

Percent Air Voids
The percentage of air voids in the asphalt mixture was taken as 5 % for the control case. Air void contents chosen for this study are shown in Table 78.

Table 78 Percentage Air Voids in Asphalt Concrete (VT)

CODE	AIR VOIDS PERCENT
V1	4.0
V2 (Control)	5.0
V3	6.0

Coefficient of Thermal Contraction (CTC or α)
The mix coefficient of thermal contraction (CTC) default value of 1.3 E-05 (in/in/^{o}F) was used for Level 3 and Level 2 sensitivity analysis in all states. This is the coefficient of thermal contraction of the AC mix, and is expressed as the change in length per unit length for unit decrease in temperature. The typical values range from 2.2 to 3.4 /^{o}C. Vermont Level 2 CTC values are listed below in Table 79.

Table 79 Vermont Mix Coefficient of Thermal Contraction Level 2 Values

CODE	COEFFICIENT OF THERMAL CONTRACTION
N1	1.0 E-05
N2 (Control)	1.3 E-05
N3	2.0 E-05

5.7.4 Unbound Layer Inputs

Base Course Resilient Modulus
The unbound materials used in this research were based on the findings from another research project conducted for the New England states (33), as well as on the State Soil Geographic database (39). The base layer material characteristics for the analysis were obtained from the DOT web sites, or when unavailable, the MEPDG default values were selected. Tables 80 and 81 present base course aggregate gradation and resilient modulus values for levels 3 and 2 in Vermont.

Table 80 Vermont Base Course Aggregate Gradations (Level 3)

CODE	M1	M2	M3
Type of course	**Crushed Gravel**	**Crushed Stone (Fine)**	**Crushed Stone (Coarse)**
Sieve Size	**Percent Passing by Weight**		
3 ½ in (90 mm)	-	-	100
3 in (75 mm)	100	-	92.5
2 in (50 mm)	97.5	100	-
1 ½ in (37.5 mm)	-	92.5	75.0
1 in (25 mm)	70.0	-	-
¾ in (19 mm)	-	60.0	55.0
#4 (4.75 mm)	39.5	27.5	27.5
#200 (0.075 mm)	6.0	2.5	2.5
Resilient Modulus Level 3	**29600**	**24370**	**33500**

Table 81 Vermont Base Course Aggregate Gradations (Level 2)

CODE	M1L2	M2L2
Type of course	**Crushed Gravel**	**Crushed Stone**
Sieve Size	**Percent Passing by Weight**	
3 ½ in (90mm)	97.6	97.6
3 in (75mm)	-	-
2 in (50mm)	91.6	91.6
1 ½ in (37.5mm)	85.8	85.8
1 in (25mm)	78.8	78.8
¾ in (19mm)	72.7	72.7
½ in (12.5mm)	63.1	63.1
3/8 in (9.5mm)	57.2	57.2
#4 (4.75mm)	44.7	44.7
#10 (2.0 mm)	33.8	33.8
#40 (0.425 mm)	20.0	20.0
#80 (0.18 mm)	12.9	12.9

#200 (0.075mm)	8.7	8.7
Resilient Modulus Level 2	**25000**	**30000**

Subgrade Resilient Modulus M_R

Table 82 presents the selected subgrade material types and resilient modulus values for level 2 and 3 sensitivity analysis in Vermont.

Table 82 Subgrade Types and Resilient Modulus Values for Vermont Level 2 & 3

CODE	SUBGRADE TYPE	Material Classification	RESILIENT MODULUS (psi)	
			Level 2	**Level 3**
E1	Clayey soils	A-7-6	11500	8000
E2	Fine sand, some silt	A-2-4	21500	32000
E3	Coarse to fine gravelly, coarse to medium sand, some fine sand	A-1-a	29500	40000

The subgrade type resilient modulus range for level 2 is much smaller than level's 3 sensitivity analysis (except the E1 subgrade type), giving more conservative approach for this research. Usually level 3 inputs should be lower than level's 2, as this level is less certain.

The following figure presents the input summaries for the state of Vermont.

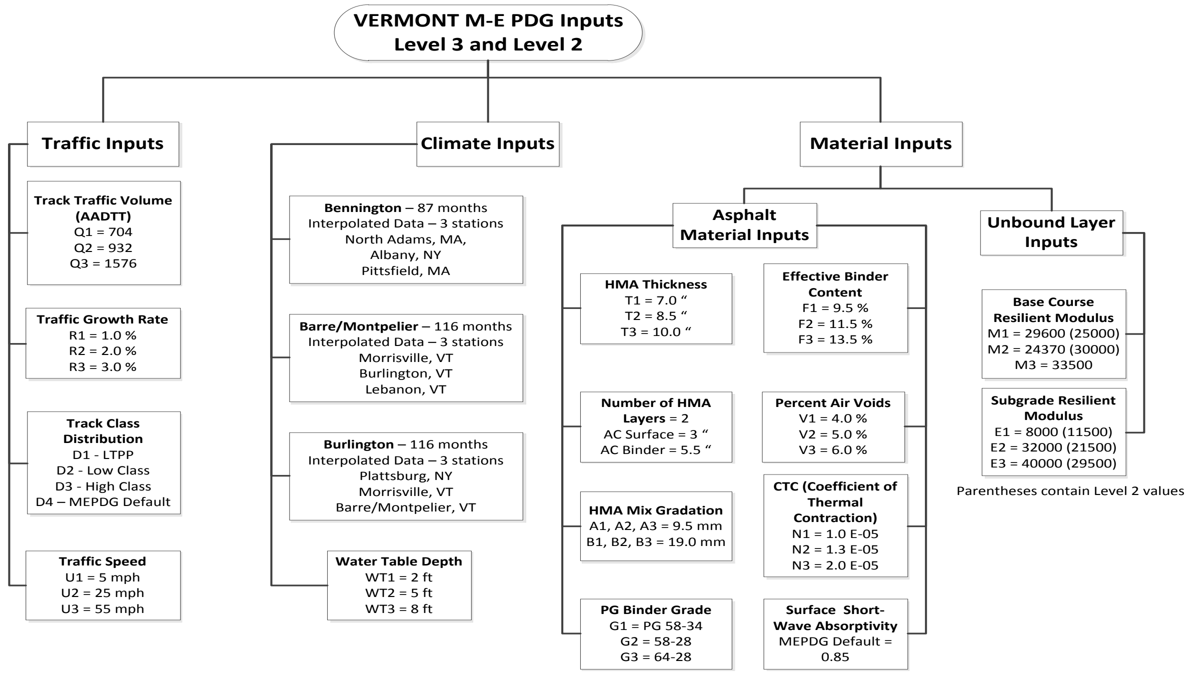

Figure 25 Vermont MEPDG Inputs Level 3 and Level 2

5.8 Input Value Selection for New York for MEPDG Runs

The variables on which various types of pavement distresses depend were identified from literature review. The default values for these variables were used in preparing the control file for Level 3 and Level 2 analysis. Tolerances found out from the state specifications for construction of flexible pavements were used to vary these parameters within the acceptable range of values.

5.8.1 Traffic Inputs

Annual Average daily Truck Traffic (AADTT)
The Annual Average Truck Traffic (AADTT) was calculated by taking 16.0 % of Annual Average Daily Traffic (AADT) as given in 2010 Traffic Data Viewer (40). The AADT for 481 Highway located in East Syracuse, NY (Onondaga County) was 26198 for the 2010 year. Control AADTT for this study is taken as 4192 (Table 83).

Table 83 Annual Average Daily Track Traffic in NY

Code	Station ID/Location	Traffic Volume (AADTT)
Q1 (Control)	East Syracuse, NY	4192
Q2	I-90 exit	6154
Q3	South of I-90	7161

Rate of Traffic Growth
Default track traffic growth rate was assumed to be 2.0% linear as calculated with base AADTT as 4192. Therefore, the three different traffic growth rates used for this study are shown in Table 84. The limits of ± 1.0% from control growth rate were chosen from the growth rates calculated for other New England states involved in this study.

Table 84 Traffic Growth Rates in NY

Code	Traffic Growth Rate
R1	1.0 % linear
R2 (Control)	2.0 % linear
R3	3.0 % linear

Truck Class Distribution

Table 85 presents four cases of truck class distribution investigated for the New York sensitivity analysis. D2 is low-class concentrated truck class distribution and D3 is high-class concentrated truck class distribution (31).

Table 85 New York Truck Class Distribution Summary

TRUCK CLASS	CODE			
	D (from LTPP)*	**D1 (low class)**	**D2 (high class)**	**D3 (Control)**
4	N/A	5.2	0.1	1.8
5	N/A	38.9	0.6	24.6
6	N/A	35.8	0.8	7.6
7	N/A	10.2	0.6	0.5
8	N/A	5.6	6.8	5.0
9	N/A	3.5	9.2	31.3
10	N/A	0.2	25.8	9.8
11	N/A	0.3	36.4	0.8
12	N/A	0.2	16.5	3.3
13	N/A	0.1	3.2	15.3

***** - LTPP Truck Class Distribution data not available

Traffic Operational Speed

Traffic operational speed for this research was analyzed in conjunction with different binder grades to observe the effects of slow and fast moving traffic. Three operational speeds were selected for the analysis: 5 mph, 25 mph and 65 mph.

5.8.2 Climate Inputs

Five climate stations were selected from available climate data base in the MEPDG. The five stations have been chosen as they were more geographically dispersed. These stations are: Albany, Buffalo, Saratoga (control), Massena and Poughkeepsie.

Sensitivity to Climate Data Interpolation

The five climate stations selected have climate data ready for use in the MEPDG software. The climate data for these stations was then interpolated from the nearest three stations to observe the difference between collected data and interpolated data (Table 86).

Table 86 New York Climate Station Selection and Interpolation

STATION	Nearest 3 Stations	Latitude	Longitude	Distance	#Months of data
Buffalo, NY	Niagara Falls, NY	43.07	-78.57	16.7	54
	Dunkirk, NY	42.29	-79.16	41.2	110
	Rochester, NY	43.07	-77.41	54.5	116
Albany, NY	Bennington, VT	42.53	-73.15	29.3	87
	North Adams, MA	42.42	-73.1	32.3	116
	Pittsfield, MA	42.26	-73.17	34.2	85
Massena, NY	Saranac Lake, NY	44.23	-74.13	49.1	93
	Plattsburgh, NY	44.41	-73.31	67.6	92
	Watertown, NY	43.59	-76.01	87.2	62
Poughkeepsie, NY	Montgomery, NY	41.31	-74.16	21.4	98
	Danbury, CT	41.22	-73.29	27.7	94
	White Plains, NY	41.04	-73.43	40.1	59
Syracuse, NY	Fulton, NY	43.21	-76.23	21.5	116
	Penn Yan, NY	42.38	-77.04	59.2	98
	Watertown, NY	43.59	-76.01	87.2	62

Water Table Depth Variation
The water table depth needs to be specified by the user. This input value affects pavement distresses such as fatigue cracking, total rutting and roughness of the pavement (IRI). Water table depths greater than 10 feet below the planned surface elevation have minimal affect on the pavement distress predictions, based on the results from the New England states. The current data for water table depths were obtained from the USGS website (37) and are listed in Table 87.

Table 87 NY Water Tables Depth Values

CODE	Depth of Water Table	Well Location	Combination with A-7-6 Subgrade
WT1	3 ft	Buffalo	E1WT1
WT2	6 ft	Massena	E1WT2
WT3 (Control)	10 ft	Syracuse	E1WT3
WT4	1 ft	Shawnee	E1WT4

5.8.3 Material Inputs – Asphalt

HMA Thickness
An HMA thickness for the control file was 9.8”. To see the effect of HMA thickness on predicting distresses values two more HMA thicknesses were selected in Table 88.

Table 88 HMA Thickness (NY)

CODE	Total HMA Thickness (in)
T1	8.0
T2 (Control)	9.8
T3	11.0

Two HMA layers were used for the MEPDG analysis:
- AC original surface – 1.2” (w/ 9.5 mm mix gradation)
- AC binder course – 8.6” (w/ 19.0 mm mix gradation)

HMA Mix Gradation
HMA mix gradation for New York State conforms to Superpave specifications (32). Table 89 presents the typical ranges of HMA mix gradation for Superpave specification.

Table 89 Range of Values of HMA Mix Gradation – Superpave Specifications

NMAS of Mix	9.5 mm (3/8”)	12.5 mm (1/2”)	19.0 mm (3/4”)	25.0 mm (1”)	37.5 mm (1.5”)
3/4” sieve	0	0 – 10	10 – NR	NR	NR
3/8” sieve	0 – 10	10 – NR	NR	NR	NR
# 4 sieve	10 – NR	NR	NR	NR	NR
#200 sieve	2 – 10	2 – 10	2 – 8	1 – 7	0 – 6

Table 90 presents the recommended HMA mix gradation inputs for New York (41).

Table 90 Recommended NY State HMA Mix Gradations Inputs

Gradation Mix Designation	Percent Retained				Percent Passing
	¾-in Sieve	½-in Sieve	3/8-in Sieve	#4-in Sieve	#200 Sieve
1-in (25.0 mm)	15	30	48	62	4
¾-in (19.0 mm)	5	20	40	58	5
½-in (12.5 mm)	0	5	25	52	6
⅜-in (9.5 mm)	0	0	5	45	6

PG Binder Grade

Five different binder grades were chosen from among the PG binders that are suitable for use in the state of New York. The PG 64-22 was used as the binder grade for the control case. The binder grade was tested in conjunction with operational speed of vehicle. Table 91 shows selected binder grades in NY.

Table 91 New York PG Binder Grades Selection

State	Binder Grades
New York	PG 58-34, PG 64-28, PG 64-22, PG 70-22, PG 76-22

Effective Binder Content Vbe

The effective binder content values were chosen to conform to the Superpave specifications. A Vbe of 11.0 was used for the control case, and the input values were taken as shown in Table 92.

Table 92 NY Effective Binder Content Input Values

CODE	In-situ VMA, percent	EFFECTIVE BINDER CONTENT
F1	13.0	9.0
F2 (Control)	15.0	11.0
F3	17.0	13.0

Percent Air Voids

The percentage of air voids in the asphalt mixture was taken as 4 % for the control case. Air void contents chosen for this study are shown in Table 93.

Table 93 NY Percentage Air Voids in Asphalt Concrete

CODE	PERCENT AIR VOIDS
V1	3.0
V2 (Control)	4.0
V3	5.5

Coefficient of Thermal Contraction (CTC or α)

The mix coefficient of thermal contraction (CTC) default value of 1.3 E-05 (in/in/°F) was used for Level 3 sensitivity analysis. This is the coefficient of thermal contraction of the AC mix, and is expressed as the change in length per unit length for unit decrease in temperature. The typical values range from 2.2 to 3.4 /°C. To see the effect of the CTC value on the sensitivity analysis the min and max ranges were selected based on the MEPDG help menu from 1.0×10^{-7} to 1.0×10^{-4} (Table 94).

Table 94 NY Mix Coefficient of Thermal Contraction Values

CODE	COEFFICIENT OF THERMAL CONTRACTION
N1	1.0 E-07
N2 (Control)	1.3 E-05
N3	1.0 E-04

5.8.4 Unbound Layer Inputs

Base Course Resilient Modulus
The unbound materials used in this research were based on the findings from another research project conducted for the New England states (33), as well as on the State Soil Geographic database (39). The base layer material characteristics for the analysis in NY were obtained from the MEPDG help menu. Table 95 presents base course resilient modulus values for level 3 in New York.

Table 95 Base Course Resilient Modulus Values (NY State)

CODE	M_R (psi)
M1 (Control)	25000
M2	30000
M3	15000

Subgrade Resilient Modulus M_R
Table 96 presents the selected subgrade material types and resilient modulus values for level 3 sensitivity analysis in New York.

Table 96 Subgrade Types and Resilient Modulus Values for New York Level 3

CODE	SUBGRADE TYPE	Material Classification	RESILIENT MODULUS (psi)	
			Level 2	**Level 3**
E1	Clayey soils	A-7-6	n/c*	8000
E2 (Control)	Fine sand, some silt	A-2-4	n/c	25000
E3	Coarse to fine gravelly, coarse to medium sand, some fine sand	A-1-a	n/c	30000

* n/c – not collected

The following figure presents the input summaries for the state of New York.

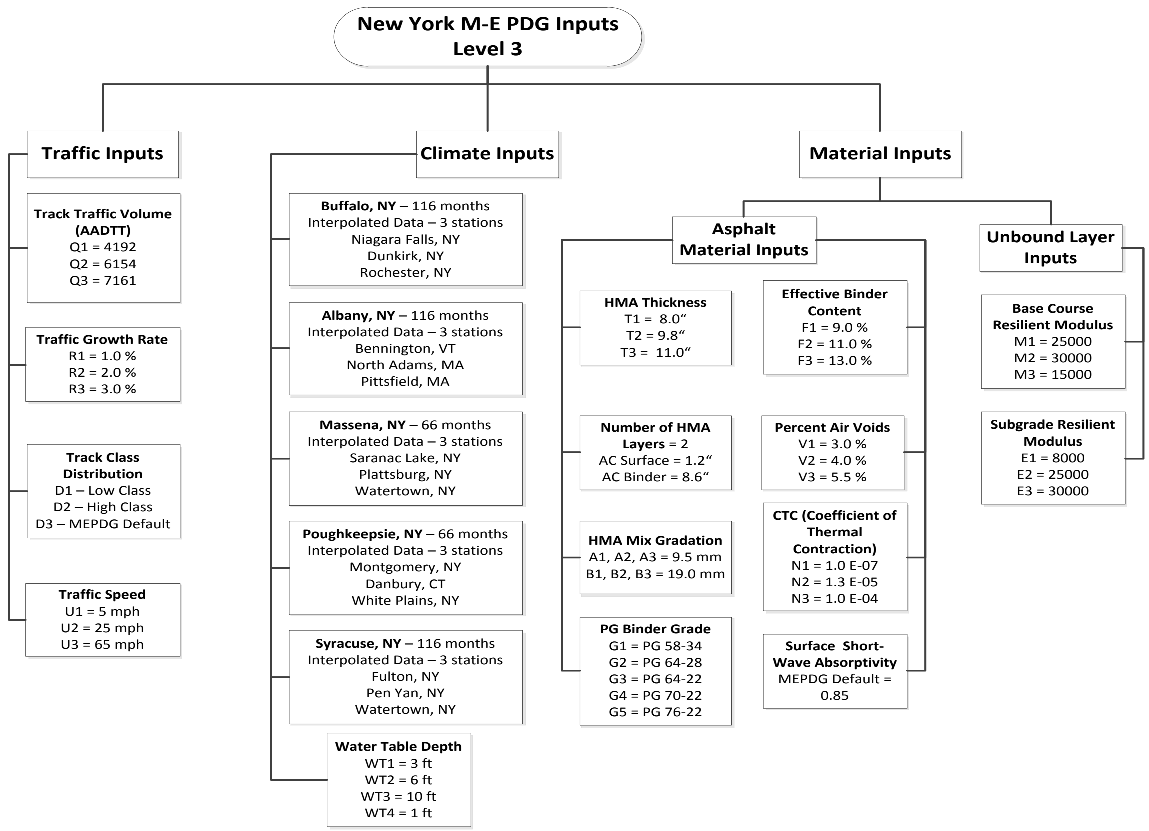

Figure 26 New York MEPDG Inputs Level 3

5.9 Input Value Selection for Massachusetts for MEPDG Runs

The variables on which various types of pavement distresses depend were identified from literature review. The default values for these variables were used in preparing the control file for Level 3 and Level 2 analysis. Tolerances found out from the state specifications for construction of flexible pavements were used to vary these parameters within the acceptable range of values.

5.9.1 Traffic Inputs

Annual Average daily Truck Traffic (AADTT)

Truck Traffic (AADTT) was calculated by taking 5.00% of AADT as given in 2005 Mass DOT Traffic Statistic (42). The 2005 year was selected, because of the higher traffic value (AADT=73,500) compared to year 2008 (AADT=64,400). Control AADTT for I-195 in New Bedford (Bristol County) for this study was taken as 3675. Table 97 presents selected AADTT for MA.

Table 97 MA Annual Average Daily Truck Traffic Values (AADTT)

Code	Station ID	Traffic Volume (AADTT)
Q1 (Control)	#6383 - New Bedford	3675
Q2	#6526 - Fall River	4080
Q3	#0007 L - Mattapoisett	1819

Rate of Traffic Growth

Default track traffic growth rate was assumed to be 2.0% linear as calculated with base AADTT as 3675. Therefore, the three different traffic growth rates used for this study are shown in Table 98. The limits of ± 1.0% from control growth rate were chosen from the growth rates calculated for other New England states involved in this study.

Table 98 Traffic Growth Rates (MA)

Code	Traffic Growth Rate
R1	1.0 % linear
R2 (Control)	2.0 % linear
R3	3.0 % linear

Truck Class Distribution
Table 99 presents four cases of truck class distribution investigated for the Massachusetts sensitivity analysis. D2 is low-class concentrated truck class distribution and D3 is high-class concentrated truck class distribution (31).

Table 99 Massachusetts Truck Class Distribution Summary

TRUCK CLASS	CODE			
	D1 (LTPP-Control)	**D2 (low class)**	**D3 (high class)**	**D4 (Level 3)**
4	3.5	5.2	0.1	1.8
5	47.2	38.9	0.6	24.6
6	9.7	35.8	0.8	7.6
7	0.5	10.2	0.6	0.5
8	8.8	5.6	6.8	5.0
9	29.8	3.5	9.2	31.3
10	0.4	0.2	25.8	9.8
11	0.1	0.3	36.4	0.8
12	0.0	0.2	16.5	3.3
13	0.0	0.1	3.2	15.3

Traffic Operational Speed
Traffic operational speed for this research was analyzed in conjunction with different binder grades (code G) to observe the effects of slow and fast moving traffic. Three operational speeds were selected for the analysis: 5 mph, 25 mph and 65 mph (Table 100). Traffic operational speed depends on the road functional classification and in Massachusetts I-195 was selected to 65 mph (Functional Class 11).

Table 100 Design Operational Speed and Binder Grade Selection in MA

Code	Traffic Operational Speed (mph)	Binder Grades
U1	5	G1, G2, G3
U2	25	G1, G2, G3
U3 (Control)	65	G1, G2, G3

5.9.2 Climate Inputs

Four climate stations were selected from the eighteen stations for which climate data is available in the MEPDG. The four stations: New Bedford (control), Boston, Westfield-Springfield and Worcester were chose as they are more geographically dispersed.

Sensitivity to Climate Data Interpolation

The four climate stations selected have climate data ready for use in the MEPDG software. The climate data for these stations was then interpolated from the nearest three stations to observe the difference between collected data and interpolated data.
Table 101 presents the virtual weather station interpolation results for the state of Massachusetts.

Table 101 Climate Station Selection and Interpolation (MA)

STATION	Nearest 3 Stations	Latitude	Longitude	Distance	#Months of data
New Bedford Lat. 41.41 Lon. -70.58 Elev. 78 ft	Taunton, MA	41.53	-71.01	14.0	99
	Newport, RI	41.32	-71.17	19.4	116
	Plymouth, MA	41.55	-70.44	20.1	116
Boston Lat. 42.22 Lon. -71.01 Elev. 180 ft	Norwood, MA	42.11	-71.1	14.8	93
	Bedford, MA	42.28	-71.17	15.2	91
	Beverly, MA	42.35	-70.55	15.8	87
Westfield/ Springfield Lat. 42.1 Lon. -72.43 Elev. 276 ft	Windsor, CT	41.56	-72.41	16.2	116
	Hartford, CT	41.44	-72.39	30.1	105
	Pittsfield, MA	42.26	-73.17	34.3	85
Worcester Lat. 42.16 Lon. -71.53 Elev. 966 ft	Fitchburg, MA	42.33	-71.46	20.4	101
	Orange, MA	42.34	-72.17	29.1	116
	Bedford, MA	42.28	-71.17	15.2	91

Water Table Depth Variation

The water table depth is another climate input parameter that needs to be specified by the user. This input value affects pavement distresses such as fatigue cracking, total rutting and roughness of the pavement (IRI). Water table depths greater than 10 feet below the planned surface elevation have minimal affect on the pavement distress predictions. The current data for water table depths were obtained from the USGS website (Table 102)

(37). The control water table depth was selected based on average values from the MA-NGW 116 New Bedford, MA well (37).

Table 102 Water Table Depth Values (MA)

CODE	Depth of Water Table	Combination with A-2-4 and A-7-6 Subgrades
WT1	2 ft	WT1 E2, WT1 E1,
WT2 (Control)	4 ft	WT2 E2, WT2 E1
WT3	6 ft	WT3 E2, WT3 E1

5.9.3 Material Inputs – Asphalt

HMA Thickness
An HMA thickness for the control file was 9.6". To see the effect of HMA thickness on predicting distresses values two more HMA thicknesses were selected in Table 103.

Table 103 HMA Thickness (MA)

CODE	Total HMA Thickness (in)
T1	8.0
T2 (Control)	9.6
T3	11.0

The two HMA layers (surface and binder) were treated as one layer with 19.0 mm asphalt mix gradation (mean).

HMA Mix Gradation
The asphalt layer thicknesses and grading were obtained from the LTPP database (27). The HMA mix grading was selected within the Superpave specification limits (32). Table 104 presents the HMA mix grading input values for the surface (9.5 mm) and the binder (19.0 mm). The mean aggregate mix values were used as the inputs for a control file in the MEPDG sensitivity analysis.

Table 104 HMA Mix Grading Input Values (MA)

% of Aggregate	9.5 mm (3/8")			19.0 mm (3/4")		
	mean	coarse	fine	mean	coarse	ne
Retained on 3/4" sieve	0	0	0	14.0	18.6	12.0
Retained on 3/8" sieve	5.0	8.2	3.6	24.0	32.4	19.8
Retained on #4 sieve	35.0	48.3	22.1	42.0	52.0	34.5
Passing #200 sieve	6.0	2.8	8.5	5.0	2.8	7.2

PG Binder Grade

Based on Mass DOT asphalt supplier list (43), three asphalt PG grades were selected: PG 52-34, PG 64-22 and PG 64-28 for the MEPDG sensitivity analysis (Table 105). The PG 64-22 was used as the binder grade for the control case. The binder grade was tested in conjunction with operational speed of vehicle (Table 100).

Table 105 PG Binder Grades (MA)

State	Binder Grades
Massachusetts	PG 52-34, PG 64-22, PG 64-28

Effective Binder Content Vbe

The effective binder content values were chosen to conform to the Superpave specifications. A Vbe of 11.0 was used for the control case, and the input values were taken as shown in Table 106.

Table 106 Effective Binder Content Input Values (MA)

CODE	In-situ VMA, percent	EFFECTIVE BINDER CONTENT
F1	14.0	10.0
F2 (Control)	15.0	11.0
F3	16.0	12.0

Percent Air Voids
The percentage of air voids in the asphalt mixture was taken as 4 % for the control case. Air void contents chosen for this study are shown in Table 107.

Table 107 Percentage Air Voids in Asphalt Concrete (MA)

CODE	PERCENT AIR VOIDS
V1 (Control)	4.0
V2	5.0
V3	6.0

Coefficient of Thermal Contraction (CTC or α)
The mix coefficient of thermal contraction (CTC) default value of 1.3 E-05 (in/in/^{o}F) was used for Level 3 sensitivity analysis in MA. This is the coefficient of thermal contraction of the AC mix, and is expressed as the change in length per unit length for unit decrease in temperature. The typical values range from 2.2 to 3.4 /^{o}C. To see the effect of the CTC value on the sensitivity analysis the broad ranges were selected based on the MEPDG help menu from 1.0×10^{-7} to 1.0×10^{-4} (Table 108).

Table 108 Mix Coefficient of Thermal Contraction Values in MA

CODE	COEFFICIENT OF THERMAL CONTRACTION
N1	1.0 E-07
N2 (Control)	1.3 E-05
N3	1.0 E-04

5.9.4 Unbound Layer Inputs

The unbound materials used in this research were based on the findings from another research project conducted for the New England states (33), as well as on the State Soil Geographic database (39).

Base Course Resilient Modulus
The base layer material characteristics for the analysis were obtained from the MA DOT web site (Table 109) (43).

Table 109 Base Course Gradation and Resilient Modulus Values (MA)

CODE	**M1 (Control)**	**M2**	**M3**
Type of course	**Crushed Gravel**	**Crushed Stone**	**River-Run Gravel**
Sieve Size	**Percent Passing by Weight**		
3 ½ in (90.0 mm)	-	-	97.6
3 in (75.0 mm)	100	-	-
2 in (50.0 mm)	97.5	100	91.6
1 ½ in (37.5 mm)	-	92.5	85.6
1 in (25.0mm)	70.0	-	78.8
¾ in (19.0 mm)	-	60.0	72.7
#4 (4.75 mm)	39.5	27.5	44.7
#200 (0.075 mm)	6.0	2.5	8.7
Resilient Modulus	**25000**	**30000**	**15000**

Subgrade Resilient Modulus M_R

Table 110 presents the selected subgrade material types and resilient modulus values for level 3 sensitivity analysis in Massachusetts.

Table 110 Subgrade Types and Resilient Modulus Values for Massachusetts Level 3

CODE	**SUBGRADE TYPE**	**Material Classification**	**RESILIENT MODULUS (psi)**	
			Level 2	**Level 3**
E1	Clayey soils	A-7-6	n/c*	8000
E2 (Control)	Fine sand, some silt	A-2-4	n/c	25000
E3	Coarse to fine gravelly, coarse to medium sand, some fine sand	A-1-a	n/c	30000

*- n/c not collected

The following figure presents the input summaries for the state of Massachusetts.

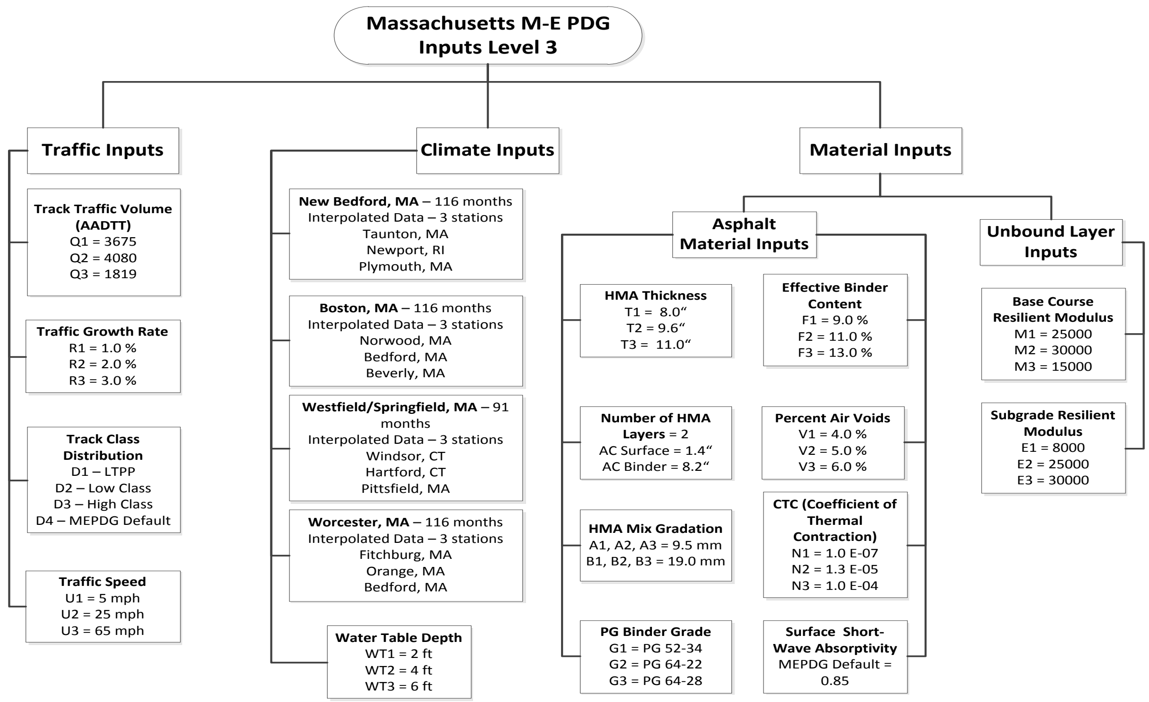

Figure 27 Massachusetts MEPDG Inputs Level 3

6. RESULTS

The runs were performed on the MEPDG using the input parameter values described in Chapter 5 of the report. MEPDG Version 1.0 was used for all the runs for Level 2 and 3 designs. Thermal cracking runs were conducted on Version 0.91 due to the failure of Version 1.0 to return results for thermal cracking module; this is a widely known problem that the thermal cracking model in the newer version of the software has errors that prevents it from predicting thermal cracking. Thermal cracking results are presented in section 6.4 and are not included in the Version 1.0 tabulated results. The results of the runs were tabulated and an analysis of variance was conducted to statistically explain the significance of the parameters on prediction of pavement distresses. Earlier research (9, 10, 11, 12) studied the effect of varying a single parameter on the predicted distresses and the results were considerably explicable by theoretical concepts. In this study, few important interactions between variable pairs such as vehicle speed – binder grade and tensile strength – coefficient of thermal contraction were studied to verify efficiency of the MEPDG in predicting distresses for combined variable effects.

The tables 111-126 that follow in this section contain the results of the runs. The various output parameters that were used for comparing performance of different pavement sections and conditions of operation are:

- Bottom-Up (Fatigue/Alligator) Cracking
- Top-Down (Longitudinal) Cracking
- Subtotal Asphalt Concrete Rutting (rutting in the surface asphalt course)
- Total Rutting (including subgrade rutting)
- International Roughness Index (IRI)
- Thermal Cracking (thermal crack length per mile, done in 0.91 Version)

Table 111 New Hampshire Level 3 Results – Predicted Pavement Performance and Failure

VARIABLE		BOTTOM-UP @ 20 years	TOP-DOWN @ 20 years	TOP-DOWN Failure Year	AC RUTTING @ 20 years	AC RUTTING Failure Year	TOTAL RUTTING @ 20 years	TOTAL RUTTING Failure Year	IRI @ 20 years
Truck Class Distribution	Default	2.98	4390	10	0.430	7	0.800	16	164.4
	LTPP	2.17	2850	15	0.385	8	0.724	No Failure	160.9
	H. Low	1.62	2000	20	0.324	12	0.676	No Failure	158.7
	H. High	3.55	3500	13	0.466	6	0.834	14	166.1
AADTT (Truck Volume)	3362	2.98	4390	10	0.430	7	0.800	16	164.4
	3655	3.26	4720	9	0.448	6	0.821	15	165.4
	6022	5.55	6690	6	0.569	4	0.963	9	172.3
Traffic Growth Rate	2.0 %	2.80	4160	11	0.418	7	0.785	17	163.7
	2.8 %	2.98	4390	10	0.430	7	0.800	16	164.4
	4.0 %	3.27	4730	10	0.448	6	0.821	15	165.4
Operational Speed 5 mph	PG 52-28	7.22	6070	7	1.161	1	1.602	2	198.6
	PG 58-28	6.26	5900	7	0.910	1	1.335	4	187.6
	PG 64-28	5.30	5680	7	0.734	1	1.145	6	179.5
Operational Speed 25 mph	PG 52-28	4.75	5360	8	0.701	2	1.104	6	177.5
	PG 58-28	3.93	5020	9	0.555	4	0.943	10	170.7
	PG 64-28	3.27	4630	9	0.454	6	0.829	14	165.7
Operational Speed 65 mph	PG 52-28	3.69	4840	9	0.540	4	0.924	11	169.8
	PG 58-28	2.98	4390	10	0.430	6	0.800	16	164.4
	PG 64-28	2.48	3920	11	0.354	10	0.710	No Failure	160.6
Water Table Depth	4 feet	3.12	3660	12	0.421	7	0.821	15	165.3
	8 feet	2.98	4390	10	0.430	7	0.800	16	164.4
	12 feet	2.98	4390	10	0.430	7	0.800	16	164.4
HMA Layer Thickness	2”	16.81	696	No Failure	0.674	2	1.492	1	206.3
	4” 12.5 mm	18.30	6790	6	0.551	4	1.063	6	185
	4” 19.0 mm	17.20	6600	6	0.530	4	1.036	6	183.2
	5”	6.94	5850	7	0.465	5	0.892	11	170.5
	6” – 1 Layer	2.98	4390	10	0.430	7	0.800	16	164.4
	6” – 2” + 4”	3.42	4460	10	0.454	7	0.828	16	165.8
Air Void Content	4 %	0.96	1420	No failure	0.375	9	0.734	18	160.6
	6 %	2.98	4390	10	0.430	7	0.800	16	164.4
	8 %	7.80	7660	4	0.509	4	0.815	15	164.8

VARIABLE		BOTTOM-UP @ 20 years	TOP-DOWN @ 20 years	TOP-DOWN Failure Year	AC RUTTING @ 20 years	AC RUTTING Failure Year	TOTAL RUTTING @ 20 years	TOTAL RUTTING Failure Year	IRI @ 20 years
Effective Binder Content	13 %	3.46	4970	9	0.416	7	0.783	18	164.0
	14 %	2.98	4390	10	0.430	7	0.800	16	164.4
	15 %	2.61	3890	11	0.443	7	0.815	15	164.8
HMA Gradation 9.5mm mix	Coarse	4.99	5900	7	0.637	3	1.041	7	175.1
	Mean	3.72	5070	9	0.501	5	0.884	12	168.2
	Fine	3.03	4470	10	0.428	6	0.798	16	164.4
HMA Gradation 19 mm mix	Coarse	3.38	4730	9	0.476	5	0.854	13	166.8
	Mean	2.96	4370	10	0.428	7	0.797	16	164.3
	Fine	2.66	4070	10	0.394	8	0.757	19	162.5
HMA Gradation 25 mm mix	Coarse	3.53	4830	9	0.495	5	0.876	13	167.7
	Mean	2.98	4360	10	0.431	7	0.801	16	164.4
	Fine	2.61	3990	11	0.390	7	0.751	20	162.3
Subgrade Type	A-7-5	3.39	1450	No failure	0.419	7	0.895	11	172.2
	A-2-4	2.98	4390	10	0.423	7	0.800x	16	164.4
	A-1-a	3.12	4480	10	0.423	7	0.805	16	162.0
Base Course Properties	Control	2.98	4390	10	0.430	7	0.800	16	164.4
	C. Gravel	2.78	3830	12	0.438	7	0.775	17	163.3
	C. Stone(C)	2.09	2360	17	0.426	7	0.797	16	163.8
	C. Stone (F)	1.05	642	No Failure	0.446	7	0.743	No Failure	161.1

Table 112 New Hampshire Level 2 Results – Predicted Pavement Performance and Failure

VARIABLE		BOTTOM-UP @ 20 years	TOP-DOWN @ 20 years	TOP-DOWN Failure Year	AC RUTTING @ 20 years	AC RUTTING Failure Year	TOTAL RUTTING @ 20 years	TOTAL RUTTING Failure Year	IRI @ 20 years
Truck Class Distribution	LTPP	0.62	238		0.244		0.584		134.5
	H. Low	0.47	159		0.47		0.562		133.5
	H. High	1.05	134		1.05		0.668		138.1
AADTT (Truck Volume)	3362	0.62	238		0.244		0.584		134.5
	3655	0.68	269		0.68		0.598		135.1
	6022	1.18	558		1.18		0.687		138.9
Traffic Growth Rate	2.0 %	0.598							
	2.8 %	0.62	238		0.244		0.584		134.5
	4.0 %	0.652							
PG 64-28	5 mph	1.1	324		0.56	2	0.94	6	148.9
	25 mph	0.8	272		0.32	6	0.68		138.3
	65 mph	0.62	238		0.244		0.584		134.5
PG 70-28	5 mph	1.1	325		0.48	3	0.85	7	145.6
	25 mph	0.7	273		0.30	7	0.65		137.3
	65 mph	0.6	237		0.23		0.57		133.9
Water Table Depth	4 feet	0.65	229		0.239		0.601		135.3
	8 feet	0.62	238		0.244		0.584		134.5
	12 feet	0.62	238		0.244		0.584		134.5
HMA Layer Thickness	2”	0.95	37.3		0.328	6	0.991	5	153.4
	4” 19.0mm	3.33	742		0.290	9	0.733		142.6
	5”	1.37	498		0.261	10	0.644		137.4
	6” – 1 Layer	0.62	238		0.244		0.584		134.5
	6” – 2” + 4”	0.668	237		0.256	10	0.598		135.1
Air Void Content	4 %	0.21	55.8		0.219		0.553		133.0
	6 %	0.62	238		0.244		0.584		134.5
	8 %	1.54	770		0.277		0.624		136.6
Effective Binder Content	13 %	0.74	305		0.24		0.58		134.4
	14 %	0.62	238		0.244		0.584		134.5
	15 %	0.53	189		0.248		0.589		134.6

Blank = no failure during analysis period

VARIABLE		BOTTOM-UP @ 20 years	TOP-DOWN @ 20 years	TOP-DOWN Failure Year	AC RUTTING @ 20 years	AC RUTTING Failure Year	TOTAL RUTTING @ 20 years	TOTAL RUTTING Failure Year	IRI @ 20 years
HMA Gradation 9.5mm mix	Coarse	0.69	272		0.268		0.613		135.7
	Mean	0.64	242		0.255		0.597		135
	Fine	0.68	271		0.262		0.607		135.4
HMA Gradation 19 mm mix	Coarse	0.62	238		0.244		0.584		134.5
	Mean	0.65	252		0.257		0.600		135.1
	Fine	0.58	222		0.231		0.568		133.8
HMA Gradation 25 mm mix	Coarse	0.64	247		0.253		0.595		135.0
	Mean	0.75	300		0.288		0.638		136.8
	Fine	0.59	227		0.235		0.576		134.1
Subgrade Type	A-7-5	0.278	17.8		0.231		0.667		139.6
	A-2-4	0.62	238		0.244		0.584		134.5
	A-1-a	0.615	417		0.234		0.517		130.9

Table 113 Connecticut Level 3 Results – Predicted Pavement Performance

VARIABLE		BOTTOM-UP @ 10 years	TOP-DOWN @ 10 years	AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
Truck Class Distribution	Default	0.09	8.05	0.093	0.255	120.3
	LTPP	0.03	1.03	0.061	0.203	118.2
	H. Low	0.05	2.6	0.070	0.223	119.0
	H. High	0.11	5.64	0.10	0.261	120.6
AADTT (Truck Volume)	752	0.09	8.05	0.093	0.255	120.3
	1036	0.12	13.1	0.108	0.277	121.2
	1400	0.17	20.7	0.125	0.300	122.2
Traffic Growth Rate	1.2 %	0.09	7.85	0.092	0.254	120.2
	1.6 %	0.09	8.05	0.093	0.255	120.3
	2.0 %	0.09	8.25	0.093	0.256	120.3
Operational Speed 5 mph	PG 52-22	0.20	23.0	0.231	0.418	126.9
	PG 58-22	0.18	19.1	0.189	0.373	125.1
	PG 64-22	0.17	16.2	0.157	0.339	123.7
Operational Speed 25 mph	PG 52-22	0.12	13.9	0.142	0.314	122.7
	PG 58-22	0.11	11.5	0.118	0.287	121.6
	PG 64-22	0.11	9.5	0.10	0.267	120.8
Operational Speed 65 mph	PG 52-22	0.09	9.6	0.108	0.272	121.0
	PG 58-22	0.09	7.8	0.091	0.252	120.2
	PG 64-22	0.08	6.3	0.078	0.237	119.6
Water Table Depth	2 feet	0.10	9.74	0.111	0.281	121.6
	4 feet	0.10	9.74	0.111	0.281	121.6
	8 feet	0.09	8.05	0.093	0.255	120.3
HMA Thickness (Wearing Course)	2” 9.5 mm	0.22	24.7	0.115	0.301	122.2
	3” 9.5 mm	0.10	10.3	0.105	0.270	120.9
	3” 19.0 mm	0.09	8.05	0.093	0.255	120.3
	4” 19.0 mm	0.04	3.37	0.085	0.232	119.3
	5” 19.0 mm	0.02	1.06	0.080	0.216	118.7
Air Void Content	3 %	0.08	3.35	0.089	0.250	120.1
	4%	0.09	8.05	0.093	0.255	120.3
	5 %	0.09	17.9	0.097	0.260	120.5

VARIABLE		BOTTOM-UP @ 10 years	TOP-DOWN @ 10 years	AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
Effective Binder Content	12 %	0.09	9.9	0.091	0.254	120.2
	13 %	0.09	8.05	0.093	0.255	120.3
	14 %	0.09	6.69	0.095	0.256	120.4
HMA Gradation 9.5mm mix	Coarse	0.11	14.4	0.129	0.297	122.0
	Mean	0.1	10.3	0.105	0.270	120.9
	Fine	0.09	8.28	0.093	0.255	120.3
HMA Gradation 19 mm mix	Coarse	0.09	8.05	0.093	0.255	120.3
	Mean	0.09	9.23	0.101	.0264	120.7
	Fine	0.08	7.23	0.087	0.248	120.0
HMA Gradation 25 mm mix	Coarse	0.09	9.61	0.104	0.268	120.8
	Mean	0.09	8.04	0.093	0.255	120.3
	Fine	0.08	7.06	0.086	0.247	120.0
Subgrade Type	A-1-b (38000)	0.09	8.05	0.093	0.255	120.3
	A-2-4 (21500)	0.102	4.38	0.091	0.309	123.0
	A-4 (16500)	0.146	0.86	0.085	0.395	127.3

Table 114 Connecticut Level 2 Results – Predicted Pavement Performance

VARIABLE		BOTTOM-UP @ 10 years	TOP-DOWN @ 10 years	AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
Truck Class Distribution	LTPP	0.033	0.97	0.056	0.196	117.9
	H. Low	0.046	2.46	0.064	0.215	118.7
	H. High	0.103	5.33	0.092	0.251	120.1
AADTT (Truck Volume)	752	0.033	0.97	0.056	0.196	117.9
	1036	0.047	1.58	0.065	0.212	118.5
	1400	0.065	2.5	0.075	0.228	119.2
Traffic Growth Rate	1.2 %	0.033	0.95	0.055	0.196	117.9
	1.6 %	0.033	0.97	0.056	0.196	117.9
	2.0 %	0.034	1.00	0.056	0.197	117.9
PG 64-28 Binder	Speed 5 mph	0.067	2.52	0.111	0.272	120.9
	Speed 25 mph	0.043	1.42	0.070	0.217	118.9
	Speed 65 mph	0.032	0.94	0.054	0.195	117.8
HMA Thickness	1.5” 9.5 mm + 4.3”	0.125	4.72	0.080	0.257	120.4
	2” 9.5 mm + 4.3”	0.079	3.12	0.069	0.231	119.3
	3” 9.5 mm + 4.3”	0.050	2.19	0.067	0.221	118.9
	3” 19.0 mm + 4.3”	0.033	0.97	0.056	0.196	117.9
Air Void Content	3 %	0.032	0.4	0.053	0.194	117.8
	4%	0.033	0.97	0.056	0.196	117.9
	5 %	0.035	2.18	0.058	0.2	118.0
Effective Binder Content	12 %	0.033	0.95	0.055	0.196	117.9
	13 %	0.033	0.97	0.056	0.196	117.9
	14 %	0.034	1.0	0.056	0.197	117.9
HMA Gradation 9.5mm mix	Coarse	0.043	1.78	0.076	0.223	119.0
	Mean	0.037	1.27	0.063	0.206	118.3
	Fine	0.034	1.01	0.056	0.197	117.9
HMA Gradation 19 mm mix	Coarse	0.035	1.12	0.06	0.202	118.1
	Mean	0.033	0.97	0.056	0.196	117.9
	Fine	0.032	0.87	0.052	0.192	117.7
HMA Gradation 25 mm mix	Coarse	0.033	1.0	0.057	0.198	118.0
	Mean	0.033	0.97	0.056	0.196	117.9
	Fine	0.031	0.85	0.052	0.191	117.7

VARIABLE		BOTTOM-UP @ 10 years	TOP-DOWN @ 10 years	AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
	A-1-b (16000)	0.046	0.38	0.054	0.276	121.1
Subgrade Type	A-2-4 (14000)	0.048	0.28	0.053	0.296	122.4
	A-4 (13000)	0.068	0.06	0.050	0.358	125.7

Table 115 Maine Results Level 3 – Predicted Pavement Performance

VARIABLE		BOTTOM-UP @ 10 years	TOP-DOWN @ 10 years	AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
Truck Class Distribution	Default	0.048	20.4	0.173	0.438	128.3
	LTPP Average	0.038	14.6	0.161	0.411	127.2
	LTPP Site Specific	0.041	16.4	0.169	0.415	127.4
AADTT (Truck Volume)	1796	0.021	6.17	0.119	0.360	125.1
	3944	0.048	20.4	0.173	0.438	128.3
	6000	0.076	38.5	0.211	0.490	130.4
Traffic Growth Rate	2.0 %	0.046	19.1	0.169	0.434	128.1
	3.0 %	0.048	20.4	0.173	0.438	128.3
	4.0 %	0.050	21.5	0.176	0.443	128.5
Operational Speed 5 mph	PG 52-28	209	0.132	0.358	0.664	137.4
	PG 58-28	191	0.127	0.341	0.645	136.6
	PG 64-28	172	0.124	0.329	0.632	136.1
Operational Speed 25 mph	PG 52-28	49.2	0.067	0.223	0.501	130.8
	PG 58-28	44.2	0.065	0.213	0.489	130.4
	PG 64-28	39.1	0.063	0.207	0.481	130.0
Operational Speed 65 mph	PG 52-28	18.5	0.046	0.171	0.434	128.1
	PG 58-28	16.4	0.044	0.164	0.426	127.8
	PG 64-28	14.3	0.043	0.160	0.420	127.6
Water Table Depth	2 feet	0.61	34.4	0.213	0.487	130.3
	4 feet	0.61	34.4	0.213	0.487	130.3
	8 feet	0.048	20.4	0.173	0.438	128.3
HMA Layer Thickness	AC 1.2" + 8.3"	0.048	20.4	0.173	0.438	128.3
	AC 2" + 7.5"	0.048	19.8	0.171	0.436	128.2
	AC 3" + 6.5"	0.049	21.3	0.183	0.449	128.7
	AC 4" + 5.5"	0.048	21.9	0.191	0.457	129.1
Air Void Content	4 %	0.047	9.75	0.171	0.435	128.2
	5 %	0.048	20.4	0.173	0.438	128.3
	6 %	0.049	39.6	0.176	0.442	128.5
Effective Binder Content	13 %	0.048	25.6	0.172	0.437	128.2
	14 %	0.048	20.4	0.173	0.438	128.3
	15 %	0.049	16.6	0.174	0.440	128.4

VARIABLE		BOTTOM-UP @ 10 years	TOP-DOWN @ 10 years	AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
HMA * Gradation 9.5mm mix	Coarse	0.052	22.9	0.184	0.453	128.9
	Mean	0.048	20.4	0.173	0.438	128.3
	Fine	0.042	18.1	0.168	0.431	128.0
HMA ** Gradation 19 mm mix	Coarse	0.045	19.0	0.180	0.444	128.5
	Mean	0.042	15.2	0.162	0.423	127.7
	Fine	0.040	12.8	0.149	0.407	127.0
Subgrade Type	A-1-a (38000)	0.045	38.3	0.173	0.394	126.0
	A-2-4 (32000)	0.048	20.4	0.173	0.438	128.3
	A-1-a (17000)	0.056	5.87	0.167	0.453	130.2

* - 9.5mm mix gradation effects were examined for 1.2” AC surface course
** - 19.0mm mix gradation effects were examined for 4” AC surface course (since larger size of maximum aggregate size cannot be chosen for 1.2” AC surface course). Mean 19.0mm gradation results are different from 4” + 5.5” HMA thickness due to difference in nominal maximum aggregate size of mix gradation.

Table 116 Maine Results Level 2 – Predicted Pavement Performance

VARIABLE		BOTTOM-UP @ 10 years	TOP-DOWN @ 10 years	AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
Truck Class Distribution	LTPP Average	0.050	5.58	0.161	0.484	130.1
	LTPP Site Specific	0.046	5.24	0.153	0.482	130.0
AADTT (Truck Volume)	1796	0.022	1.69	0.111	0.404	126.9
	3944	0.050	5.58	0.161	0.484	130.1
	6000	0.079	10.6	0.197	0.535	132.2
Traffic Growth Rate	2.0 %	0.048	5.26	0.158	0.479	129.9
	3.0 %	0.050	5.58	0.161	0.484	130.1
	4.0 %	0.052	5.91	0.164	0.488	130.3
Operational Speed 5 mph	PG 64-28	0.133	90.7	0.332	0.701	138.8
	PG 70-28	0.132	88	0.325	0.693	138.5
Operational Speed 25 mph	PG 64-28	0.070	16.4	0.206	0.543	132.5
	PG 70-28	0.067	15.7	0.202	0.539	132.3
Operational Speed 65 mph	PG 64-28	0.049	5.03	0.158	0.479	129.9
	PG 70-28	0.048	4.77	0.155	0.476	129.8
HMA Layer Thickness	AC 1.2” + 8.3”	0.050	5.58	0.161	0.484	130.1
	AC 2” + 7.5”	0.050	5.27	0.161	0.483	130.1
	AC 3” + 6.5”	0.052	5.74	0.173	0.496	130.6
	AC 4” + 5.5”	0.050	5.89	0.180	0.503	130.9
Air Void Content	4 %	0.050	2.63	0.159	0.480	130.0
	5 %	0.050	5.27	0.161	0.483	130.1
	6 %	0.051	11.1	0.164	0.488	130.3
Effective Binder Content	13 %	0.050	6.96	0.158	0.482	130.1
	14 %	0.050	5.27	0.161	0.483	130.1
	15 %	0.050	4.57	0.164	0.485	130.2
HMA * Gradation 19.0mm mix	Coarse	0.056	7.35	0.168	0.495	130.6
	Mean	0.050	5.27	0.161	0.483	130.1
	Fine	0.045	4.0	0.156	0.474	129.7
HMA ** Gradation 25.0mm mix	Coarse	0.058	8.19	0.171	0.499	130.7
	Mean	0.050	5.36	0.161	0.483	130.1
	Fine	0.443	3.82	0.155	0.472	129.7

VARIABLE		BOTTOM-UP @ 10 years	TOP-DOWN @ 10 years	AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
Subgrade Type	A-1-b (17000)	0.048	7.78	0.162	0.462	128.7
	A-2-4 (14000)	0.050	5.58	0.161	0.484	130.1
	A-6 (11000)	0.061	0.81	0.154	0.508	132.8

* - 19.0 mm mix gradation effects on AC binder course
** - 25.0 mm mix gradation effects on AC binder course – Both cases have layer structure similar to the control case

Table 117 Rhode Island Level 3 Results – Predicted Performance for Asphalt Concrete Overlay over Fractured Jointed Plain Concrete Pavement

VARIABLE		AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
Truck Class Distribution	Default	0.171	0.398	126.0
	LTPP	0.151	0.342	123.7
	H. Low	0.167	0.344	123.8
	H. High	0.171	0.408	126.3
AADTT (Truck Volume)	1500	0.145	0.362	124.5
	2120	0.171	0.398	126.0
	2500	0.186	0.417	126.7
	4000	0.232	0.477	129.1
Traffic Growth Rate	1.5%	0.164	0.388	125.6
	2.5%	0.167	0.392	125.7
	4.0%	0.171	0.398	126.0
Operational Speed 5 mph	PG 52-28	0.472	0.705	138.9
	PG 58-28	0.391	0.623	134.9
	PG 64-28	0.340	0.572	132.9
Operational Speed 25 mph	PG 52-28	0.260	0.489	129.6
	PG 58-28	0.227	0.455	128.2
	PG 64-28	0.209	0.437	127.5
Operational Speed 65 mph	PG 52-28	0.185	0.412	126.5
	PG 58-28	0.167	0.394	125.8
	PG 64-28	0.159	0.386	125.5
Climate	Providence	0.171	0.398	126.0
	Newport	0.188	0.409	126.4
	Westerly	0.193	0.414	126.6
Water Table Depth	2 ft	0.13	0.503	130.1
	4 ft	0.175	0.498	130.2
	8 ft	0.171	0.398	126.0
HMA Layer Thickness	2” AC Overlay	0.097	0.337	123.5
	2.5” AC Overlay	0.145	0.380	125.2
	3” AC Overlay	0.171	0.398	126.0
Air Void Content	3 %	0.163	0.389	125.6
	4 %	0.171	0.398	126.0
	5 %	0.181	0.409	126.4
HMA Gradation 9.5mm mix	Coarse	0.161	0.387	125.5
	Mean	0.171	0.398	126.0
	Fine	0.168	0.395	125.8
Subgrade	A-1-b	0.171	0.398	126.0
	A-3	0.174	0.463	128.1
Effective Binder Content	12%	0.171	0.398	126.0
	13%	0.168	0.395	125.8
	14 %	0.174	0.401	126.1
	15 %	0.176	0.403	126.3

Table 118 Rhode Island Level 3 Results – Predicted Performance for Asphalt Concrete Overlay over Stabilized Cement Base Pavement

VARIABLE		AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
Truck Class Distribution	Default	0.145	0.463	128.5
	LTPP	0.127	0.403	126.1
	H. Low	0.109	0.411	126.4
	H. High	0.157	0.470	128.8
AADTT (Truck Volume)	1500	0.123	0.427	127.1
	2120	0.145	0.463	128.5
	2500	0.156	0.481	129.2
	4000	0.196	0.539	131.6
Traffic Growth Rate	1.5%	0.138	0.452	128.1
	2.5%	0.141	0.457	128.3
	4.0%	0.145	0.463	128.5
Operational Speed 5 mph	PG 52-28	0.373	0.701	138.0
	PG 58-28	0.332	0.653	136.1
	PG 64-28	0.268	0.593	133.7
Operational Speed 25 mph	PG 52-28	0.210	0.531	131.2
	PG 58-28	0.199	0.514	130.6
	PG 64-28	0.172	0.492	129.7
Operational Speed 65 mph	PG 52-28	0.156	0.474	129.0
	PG 58-28	0.154	0.467	128.7
	PG 64-28	0.137	0.455	128.2
Climate	Providence	0.145	0.463	128.5
	Newport	0.175	0.498	130.2
	Westerly	0.175	0.498	147.1
HMA Layer Thickness	2” AC Overlay	0.093	0.439	127.6
	2.5” AC Overlay	0.129	0.462	128.5
	3” AC Overlay	0.145	0.463	128.5
Air Void Content	3 %	0.139	0.456	128.3
	4 %	0.145	0.463	128.5
	5 %	0.151	0.470	128.8
HMA Gradation 9.5mm mix	Coarse	0.138	0.454	128.2
	Mean	0.145	0.463	128.5
	Fine	0.142	0.461	128.4
Effective Binder Content	12%	0.143	0.461	128.4
	13 %	0.145	0.463	128.5
	14 %	0.146	0.465	128.6
	15 %	0.148	0.466	128.6

Table 119 Rhode Island Level 2 Results – Predicted Performance
Asphalt Concrete Overlay over Fractured Jointed Plain Concrete Pavement

VARIABLE		AC RUTTING @ 10 years	TOTAL RUTTING @ 10 years	IRI @ 10 years
Truck Class Distribution	LTPP	0.156	0.347	123.9
	H. Low	0.149	0.338	123.6
	H. High	0.192	0.414	126.6
AADTT (Truck Volume)	1500	0.132	0.316	122.7
	2120	0.156	0.347	123.9
	2500	0.169	0.364	124.7
	4000	0.211	0.417	126.7
Traffic Growth Rate	1.5%	0.149	0.338	123.6
	2.5%	0.152	0.342	123.7
	4.0%	0.156	0.347	123.9
Operational Speed 5 mph	PG 64-28	0.343	0.539	131.6
	PG 70-28	0.338	0.535	131.4
Operational Speed 25 mph	PG 64-28	0.204	0.397	125.9
	PG 70-28	0.208	0.412	126.5
Operational Speed 65 mph	PG 64-28	0.152	0.344	123.8
	PG 70-28	0.171	0.363	124.5
Climate	Providence	0.156	0.347	123.9
	Newport	0.163	0.349	124.0
	Westerly	0.244	0.423	125.9
Water Table Depth	2 ft	0.141	0.360	124.4
	4 ft	0.147	0.350	124.0
	8 ft	0.156	0.347	123.9
HMA Layer Thickness	2” AC Overlay	0.088	0.292	121.7
	2.5” AC Overlay	0.132	0.330	123.2
	3” AC Overlay	0.156	0.347	123.9
Air Void Content	3 %	0.148	0.339	123.6
	4 %	0.156	0.347	123.9
	5 %	0.165	0.357	124.3
HMA Gradation 9.5mm mix	Coarse	0.147	0.337	123.5
	Mean	0.156	0.347	123.9
	Fine	0.153	123.5	123.8
Effective Binder Content	12%	0.154	0.345	123.8
	13 %	0.156	0.347	123.9
	14 %	0.158	0.349	124.0
	15 %	0.160	0.351	124.1
Subgrade Soil Type	A-1-b M_R 16000	0.156	0.347	123.9
	A-1-b M_R 13400	0.155	0.367	124..7
	A-1-b M_R 12000	0.155	0.381	125.3
	A-3 M_R 9800	0.157	0.413	125.9

Table 120 Vermont Level 3 Results

VERMONT LEVEL 3										
Input Value	**Bottom-Up Cracking**		**Top-Down Cracking**		**AC Rutting**		**Total Rutting**		**IRI**	
	Value	Rank	Value	Rank	Value	Rank	Value	Rank	Value	Rank
HMA thickness	0.079	1	0.997	1	0.145	8	0.218	4	0.027	4
HMA mix gradation	0.013	6	0.258	9	0.395	3	0.198	5	0.025	6
HMA air voids	0.053	2	0.546	3	0.125	9	0.069	12	0.009	11
HMA effective binder content	0.033	5	0.28	8	0.151	7	0.085	10	0.007	12
HMA binder grade	0.013	6	0.242	10	0.296	5	0.157	7	0.019	8
Base type/modulus	0.013	6	0.302	7	0.039	13	0.061	13	0.007	12
Subgrade type/modulus	0.013	6	0.768	2	0.046	12	0.303	2	0.1	2
Ground water table	0.007	7	0.203	11	0.066	11	0.102	9	0.012	10
WT with weakest subgrade	0.013	6	0.018	14	0.033	14	0.074	11	0.009	11
Climate	0.007	7	0.083	12	0.263	6	0.118	8	0.013	9
AADTT value	0.053	2	0.423	5	0.474	2	0.259	3	0.032	5
Operational speed	0.046	3	0.433	4	0.98	1	0.488	1	0.059	3
Traffic growth rate	0.007	7	0.076	13	0.079	10	0.044	14	0.007	12
Traffic distribution	0.04	4	0.409	6	0.309	4	0.171	6	0.022	7
HMA CTC	0	8	0	15	0	15	0	15	0.005	13
Initial IRI	0	8	0	15	0	15	0	15	0.615	1

Table 121 VT Ranking Summary of Significance of Each Input Parameter on the Performance of Flexible Pavement

VERMONT LEVEL 3							
Input Variable	**Bottom-Up**	**Top-Down**	**AC Rutting**	**Total Rutting**	**IRI**	**Total Ranking Points**	**Overall Order of Significance**
	Rank	**Rank**	**Rank**	**Rank**	**Rank**		
HMA thickness	1	1	8	4	4	**18**	**3**
HMA mix gradation	6	9	3	5	6	**29**	**6**
HMA air voids	2	3	9	12	11	**37**	**8**
HMA effective binder content	5	8	7	10	12	**42**	**9**
HMA binder grade	6	10	5	7	8	**36**	**7**
Base type/modulus	6	7	13	13	12	**51**	**11**
Subgrade type/modulus	6	2	12	2	2	**24**	**4**
Ground water table	7	11	11	9	10	**48**	**10**
WT with weakest subgrade	6	14	14	11	11	**56**	**13**
Climate	7	12	6	8	9	**42**	**9**
AADTT value	2	5	2	3	5	**17**	**2**
Operational speed	3	4	1	1	3	**12**	**1**
Traffic growth rate	7	13	10	14	12	**56**	**13**
Traffic distribution	4	6	4	6	7	**27**	**5**
HMA CTC	8	15	15	15	13	**66**	**14**
Initial IRI	8	15	15	15	1	**54**	**12**

Table 122 Vermont Level 2 Results

VERMONT LEVEL 2										
Input Variable	Bottom-Up Cracking		Top-Down Cracking		AC Rutting		Total Rutting		IRI	
	Value	Rank	Value	Rank	Value	Rank	Value	Rank	Value	Rank
HMA air voids	0.065	3	0.551	3	0.126	5	0.066	6	0.009	8
HMA effective binder content	0.033	5	0.297	6	0.153	3	0.080	5	0.011	7
HMA CTC	0.000	8	0.000	9	0.000	8	0.000	9	0.098	2
Base type/modulus	0.013	7	0.185	7	0.022	7	0.032	8	0.005	10
Subgrade type/modulus	0.020	6	0.672	2	0.033	6	0.096	4	0.075	3
WT with weakest subgrade	0.020	6	0.022	8	0.033	6	0.055	7	0.008	9
AADTT value	0.072	2	0.436	4	0.470	1	0.261	1	0.037	4
Traffic distribution	0.039	4	0.307	5	0.333	2	0.172	3	0.024	6
Initial IRI	0.000	8	0.000	9	0.000	8	0.000	9	0.600	1
HMA thickness	0.131	1	1.153	1	0.148	4	0.222	2	0.032	5

Table 123 New York Level 3 Results

NEW YORK LEVEL 3										
Input Variable	**Bottom-Up**		**Top-Down**		**AC Rutting**		**Total Rutting**		**IRI**	
	Value	Rank	Value	Rank	Value	Rank	Value	Rank	Value	Rank
HMA thickness	0.175	1	2.550	1	0.155	9	0.208	5	0.024	7
HMA mix gradation	0.019	11	0.417	8	0.244	6	0.134	8	0.015	10
HMA air voids	0.130	2	1.047	5	0.113	11	0.063	11	0.002	14
HMA effective binder content	0.026	10	0.141	12	0.167	8	0.089	9	0.020	8
HMA binder grade	0.065	5	1.409	3	0.768	3	0.411	2	0.046	6
Base type/modulus	0.032	9	0.295	10	0.065	13	0.066	10	0.008	12
Subgrade type/modulus	0.078	4	1.206	4	0.173	7	0.395	3	0.102	4
Ground water table	0.058	6	0.436	7	0.137	10	0.134	8	0.008	12
WT with weakest subgrade	0.078	4	0.010	14	0.077	12	0.061	12	0.009	11
Climate	0.045	8	0.881	6	0.786	2	0.392	4	0.149	2
AADTT value	0.045	8	0.326	9	0.292	5	0.161	7	0.019	9
Operational speed	0.091	3	1.633	2	1.024	1	0.529	1	0.061	5
Traffic growth rate	0.013	12	0.082	13	0.137	10	0.045	13	0.005	13
Traffic distribution	0.052	7	0.224	11	0.327	4	0.166	6	0.019	9
HMA CTC	0.000	13	0.000	15	0.000	14	0.000	14	0.115	3
Initial IRI	0.000	13	0.000	15	0.000	14	0.000	14	0.577	1

Table 124 NY Ranking Summary of Significance of Each Input Parameter on the Performance of Flexible Pavement

NEW YORK LEVEL 3							
Input Variable	**Bottom-Up**	**Top-Down**	**AC Rutting**	**Total Rutting**	**IRI**	**Total Ranking Points**	**Overall Order of Significance**
	Rank	Rank	Rank	Rank	Rank		
HMA thickness	1	1	9	5	7	**23**	**4**
HMA mix gradation	11	8	6	8	10	**43**	**7**
HMA air voids	2	5	11	11	14	**43**	**7**
HMA effective binder content	10	12	8	9	8	**47**	**8**
HMA binder grade	5	3	3	2	6	**19**	**2**
Base type/modulus	9	10	13	10	12	**54**	**10**
Subgrade type/modulus	4	4	7	3	4	**22**	**3**
Ground water table	6	7	10	8	12	**43**	**7**
WT with weakest subgrade	4	14	12	12	11	**53**	**9**
Climate	8	6	2	4	2	**22**	**3**
AADTT value	8	9	5	7	9	**38**	**6**
Operational speed	3	2	1	1	5	**12**	**1**
Traffic growth rate	12	13	10	13	13	**61**	**13**
Traffic distribution	7	11	4	6	9	**37**	**5**
HMA CTC	13	15	14	14	3	**59**	**12**
Initial IRI	13	15	14	14	1	**57**	**11**

Table 125 Massachusetts Level 3 Results

MASSACHUSETTS LEVEL 3										
Input Variable	**Bottom-Up**		**Top-Down**		**AC Rutting**		**Total Rutting**		**IRI**	
	Value	Rank	Value	Rank	Value	Rank	Value	Rank	Value	Rank
HMA thickness	0.106	1	1.465	1	0.153	7	0.261	4	0.034	8
HMA mix gradation	0.013	7	0.289	11	0.235	6	0.134	8	0.018	10
HMA air voids	0.086	2	0.942	3	0.133	8	0.075	11	0.010	13
HMA effective binder content	0.013	7	0.103	12	0.071	10	0.042	13	0.005	15
HMA binder grade	0.033	4	0.902	5	0.755	2	0.406	2	0.053	5
Base type/modulus	0.026	5	0.360	9	0.087	9	0.101	9	0.013	11
Subgrade type/modulus	0.013	7	0.623	6	0.046	13	0.202	7	0.084	3
Ground water table	0.013	7	0.349	10	0.061	12	0.096	10	0.012	12
WT with weakest subgrade	0.020	6	0.015	14	0.046	13	0.063	12	0.008	14
Climate	0.026	5	0.506	7	0.469	3	0.235	5	0.039	7
AADTT value	0.026	5	0.411	8	0.332	5	0.207	6	0.027	9
Operational speed	0.046	3	1.096	2	1.051	1	0.556	1	0.072	4
Traffic growth rate	0.007	8	0.069	13	0.066	11	0.040	14	0.005	15
Traffic distribution	0.086	2	0.908	4	0.429	4	0.289	3	0.044	6
HMA CTC	0.000	9	0.000	15	0.000	14	0.000	15	0.087	2
Initial IRI	0.000	9	0.000	15	0.000	14	0.000	15	0.588	1

Table 126 MA Ranking Summary of Significance of Each Input Parameter on the Performance of Flexible Pavement

MASSACHUSETTS LEVEL 3							
Input Variable	**Bottom-Up Rank**	**Top-Down Rank**	**AC Rutting Rank**	**Total Rutting Rank**	**IRI Rank**	**Total Ranking Points**	**Overall Order of Significance**
HMA thickness	1	1	7	4	8	**21**	**4**
HMA mix gradation	7	11	6	8	10	**42**	**9**
HMA air voids	2	3	8	11	13	**37**	**8**
HMA effective binder content	7	12	10	13	15	**57**	**14**
HMA binder grade	4	5	2	2	5	**18**	**2**
Base type/modulus	5	9	9	9	11	**43**	**10**
Subgrade type/modulus	7	6	13	7	3	**36**	**7**
Ground water table	7	10	12	10	12	**51**	**11**
WT with weakest subgrade	6	14	13	12	14	**59**	**15**
Climate	5	7	3	5	7	**27**	**5**
AADTT value	5	8	5	6	9	**33**	**6**
Operational speed	3	2	1	1	4	**11**	**1**
Traffic growth rate	8	13	11	14	15	**61**	**16**
Traffic distribution	2	4	4	3	6	**19**	**3**
HMA CTC	9	15	14	15	2	**55**	**13**
Initial IRI	9	15	14	15	1	**54**	**12**

6.1 Interpretation of Results – Graphical Method

The results obtained from the runs were interpreted graphically to obtain a relationship between each input variable used in the study and each predicted pavement distress. A few general observations made from the results are as follows:

- The trends for all the input parameters and their effects on predicted pavement distresses are the same for all the states studied, and are similar to the performance expected from theoretical explanation
- Level 2 analysis resulted in predicted performance that is lower than Level 3 values, with similar trends being repeated for each of the input variables considered
- The magnitude of variability is different for different states studied due to differences in pavement layer structure, traffic and environmental conditions and the material properties used in the asphalt and unbound layers
- Pavement distresses exceeded the pre-defined failure limits only in the case of New Hampshire, and were very much below the limits for Connecticut, Maine and Rhode Island
- The AC overlay over fractured JPCP structure selected for the state of Rhode Island based on LTPP data returned zero predicted distress values for bottom-up and longitudinal cracking. The reason for zero prediction of cracking could result from a very high modulus fractured concrete course underlying the asphalt concrete layer.
- Massachusetts data is implemented in a case study using an experimental design – response surface methodology to study two-factor interactions, which is a significant improvement over the currently studied single factor effects on predicted performance

The following sections explain the effect of individual input parameters on the pavement distresses and roughness. The graphs depict the predicted pavement distresses plotted on the Y-axis (dependent variable) versus time, over the entire design life of the pavement.

6.1.1 Effect of Traffic Inputs on Pavement Distresses

The traffic inputs which were considered in this study are

- Annual Average Daily Truck Traffic (AADTT)
- Traffic Growth Rate
- Truck Class Distribution
- Traffic Operational Speed

Traffic load distribution spectra were not studied due to unavailability of data sources for the purpose.

Annual Average Daily Truck Traffic (AADTT)

The predicted pavement distresses increase with an increase in the AADTT. AADTT was not considered as a factor while studying thermal cracking, which is not a load-associated phenomenon. Longitudinal cracking, bottom-up cracking, rutting and roughness increase with an increase in AADTT. The sensitivity of the distresses to AADTT can be explained graphically as follows.

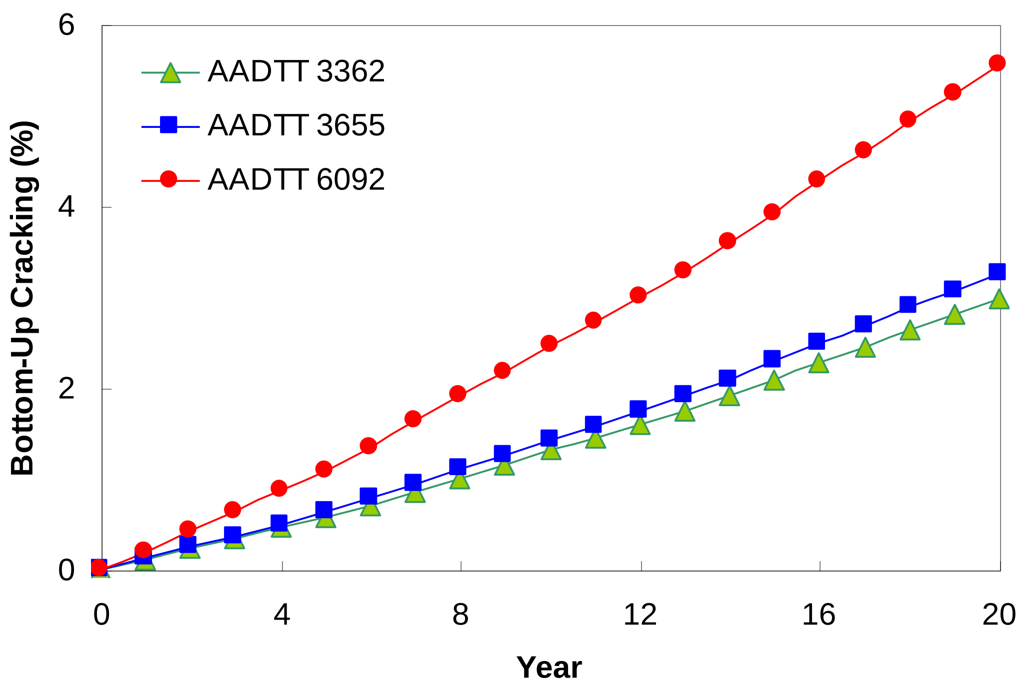

Figure 28 Effect of AADTT on Bottom-Up Cracking – New Hampshire Level 3

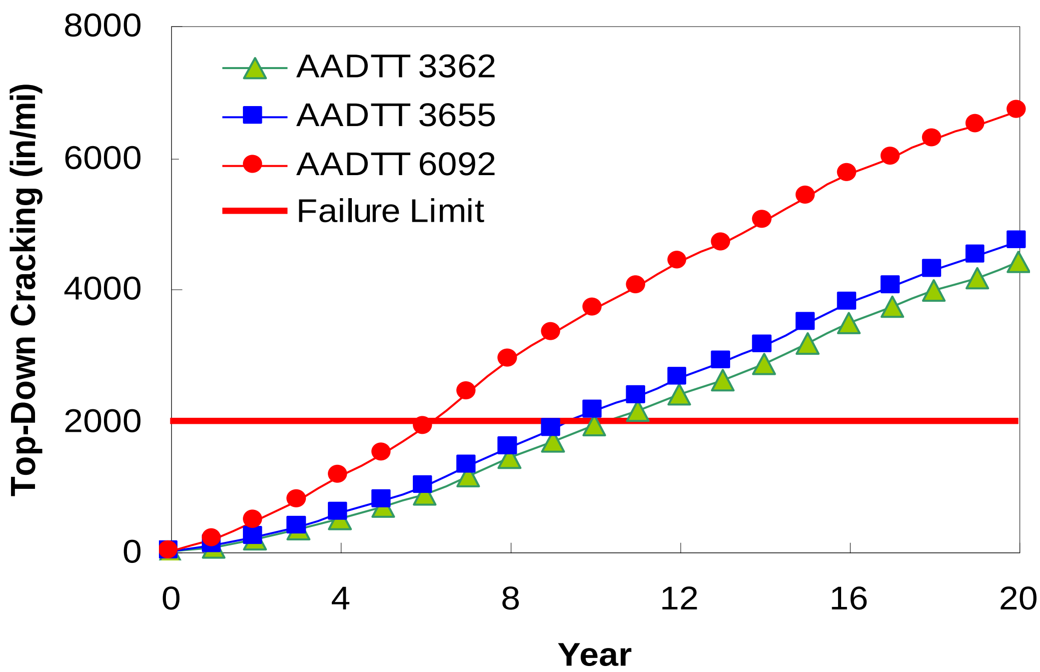

Figure 29 Effect of AADTT on Top-Down Cracking – New Hampshire Level 3

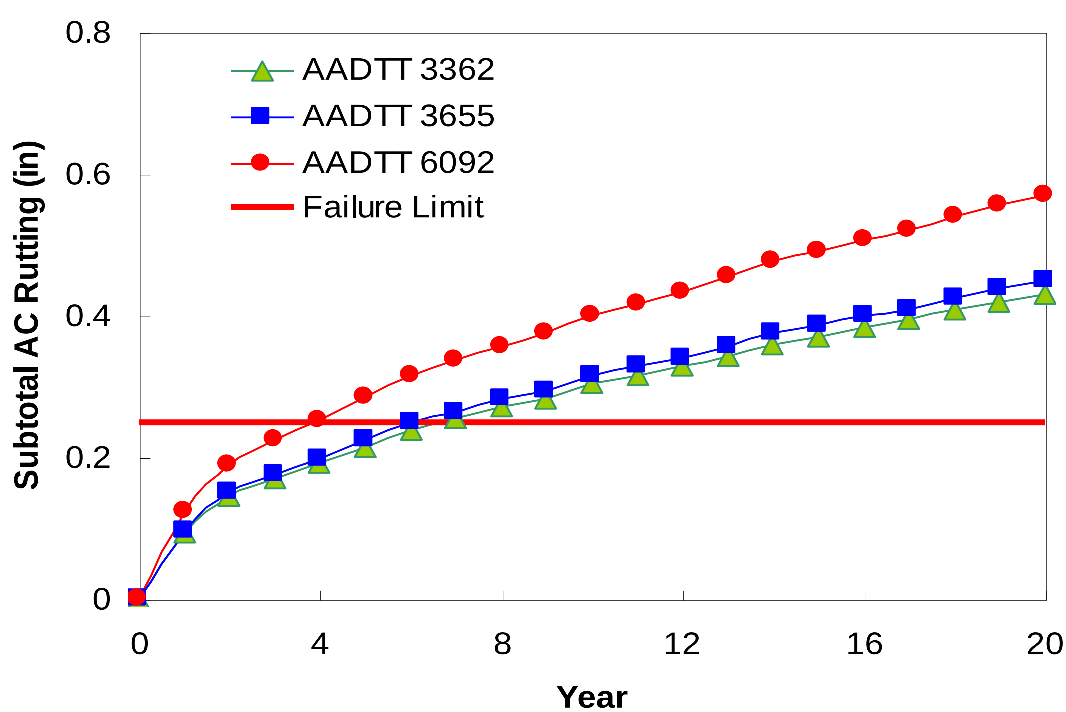

Figure 30 Effect of AADTT on Subtotal AC Rutting – New Hampshire Level 3

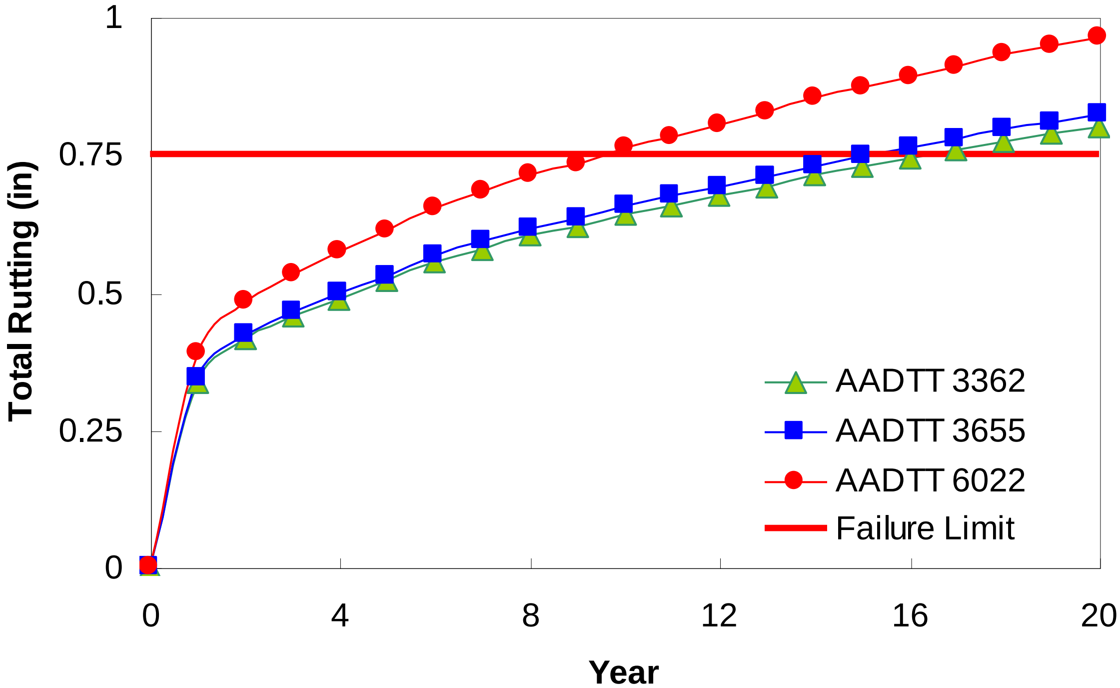

Figure 31 Effect of AADTT on Total Rutting – New Hampshire Level 3

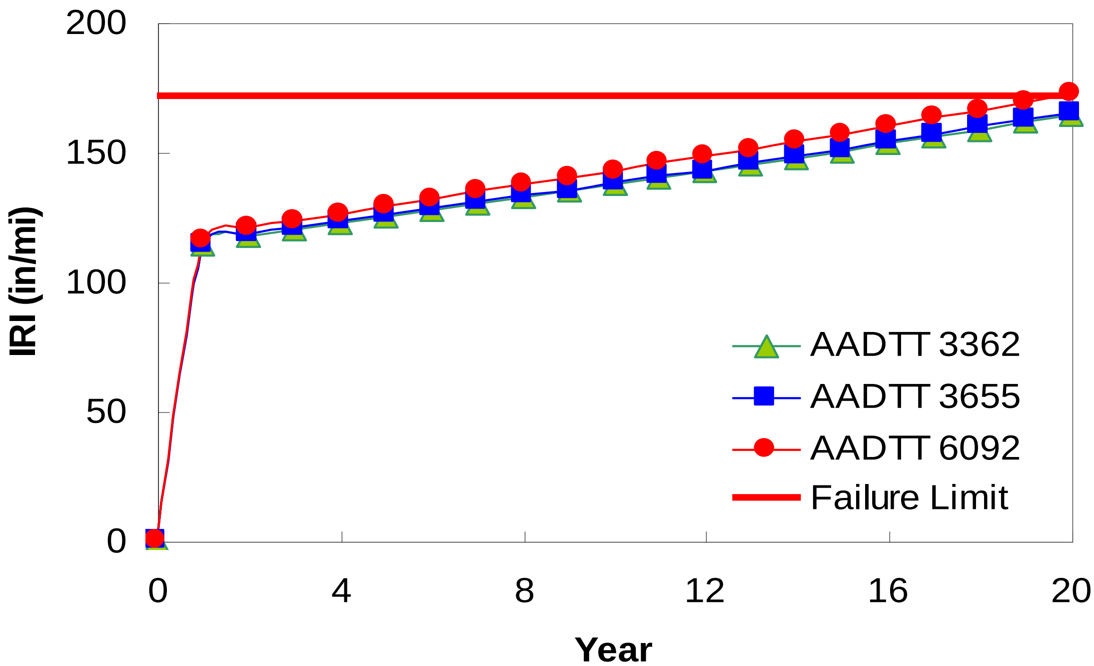

Figure 32 Effect of AADTT on IRI – New Hampshire Level 3

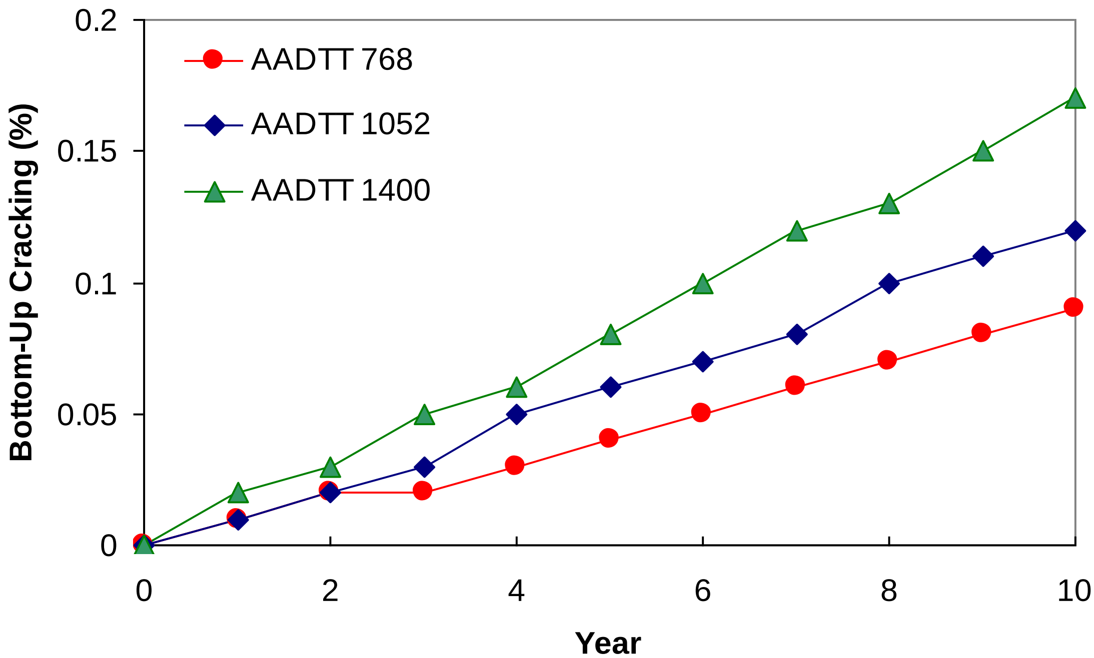

Figure 33 Effect of AADTT on Bottom-Up Cracking – Connecticut Level 3

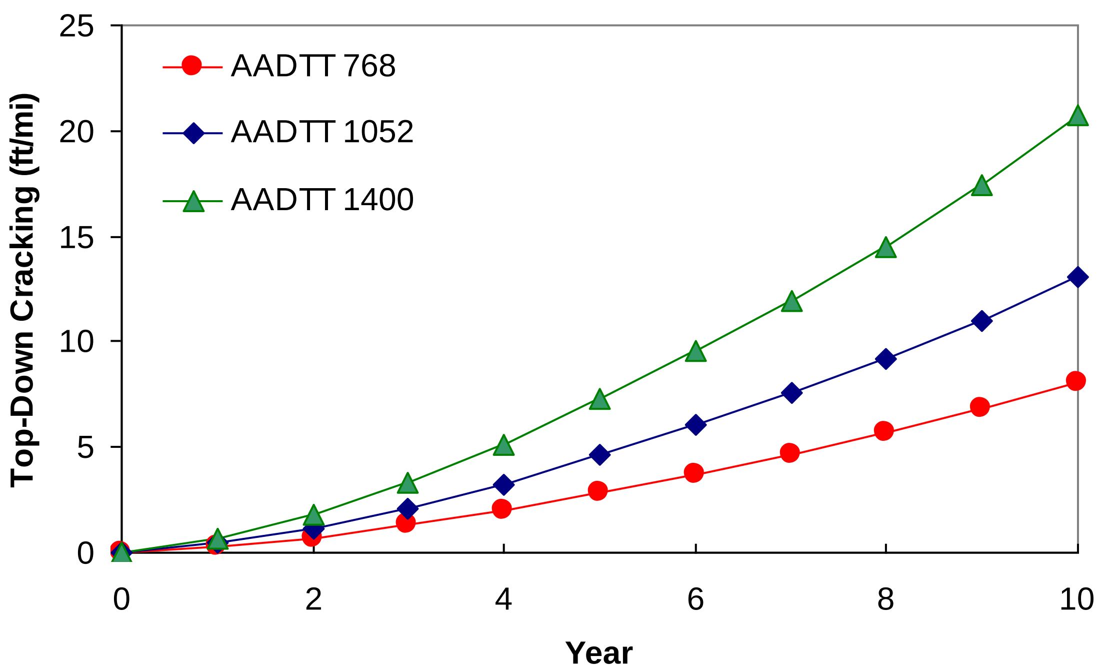

Figure 34 Effect of AADTT on Top-Down Cracking – Connecticut Level 3

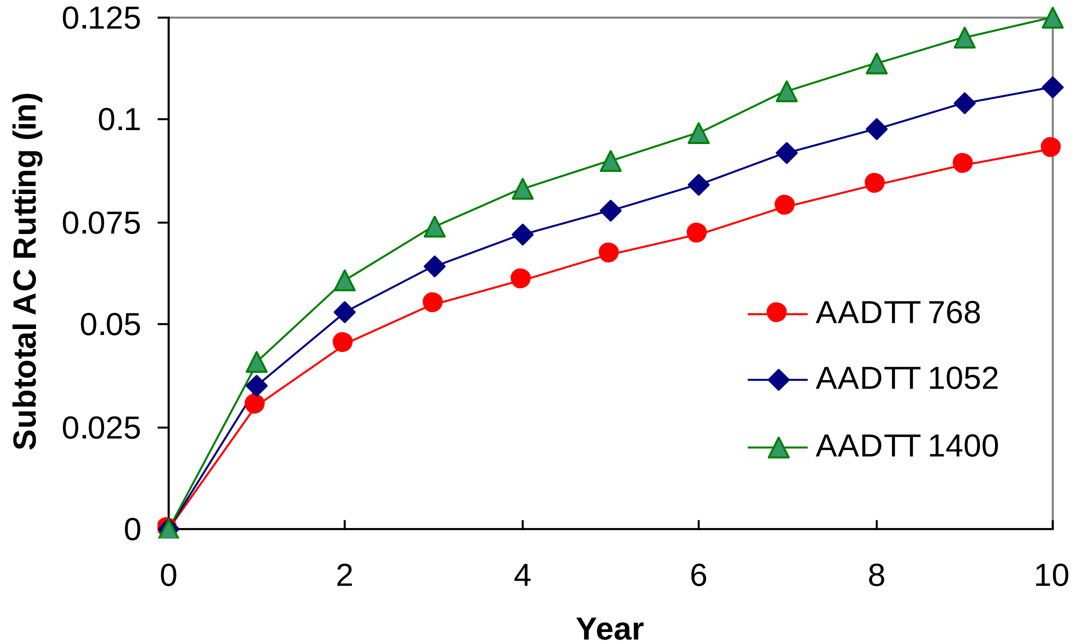

Figure 35 Effect of AADTT on Subtotal AC Rutting – Connecticut Level 3

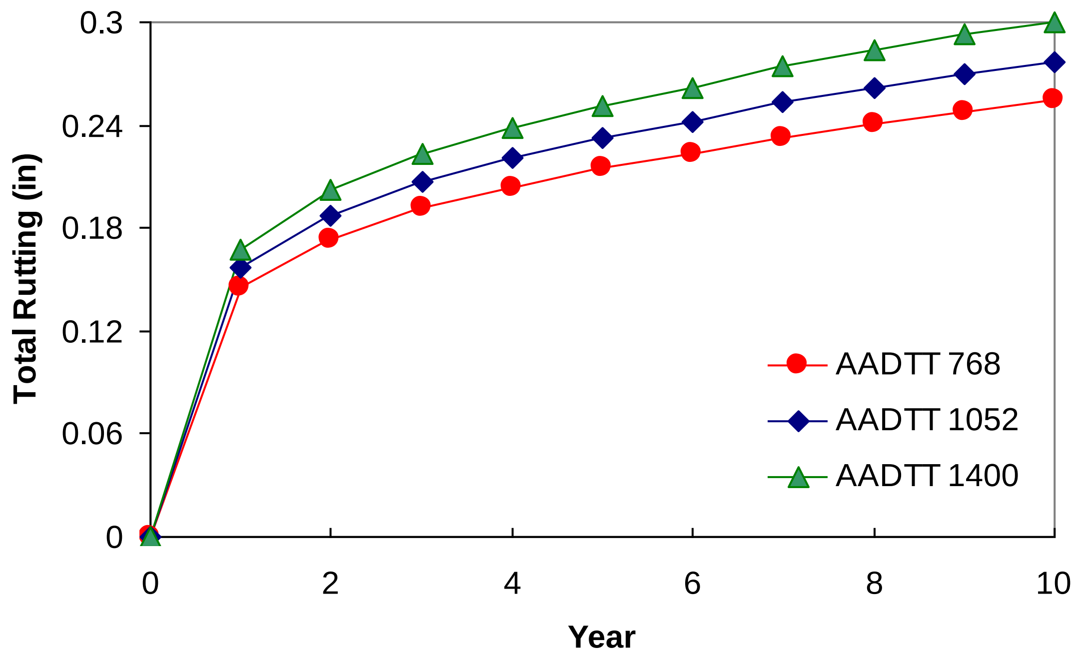

Figure 36 Effect of AADTT on Total Rutting – Connecticut Level 3

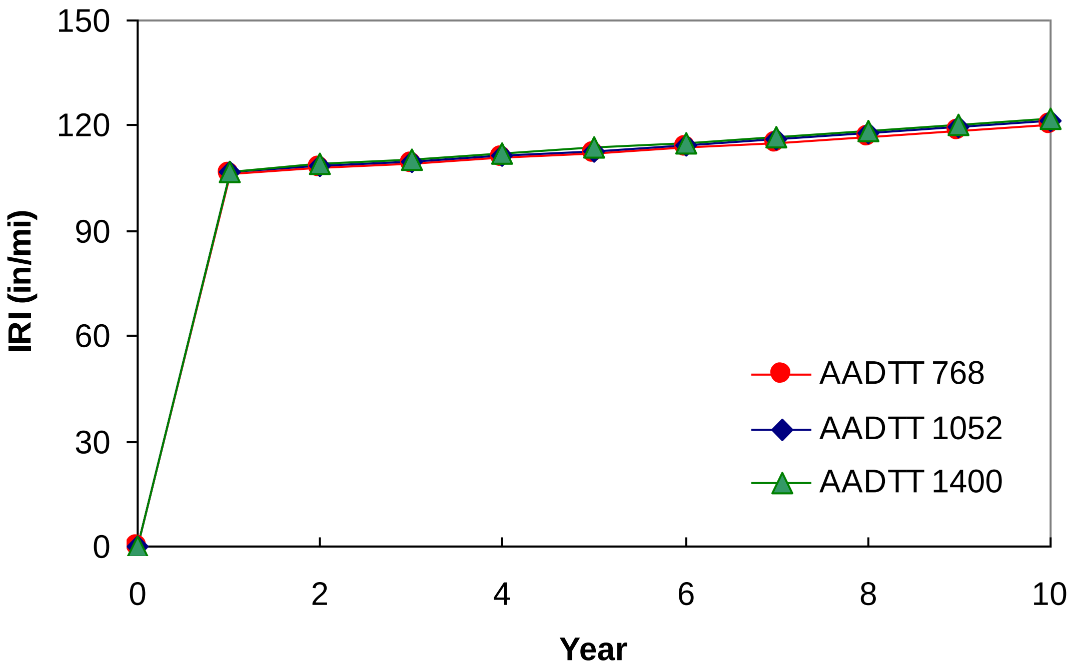

Figure 37 Effect of AADTT on IRI – Connecticut Level 3

The predicted distresses are higher for a higher AADTT as compared to a lower AADTT. However, at higher AADTT levels, the predicted distresses increase more slowly as compared to those at lower AADTT levels. Therefore, it can be inferred from the study that pavements which handle lower truck traffic volumes should be designed with a more precise estimate of the design AADTT than for higher volumes. Failure periods obtained for New Hampshire pavement structure (Table 111) do not show much variation for all distresses for the lower and median AADTT values, therefore extreme caution need not be exercised for estimating the AADTT value at high truck traffic volumes.

The results for Level 2 analysis also show similar trends and the predicted distresses can be considered a more conservative prediction of the pavement performance compared to Level 3. The following graphs show the performance trends over the design life of the pavement for New Hampshire, Maine, Vermont, New York and Massachusetts pavement structures. Inference can be made from Figure 38 – Figure 67 that the variation of predicted distresses with AADTT follows the same trend with increase in magnitude despite difference in pavement structures.

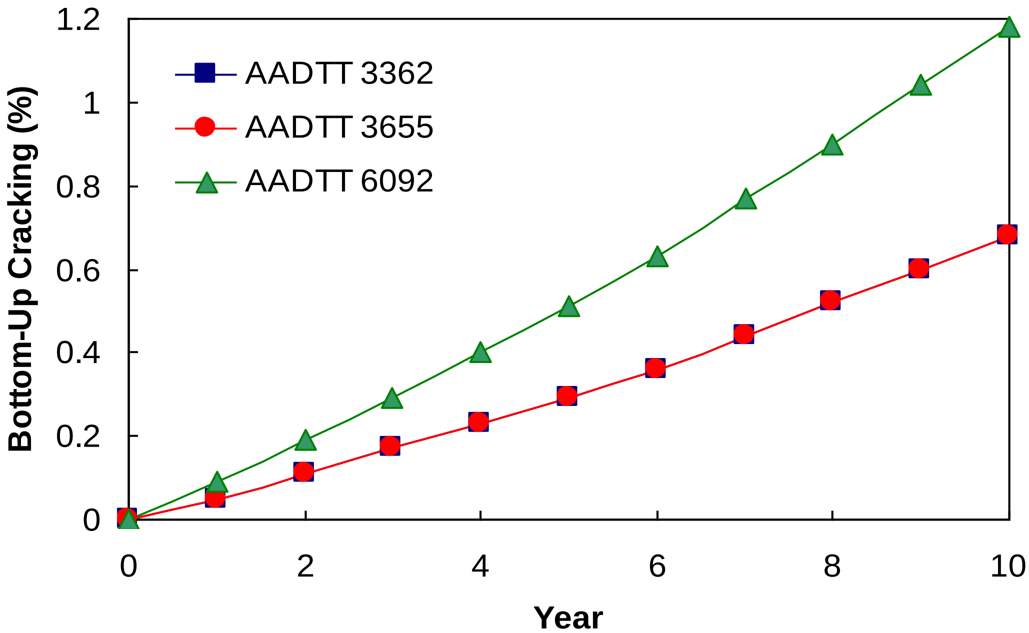

Figure 38 Effect of AADTT on Bottom-Up Cracking – New Hampshire Level 2

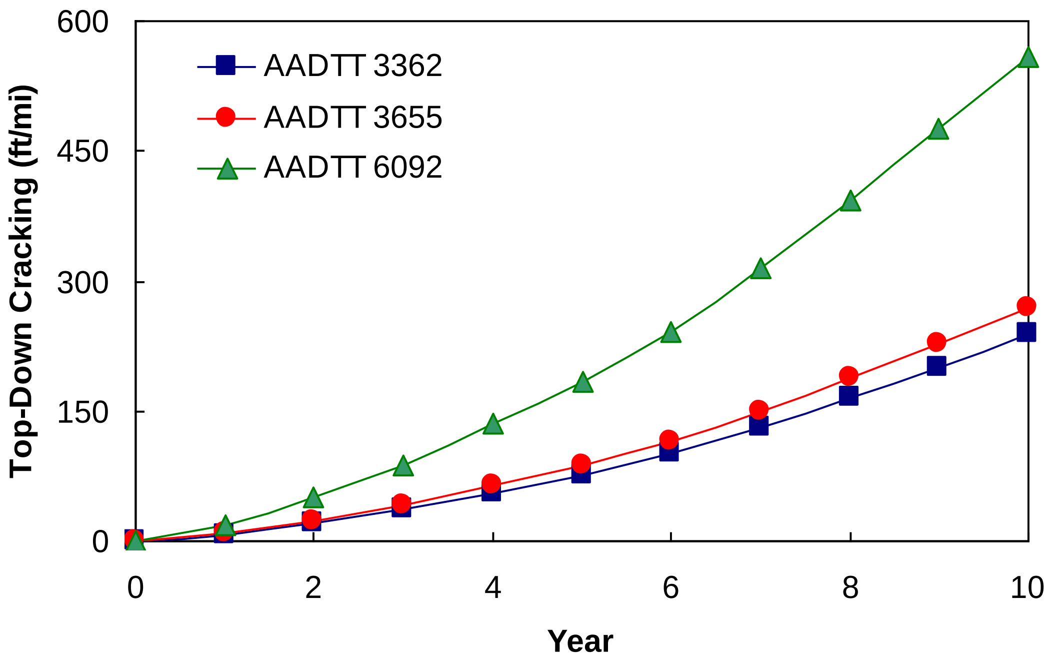

Figure 39 Effect of AADTT on Top-Down Cracking – New Hampshire Level 2

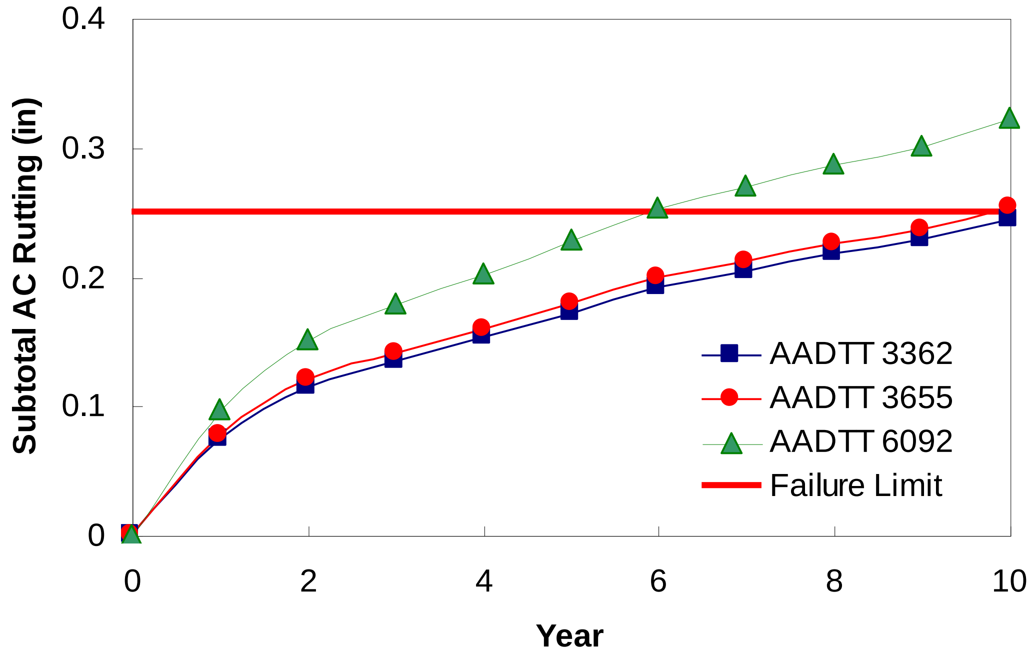

Figure 40 Effect of AADTT on Subtotal AC Rutting – New Hampshire Level 2

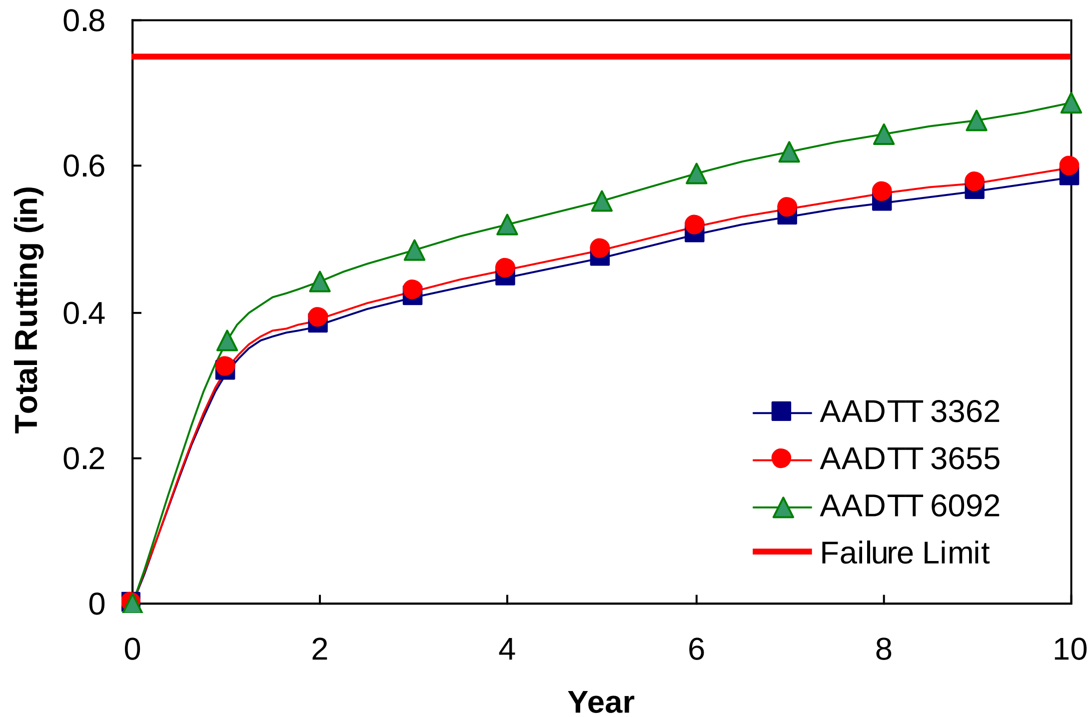

Figure 41 Effect of AADTT on Total Rutting – New Hampshire Level 2

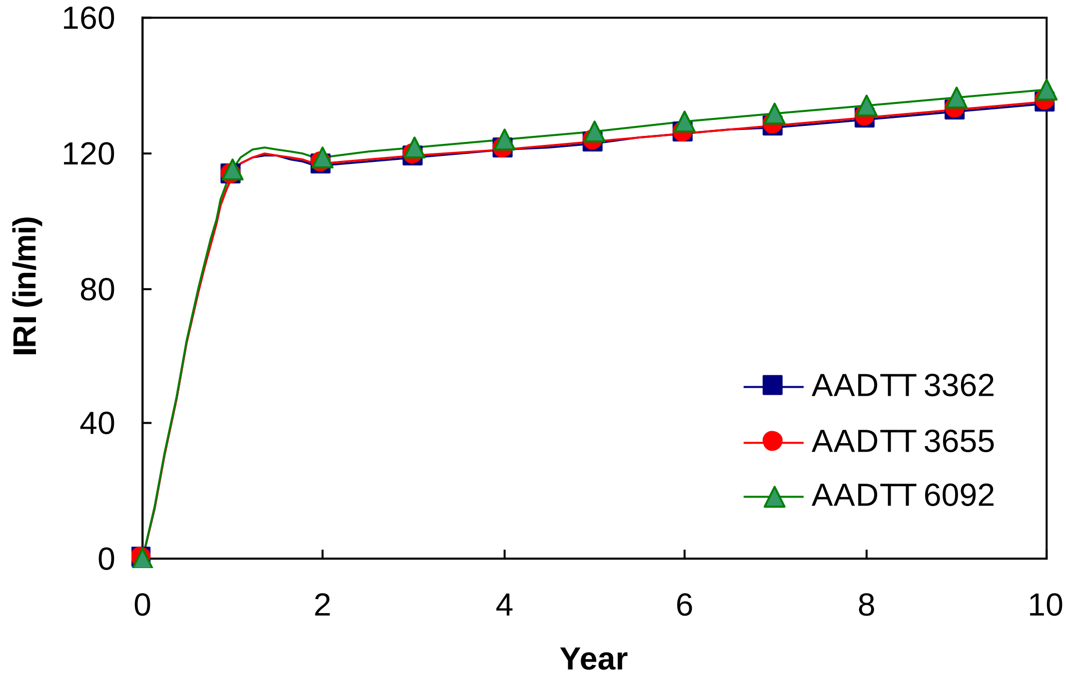

Figure 42 Effect of AADTT on IRI – New Hampshire Level 2

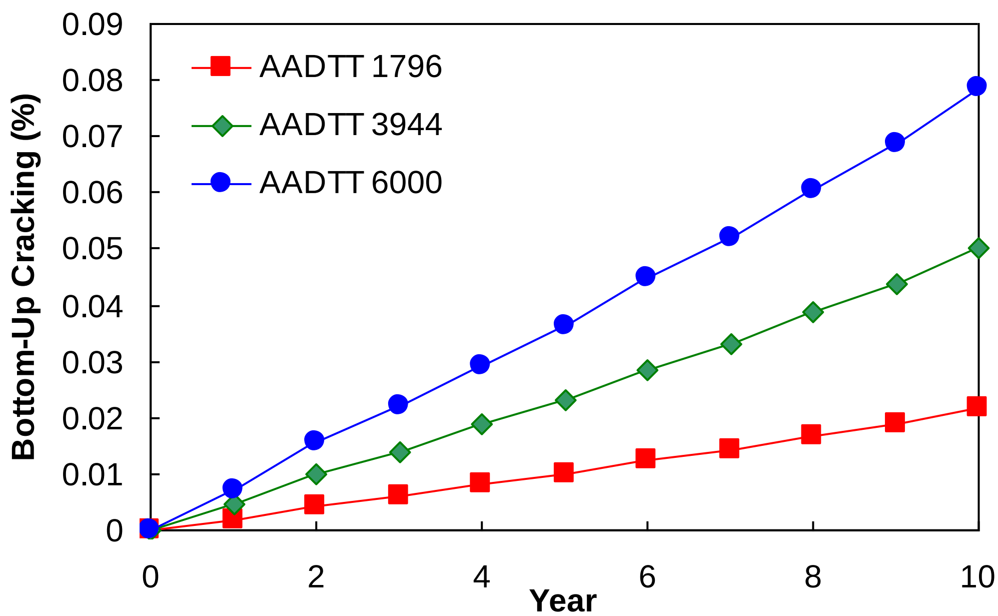

Figure 43 Effect of AADTT on Bottom-Up Cracking – Maine Level 2

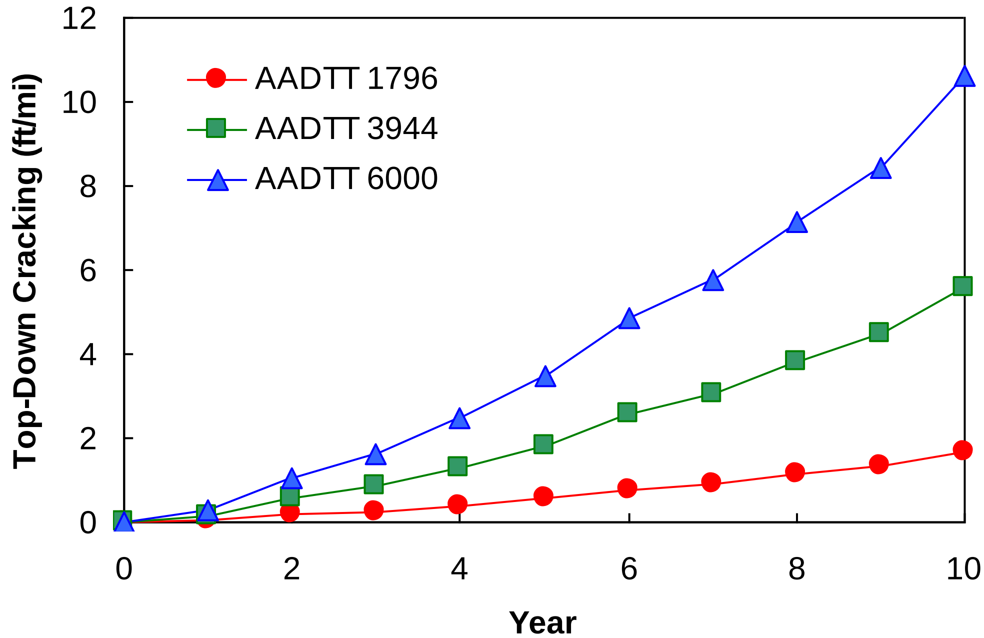

Figure 44 Effect of AADTT on Top-Down Cracking – Maine Level 2

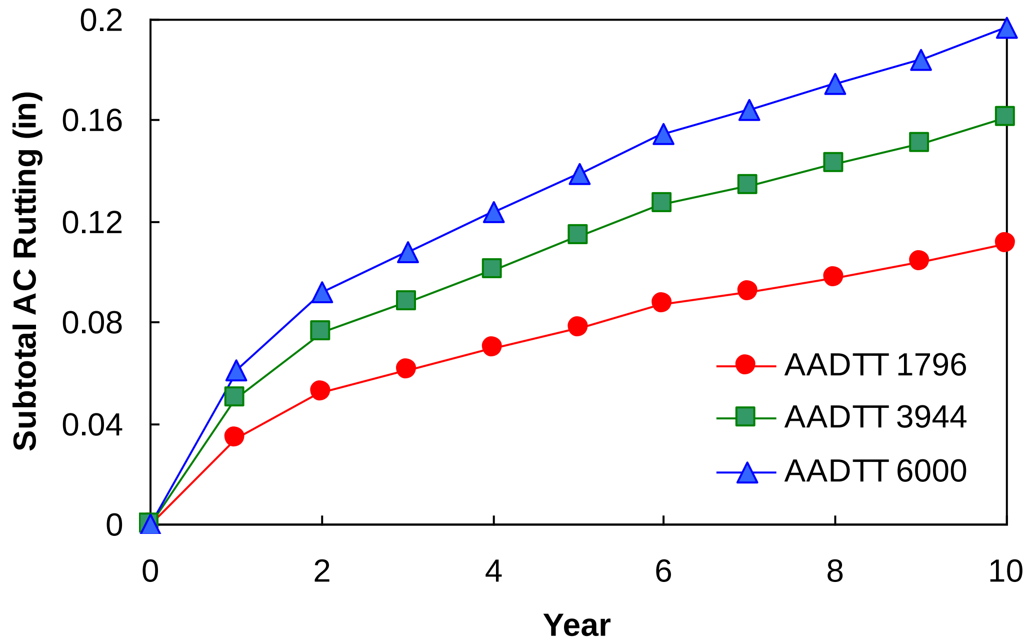

Figure 45 Effect of Subtotal AC Rutting – Maine Level 2

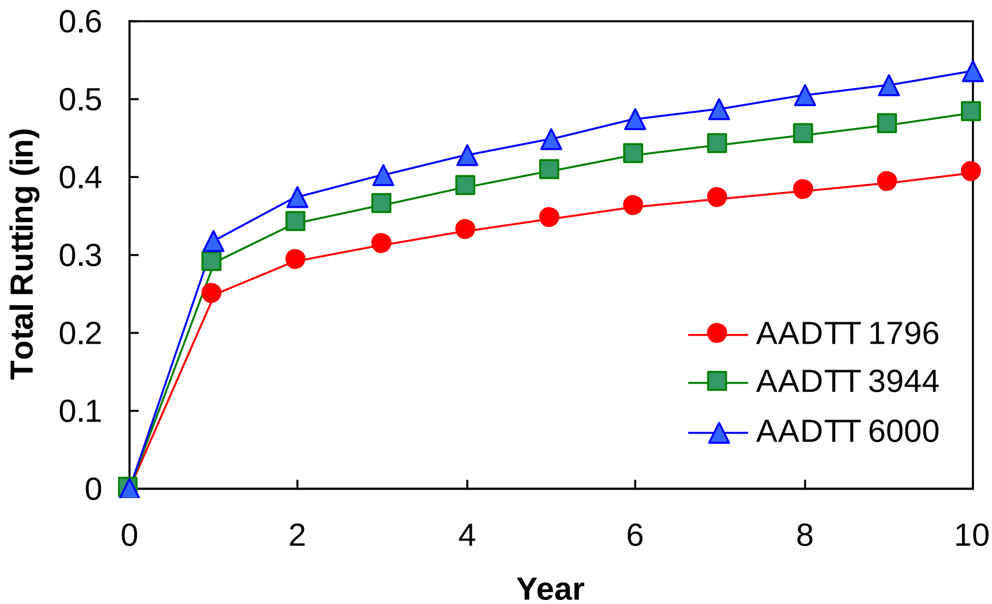

Figure 46 Effect of AADTT on Total Rutting – Maine Level 2

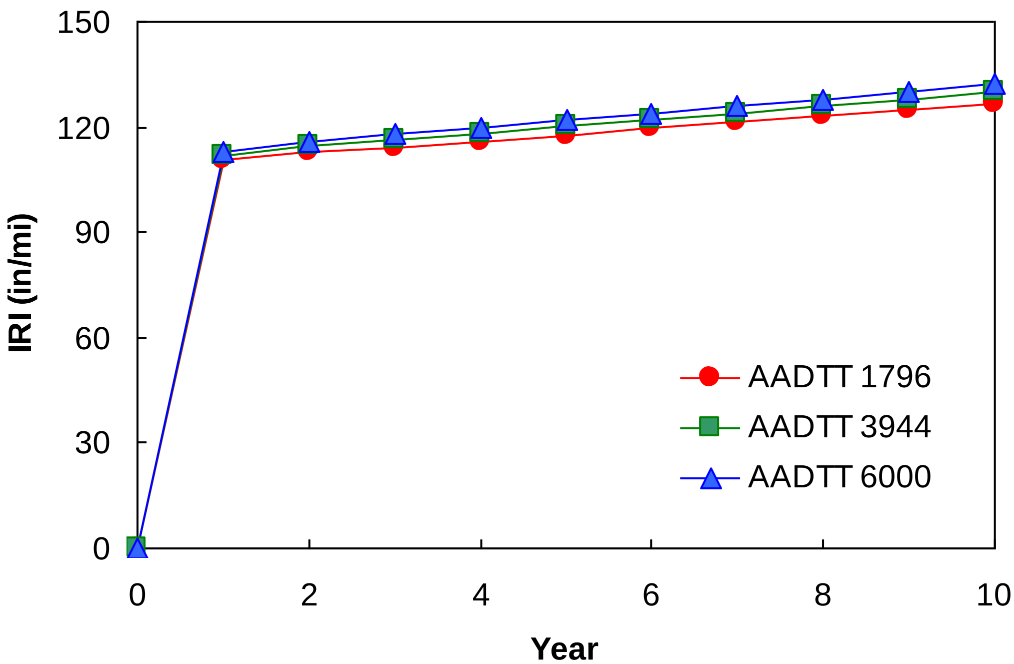

Figure 47 Effect of AADTT on IRI – Maine Level 2

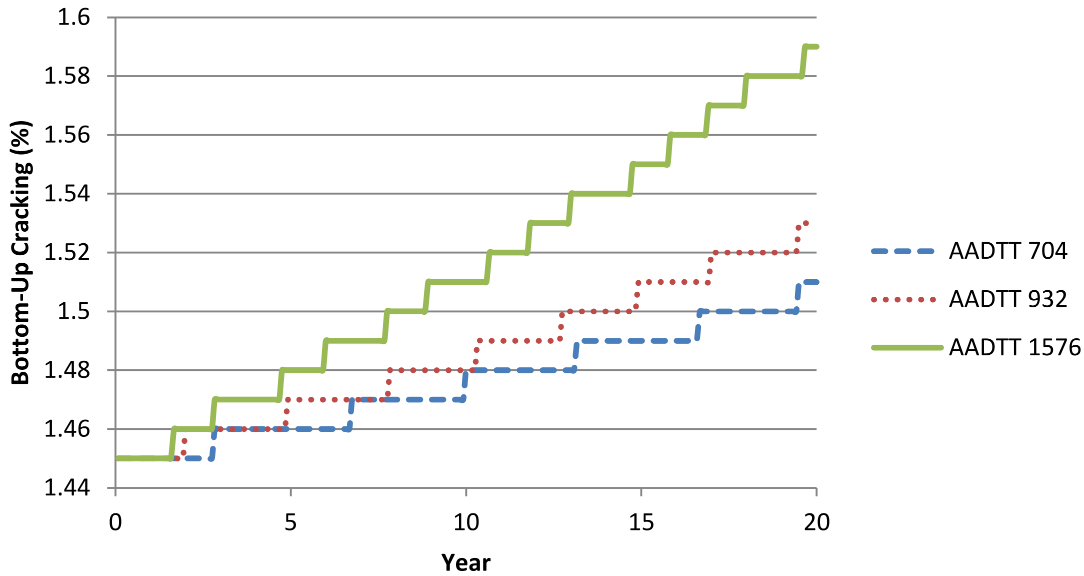

Figure 48 Effect of AADTT on Bottom-Up Cracking - Vermont level 3

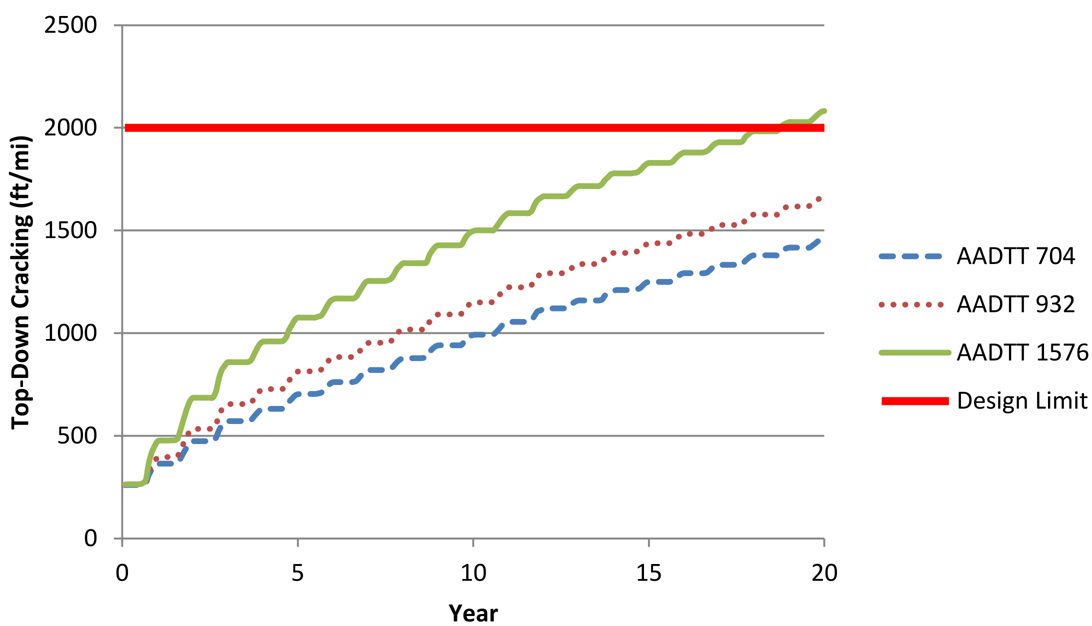

Figure 49 Effect of AADTT on Top-Down Cracking - Vermont level 3

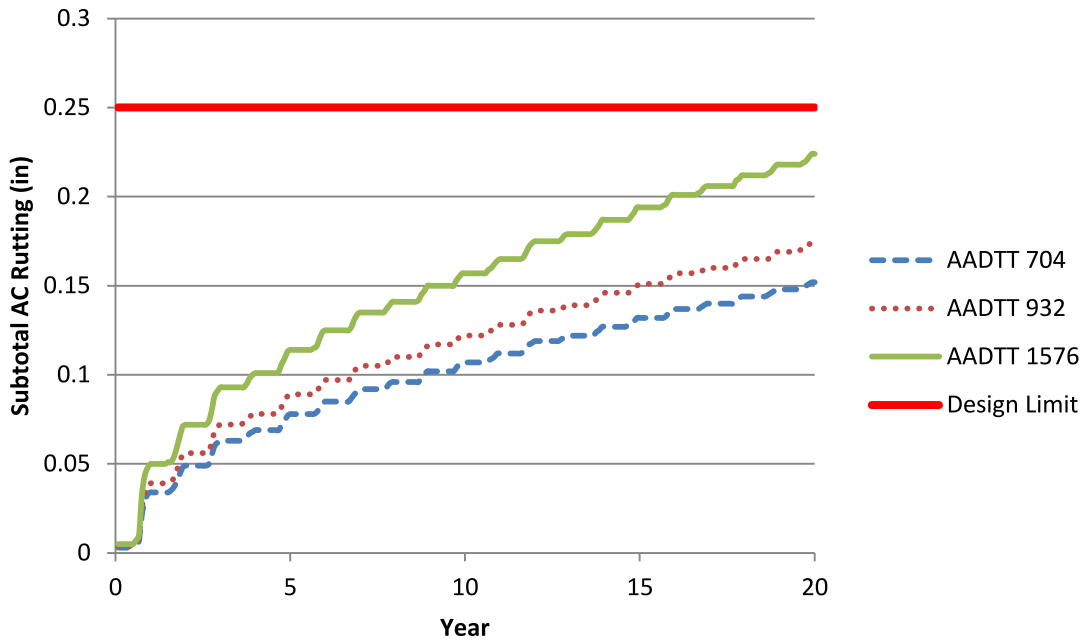

Figure 50 Effect of AADTT on Subtotal AC Rutting -Vermont level 3

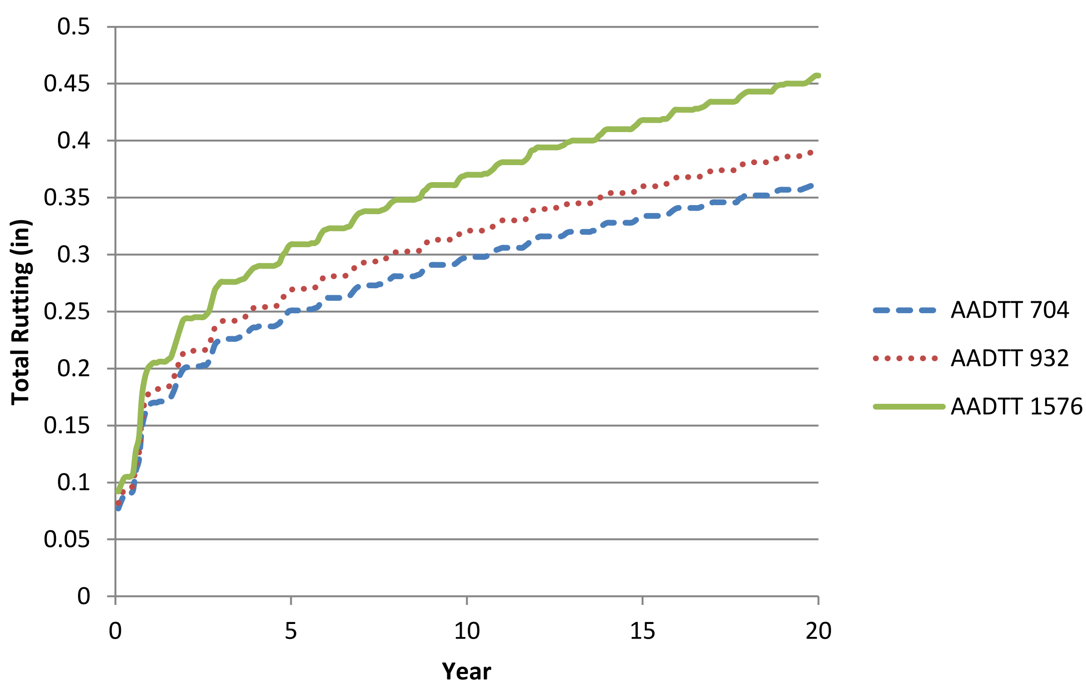

Figure 51 Effect of AADTT on Total Rutting - Vermont level 3

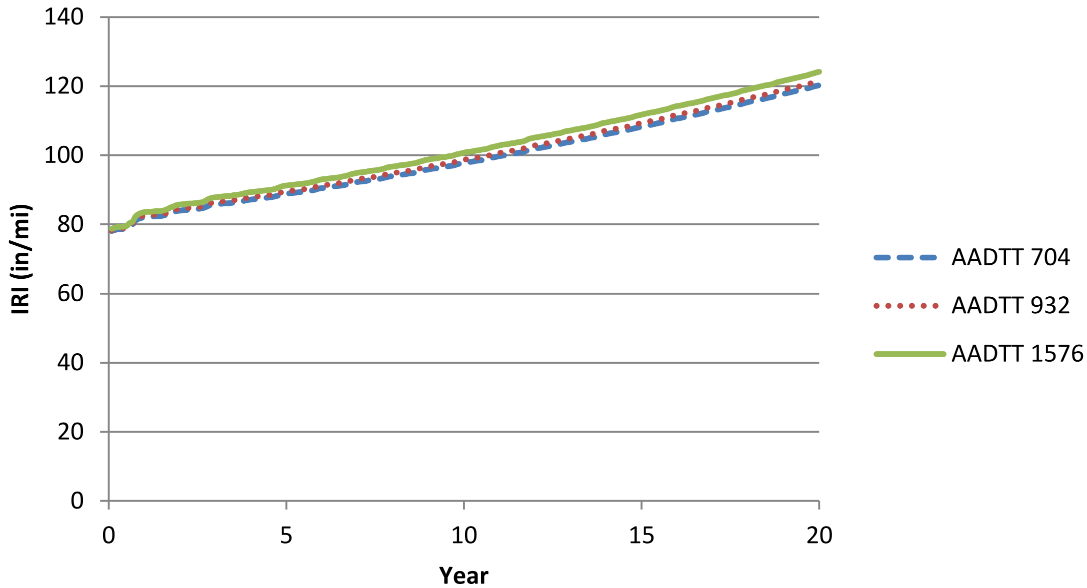

Figure 52 Effect of AADTT on IRI - Vermont level 3 - Vermont level 3

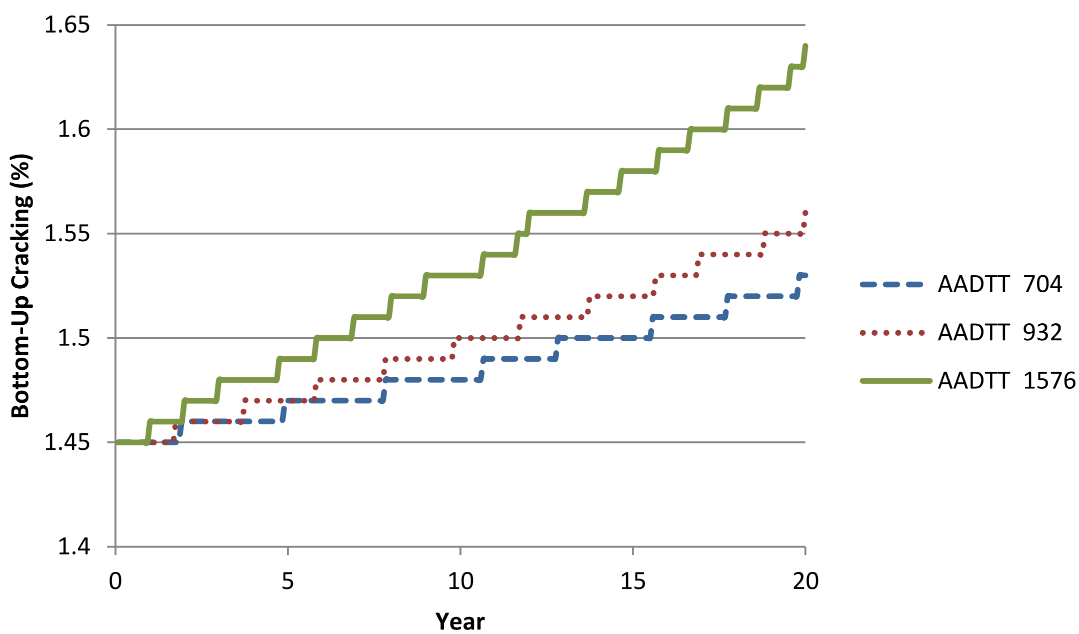

Figure 53 Effect of AADTT on Bottom-Up Cracking - Vermont level 2

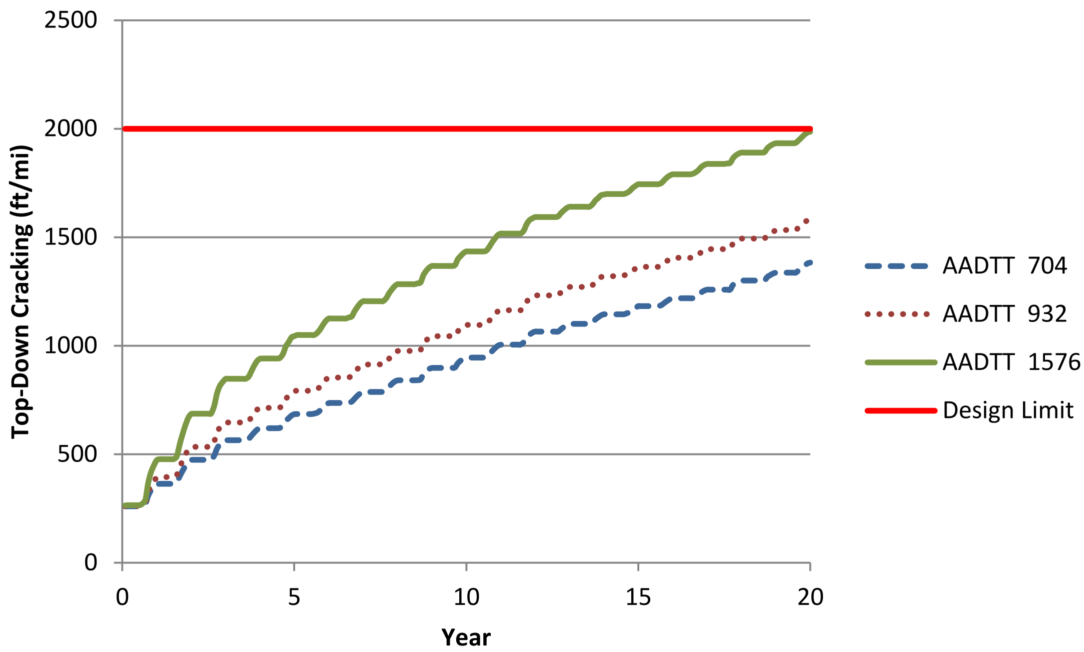

Figure 54 Effect of AADTT on Top-Down Cracking - Vermont level 2

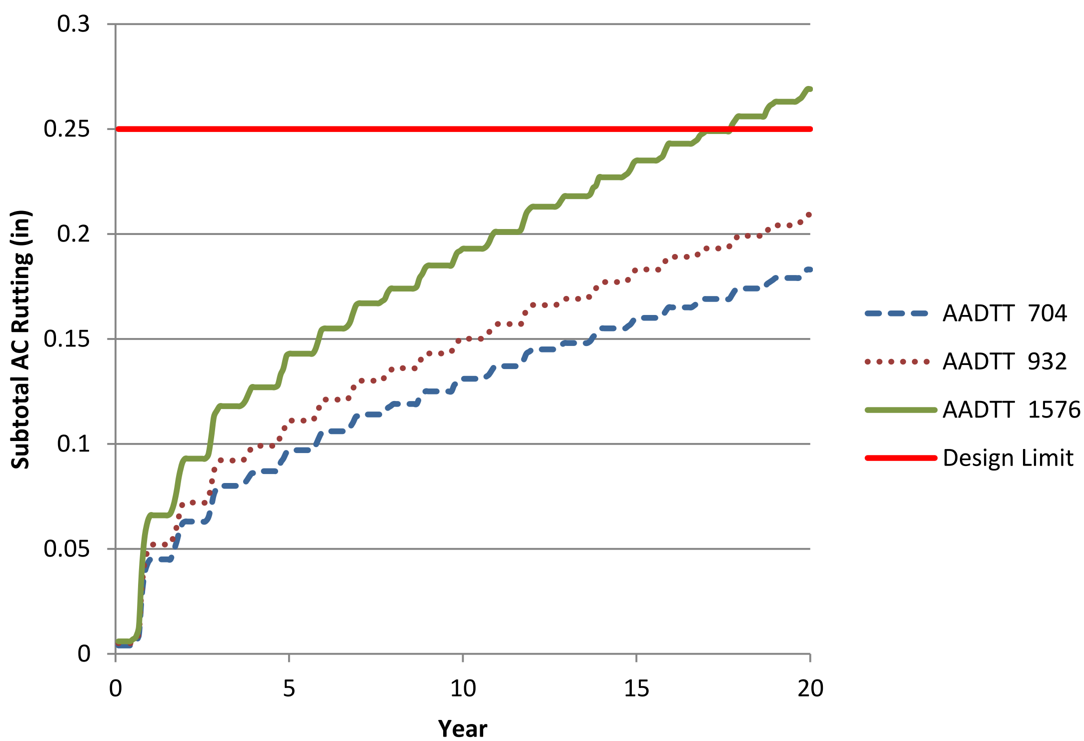

Figure 55 Effect of AADTT on Subtotal AC Rutting - Vermont level 2

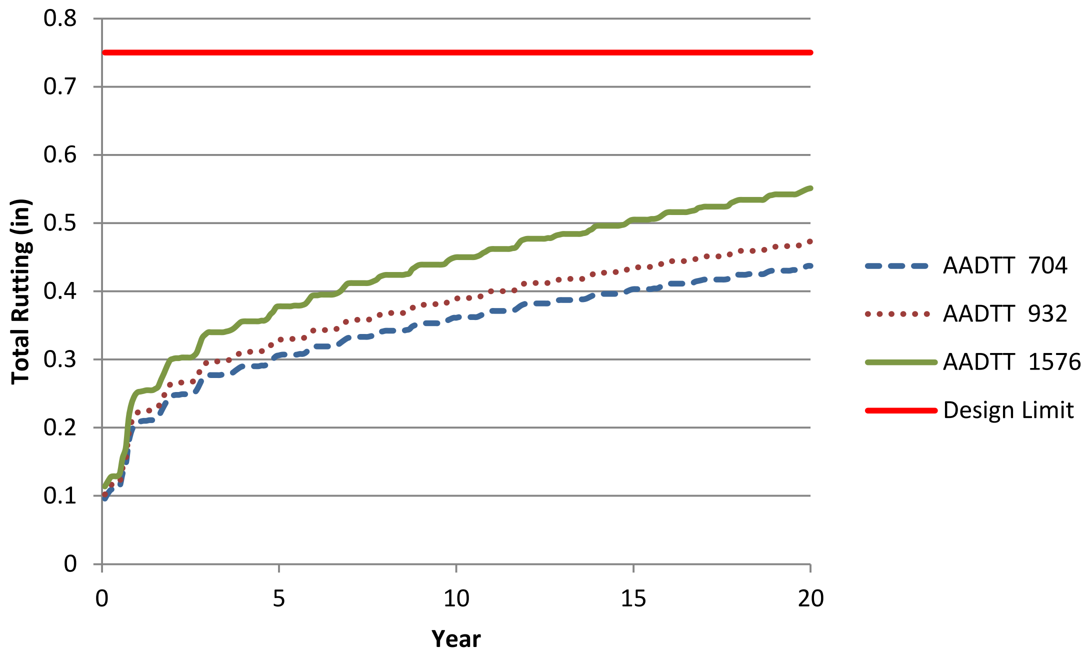

Figure 56 Effect of AADTT on Total Rutting - Vermont level 2

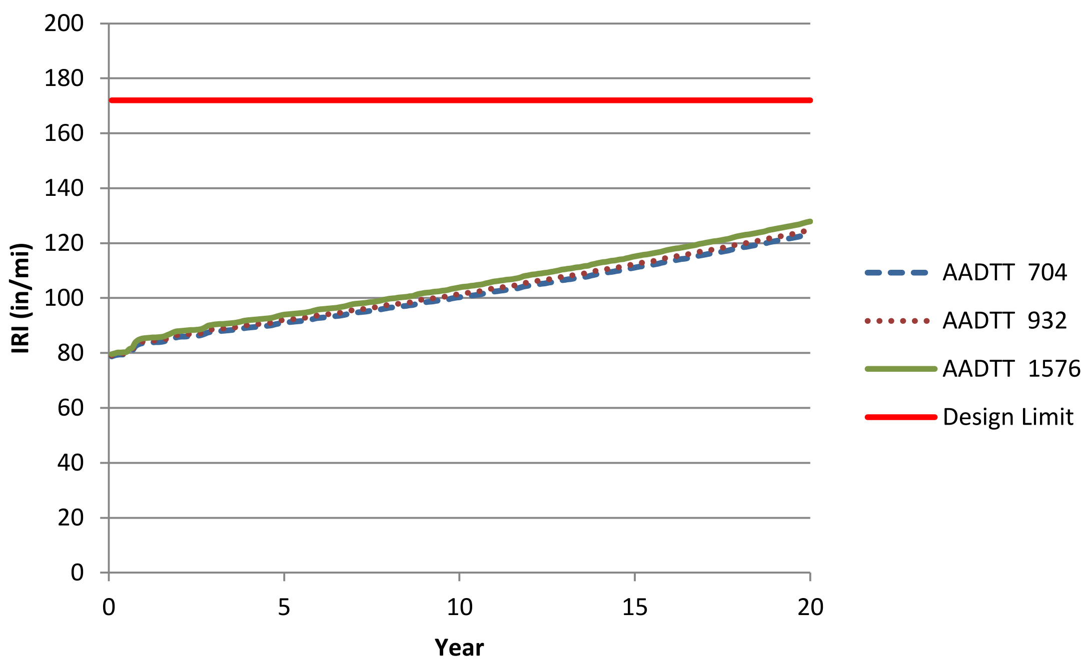

Figure 57 Effect of AADTT on IRI - Vermont level 2

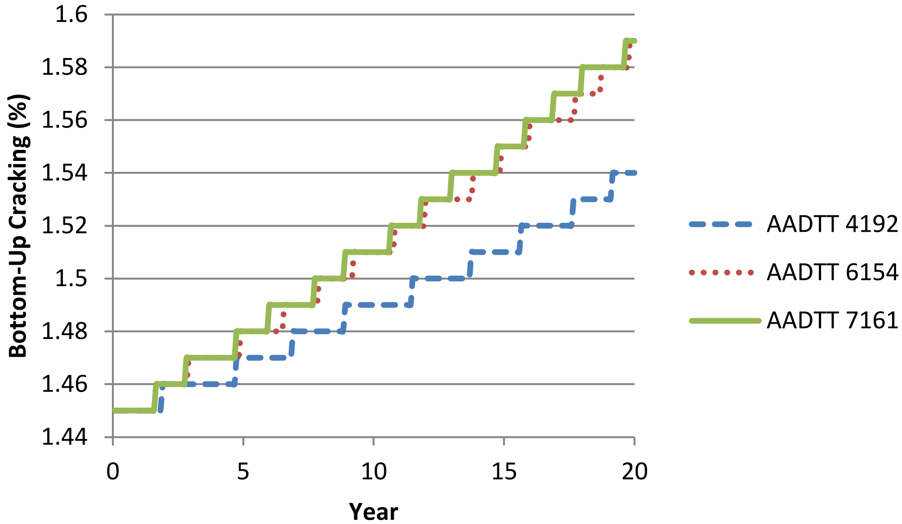

Figure 58 Effect of AADTT on Bottom-Up Cracking - NY Level 3

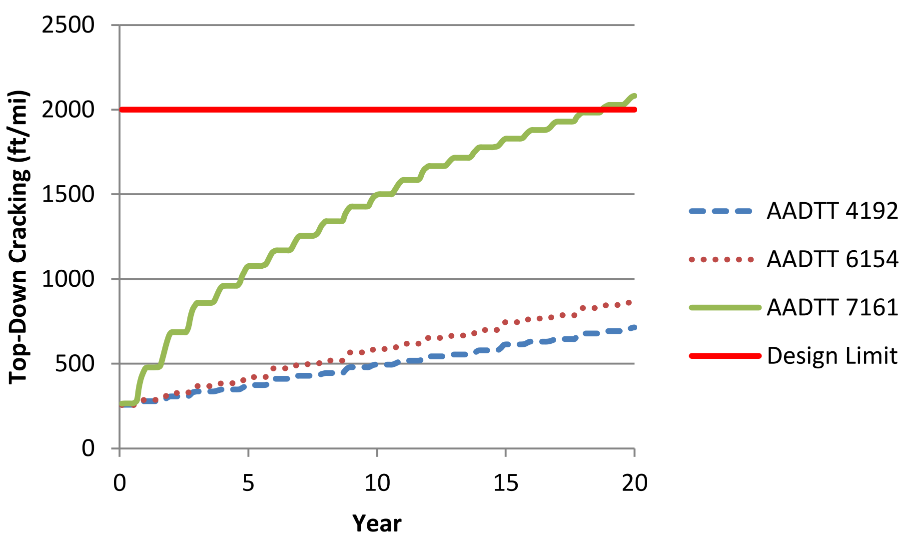

Figure 59 Effect of AADTT on Top-Down Cracking - NY Level 3

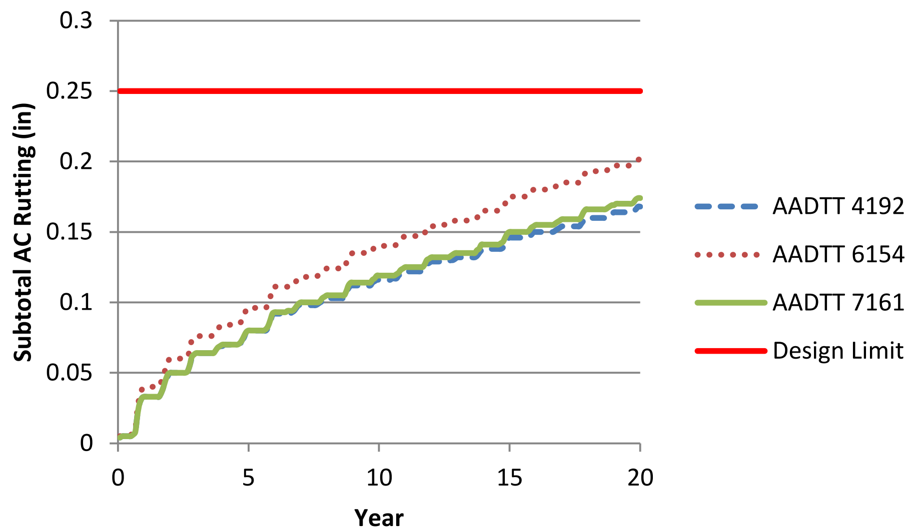

Figure 60 Effect of AADTT on Subtotal AC Rutting - NY Level 3

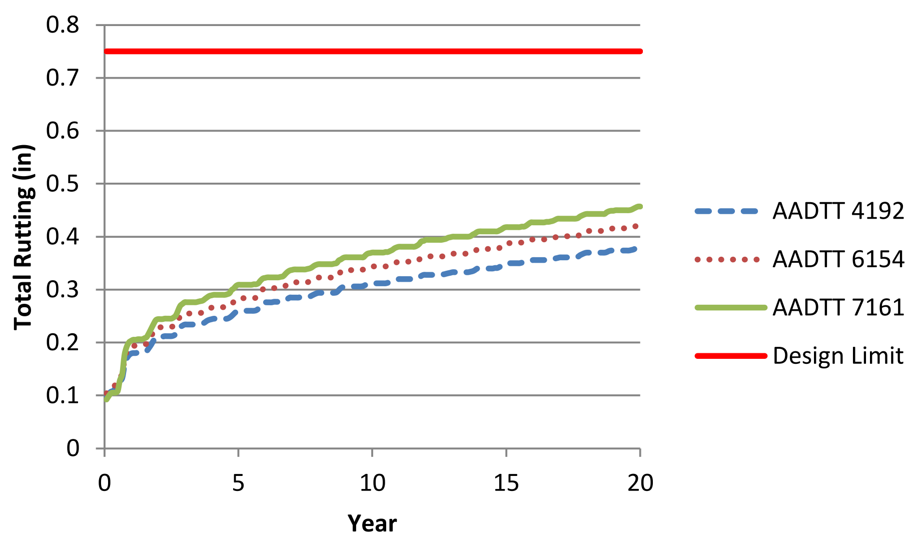

Figure 61 Effect of AADTT on Total Rutting - NY Level 3

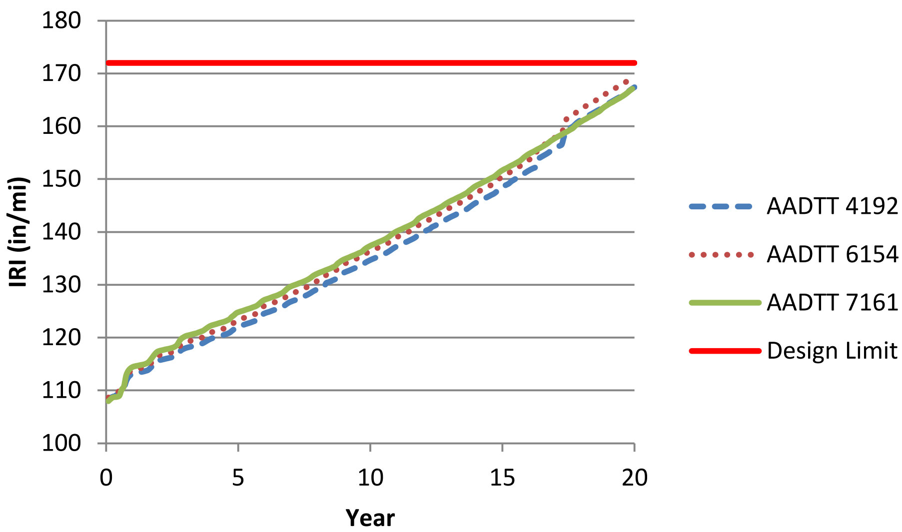

Figure 62 Effect of AADTT on IRI - NY Level 3

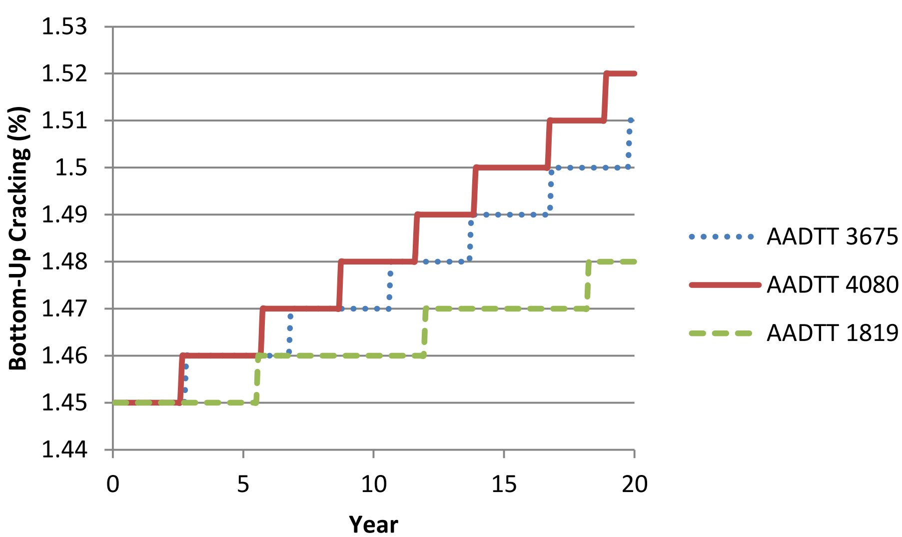

Figure 63 Effect of AADTT on Bottom-Up Cracking - Massachusetts Level 3

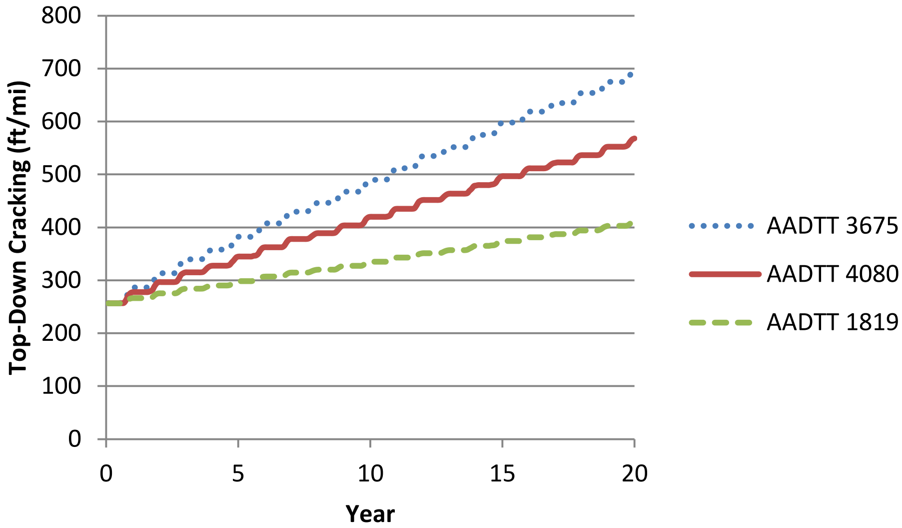

Figure 64 Effect of AADTT on Top-Down Cracking - Massachusetts Level 3

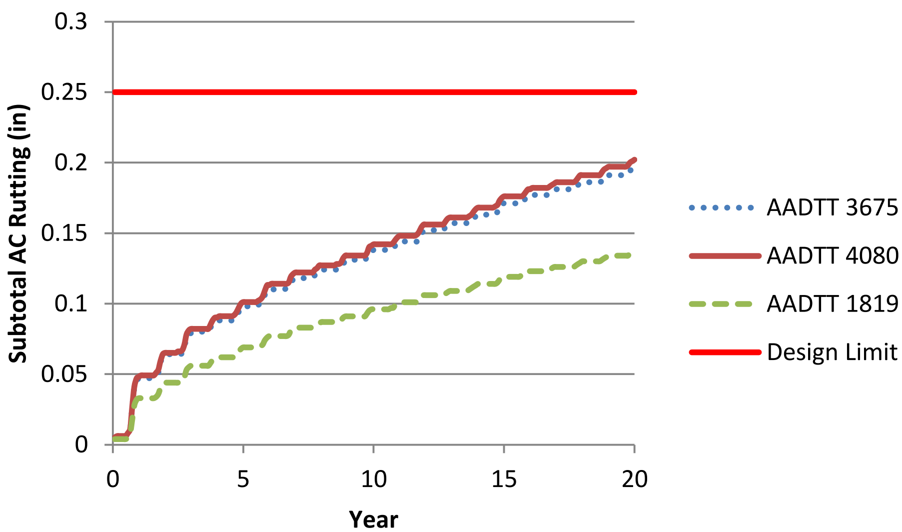

Figure 65 Effect of AADTT on Subtotal AC Rutting - Massachusetts Level 3

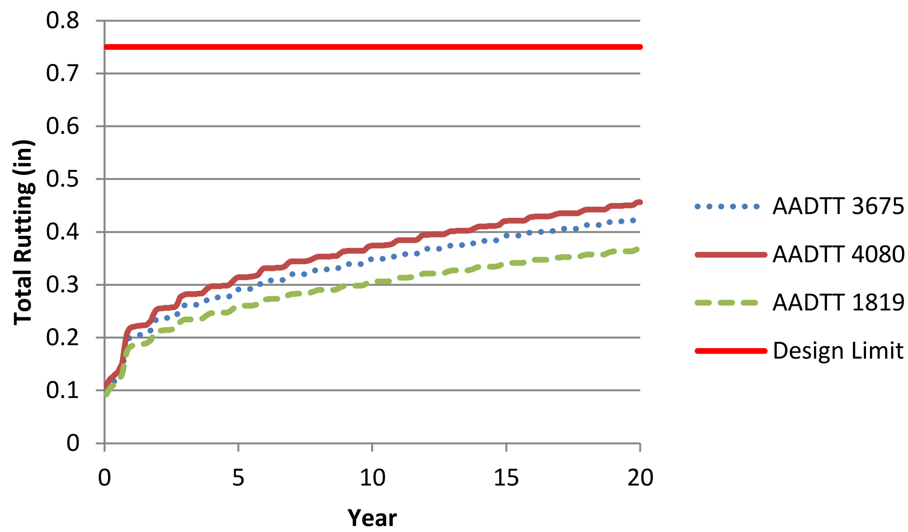

Figure 66 Effect of AADTT on Total Rutting - Massachusetts Level 3

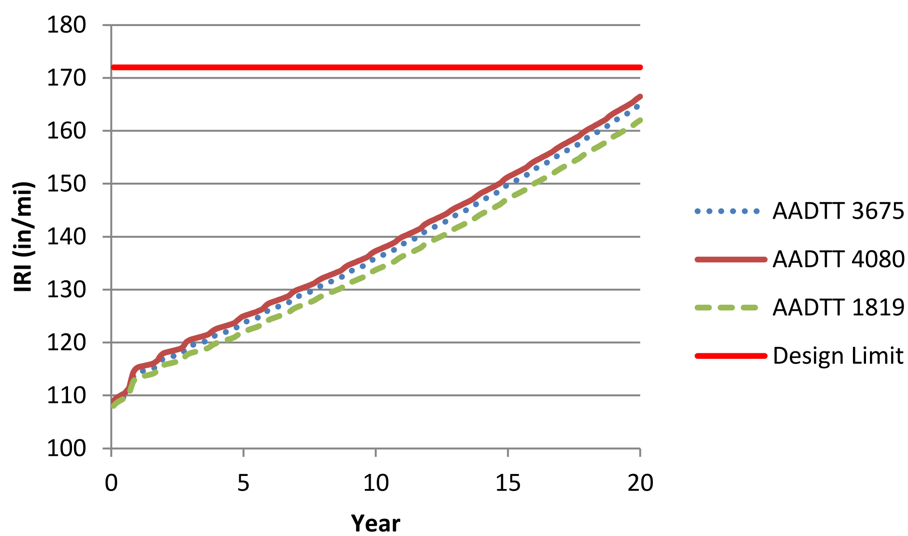

Figure 67 Effect of AADTT on IRI - Massachusetts Level 3

Rate of Growth of Traffic

Traffic growth rate was found to not influence the predicted distresses significantly. Graphically, this can be observed from the closeness of the performance prediction curves for different pavement structures. Truck volume plays a more significant role in the performance prediction rather than the actual growth of traffic. Traffic growth rate values used in the study have been obtained from actual pavement sections, and hence are representative of the values assumed for real road design. Therefore, an assumed traffic growth rate will suffice for low-significance roads, or growth rate can be calculated from traffic data of roads with similar structure, traffic and service level for achieving greater reliability in results.

The following graphs present the prediction of Level 3 bottom-up cracking, total rutting and roughness for the states of New Hampshire, Connecticut, Maine, Vermont, New York and Massachusetts. Level 2 results also showed no significant effect of traffic growth rate.

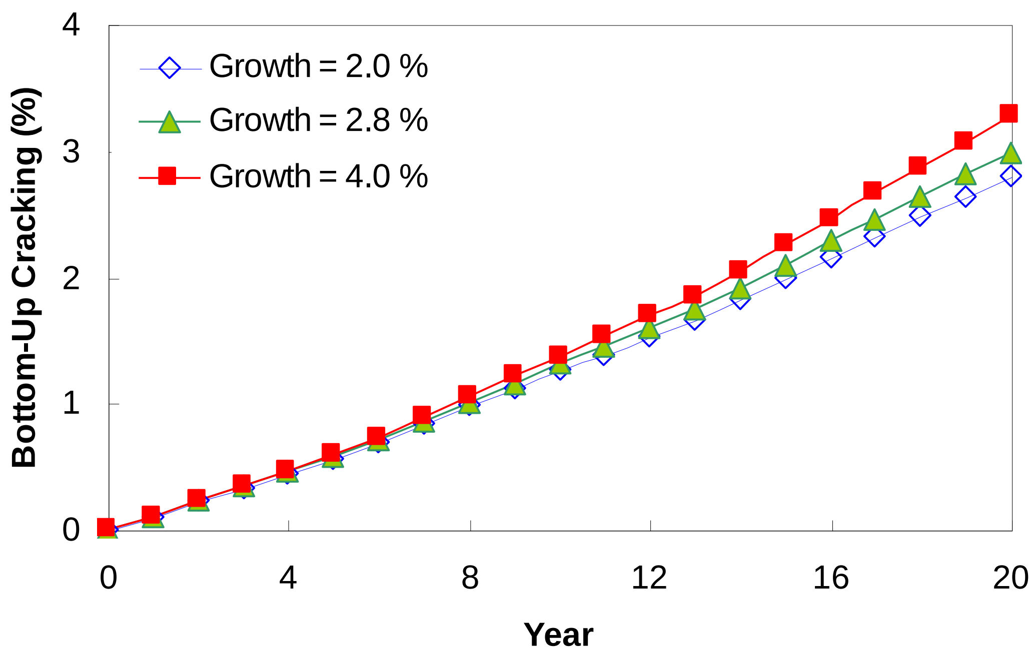

Figure 68 Effect of Traffic Growth Rate at Bottom-Up Cracking – New Hampshire

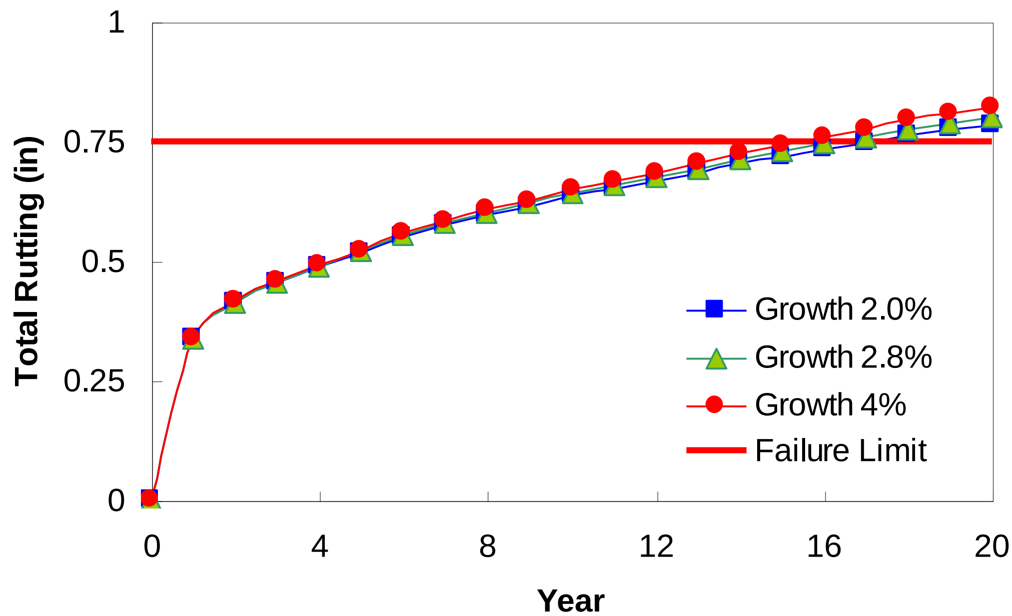

Figure 69 Effect of Traffic Growth Rate on Total Rutting – New Hampshire

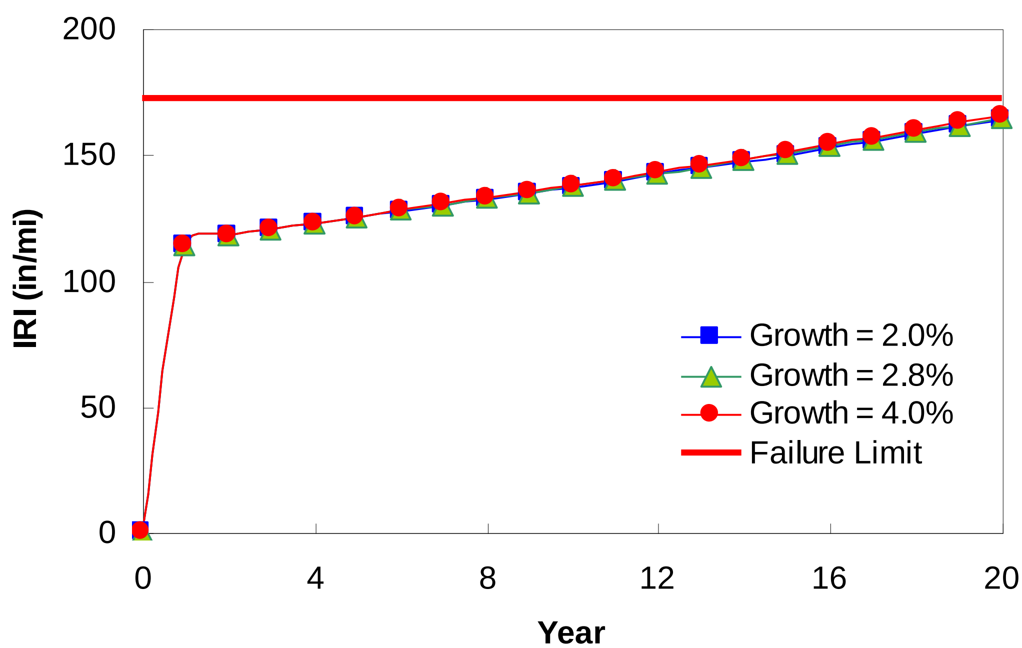

Figure 70 Effect of Traffic Growth Rate on IRI – New Hampshire

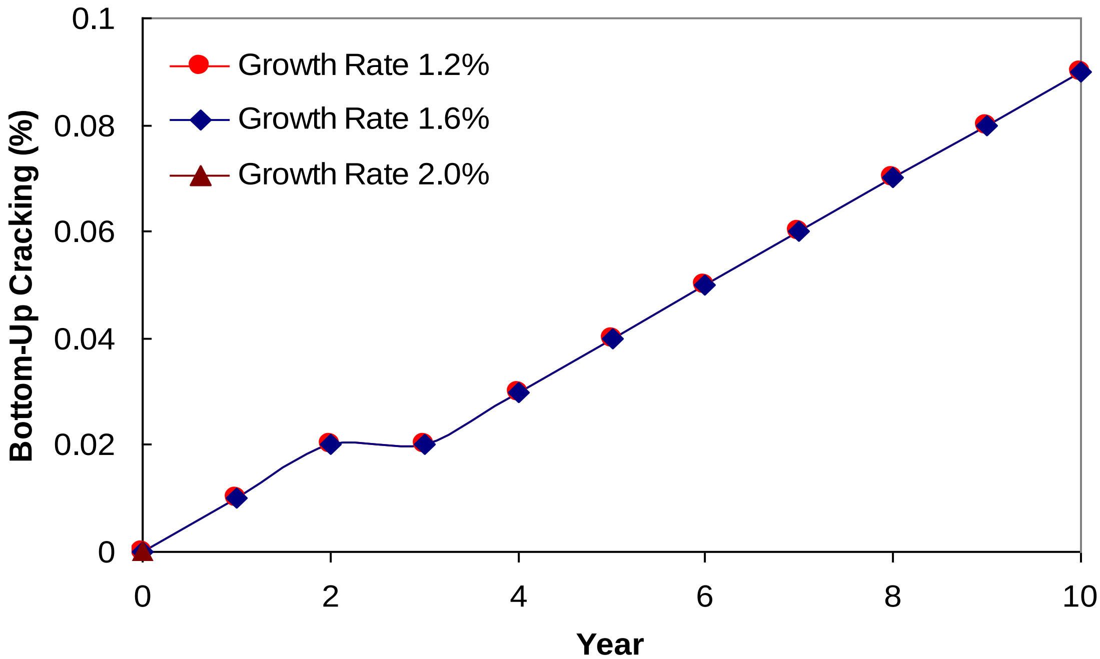

Figure 71 Effect of Traffic Growth Rate on Bottom-Up Cracking – Connecticut

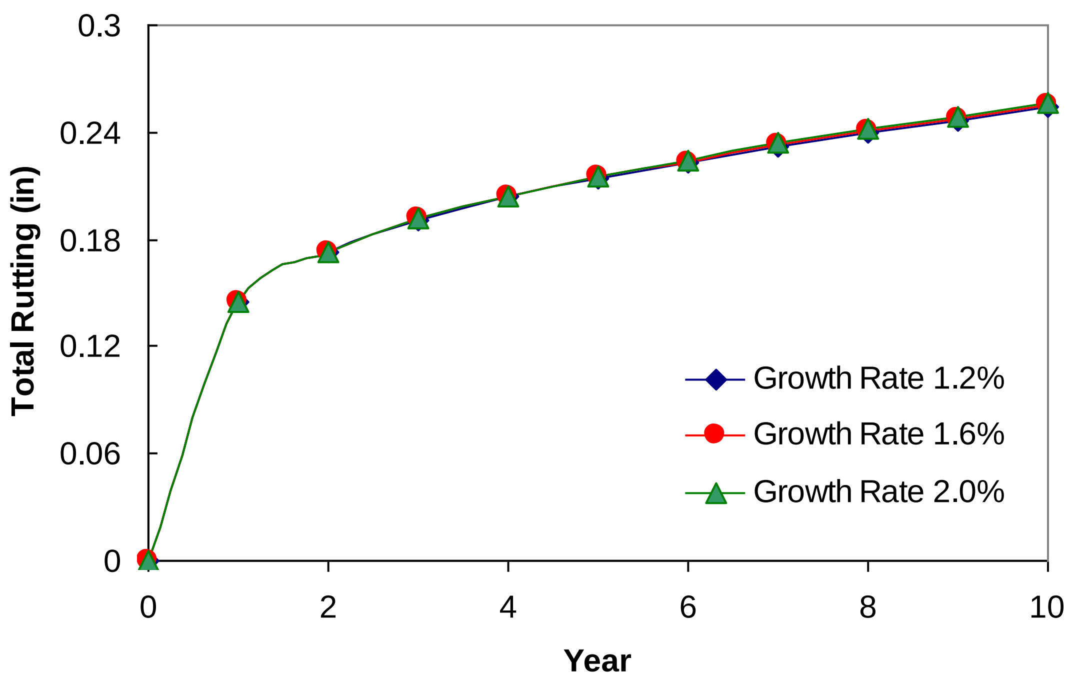

Figure 72 Effect of Traffic Growth Rate on Total Rutting – Connecticut

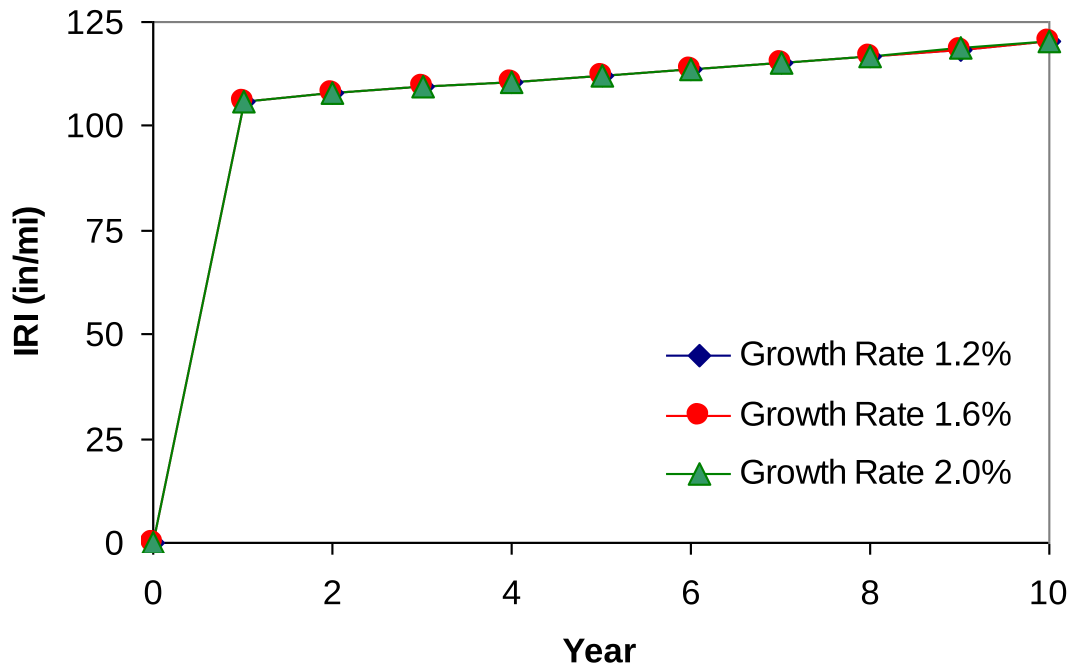

Figure 73 Effect of Traffic Growth Rate on IRI – Connecticut

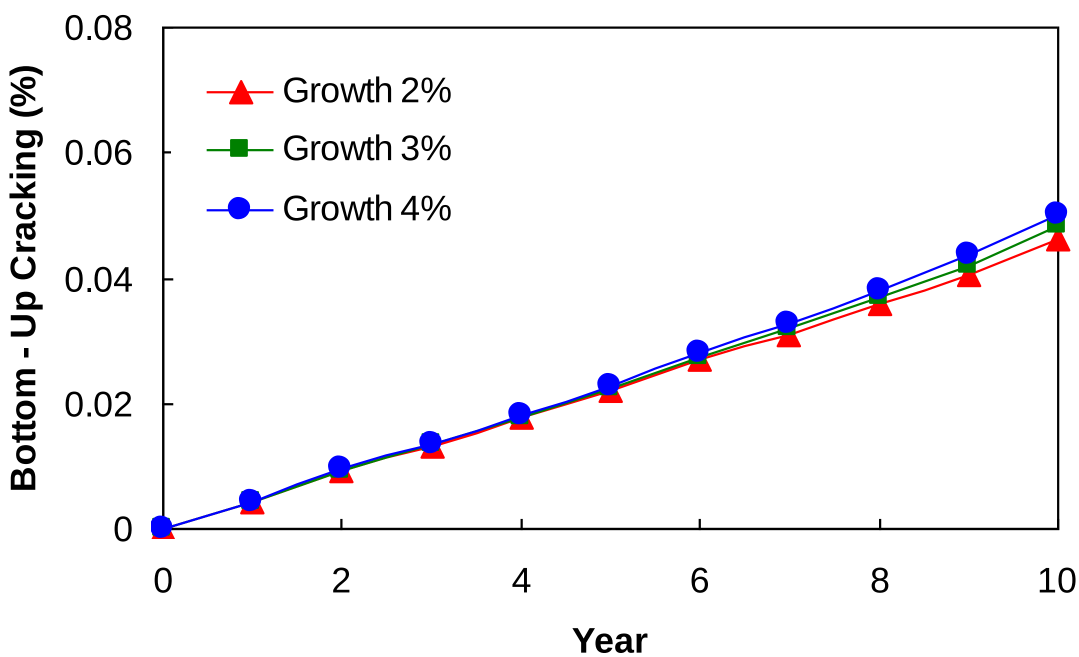

Figure 74 Effect of Traffic Growth Rate on Bottom-Up Cracking – Maine

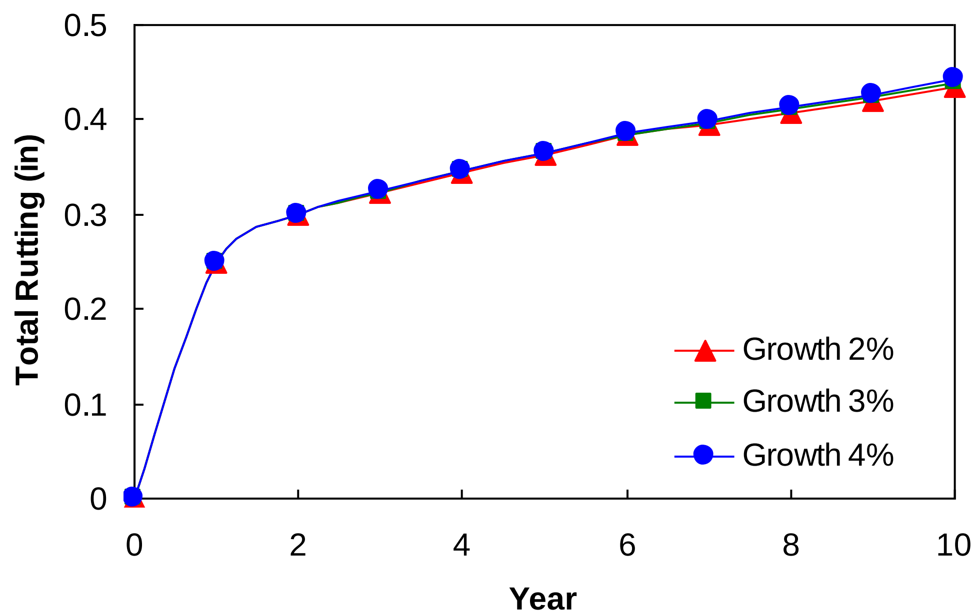

Figure 75 Effect of Traffic Growth Rate on Total Rutting – Maine

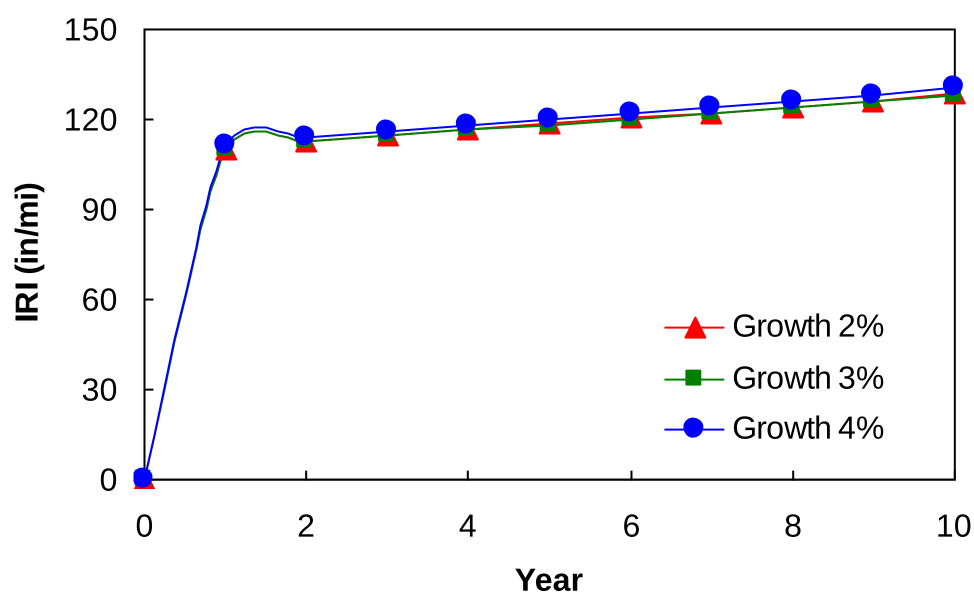

Figure 76 Effect of Traffic Growth Rate on IRI – Maine

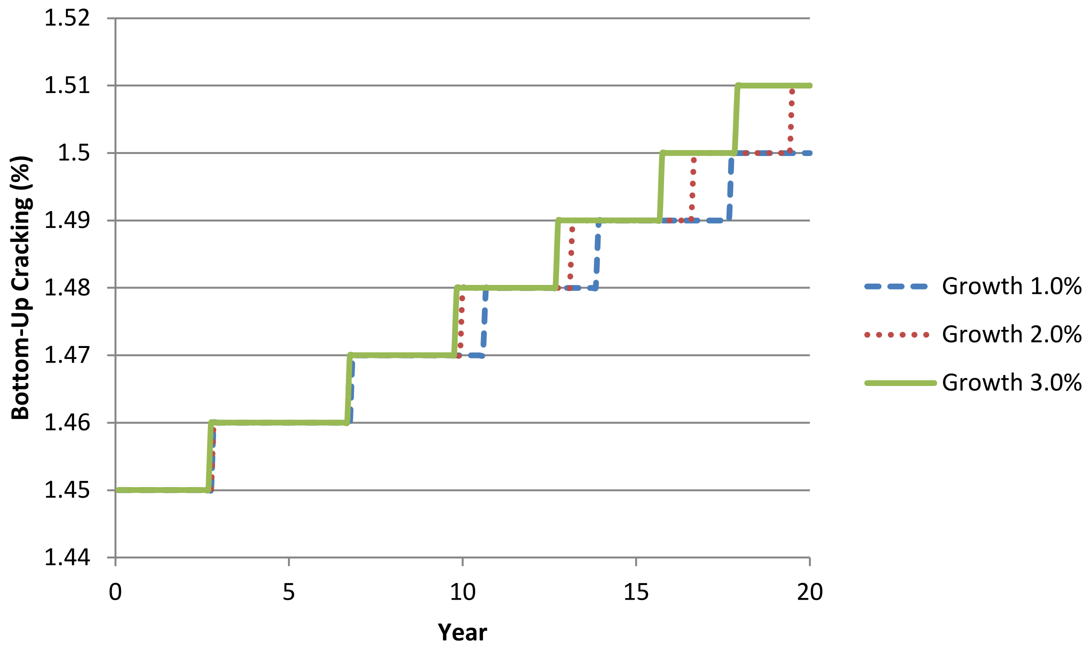

Figure 77 Effect of Traffic Growth Rate on Bottom-Up Cracking – Vermont

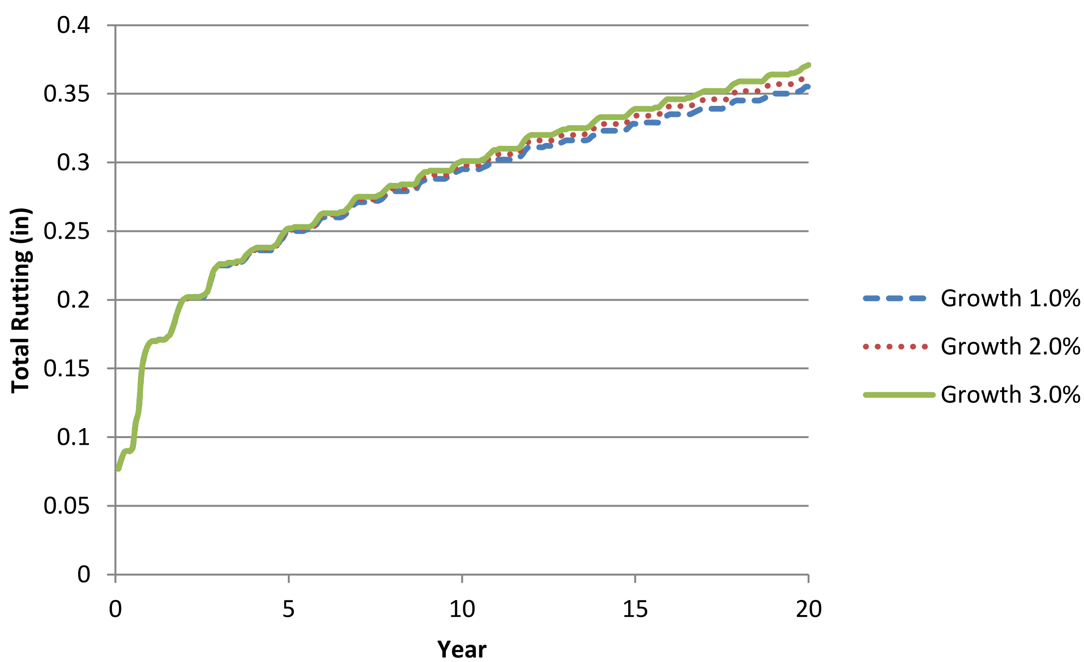

Figure 78 Effect of Traffic Growth Rate on Total Rutting – Vermont

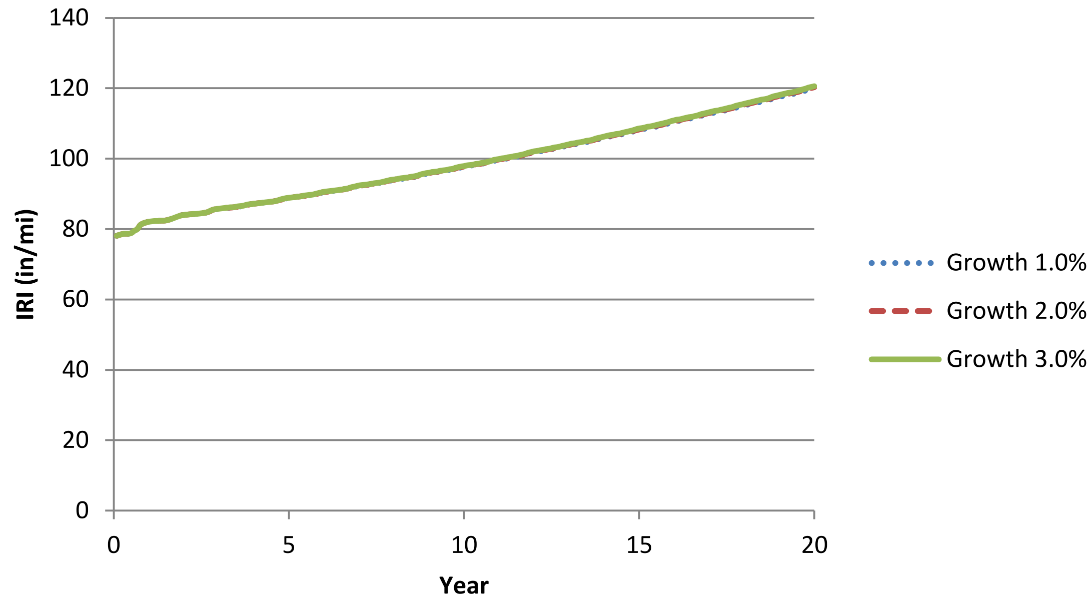

Figure 79 Effect of Traffic Growth Rate on IRI – Vermont

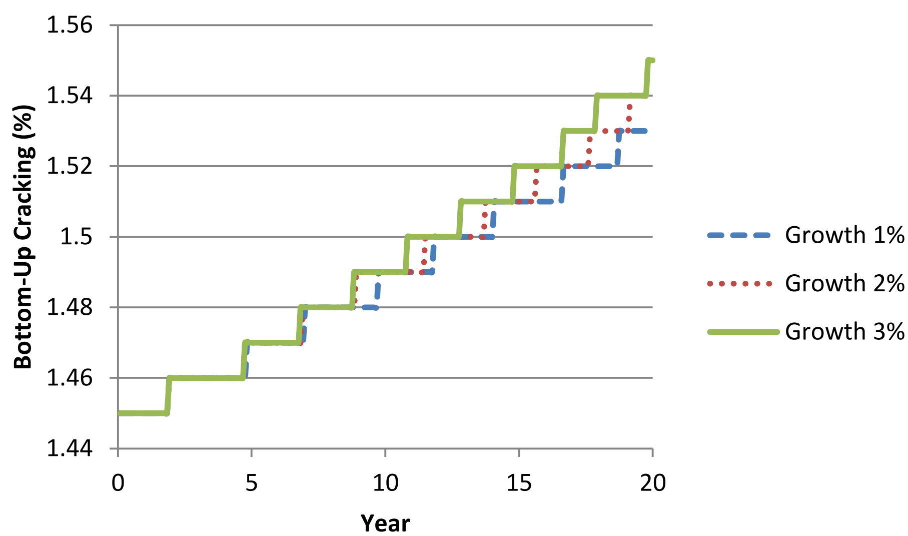

Figure 80 Effect of Traffic Growth Rate on Bottom-Up Cracking – New York

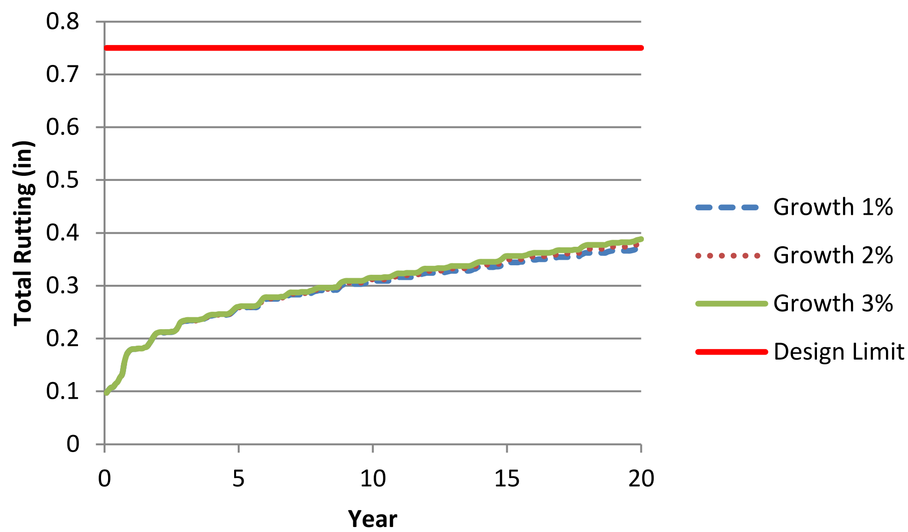

Figure 81 Effect of Traffic Growth Rate on Total Rutting – New York

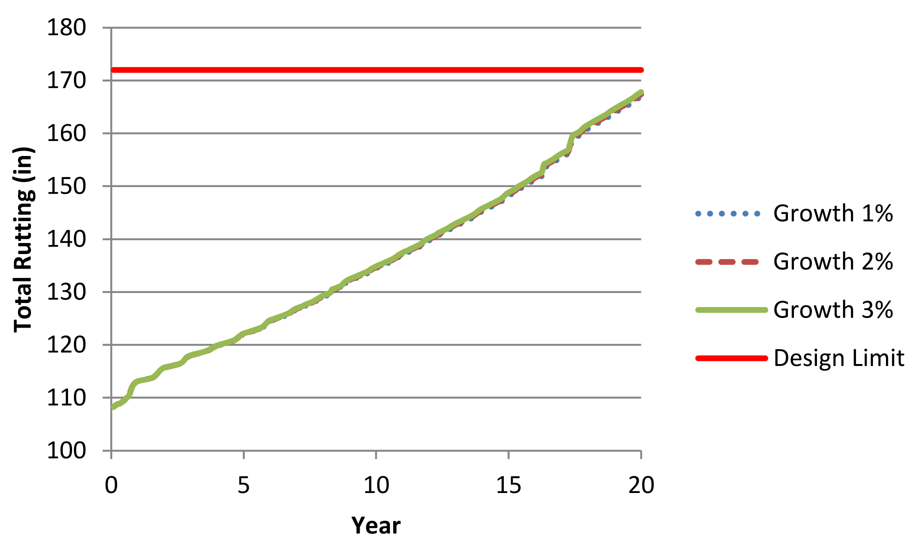

Figure 82 Effect of Traffic Growth Rate on IRI – New York

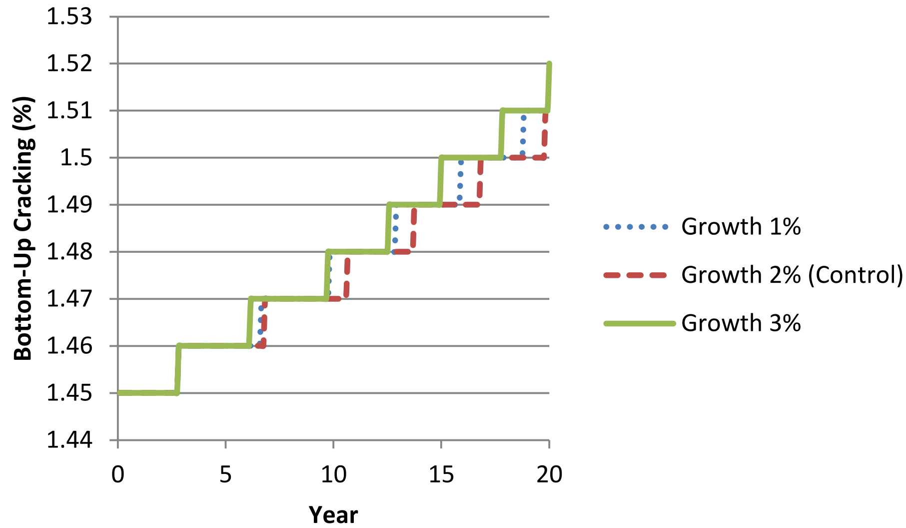

Figure 83 Effect of Traffic Growth Rate on Bottom-Up Cracking – Massachusetts

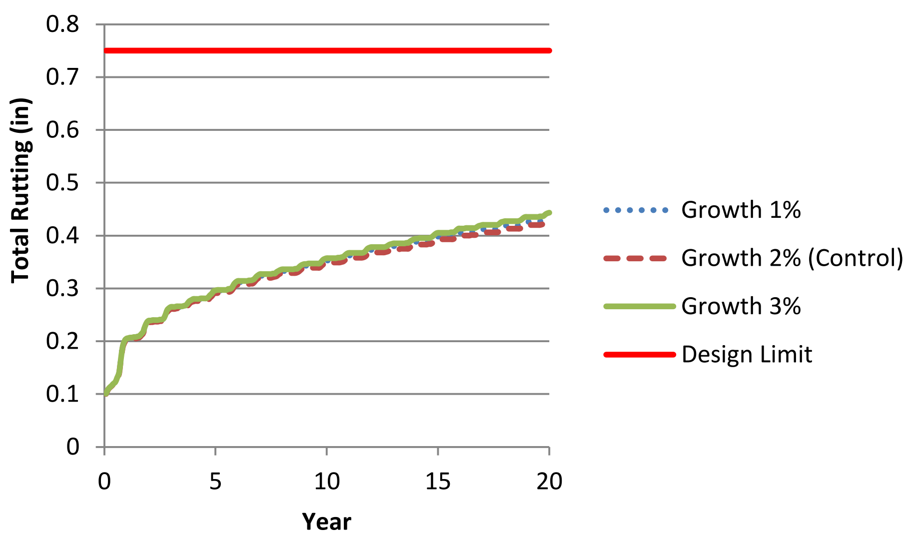

Figure 84 Effect of Traffic Growth Rate on Total Rutting – Massachusetts

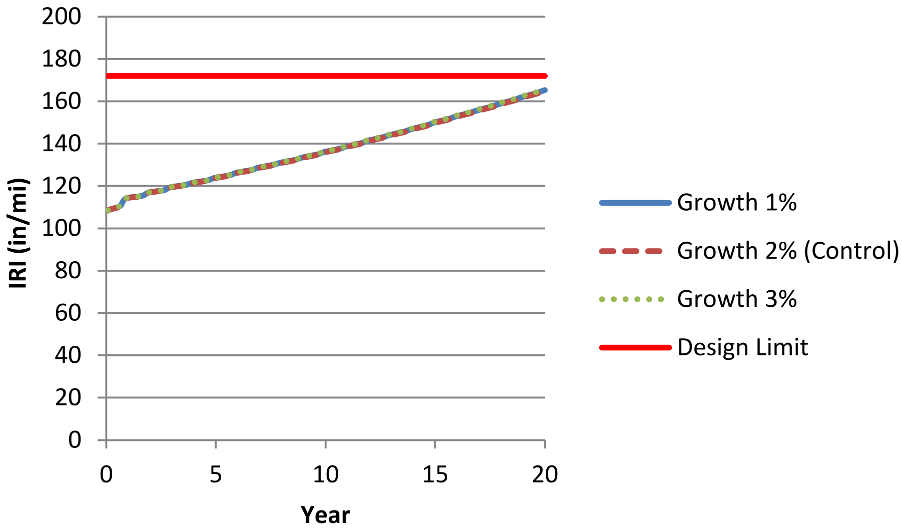

Figure 85 Effect of Traffic Growth Rate on IRI – Massachusetts

Truck Class Distribution

Truck class distributions were obtained from LTPP monitored truck class counts for the sections studied. This data was used as a Level 2 default, representing data obtained from the construction site. MEPDG software contains national LTPP averages for different classes of roads described below:

- Principal arterial – Interstate and defense routes
- Principal arterials – others
- Minor arterials
- Major Collectors
- Minor Collectors
- Local routes and streets

The appropriate default distribution for Level 3 design can be selected from the available distributions given in the M-E design guide software using “load distribution” feature in vehicle class distribution screen (Figure 6).

Truck class distribution significantly affects the predicted pavement distresses for both Level 2 and Level 3. IRI was not found to be affected by the truck distribution. A higher percentage of high-class trucks significantly increases the distresses on a pavement due to incremental damage caused to the roads due to heavier loads. Therefore, for pavements of low importance, design can be done by using default values and a more conservative design can be obtained by using the available default distributions described above having a higher percentage of trucks.

The performance prediction plots versus time are shown below. The general trend observed is as explained above. It is observed that LTPP distributions contain a lower percentage of high-class trucks than the assumed high percentage of high-class truck distribution (31).

For the state of New Hampshire, where failure was observed for the gathered input data and pavement structure, the year in which the pavement failed advanced significantly with changing truck class distributions. Default distribution for interstate routes (I-393 for New Hampshire study) yielded a more conservative design as compared to LTPP distribution. Such observation is strictly subjected to the percentage of high-class trucks, and therefore default distributions must be compared to available LTPP distributions for the state to determine a reliable truck class distribution for design.

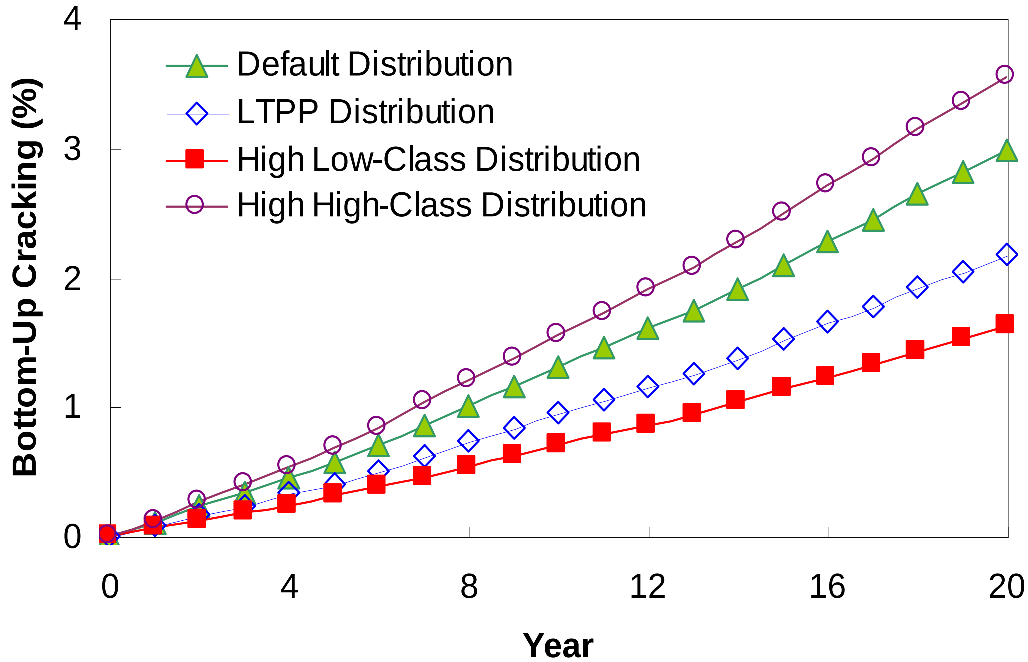

Figure 86 Effect of Truck Class Distribution on Bottom-Up Cracking – New Hampshire

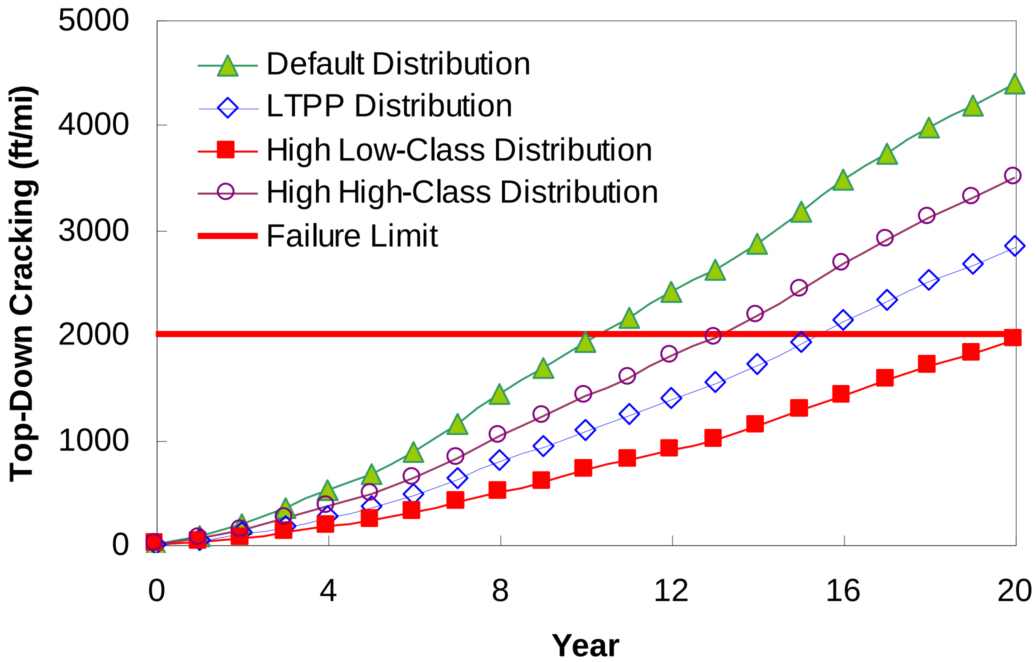

Figure 87 Effect of Truck Class Distribution on Top-Down Cracking – New Hampshire

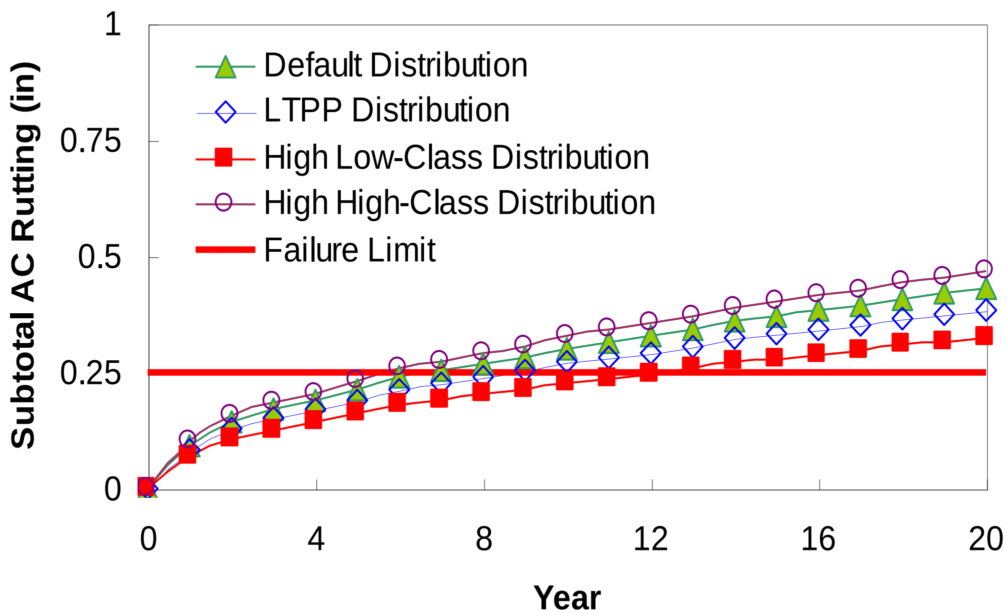

Figure 88 Effect of Truck Class Distribution on Subtotal AC Rutting – New Hampshire

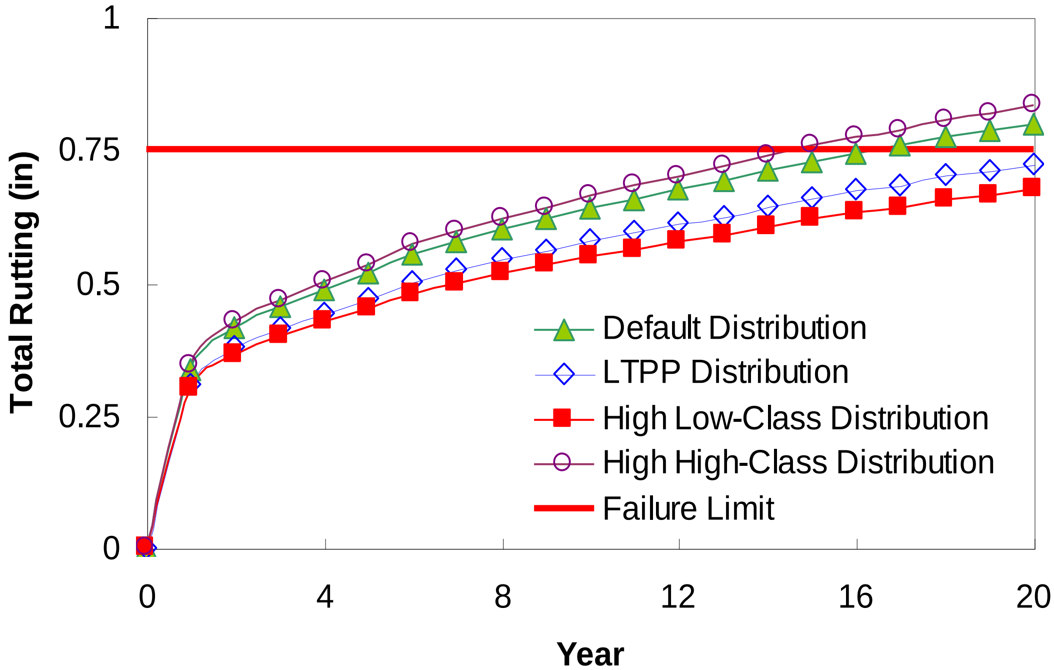

Figure 89 Effect of Truck Class Distribution on Total Rutting – New Hampshire

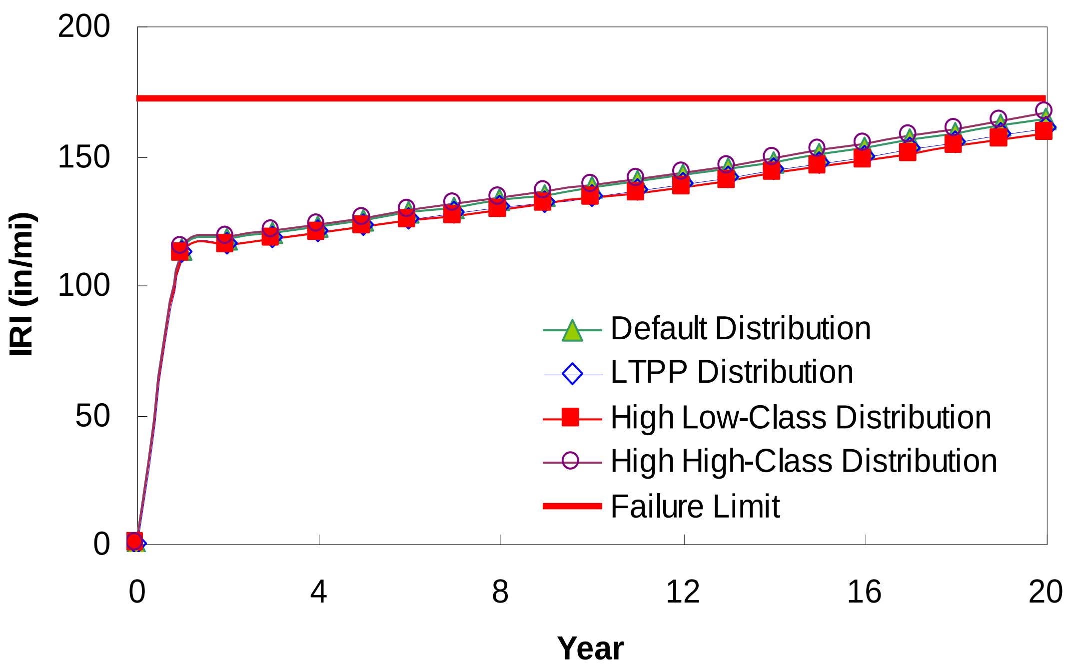

Figure 90 Effect of Truck Class Distribution on IRI – New Hampshire

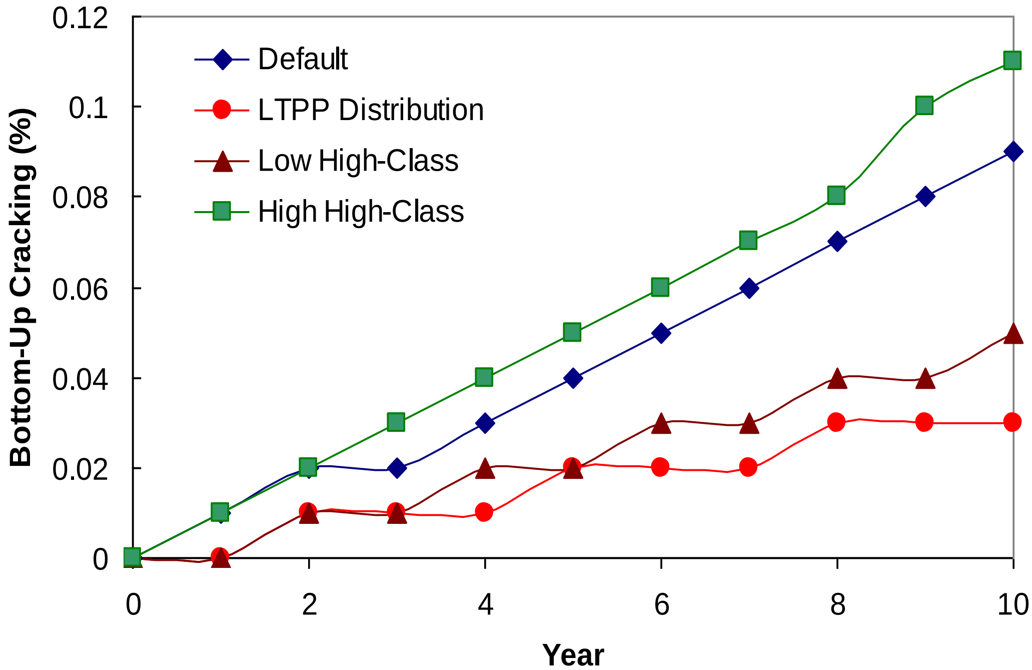

Figure 91 Effect of Truck Class Distribution on Bottom-Up Cracking – Connecticut

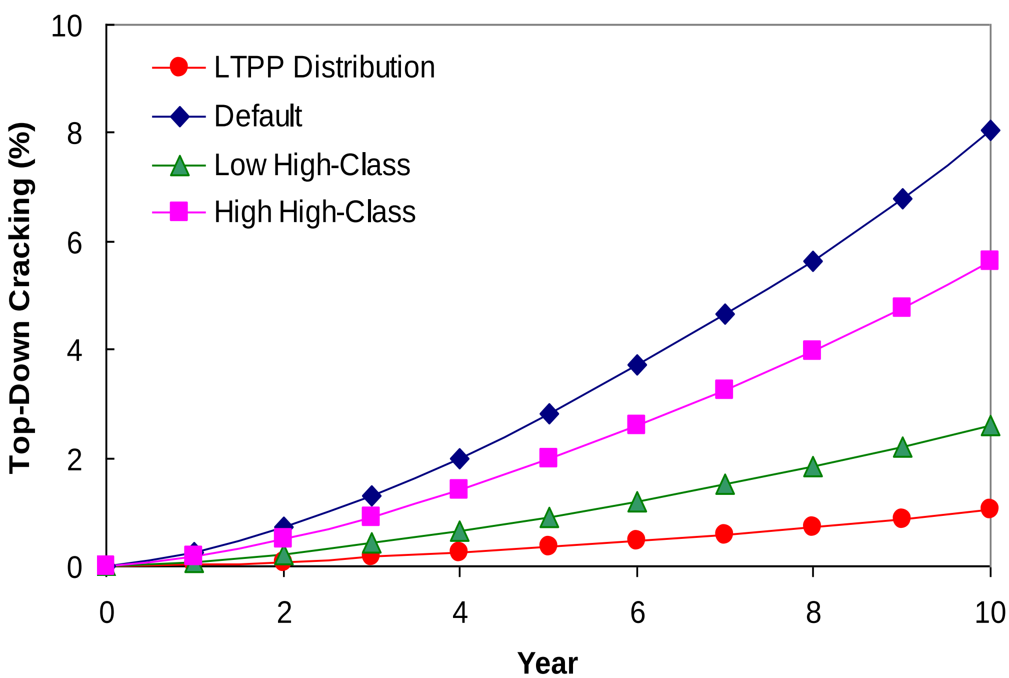

Figure 92 Effect of Truck Class Distribution on Top-Down Cracking – Connecticut

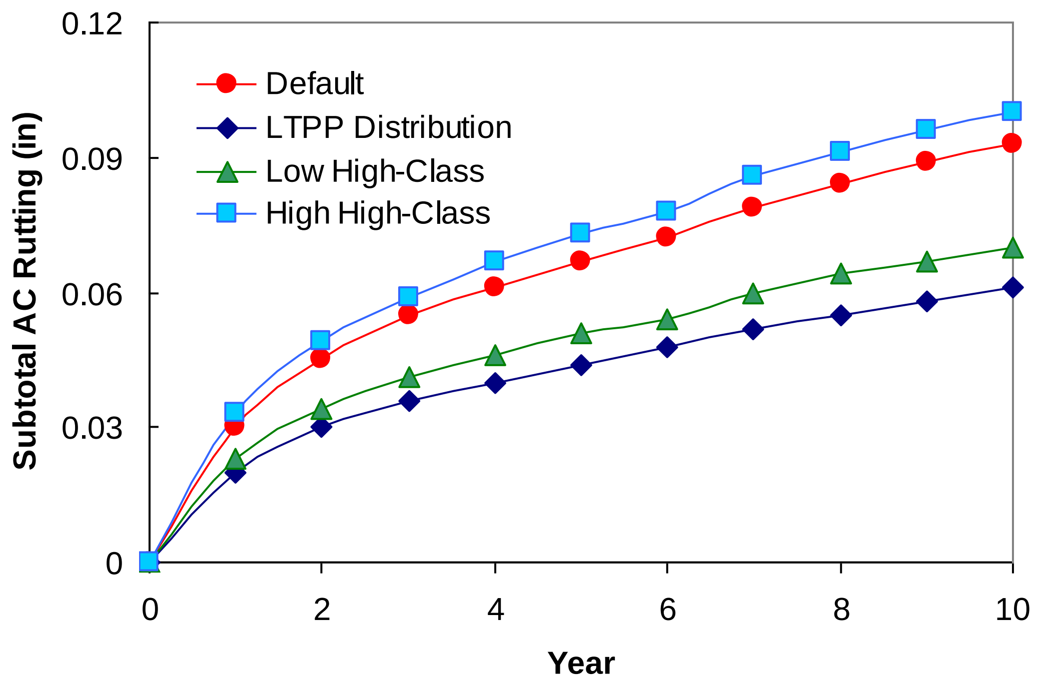

Figure 93 Effect of Truck Class Distribution on Subtotal AC Rutting – Connecticut

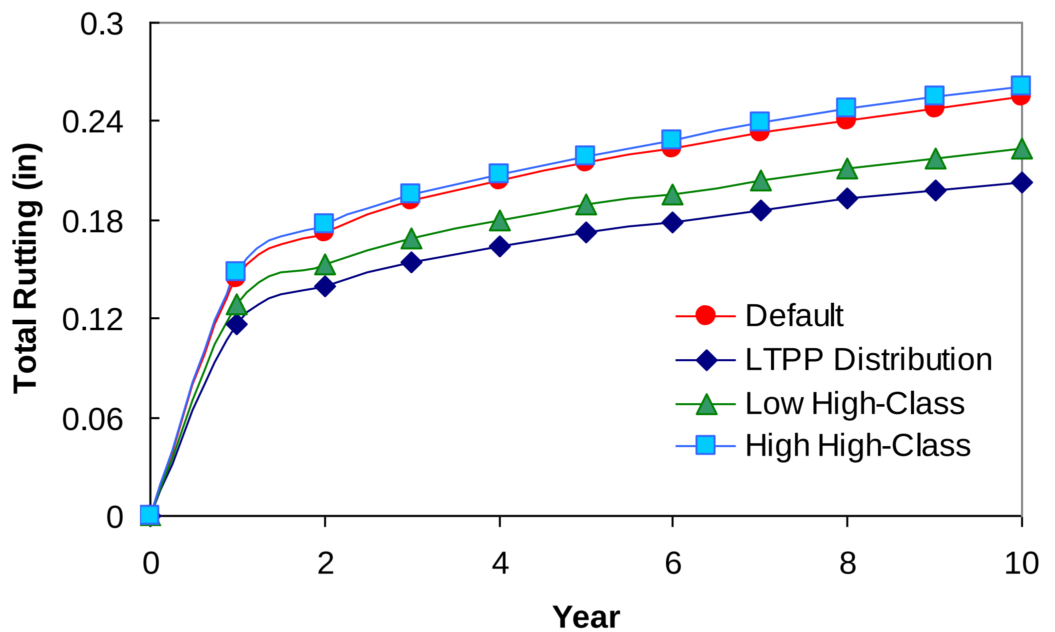

Figure 94 Effect of Truck Class Distribution on Total Rutting – Connecticut

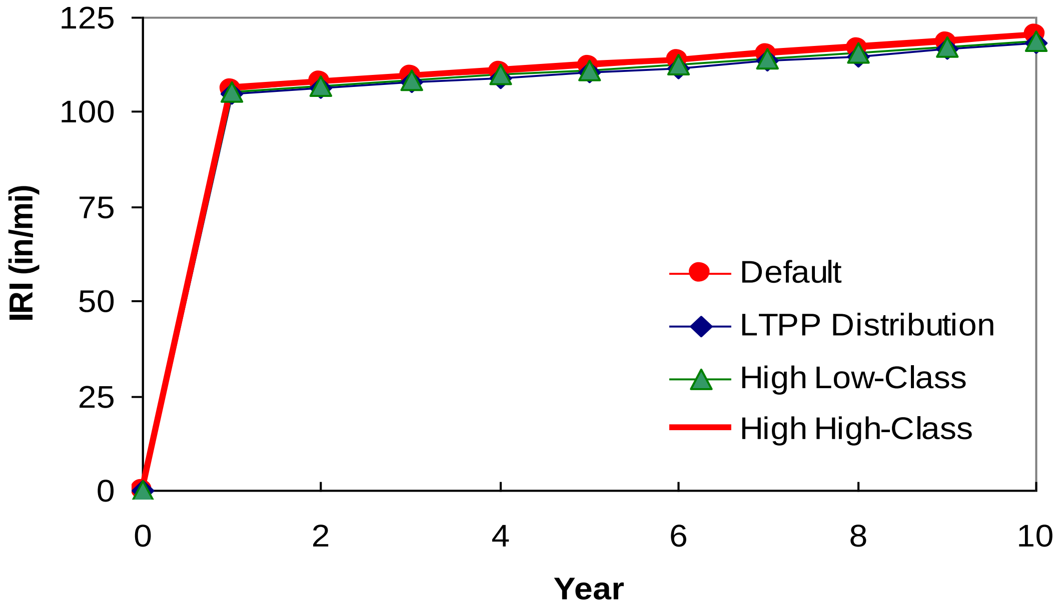

Figure 95 Effect of Truck Class Distribution on IRI – Connecticut

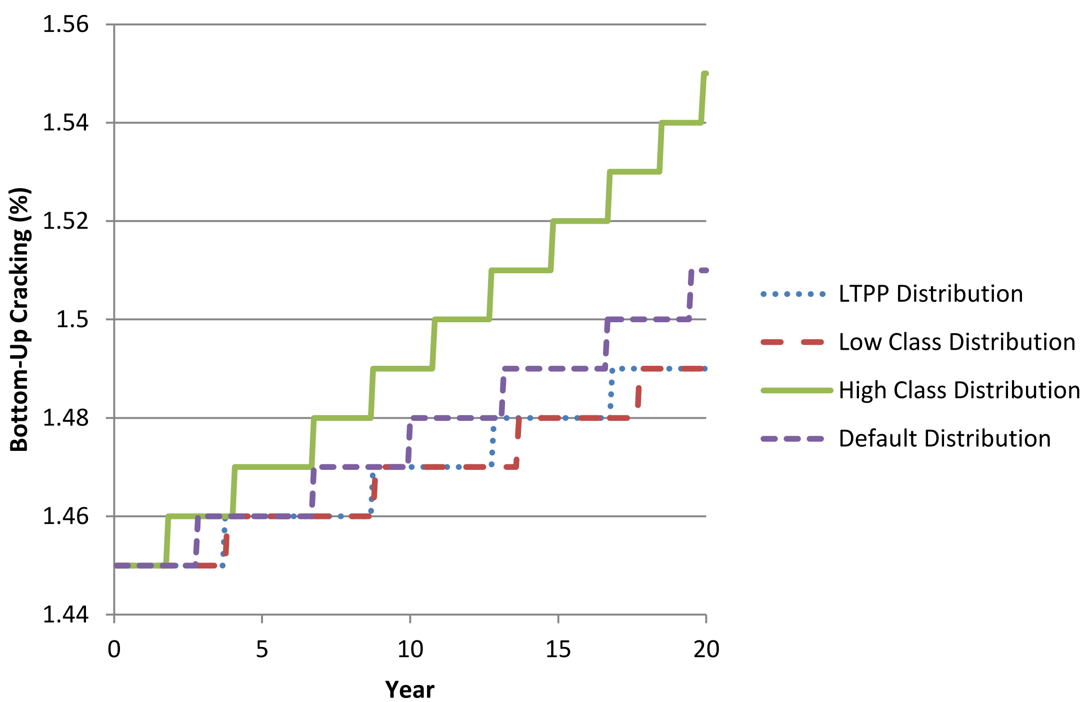

Figure 96 Effect of Truck Class Distribution on Bottom-Up Cracking - Vermont

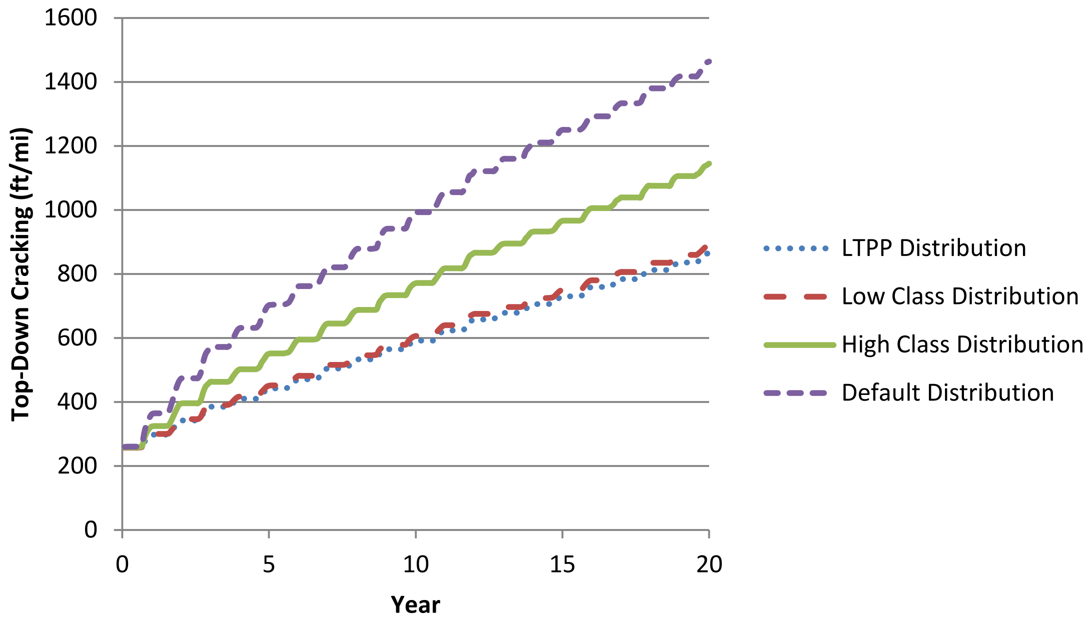

Figure 97 Effect of Truck Class Distribution on Top-Down Cracking - Vermont

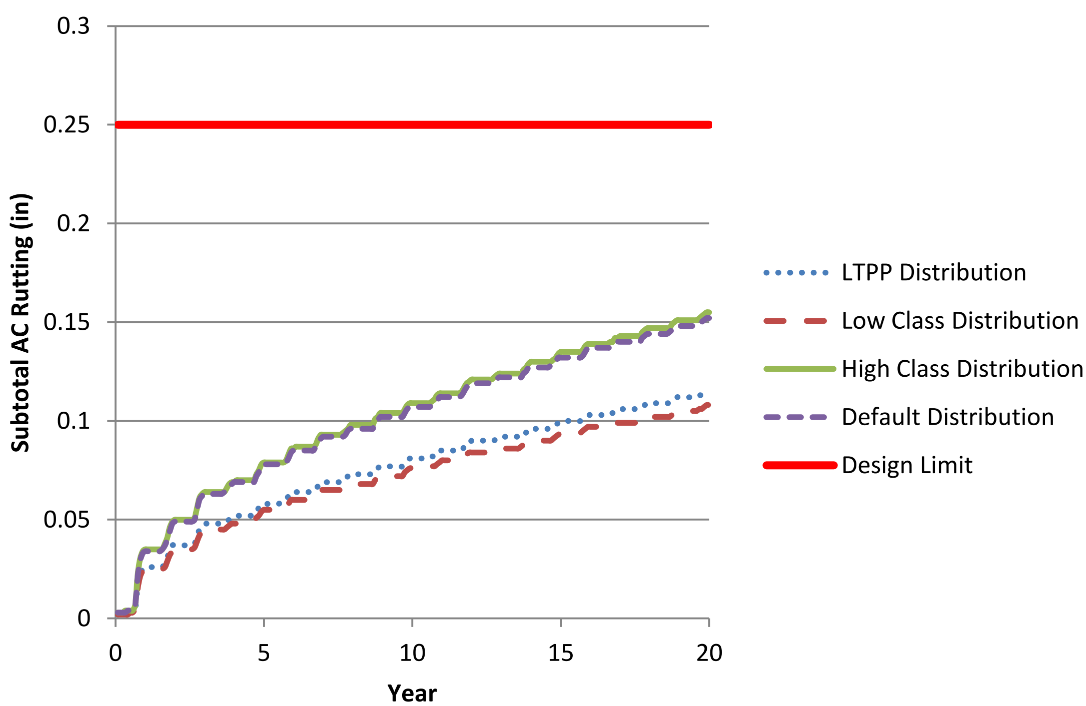

Figure 98 Effect of Truck Class Distribution on Subtotal AC Rutting - Vermont

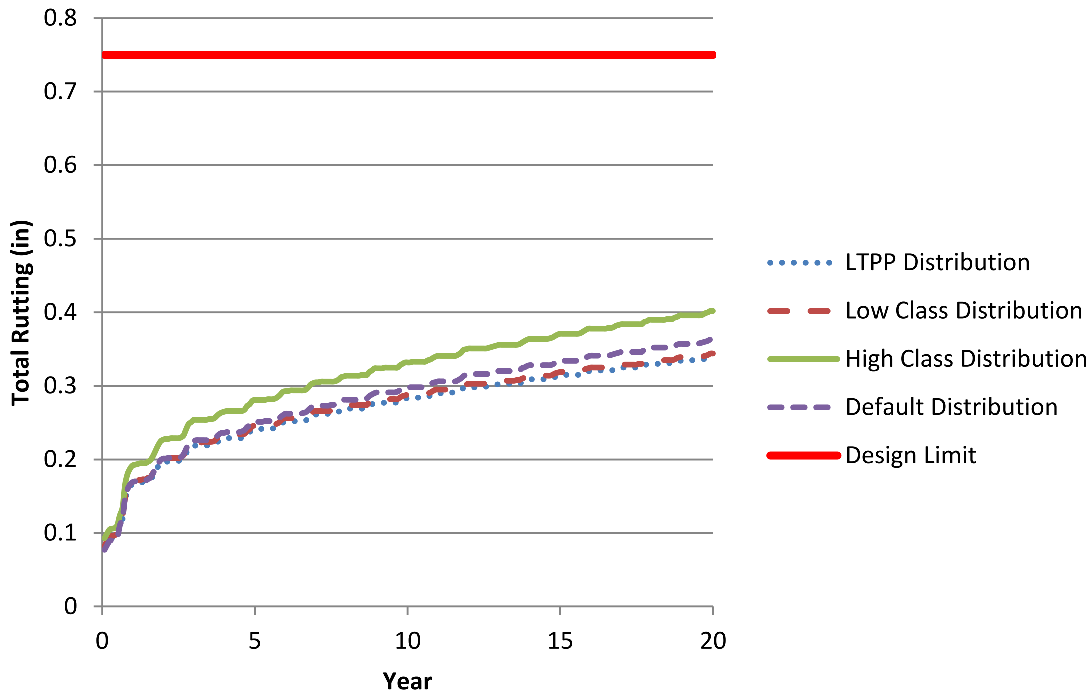

Figure 99 Effect of Truck Class Distribution on Total Rutting - Vermont

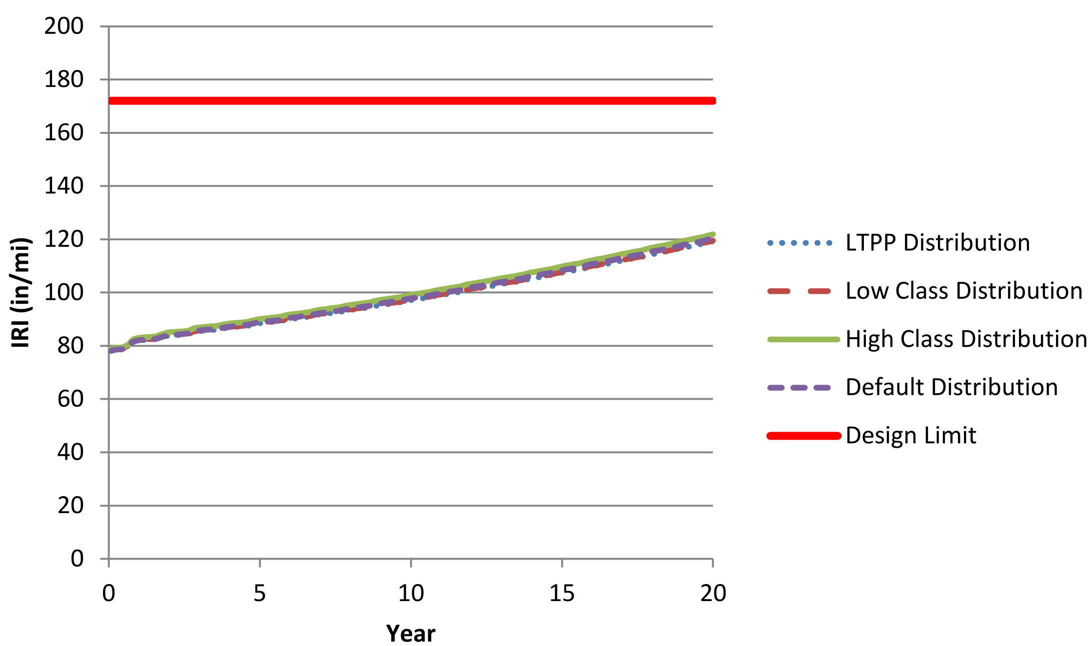

Figure 100 Effect of Truck Class Distribution on IRI - Vermont

Traffic Operational Speed

Traffic operational speed was studied in conjunction with the performance grade of binder used in the asphalt concrete surface layer. The purpose of this activity was to analyze the interaction between the two input parameters in predicting pavement performance, as well as to demonstrate and verify the recommendation made by Superpave specifications (32).

Design operational speeds of 5 mph, 25 mph and 65 mph (55 mph in Vermont) were selected for the study. These speeds simulate the speeds of slow-moving traffic (at intersections and heavy traffic sections), common speed limit in a residential area – local highways (medium operational speed level) and typical restricted-access interstate speed limit (high speed level) respectively.

This research did not investigate how realistic ranking of vehicle speed is as a variable for pavement performance predictions. It is up to the state agency to decide if the change of vehicle speed and its range could really affect the pavement performance.

Figures 101 to 105 present an example of performance prediction plots in New York Level 3 sensitivity analysis in conjunction with the PG 64-22 binder grade.

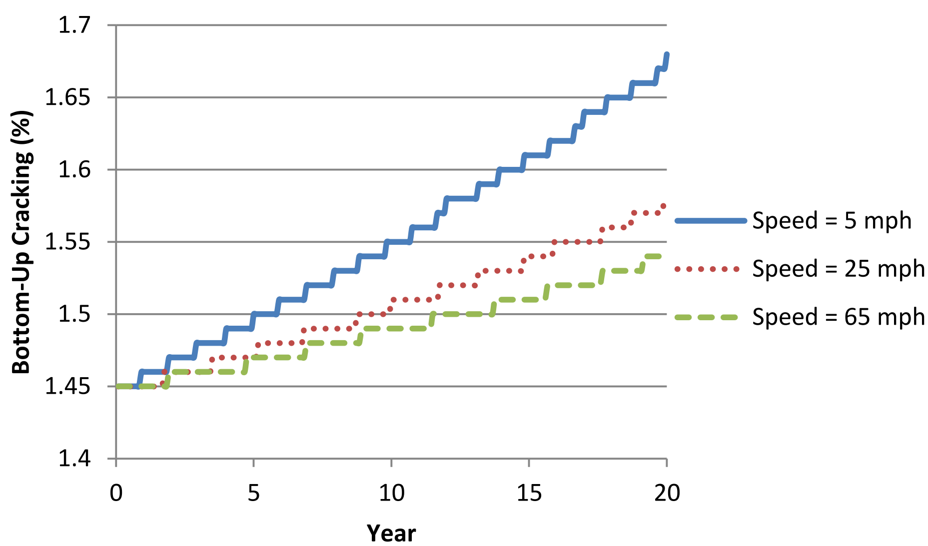

Figure 101 Effect of Traffic Speed on Bottom-Up Cracking with PG 64-22 in NY

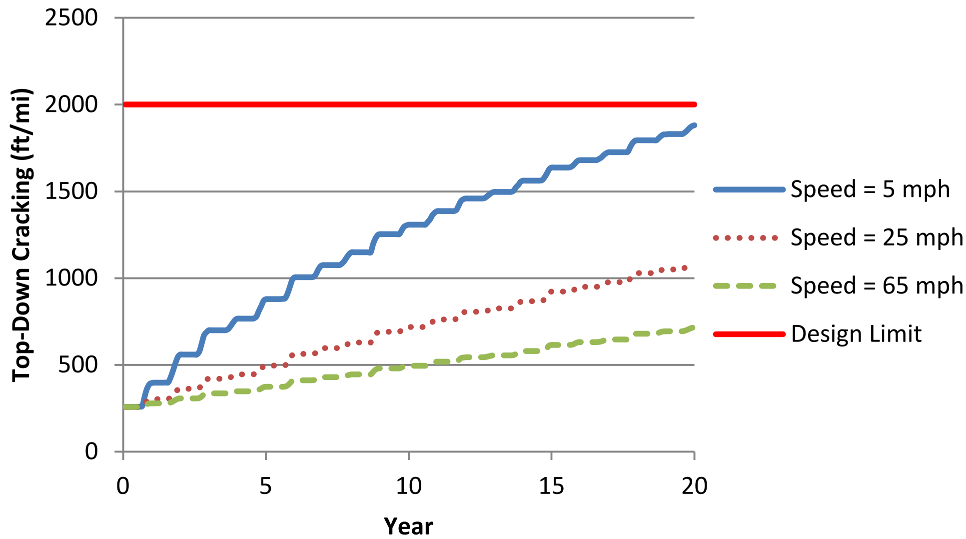

Figure 102 Effect of Traffic Speed on Top-Down Cracking with PG 64-22 in NY

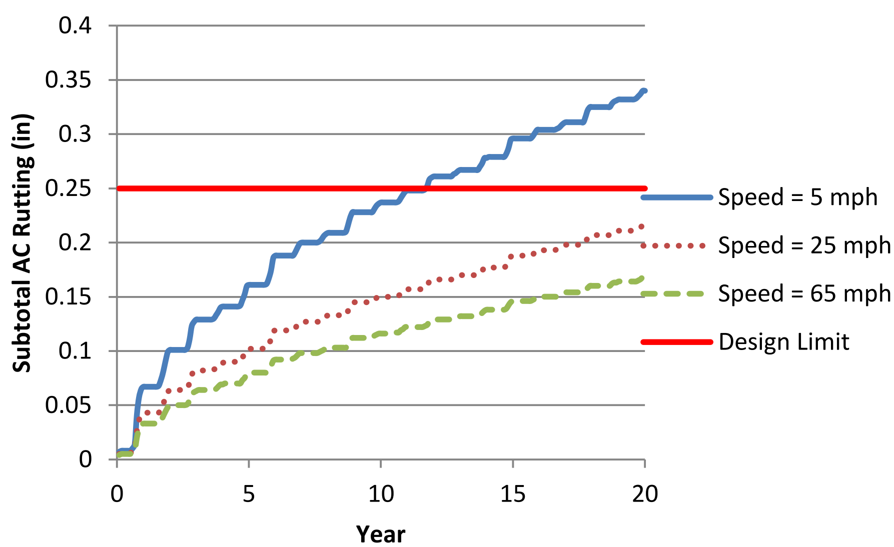

Figure 103 Effect of Traffic Speed on Subtotal AC Rutting with PG 64-22 in NY

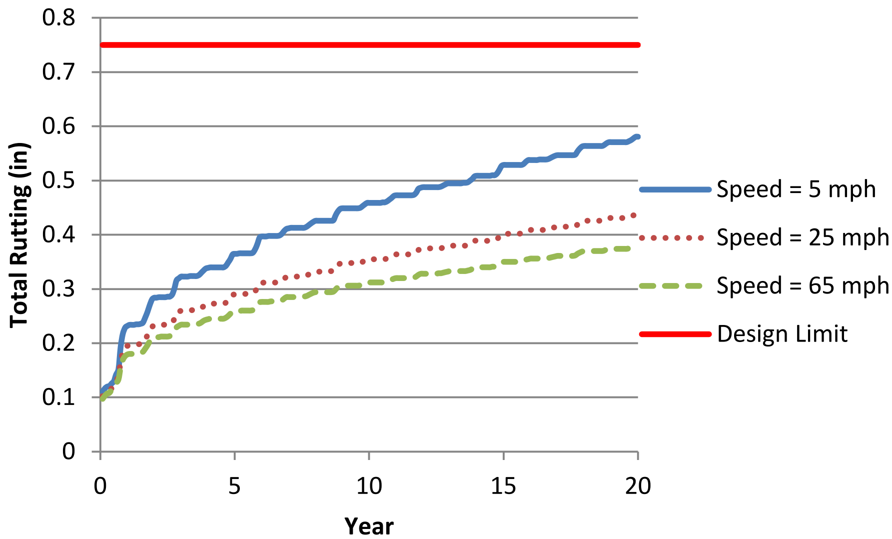

Figure 104 Effect of Traffic Speed on Total Rutting with PG 64-22 in NY

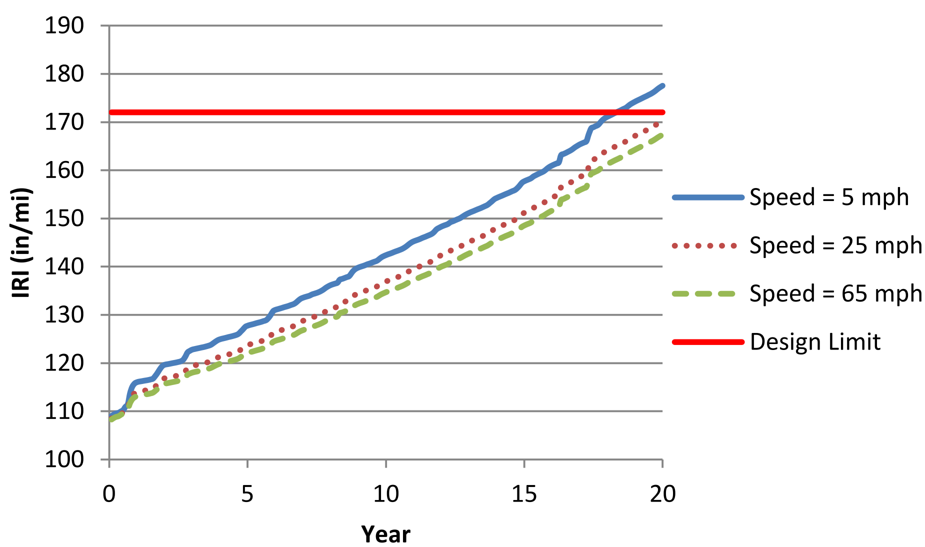

Figure 105 Effect of Traffic Speed on IRI with PG 64-22 in NY

6.1.2 Effect of Climate Inputs on Predicted Distresses

Climate inputs studied in this project consist of two main input variables – climate data from the MEPDG climate database and water table depth. New England states have very few climate stations integrated into the climate database of the MEPDG. Therefore, there is a need to analyze the sufficiency of the existing climate data for use in designing flexible pavements and overlays using the MEPDG. Climate data for the construction locations can be obtained in the following ways:

1. Using the climate data available in the MEPDG if there already exists a climate station for the location
2. Interpolating climate data by choosing an appropriate number of stations from a list of surrounding six closest weather stations provided by the MEPDG. This activity requires the latitude, longitude and elevation of the construction site for the MEPDG to select and display six surrounding weather stations

This activity is also used as a basis to advocate the need for setting up weather stations to collect climate data or recommend interpolation from surrounding weather stations.

All of the climate data necessary for the MEPDG sensitivity analysis is available from over 800 weather stations located across the U.S. The designer must specify the project location (longitude and latitude) to obtain the six closest weather stations. One weather station can be selected for the MEPDG sensitivity analysis if the project is located less than 50 miles from the station. At least three weather stations must be chosen for each project location (to create a virtual weather station) if the project is located more than 50 miles from the weather station. The purpose of choosing more water stations was to avoid the possibility of missing data and of obtaining errors from a single weather station. The climate variable was found to have a significant effect on the AC and total rutting predictions.

The MEPDG Version 1.0 and 1.1 studies to predict cracking, rutting and roughness showed that climate data can be interpolated from surrounding weather stations provided by the MEPDG, given the following conditions are satisfied:

- The latitude, longitude and elevation of the construction site are available
- At least three stations can be selected such that their positional average represents the location under consideration
- There are no significant geographical obstructions like mountains, sea-inlets and valleys

Longitudinal and fatigue cracking, rutting and IRI are insensitive to the variation in climate data.

Climate has a greater effect on thermal cracking than other distresses as thermal cracking is most sensitive to temperature than other types of distresses. Roughness is insensitive to climate. Effect of climate data has been analyzed graphically only due to difficulty in incorporating the variable both quantitatively and qualitatively in the statistical model.

For the state of New Hampshire, interpolated data predicted slightly lower distresses for Lebanon station because the weather station's location is in a valley surrounded by contours of approximately 800 ft (dark brown contours in the map) as shown in Figure 106.

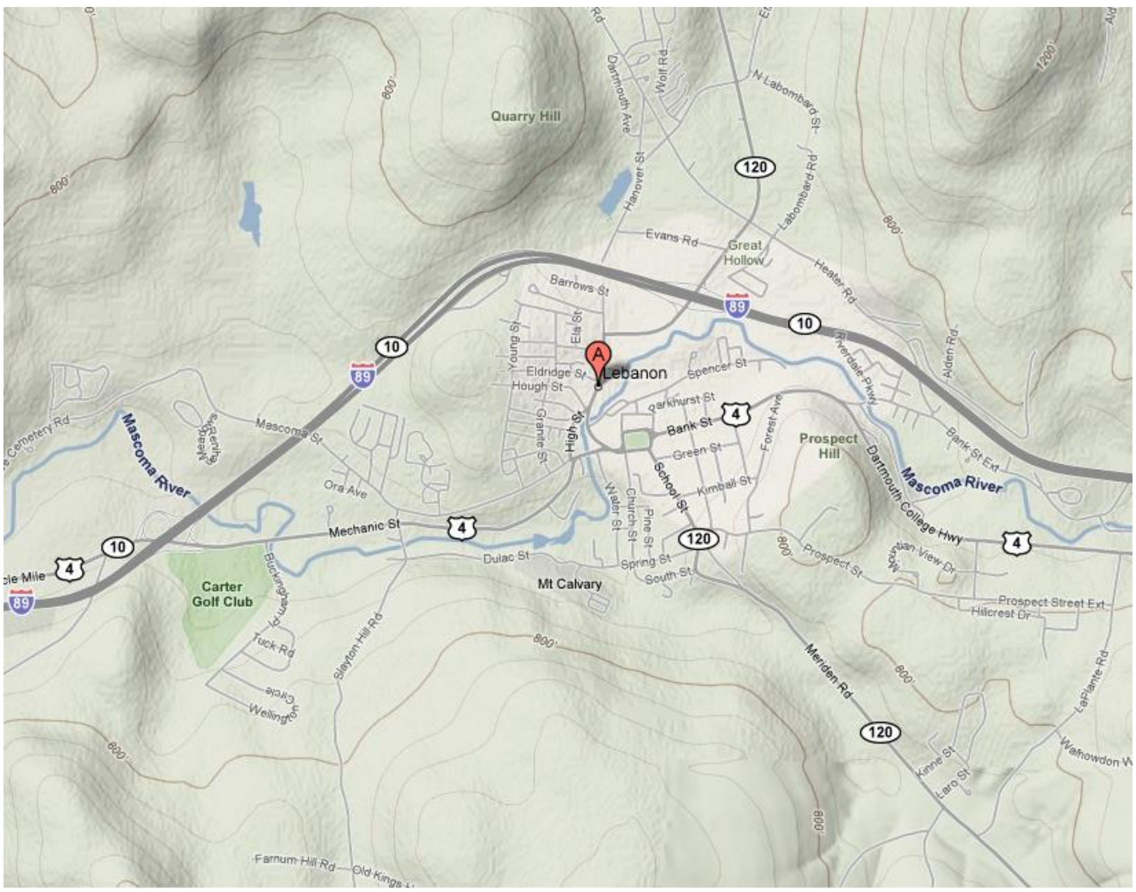

Figure 106 Contour Map - Lebanon, NH

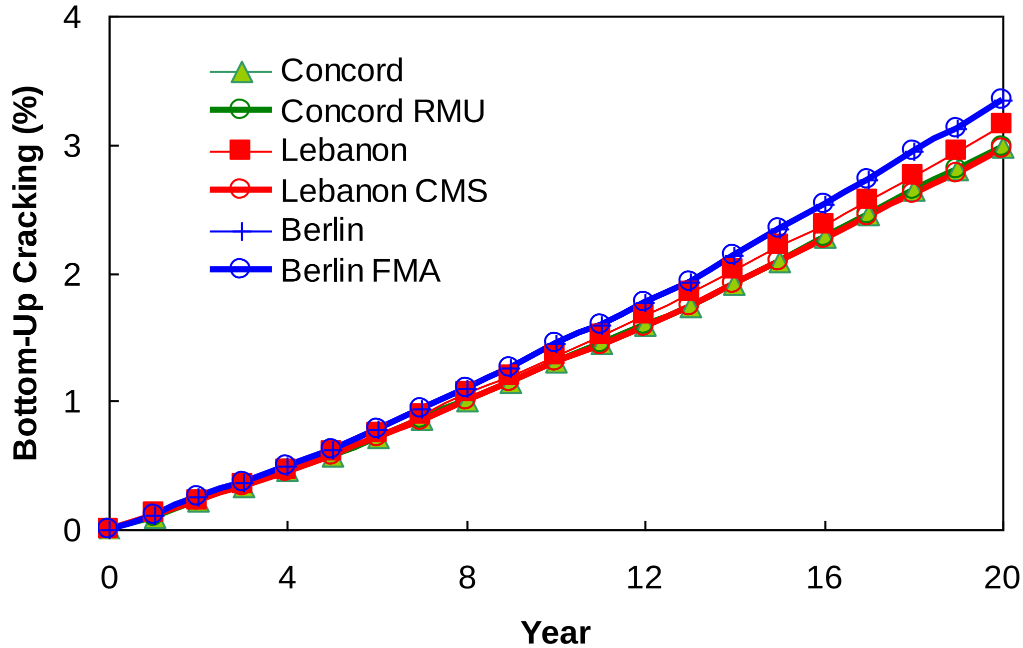

Figure 107 Effect of Climate on Bottom-Up Cracking – New Hampshire

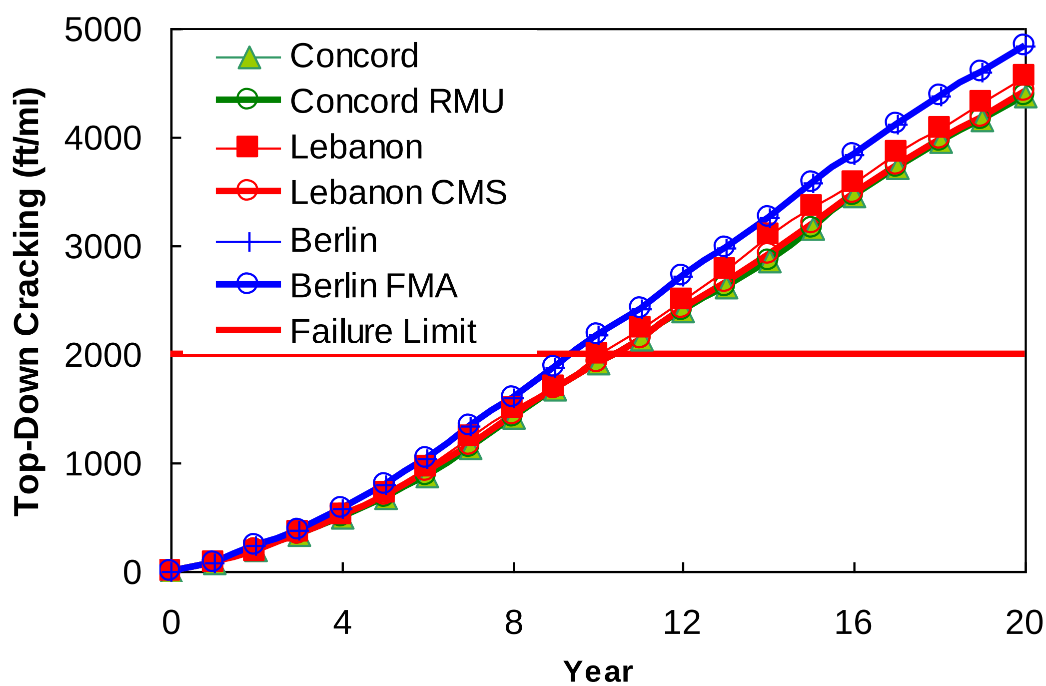

Figure 108 Effect of Climate on Top-Down Cracking – New Hampshire

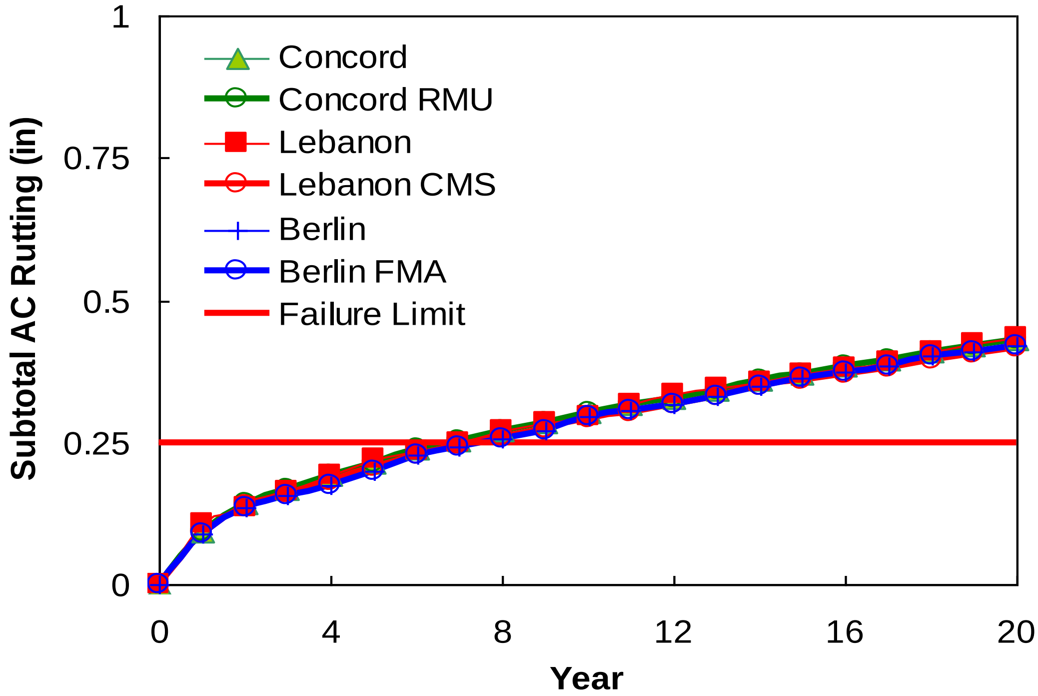

Figure 109 Effect of Climate on Subtotal AC Rutting – New Hampshire

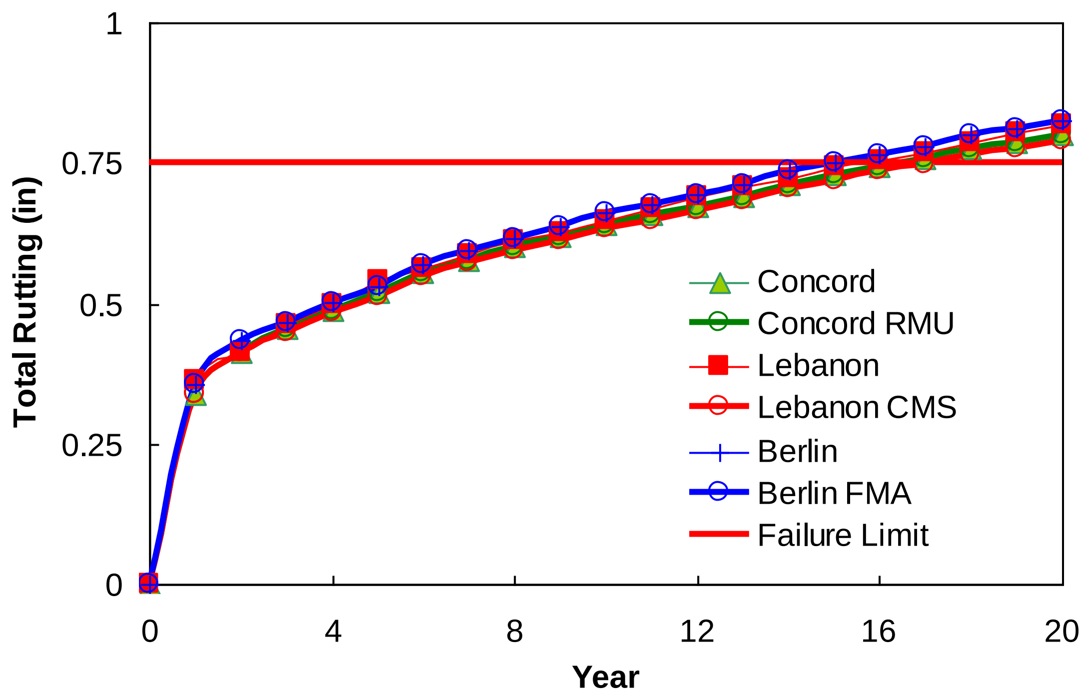

Figure 110 Effect of Climate on Total Rutting – New Hampshire

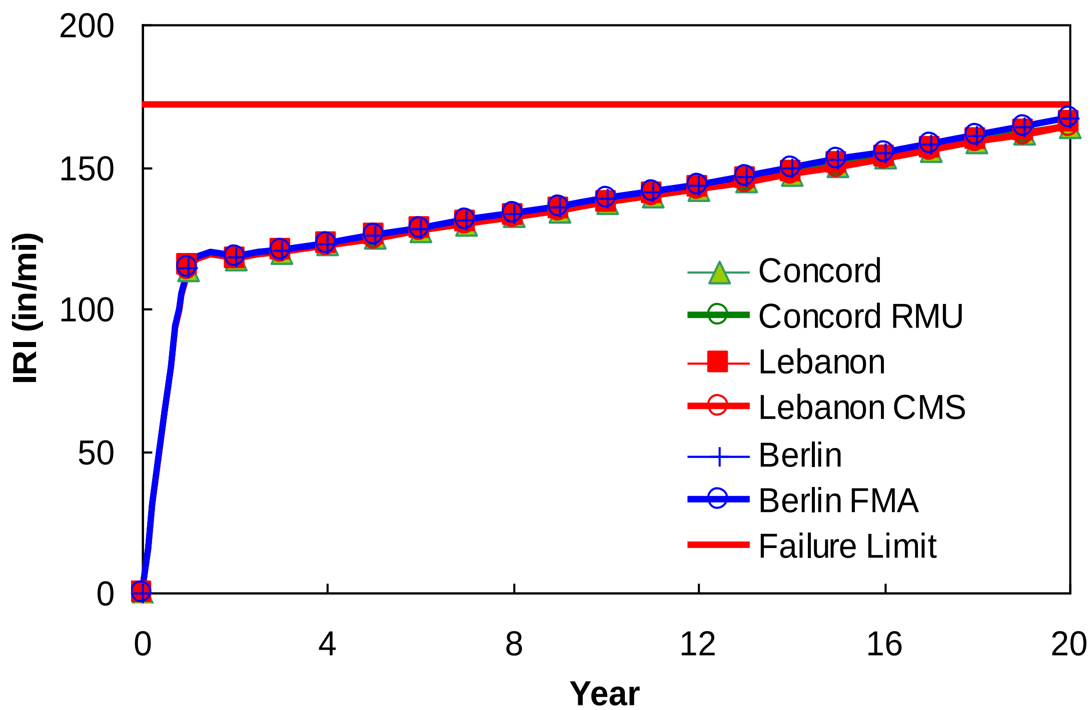

Figure 111 Effect of Climate on IRI – New Hampshire

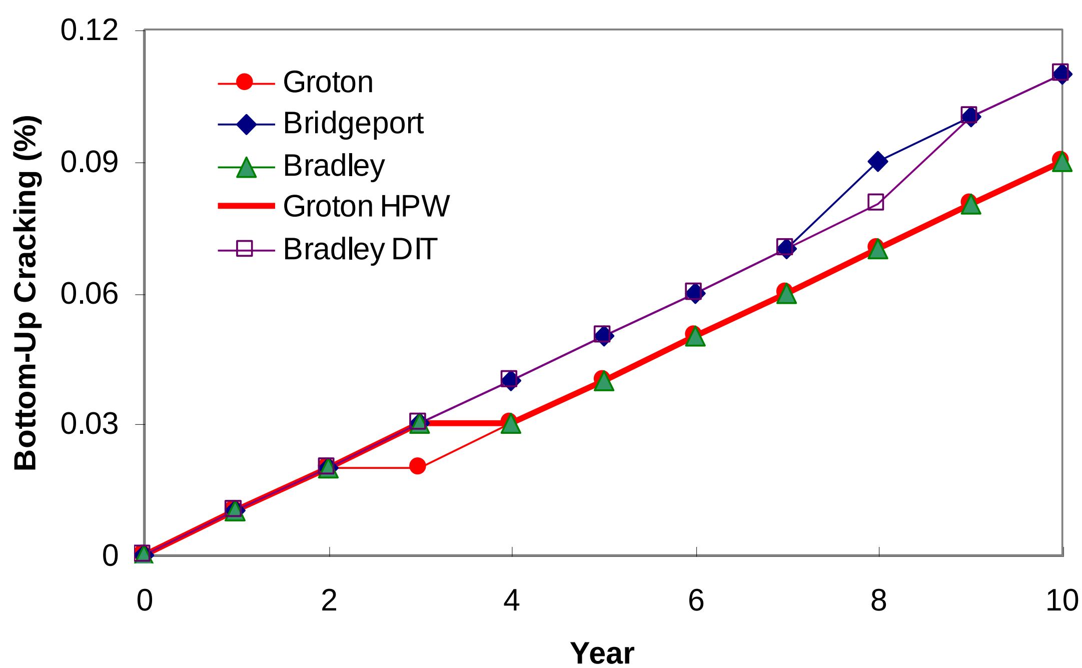

Figure 112 Effect of Climate on Bottom-Up Cracking – Connecticut

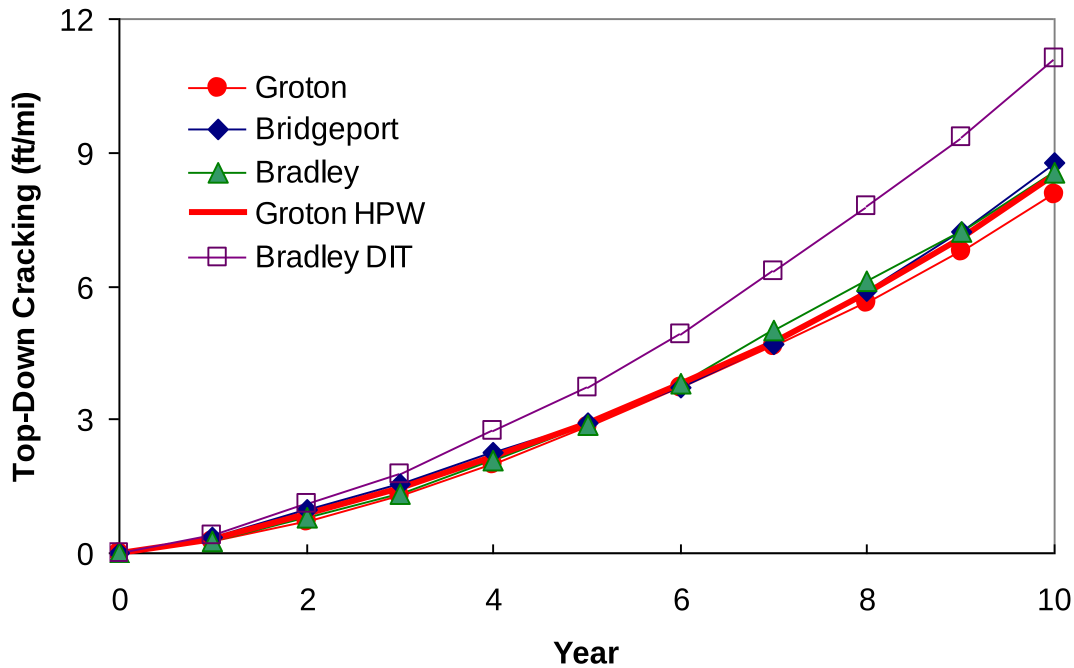

Figure 113 Effect of Climate on Top-Down Cracking – Connecticut

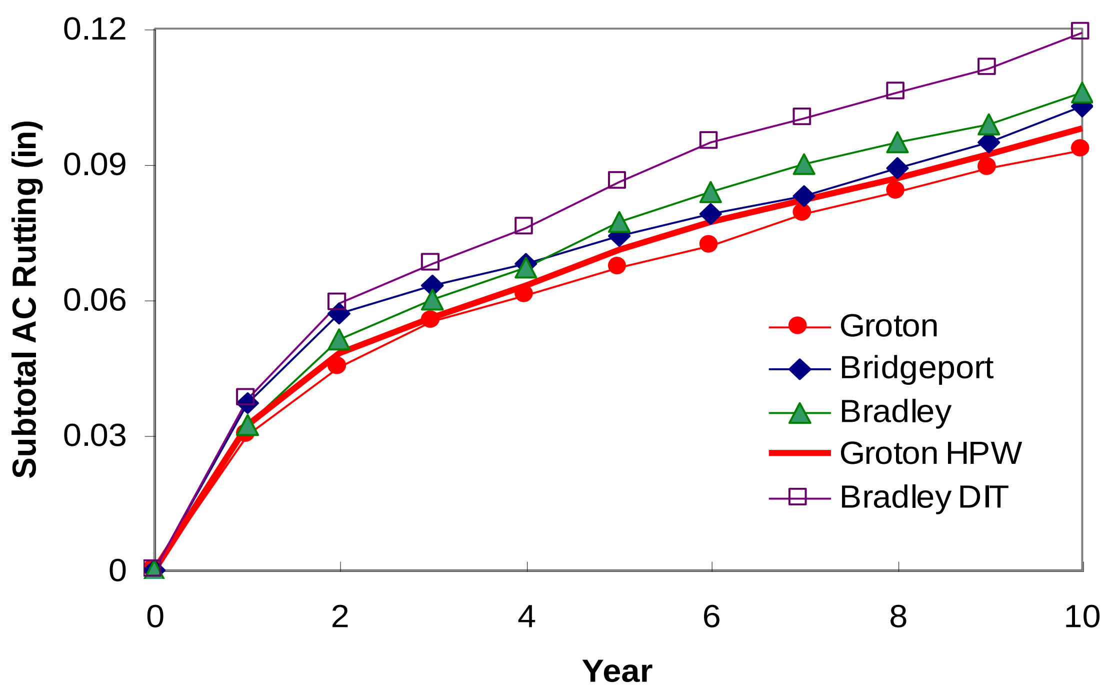

Figure 114 Effect of Climate on Subtotal AC Rutting – Connecticut

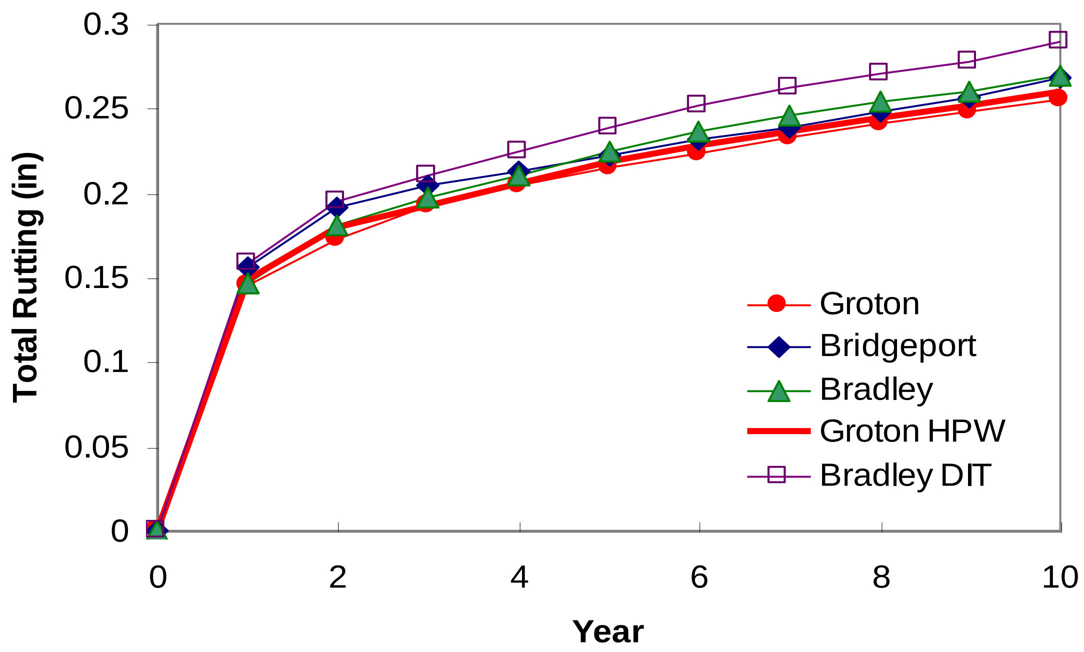

Figure 115 Effect of Climate on Total Rutting – Connecticut

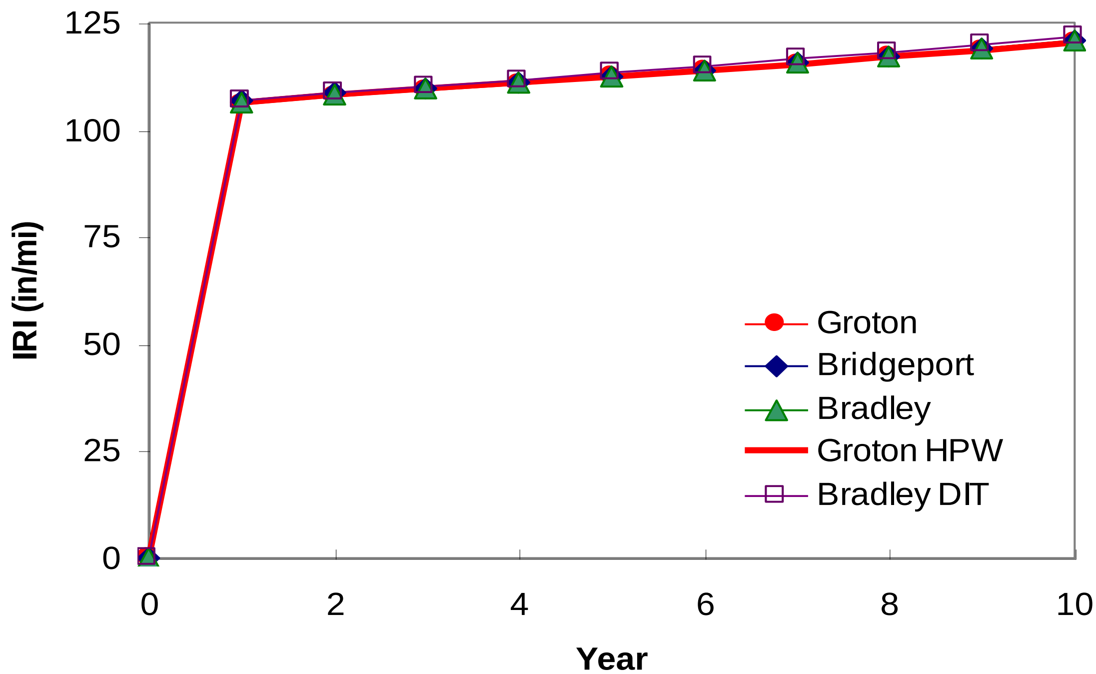

Figure 116 Effect of Climate on IRI – Connecticut

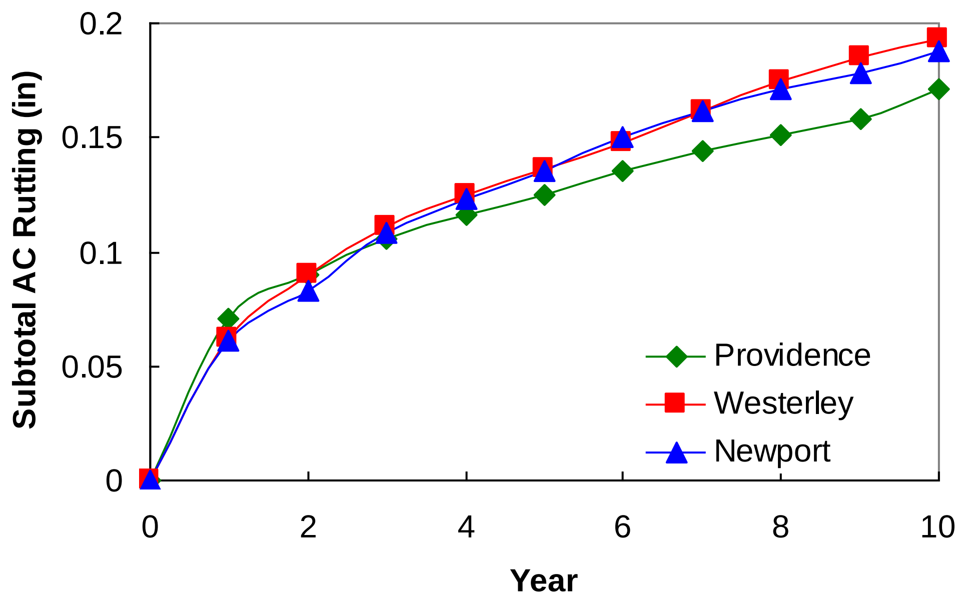

Figure 117 Effect of Climate on Subtotal AC Rutting – Rhode Island

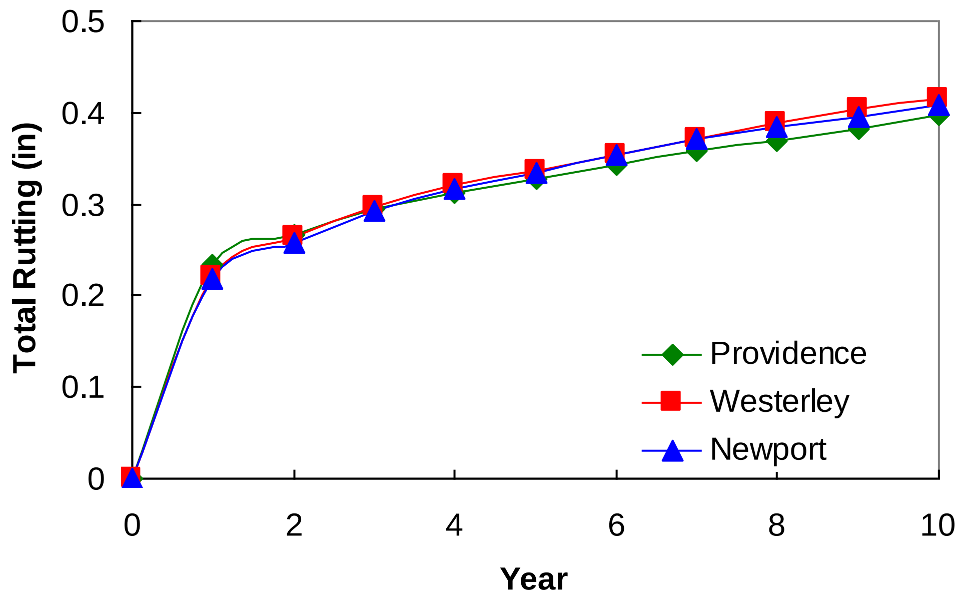

Figure 118 Effect of Climate on Total Rutting – Rhode Island

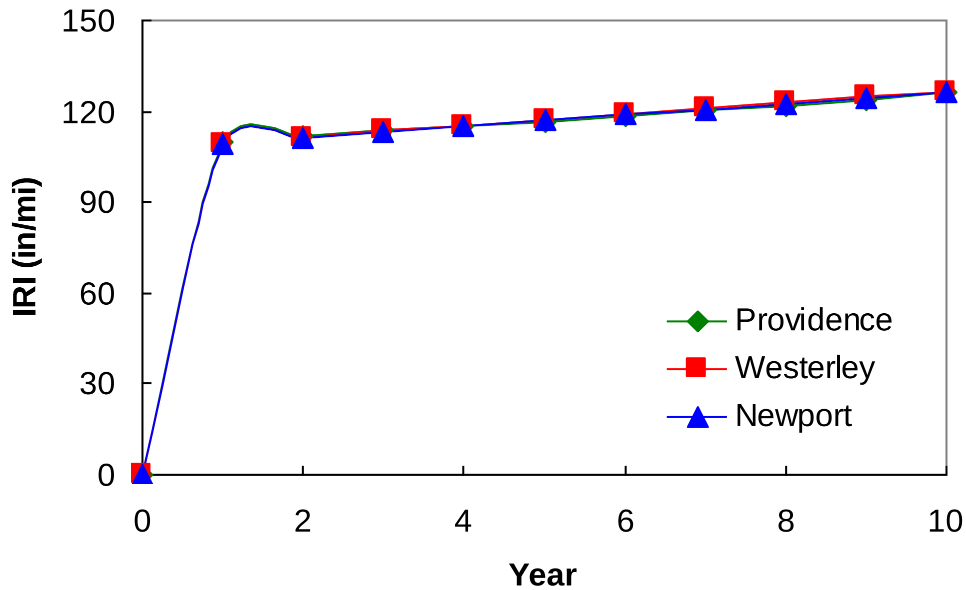

Figure 119 Effect of Climate on IRI – Rhode Island

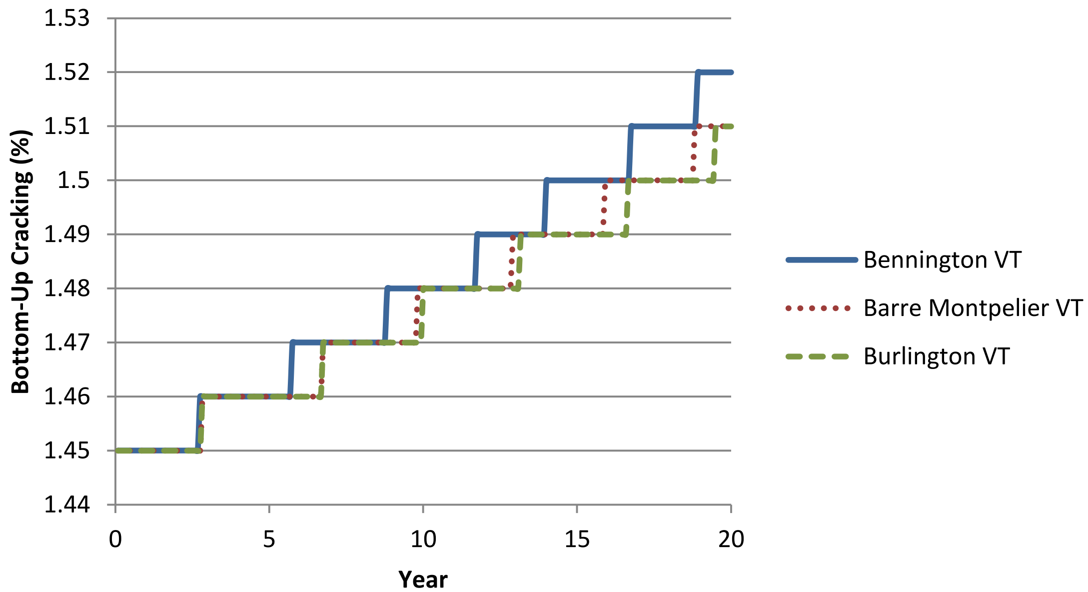

Figure 120 Effect of Climate on Bottom-Up Cracking – Vermont

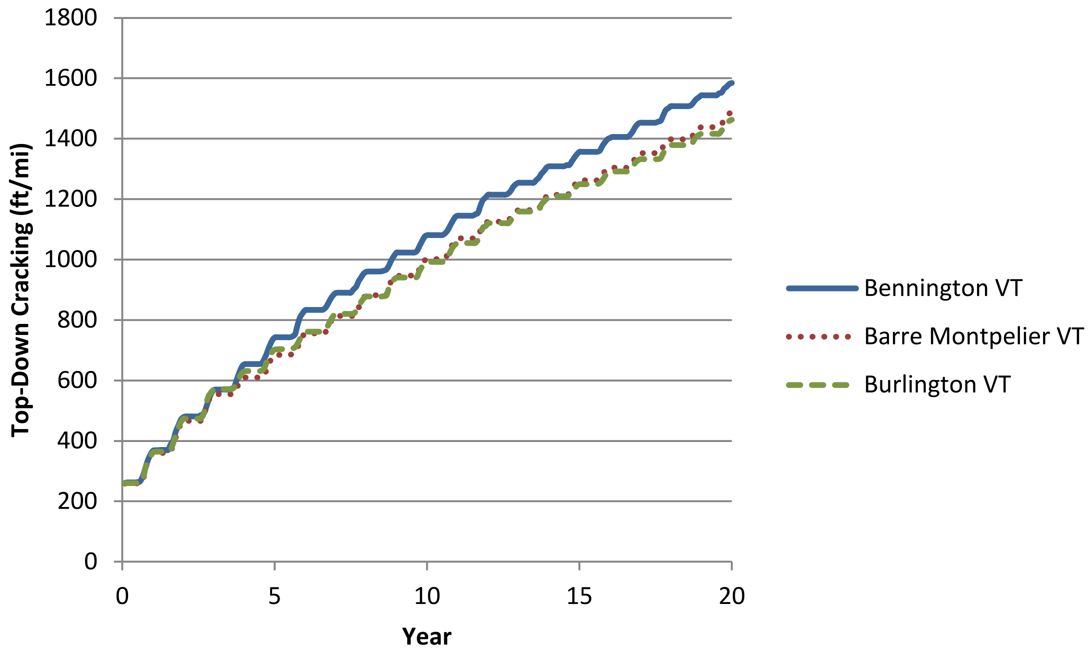

Figure 121 Effect of Climate on Top-Down Cracking – Vermont

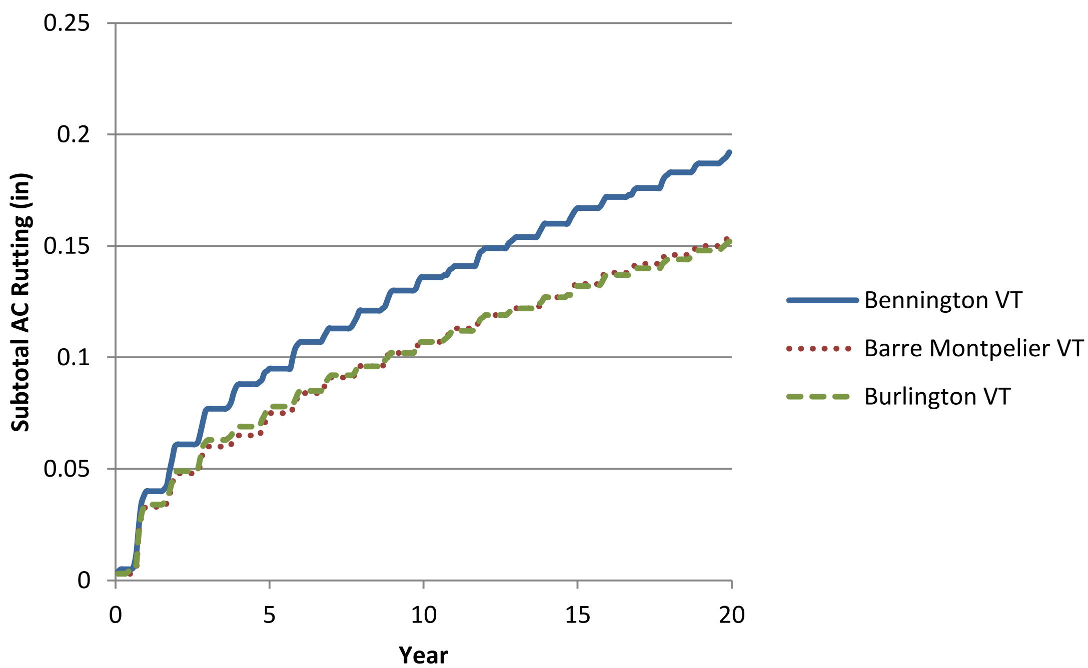

Figure 122 Effect of Climate on Subtotal AC Rutting – Vermont

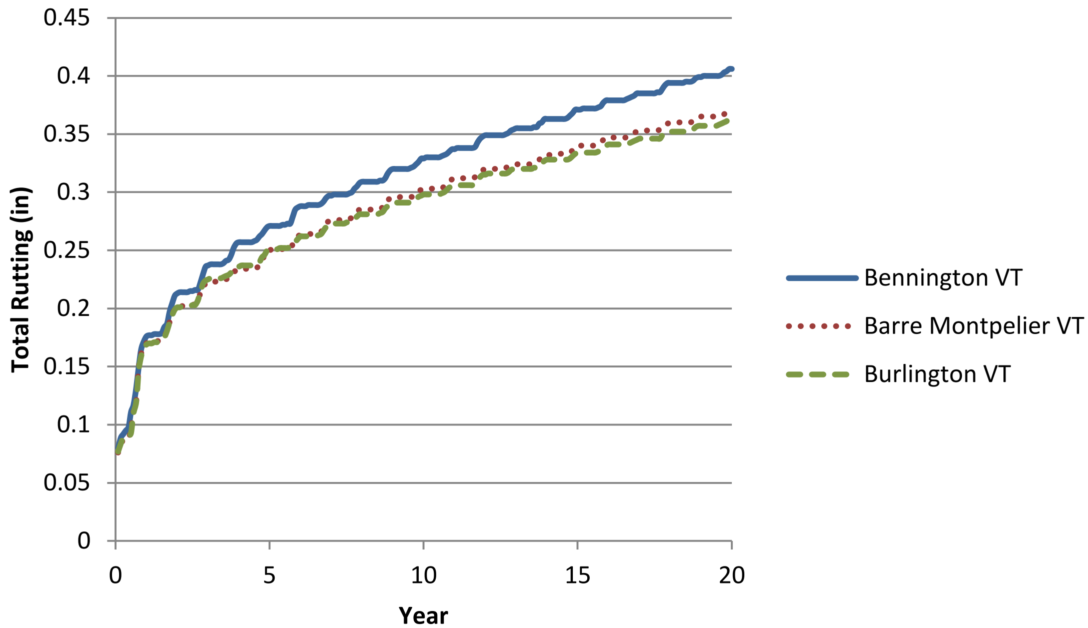

Figure 123 Effect of Climate on Total Rutting – Vermont

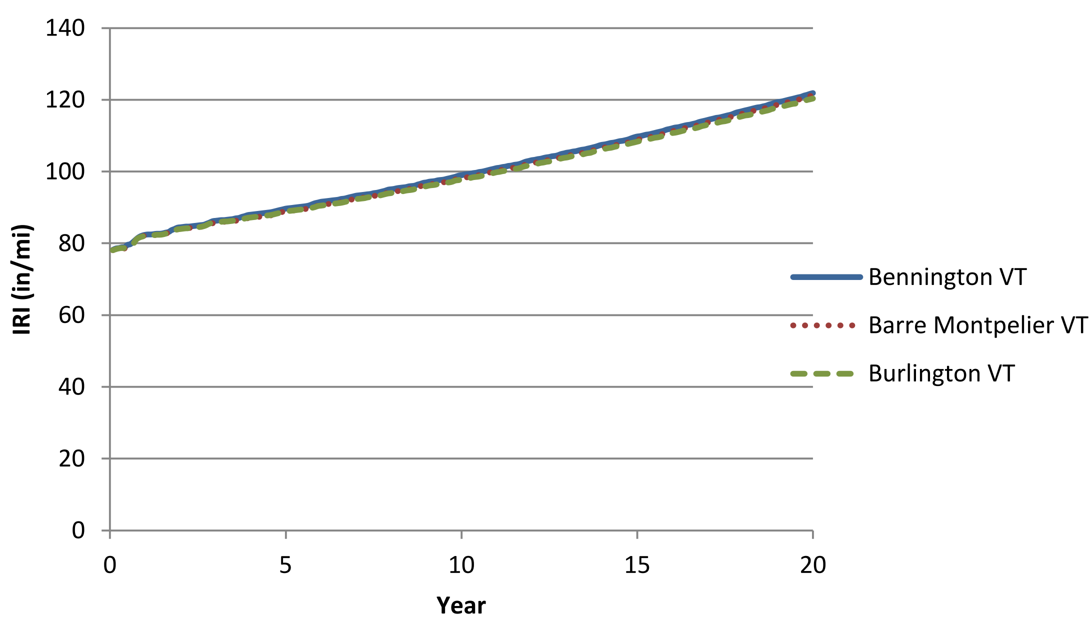

Figure 124 Effect of Climate on IRI – Vermont

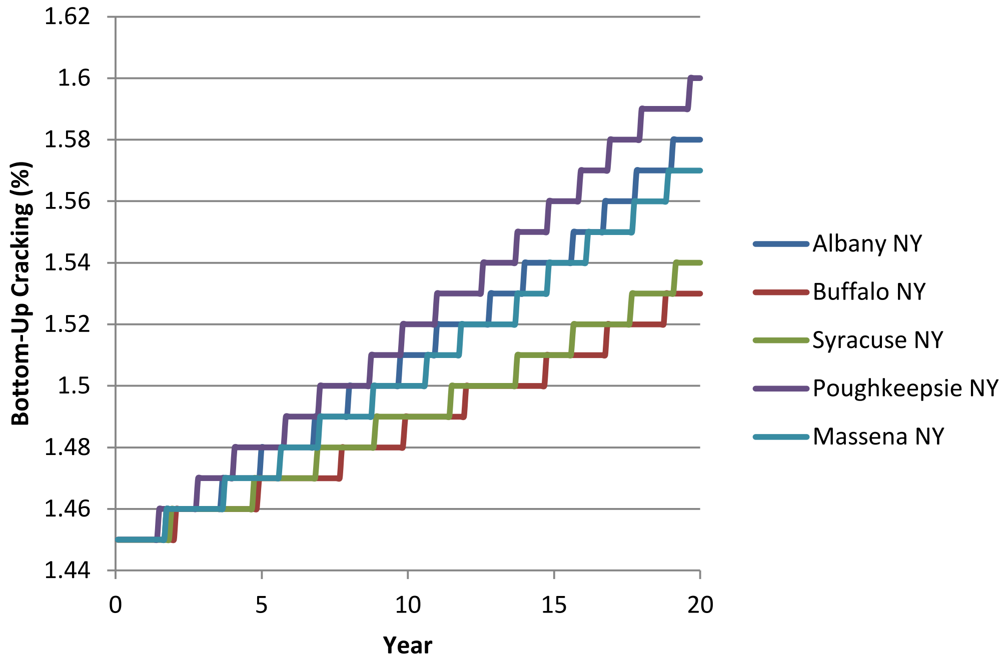

Figure 125 Effect of Climate on Bottom-Up Cracking – New York

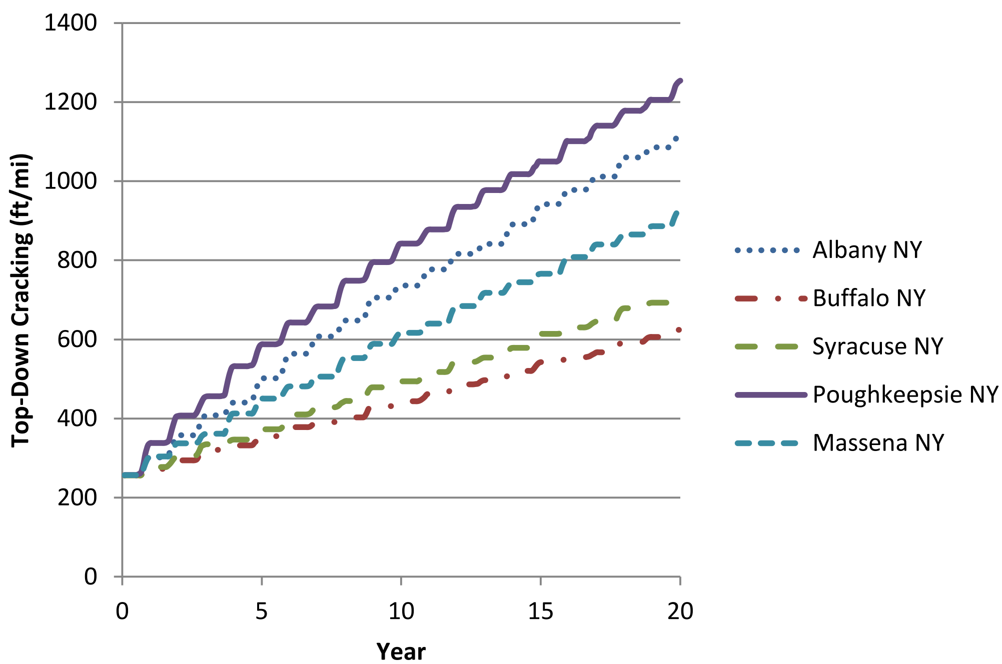

Figure 126 Effect of Climate on Top-Down Cracking – New York

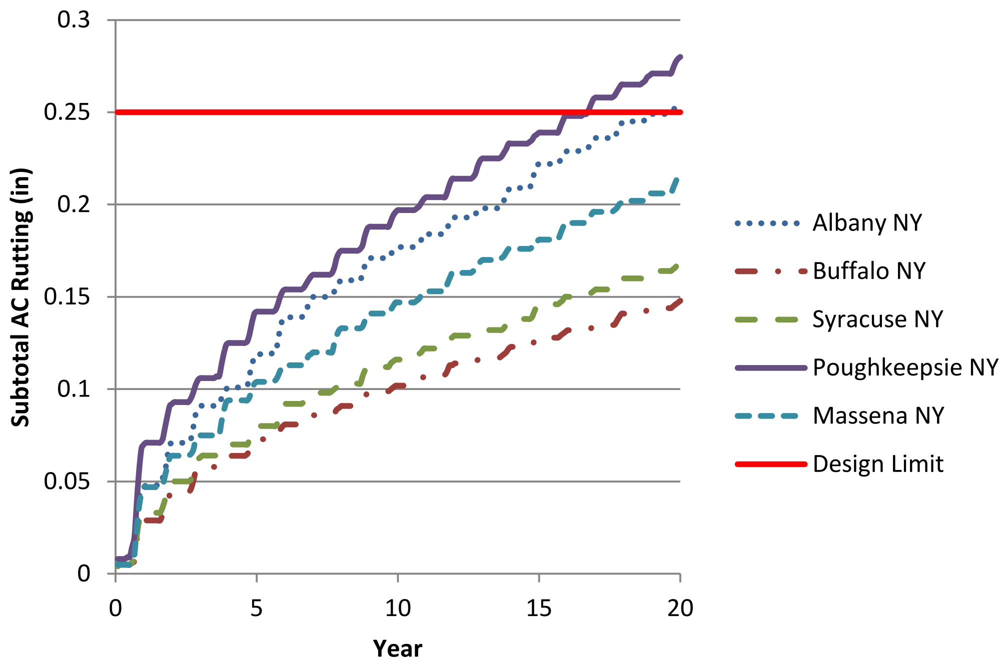

Figure 127 Effect of Climate on Subtotal AC Rutting – New York

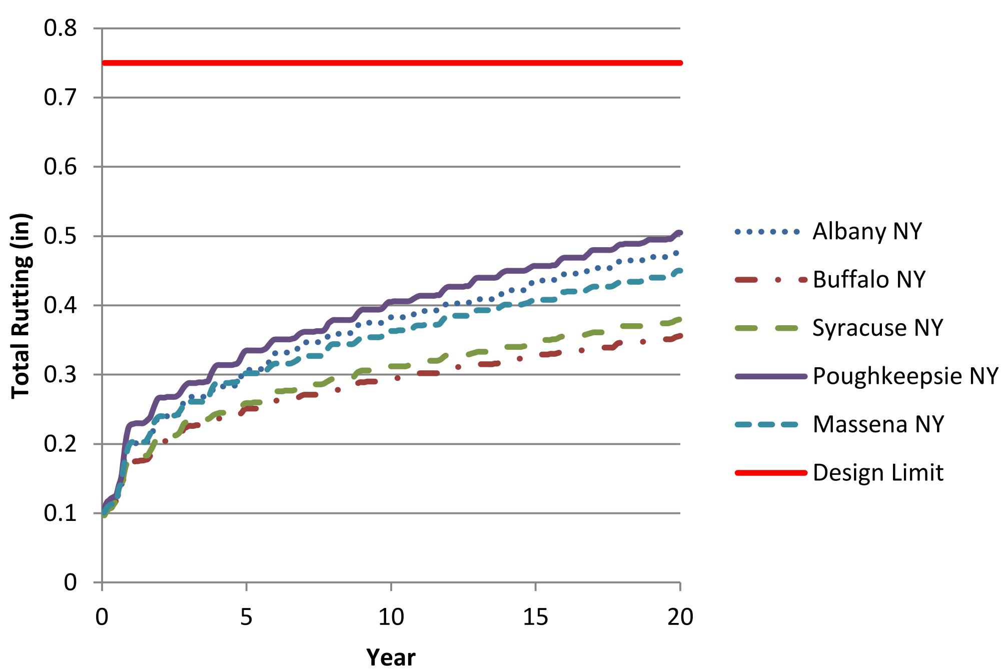

Figure 128 Effect of Climate on Total Rutting – New York

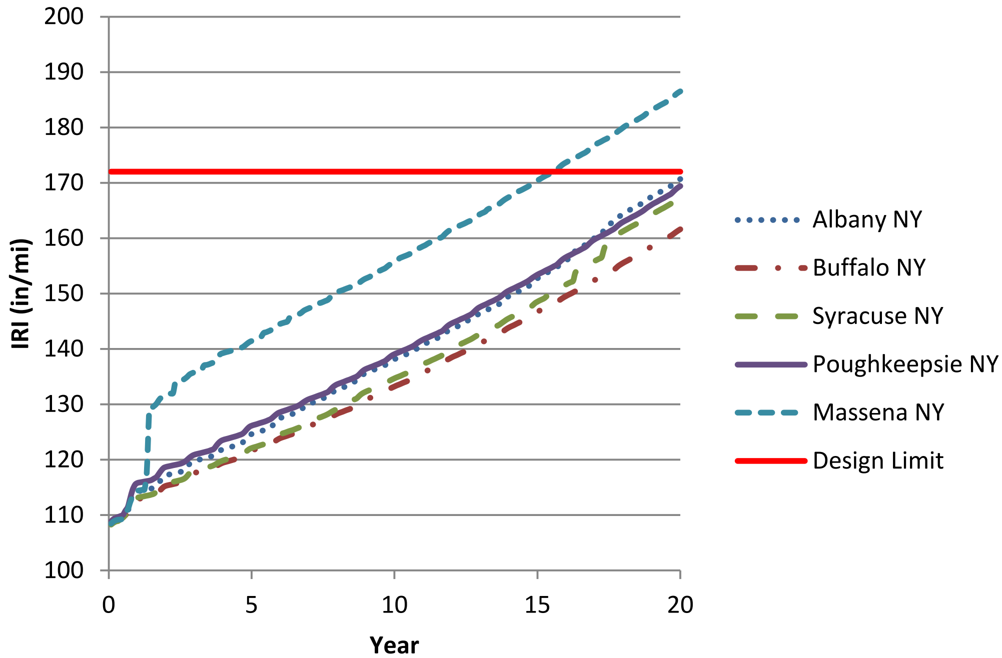

Figure 129 Effect of Climate on IRI – New York

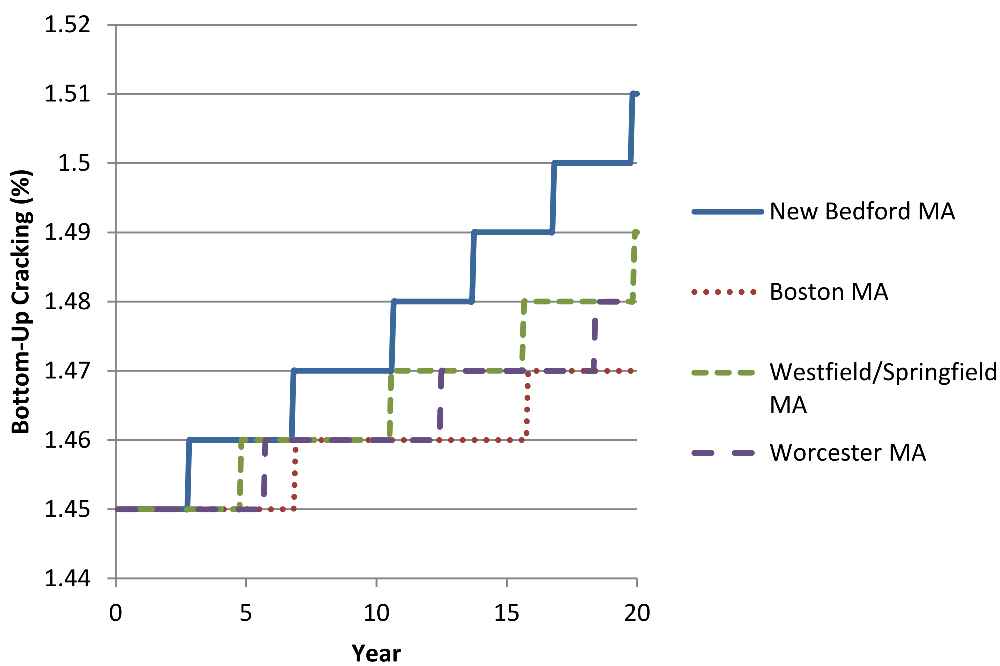

Figure 130 Effect of Climate on Bottom-Up Cracking – Massachusetts

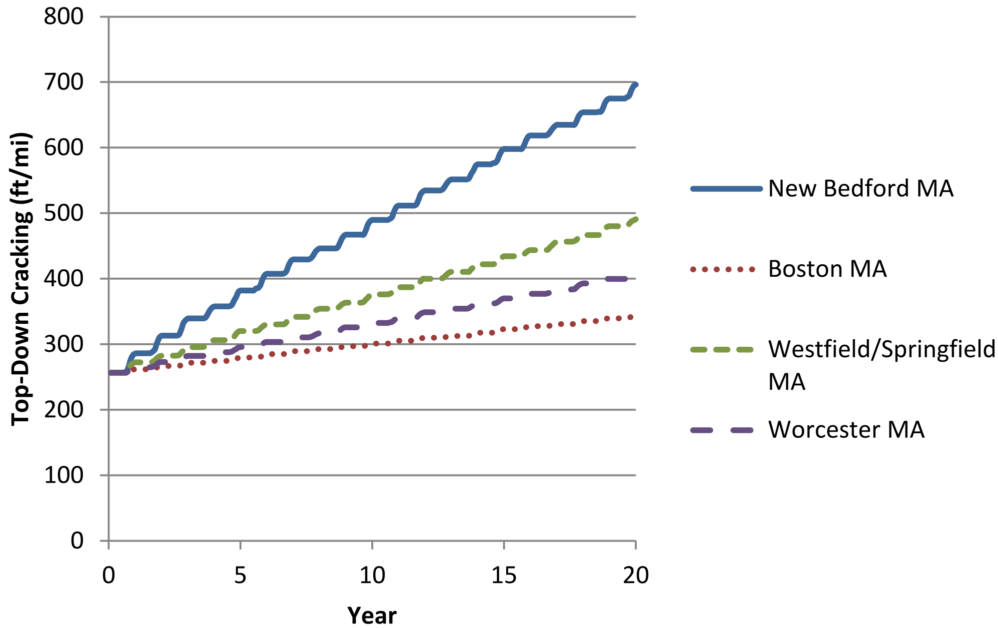

Figure 131 Effect of Climate on Top-Down Cracking – Massachusetts

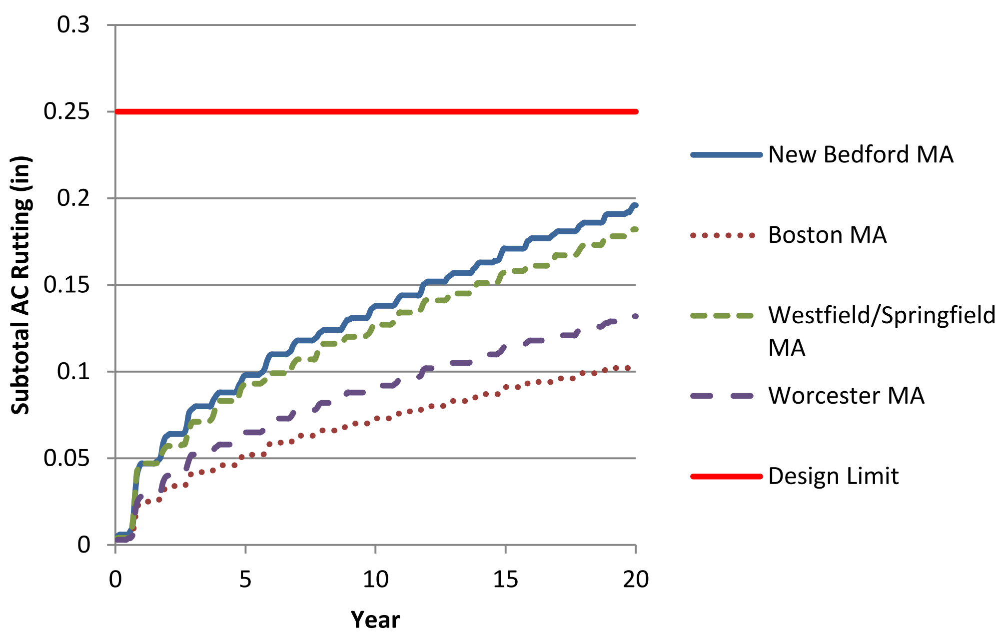

Figure 132 Effect of Climate on Subtotal AC Rutting – Massachusetts

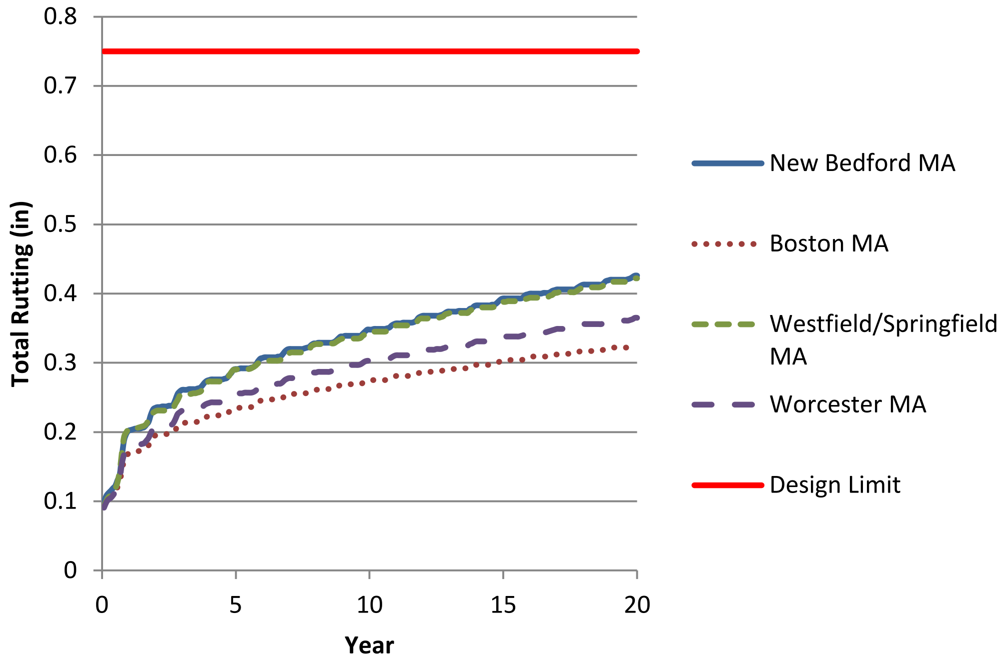

Figure 133 Effect of Climate on Total Rutting – Massachusetts

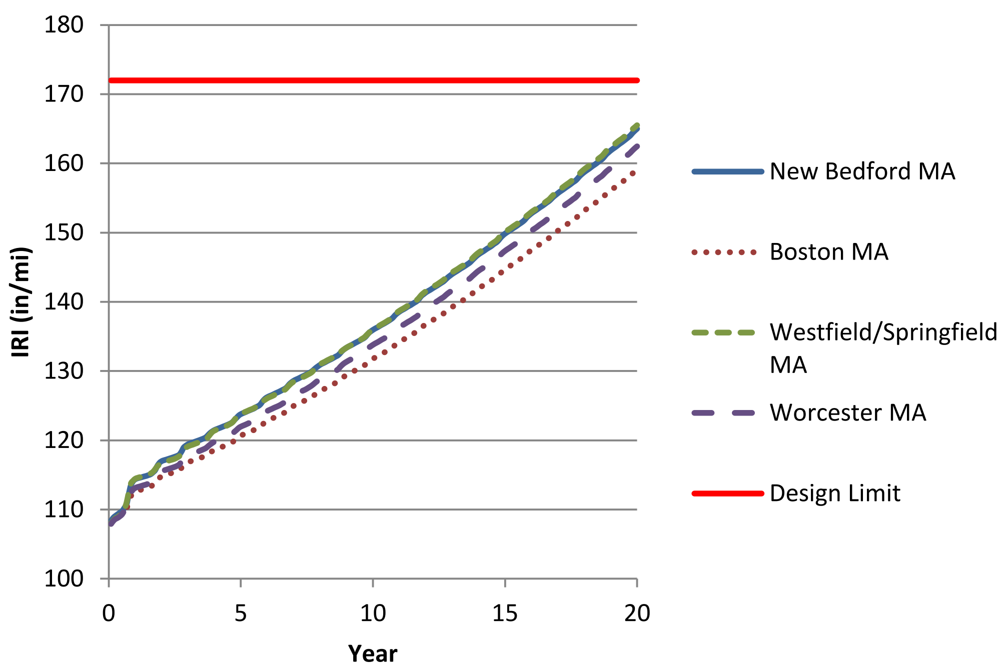

Figure 134 Effect of Climate on IRI – Massachusetts

The literature review also shows that the climatic data have a significant effect on the thermal cracking predictions. The occurrence was only observed in New York State, where the thermal cracking model worked well, except for the Buffalo, NY location, where the thermal crack length values decreased with the increase of time. In the other states (VT and MA) the task could not be completed due to the MEPDG version 1.1 software shortcoming (transverse cracking values equal to “0”). The example of the climate effect on the thermal cracking distress is seen in Figure 135.

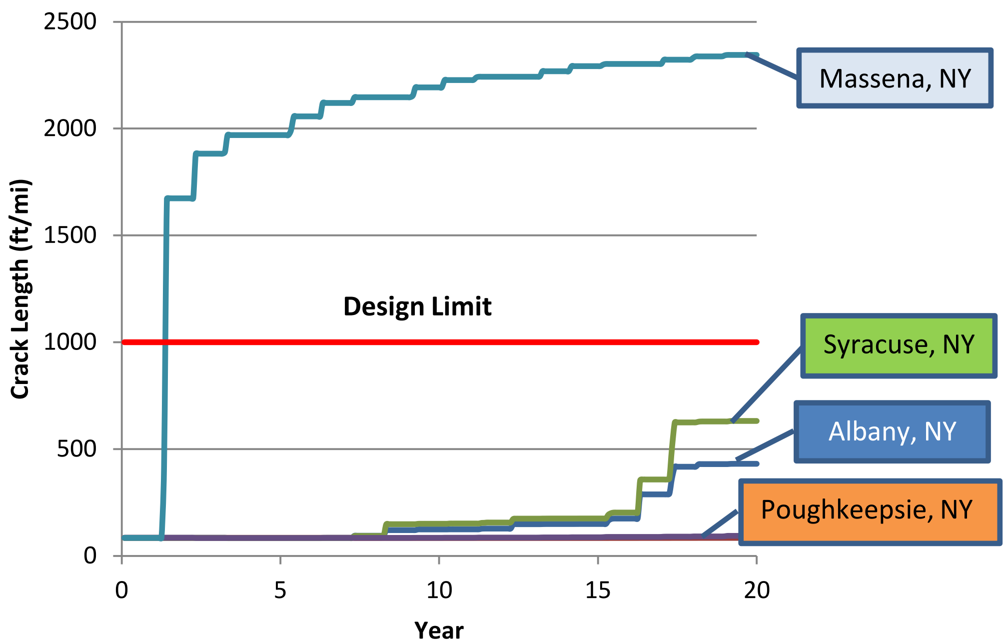

Figure 135 Effect of Climate on Thermal Cracking in NY State

The inference drawn from Level 2 analysis is similar to that obtained from Level 3 results. Since climate data is obtained in a similar manner for all levels of design in the MEPDG, the variation of distresses with change in climate follows a trend similar to that with respect to any other input parameter.

Inference: Climate data for a construction location can be interpolated without major deviation from expected performance prediction for Levels 2 and 3 of design in the MEPDG. Proper care should be taken during interpolation of climate from surrounding stations, and triangulation is an effective method of obtaining the resultant climate data.

Effect of Water Table Depth

Water table depth was kept at a default level of 8 ft for the control case for all pavement structures studied. Results of the study show that water table depths greater than 8 ft have no effect on the predicted distresses, whereas distresses like bottom-up (fatigue) cracking and total rutting (which includes subgrade rutting) increase with a decrease in the depth of the water table. Level 2 results predicted lower values of distresses than Level 3; hence water table depth variation is less significant for Level 2 design as compared to Level 3.

Therefore, in areas with water table depth greater than 12 ft, water table depth need not be measured with great precision and may be obtained by averaging data from surrounding stations whose data is collected by the United States Geological Survey website. The conclusion above was the result of a number of trial runs on varying water table depths greater than 12 feet, which showed no change in predicted distress values. An attempt has been made to compile the water table depth data for the states of New Hampshire and the data is presented in the Appendix of the report.

An interesting observation from Figure 137 is that even though the performance prediction trend of total rutting with time shows no serious deviation for increasing water table depth in terms of magnitude, the year of operation in which the pavement failed by reaching the criterion is different for each water table depth. Therefore, water table depth is an important parameter to which rutting is sensitive under condition of failure.

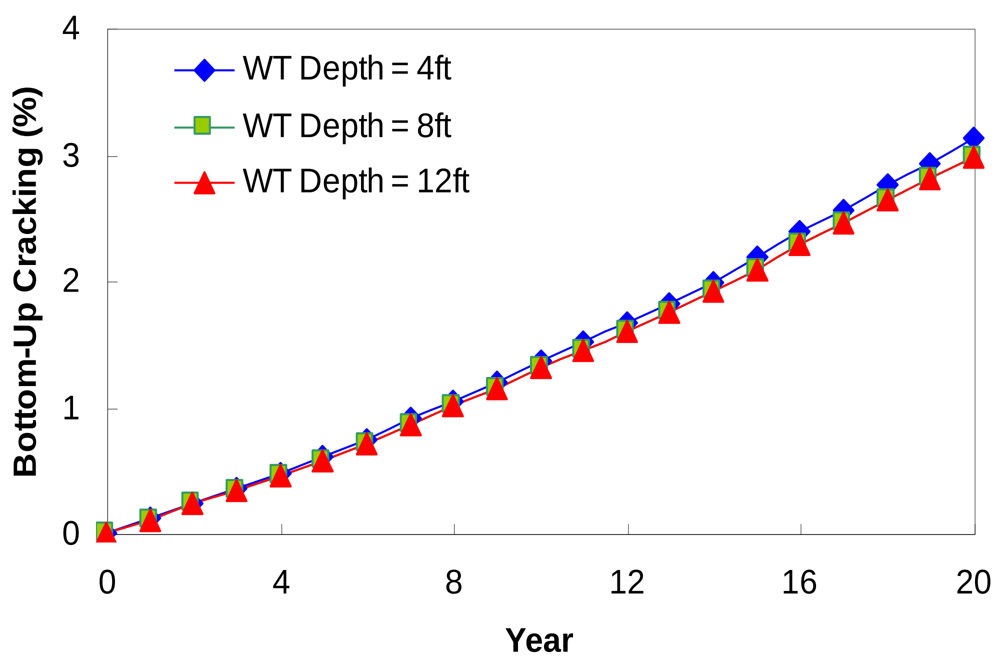

Figure 136 Effect of Water Table Depth on Bottom-Up Cracking – New Hampshire

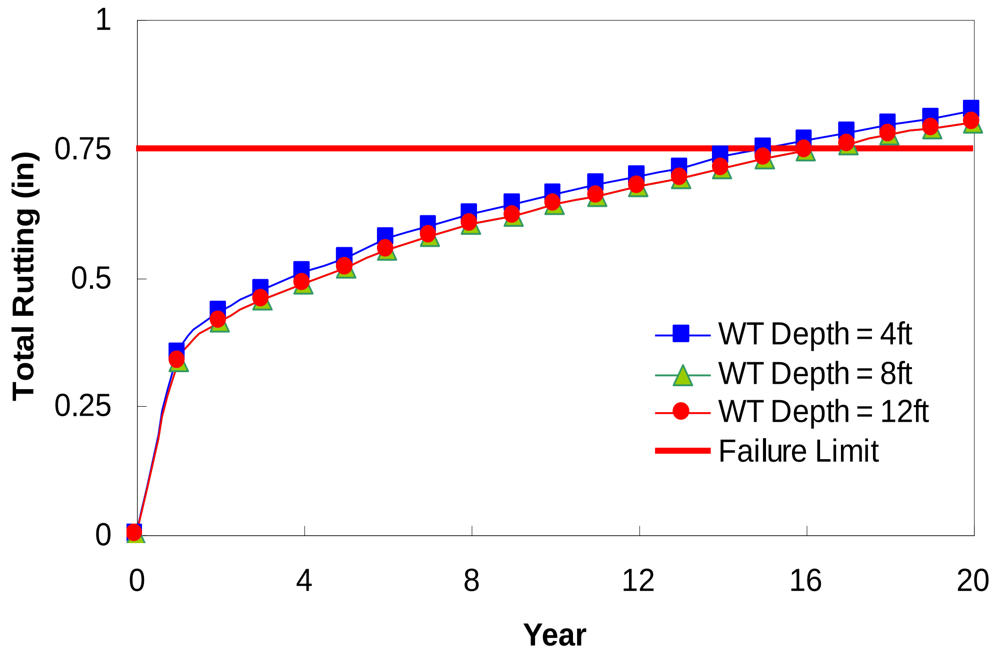

Figure 137 Effect of Water Table Depth on Total Rutting – New Hampshire

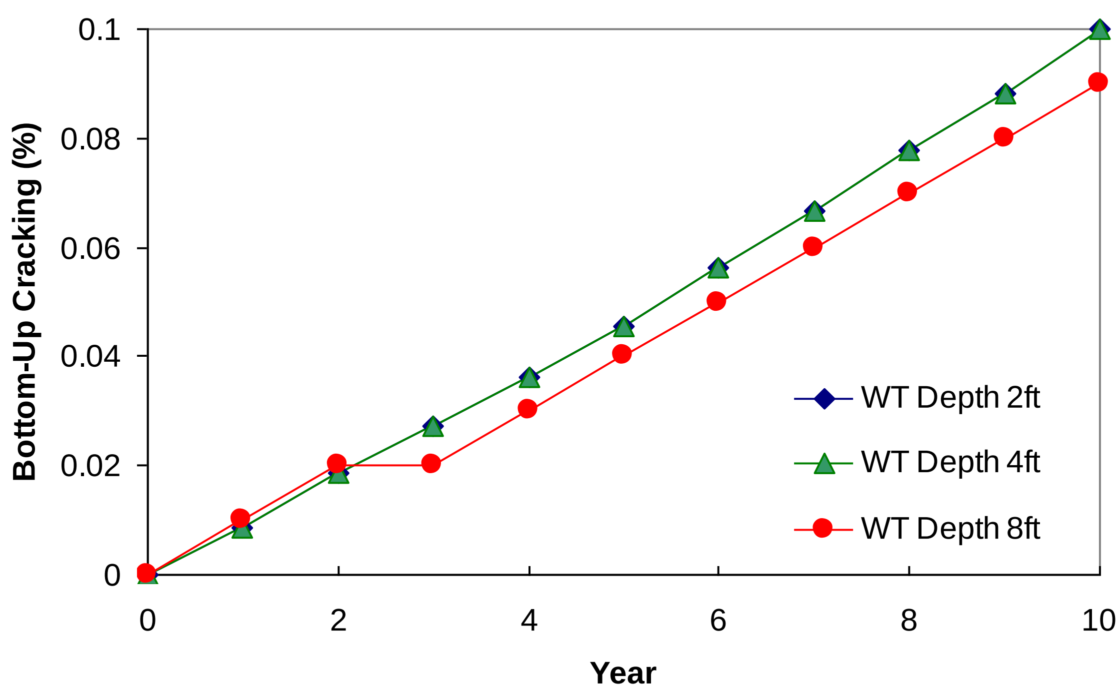

Figure 138 Effect of Water Table Depth on Bottom-Up Cracking – Connecticut

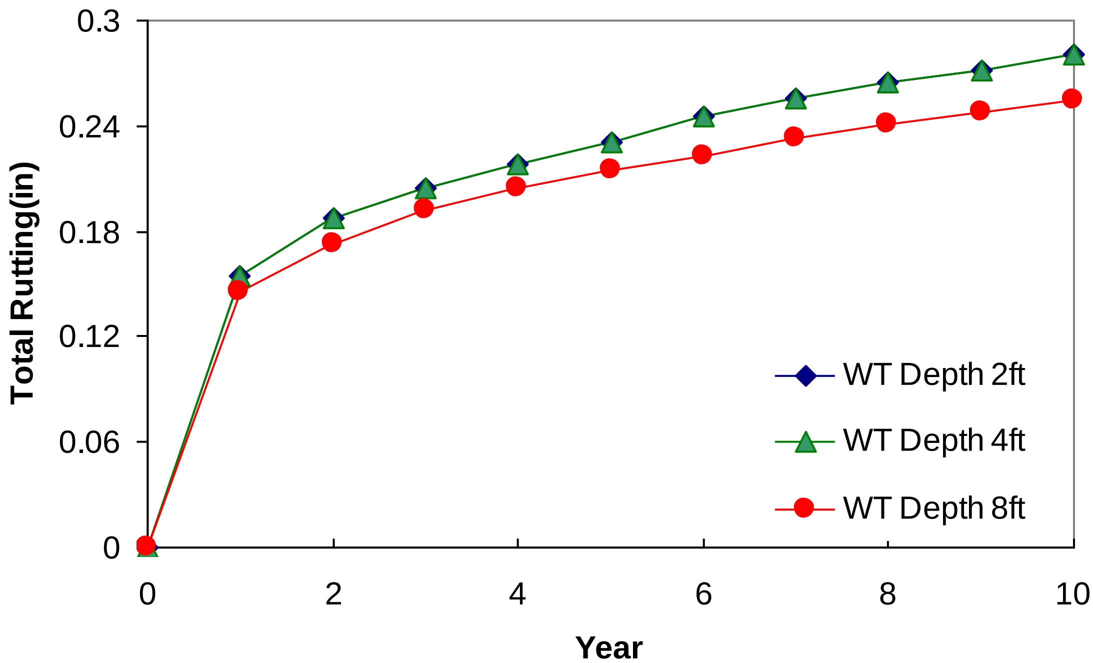

Figure 139 Effect of Water Table Depth on Total Rutting – Connecticut

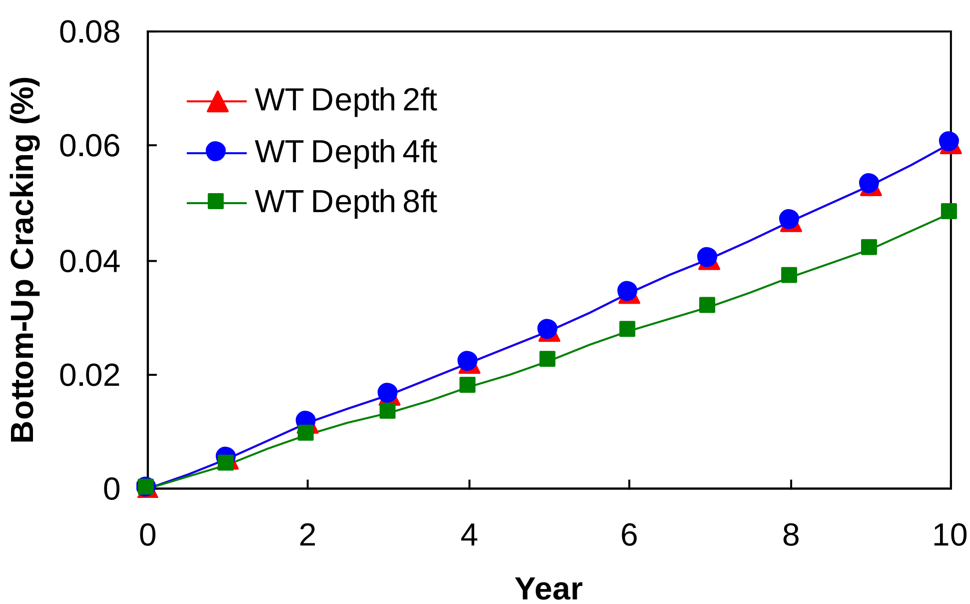

Figure 140 Effect of Water Table Depth on Bottom-Up Cracking – Maine

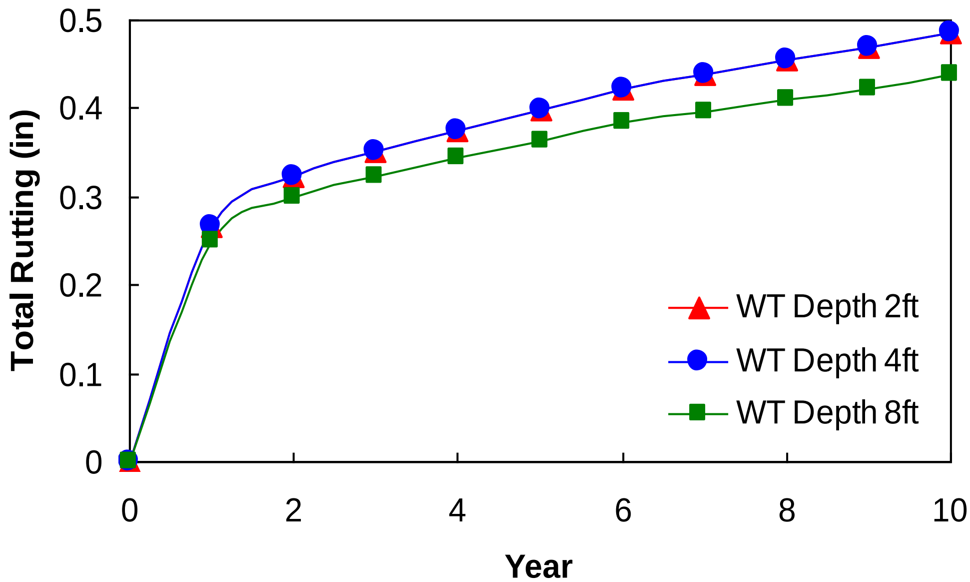

Figure 141 Effect of Water Table Depth on Total Rutting – Maine

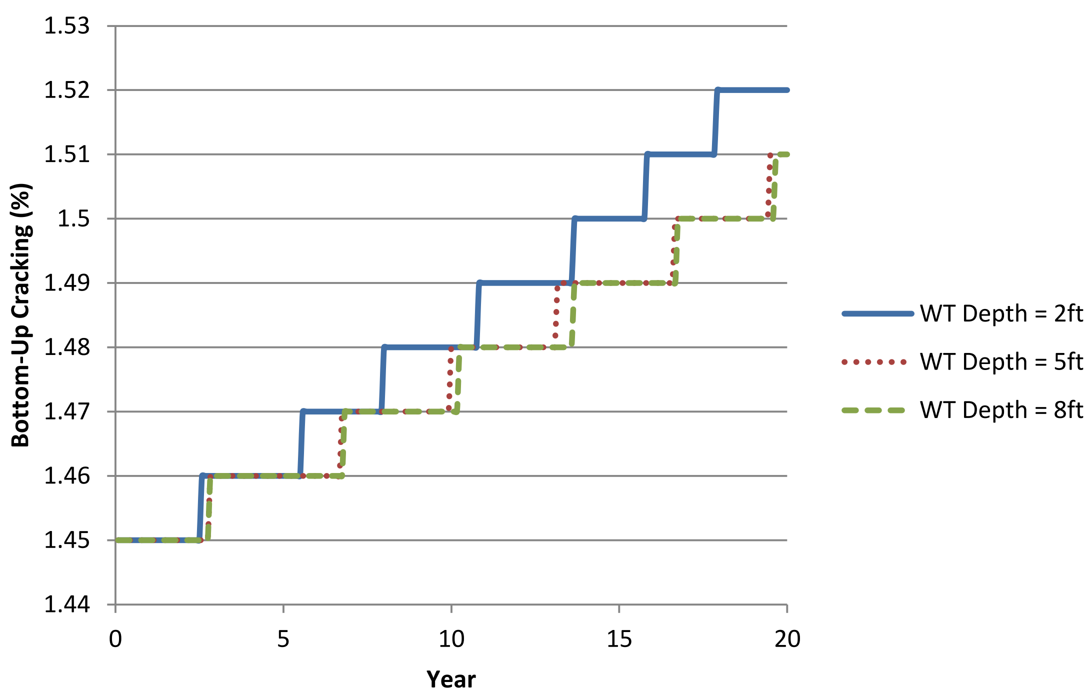

Figure 142 Effect of Water Table Depth on Bottom-Up Cracking - Vermont

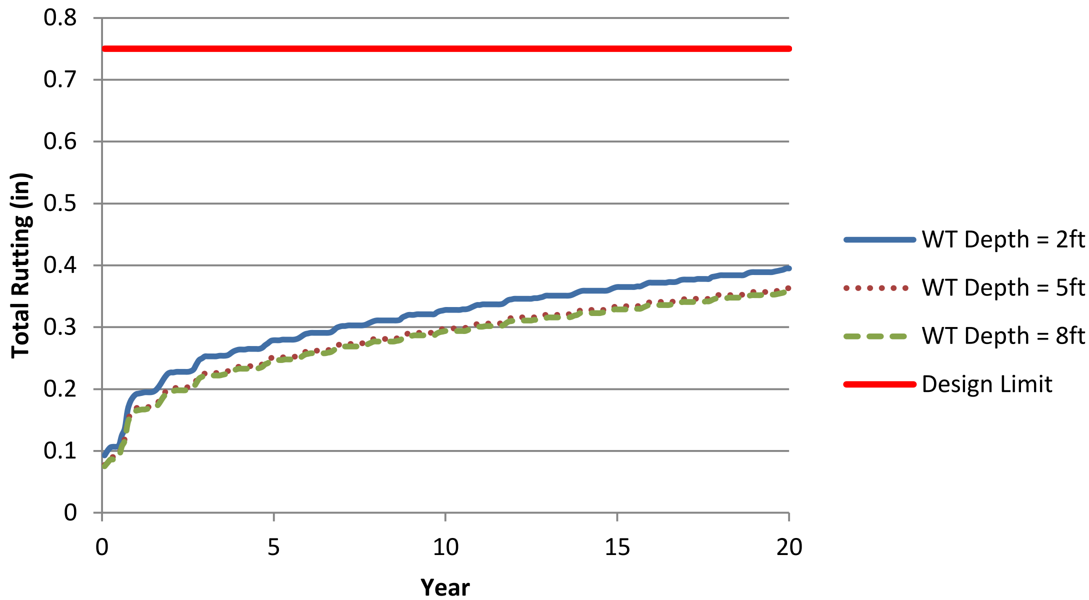

Figure 143 Effect of Water Table Depth on Total Rutting - Vermont

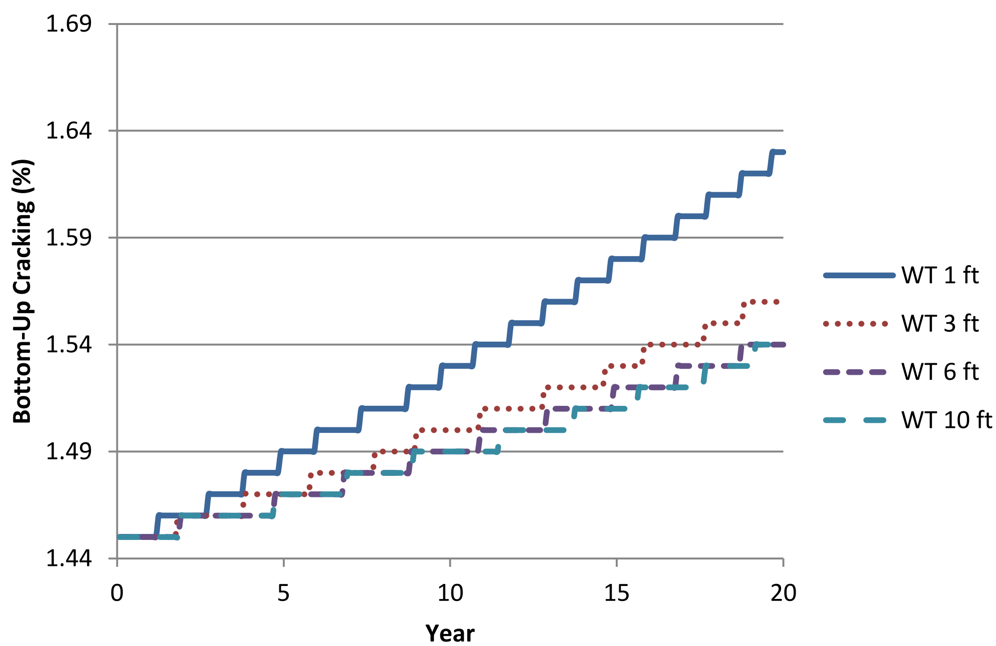

Figure 144 Effect of Water Table Depth on Bottom-Up Cracking - New York

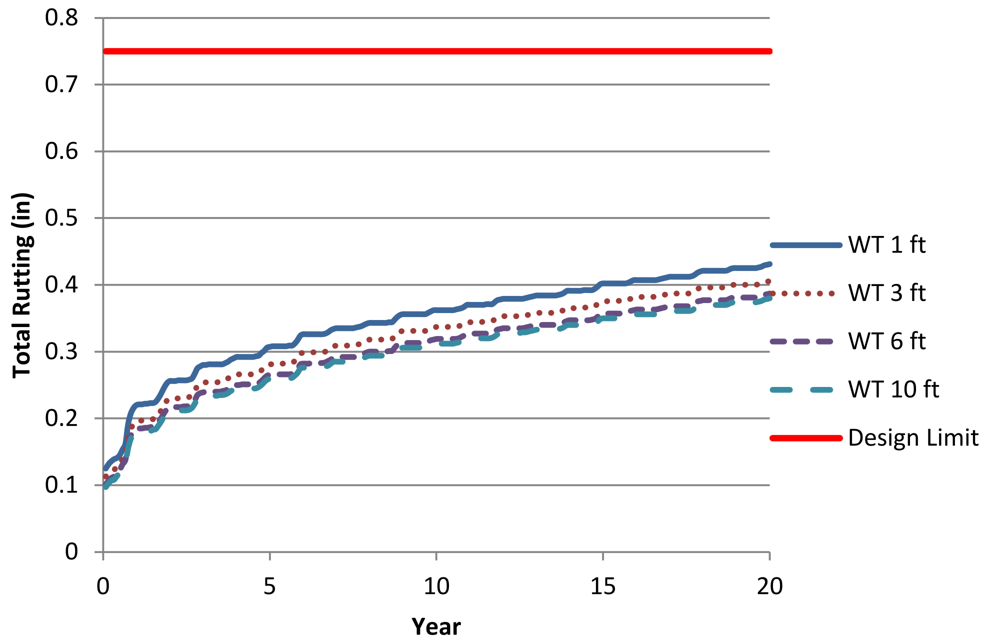

Figure 145 Effect of Water Table Depth on Total Rutting - New York

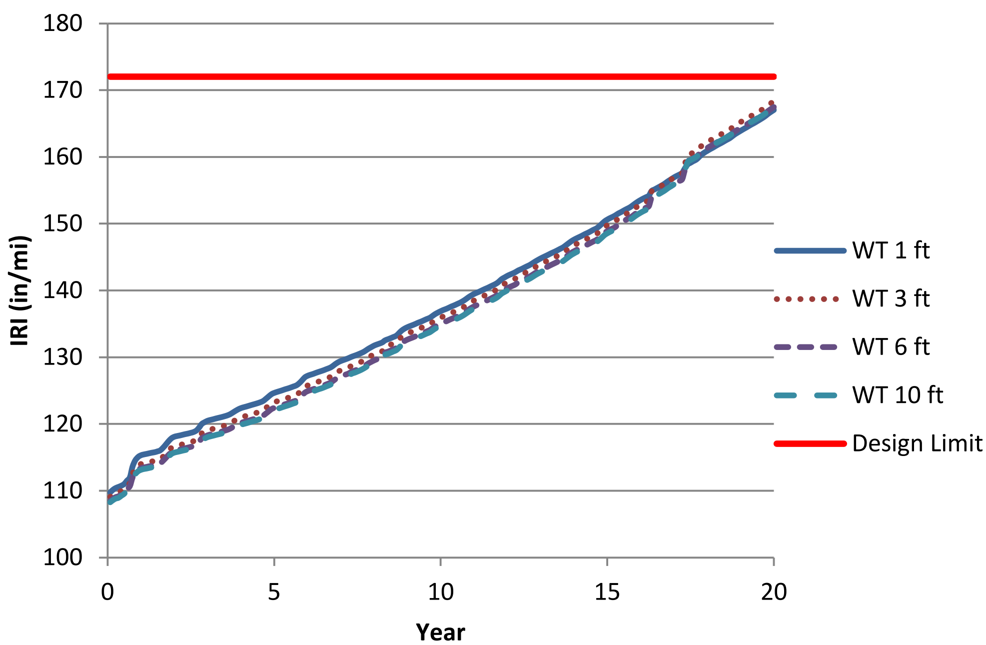

Figure 146 Effect of Water Table Depth on IRI - New York

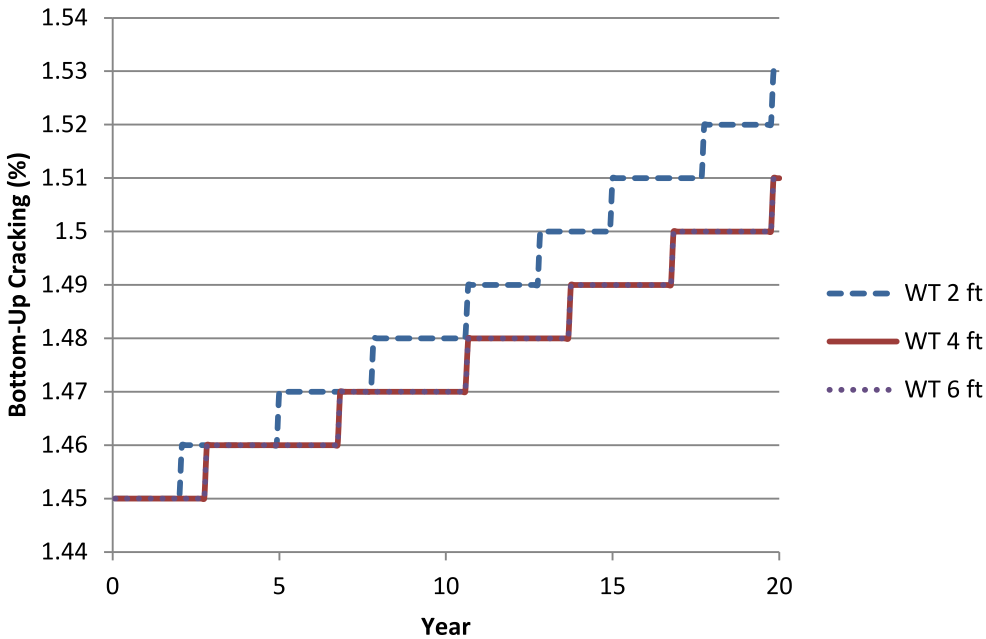

Figure 147 Effect of Water Table Depth on Bottom-Up Cracking - Massachusetts

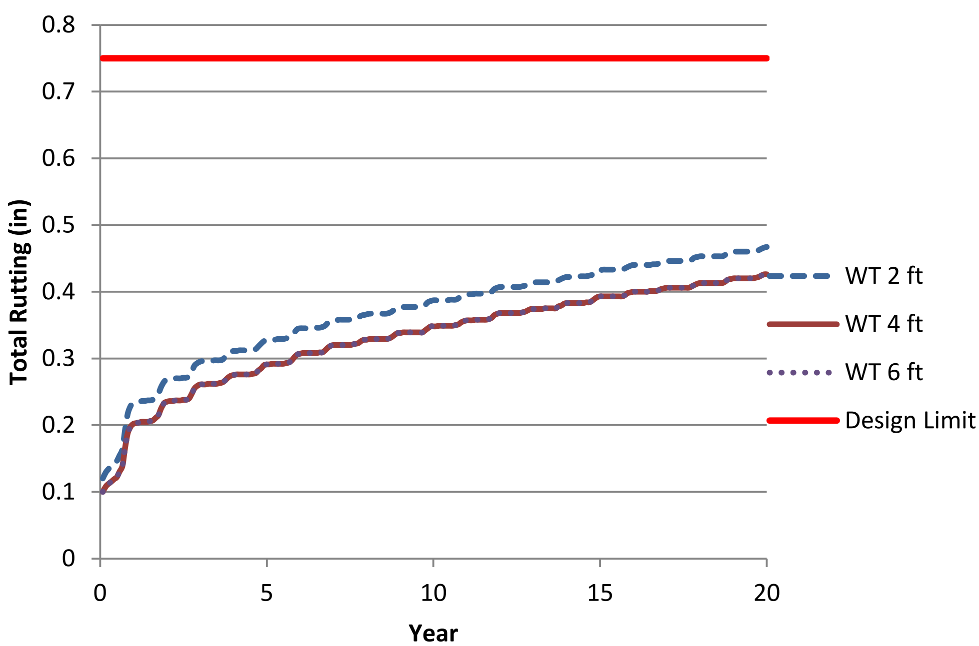

Figure 148 Effect of Water Table Depth on Total Rutting - Massachusetts

Additional runs were conducted to verify the results from significance studies provided in the M-E design guide for permanent deformation in flexible pavements (44). The sensitivity study conducted on total rutting (including subgrade rutting) concluded that water table depth plays a very significant role in prediction of rutting, particularly for subgrade soils of low strength (low resilient modulus values). Therefore, a series of runs was conducted for the selected water table depths using A-7-5 and A-7-6 subgrades having the lowest bearing capacity among all provided AASHTO soil classes. The results showed that effect of water table depth slightly increases with a weaker subgrade as compared to one with higher strength. Therefore, pavements constructed on weaker subgrades with high water tables must be provided additional structural capacity to support the traffic without undergoing a large magnitude of subgrade rutting.

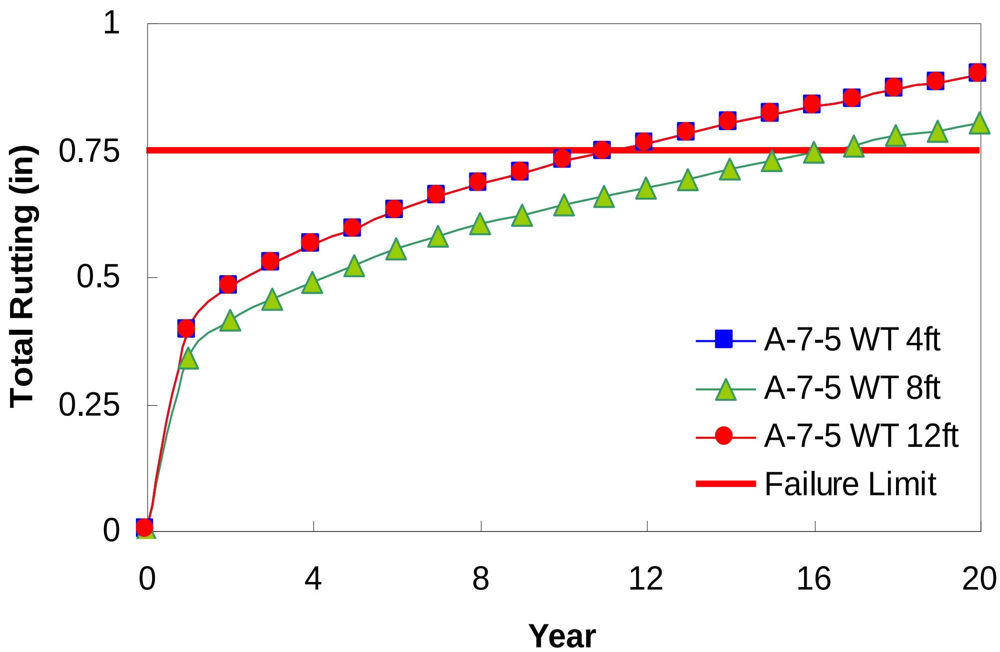

Figure 149 Effect of Water Table on Total Rutting with Weakest Subgrade - NH

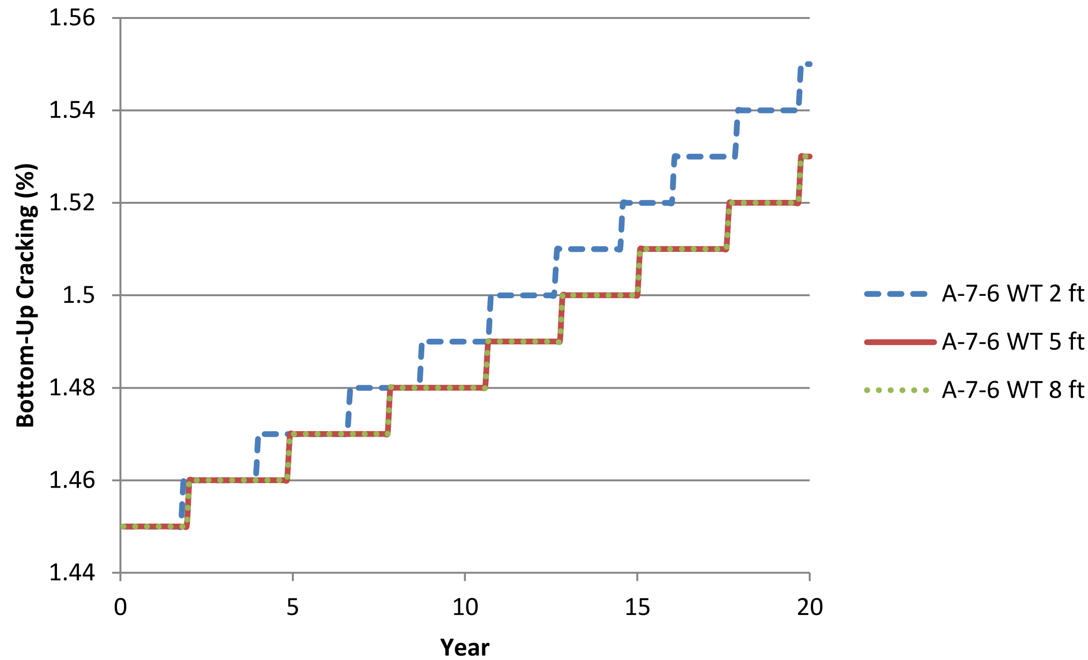

Figure 150 Effect of Water Table on Bottom-Up Cracking with Weakest Subgrade (VT)

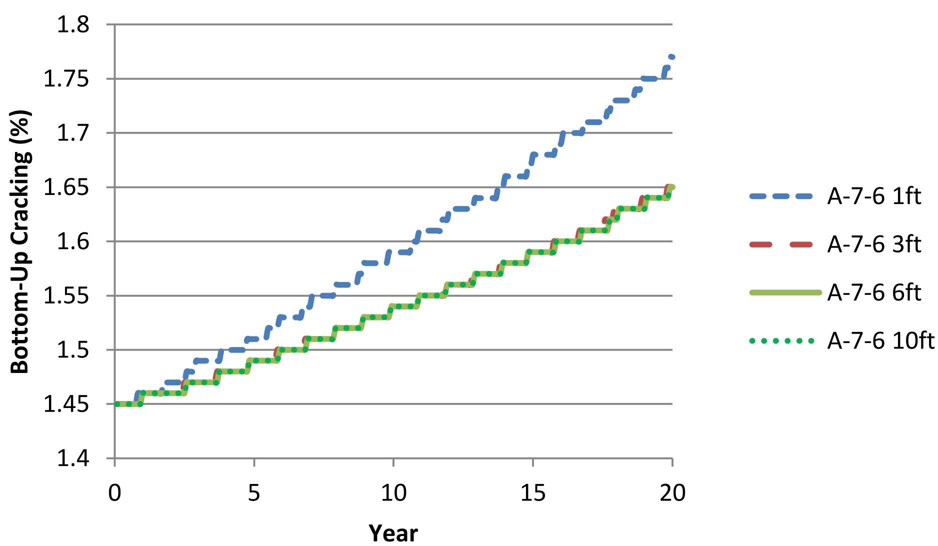

Figure 151 Effect of Water Table on Bottom-Up Cracking with Weakest Subgrade (NY)

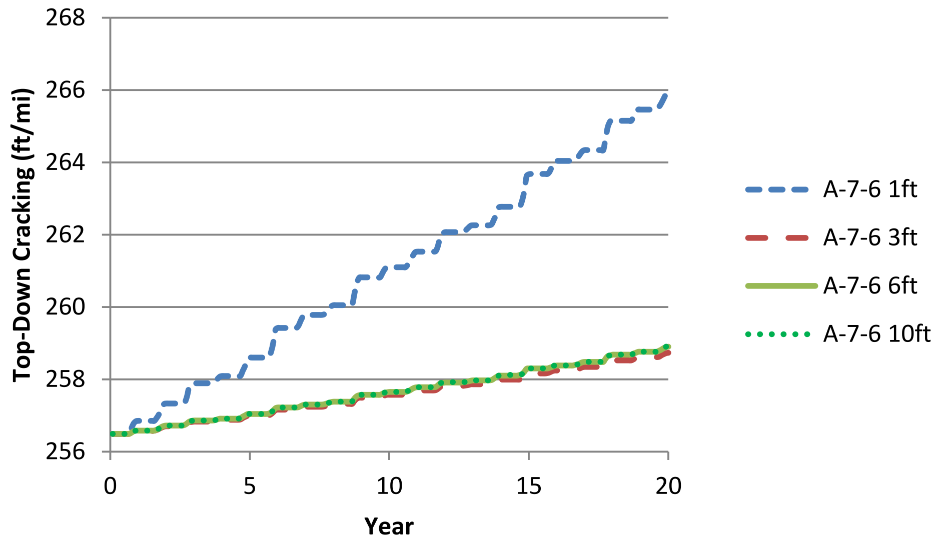

Figure 152 Effect of Water Table on Top-Down Cracking with Weakest Subgrade (NY)

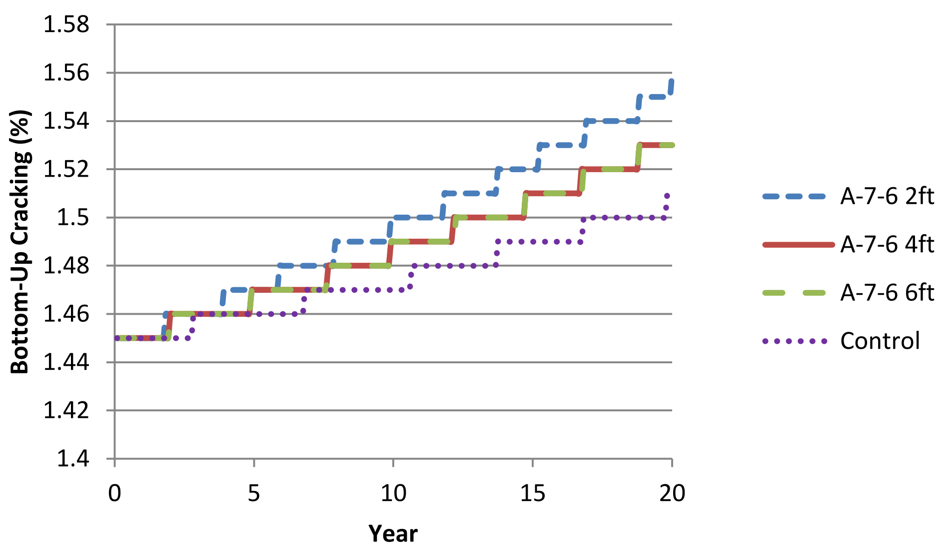

Figure 153 Effect of Water Table on Bottom-Up Cracking with Weakest Subgrade (MA)

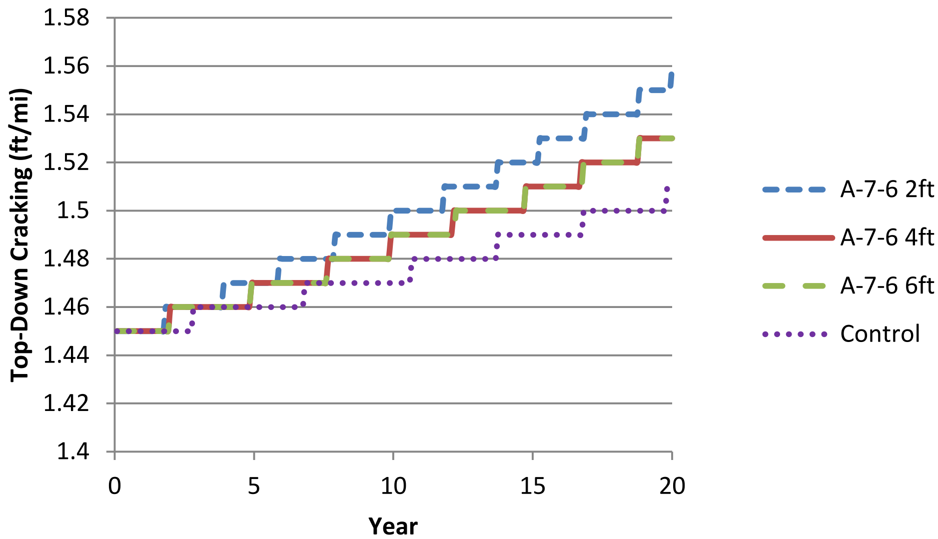

Figure 154 Effect of Water Table on Top-Down Cracking with Weakest Subgrade (MA)

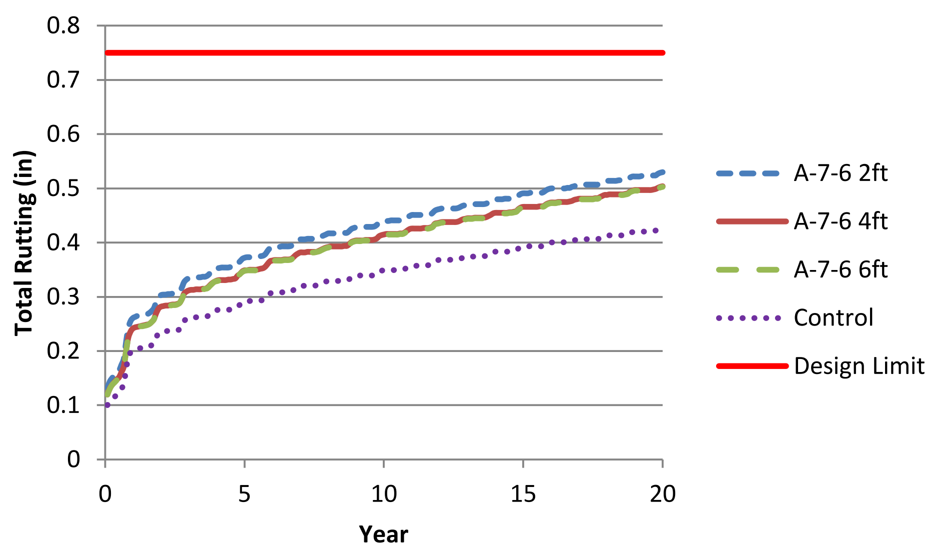

Figure 155 Effect of Water Table on Total Rutting with Weakest Subgrade (MA)

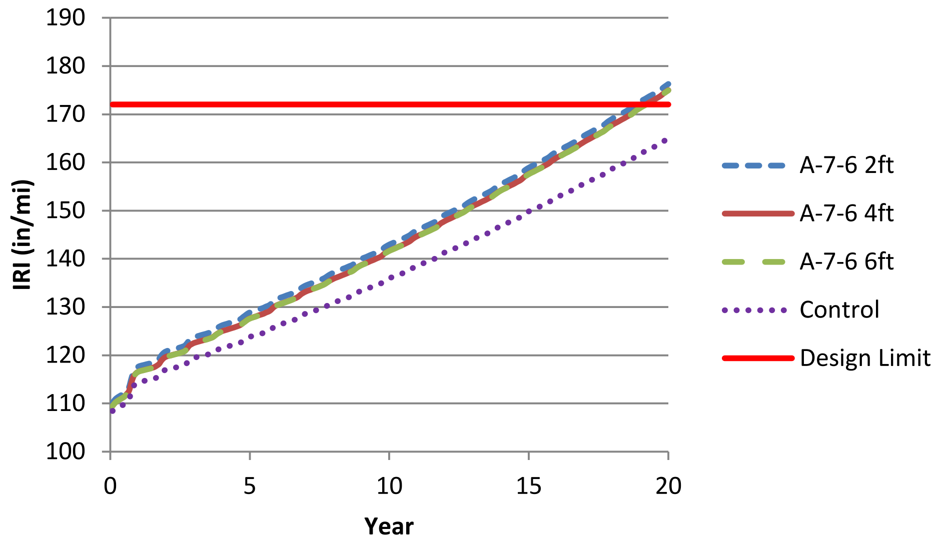

Figure 156 Effect of Water Table on IRI with Weakest Subgrade (MA)

6.1.3 Effect of Material Inputs on Pavement Distresses – Asphalt Concrete

Material inputs that have been identified from literature review to affect pavement performance can be broadly classified as asphalt concrete material properties and unbound layer inputs. Layer thicknesses are a characteristic of pavement layer structure. Material properties are varied using tolerances obtained from the state construction specifications; hence the sensitivity of distresses to each of the parameters is a measure of the adequacy of existing tolerances and suggestions are made using the results of graphical analysis as well as statistical analysis.

Air Void Content of Asphalt Concrete

Air voids in the asphalt concrete layer were obtained from testing data provided in the LTPP database. The tolerances for air void content for all the states studied were obtained and the results are graphically explained below. Pavement distresses were found to be significantly affected by change in air void content, particularly cracking (fatigue and longitudinal) followed by rutting. IRI also increased with an increase in air voids, but the significance of the effect was much less as compared to that on cracking and rutting. Air voids was also found to affect thermal cracking, which is described in later sections on thermal cracking.

The following plots of performance prediction with time show the results for Level 2 analysis, where the predicted values are much lower than those for Level 3.

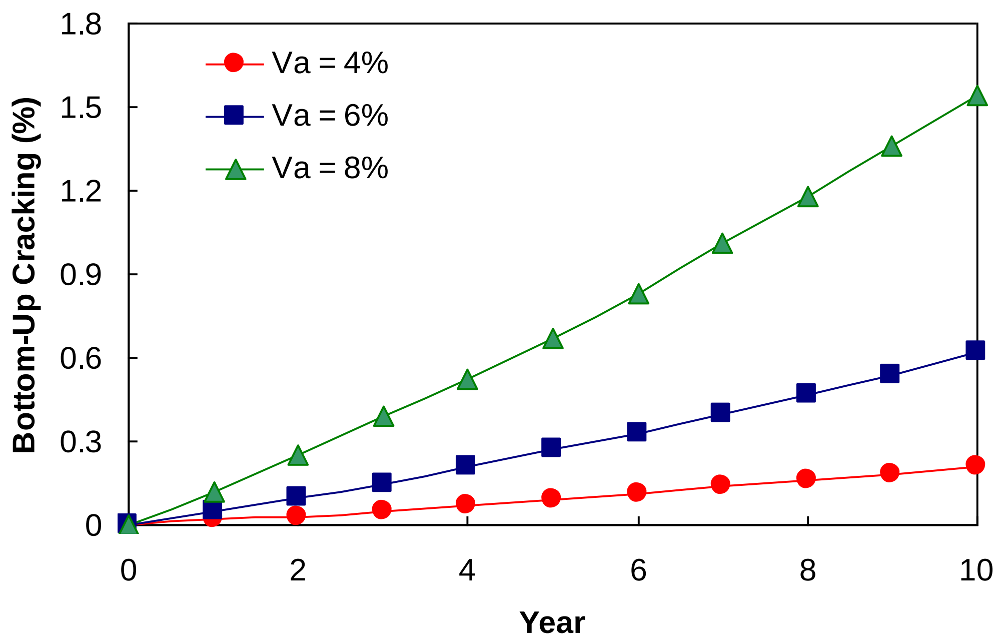

Figure 157 Effect of Air Void Content on Bottom-Up Cracking – New Hampshire

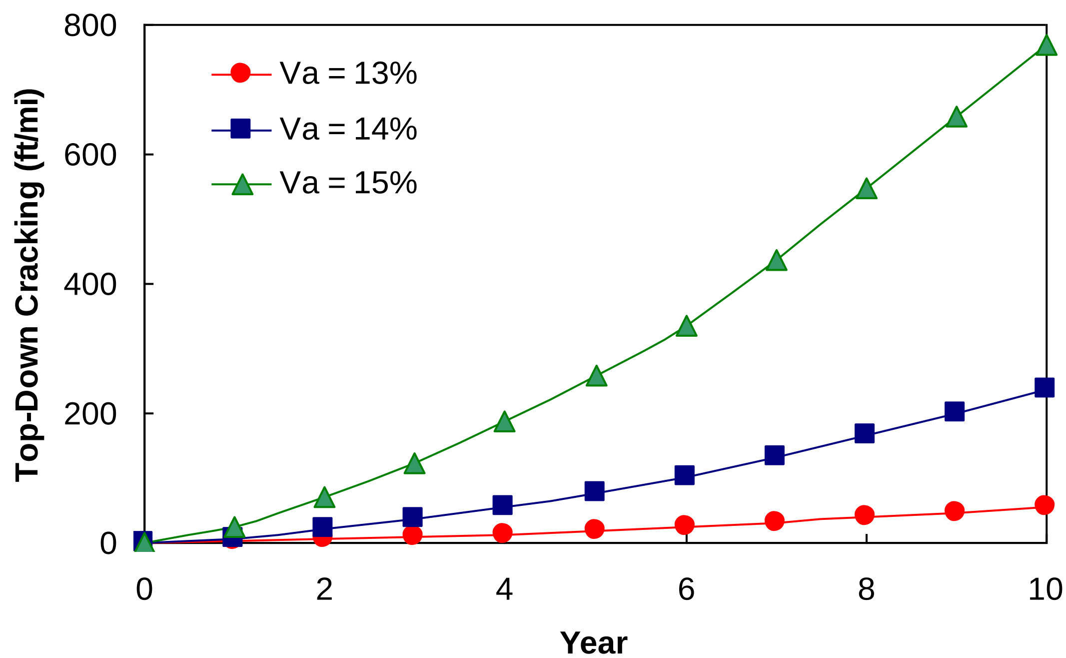

Figure 158 Effect of Air Void Content on Top-Down Cracking – New Hampshire

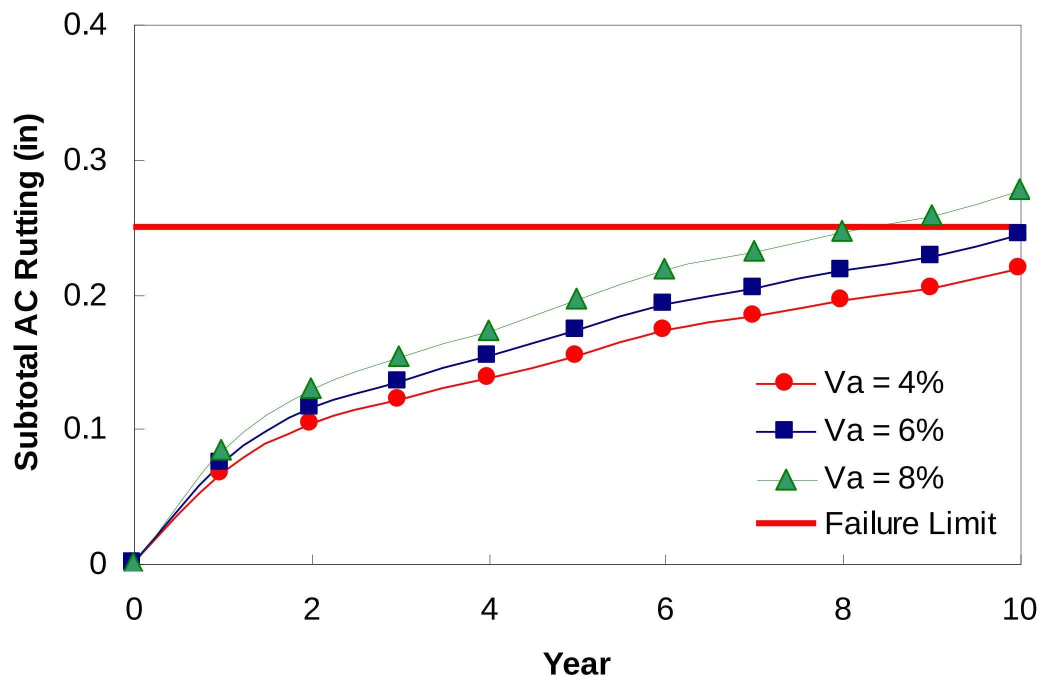

Figure 159 Effect of Air Void Content on Subtotal AC Rutting – New Hampshire

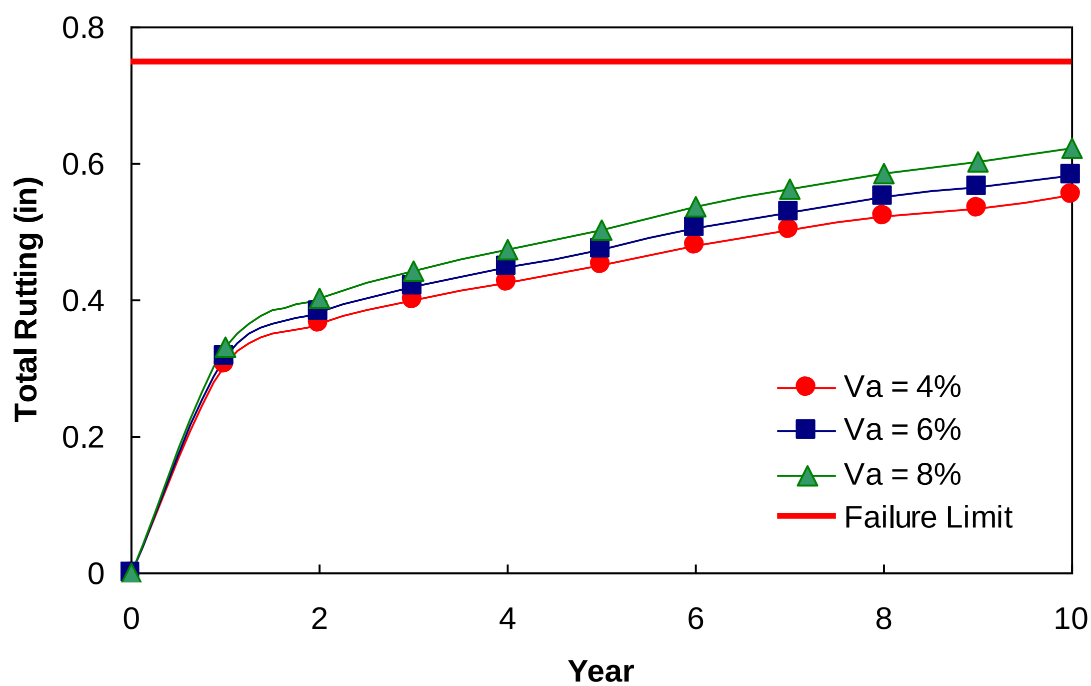

Figure 160 Effect of Air Void Content on Total Rutting – New Hampshire

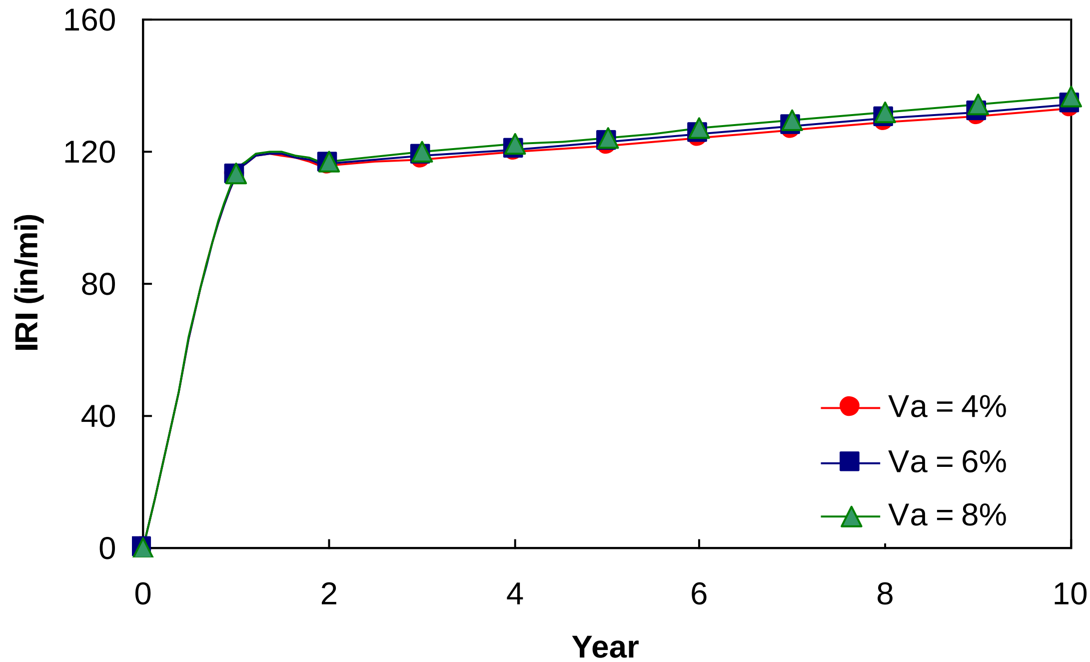

Figure 161 Effect of Air Void Content on IRI – New Hampshire

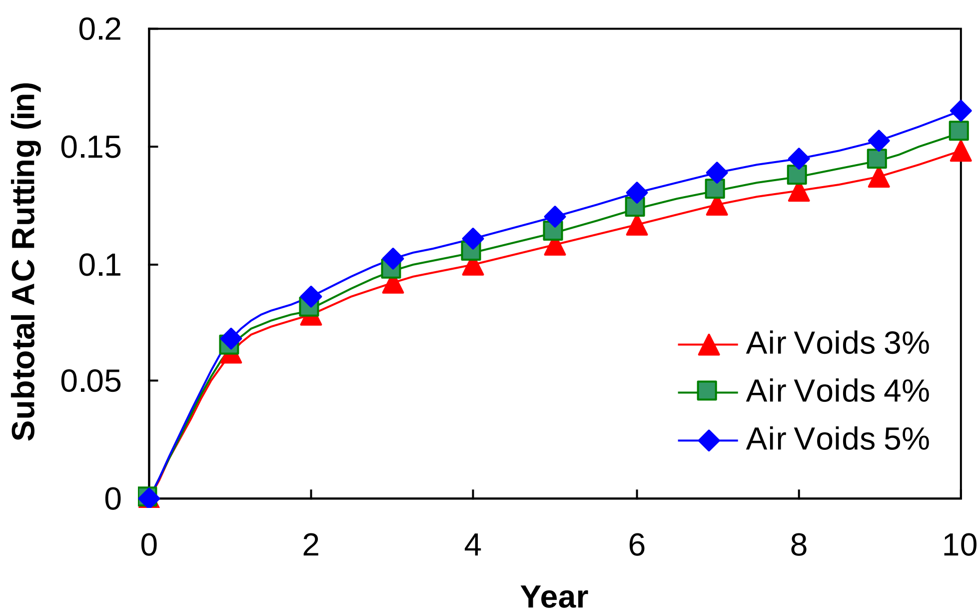

Figure 162 Effect of Air Void Content on Subtotal AC Rutting – Rhode Island

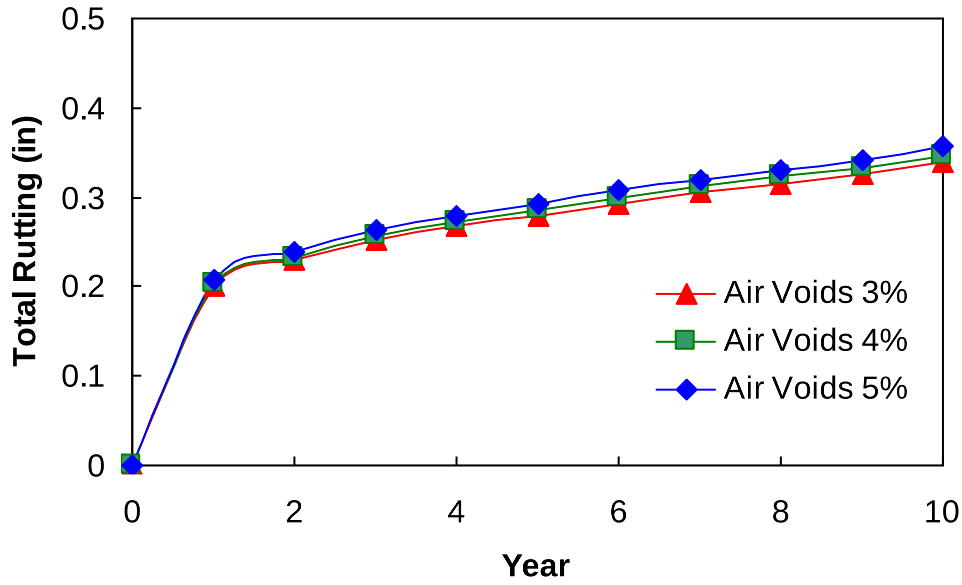

Figure 163 Effect of Air Void Content on Total Rutting – Rhode Island

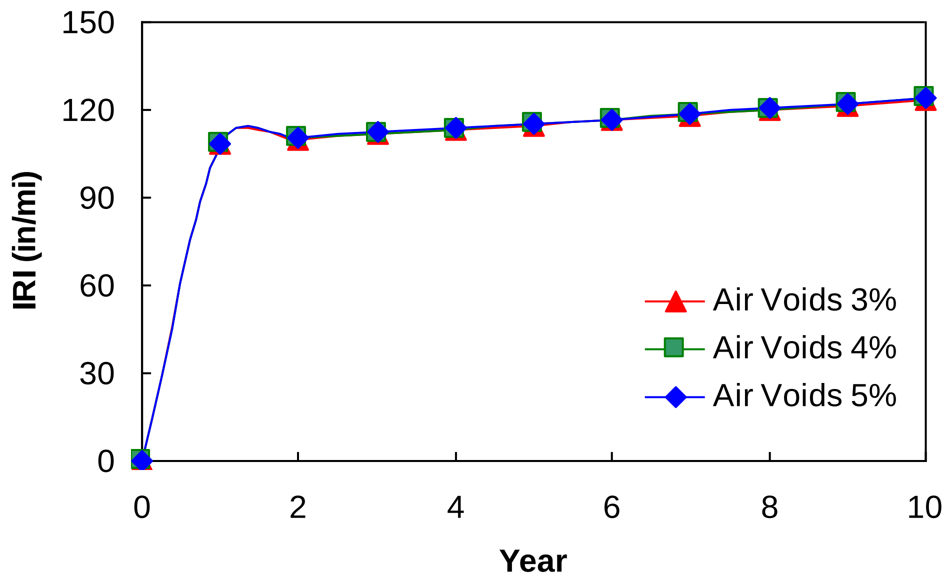

Figure 164 Effect of Air Void Content on IRI – Rhode Island

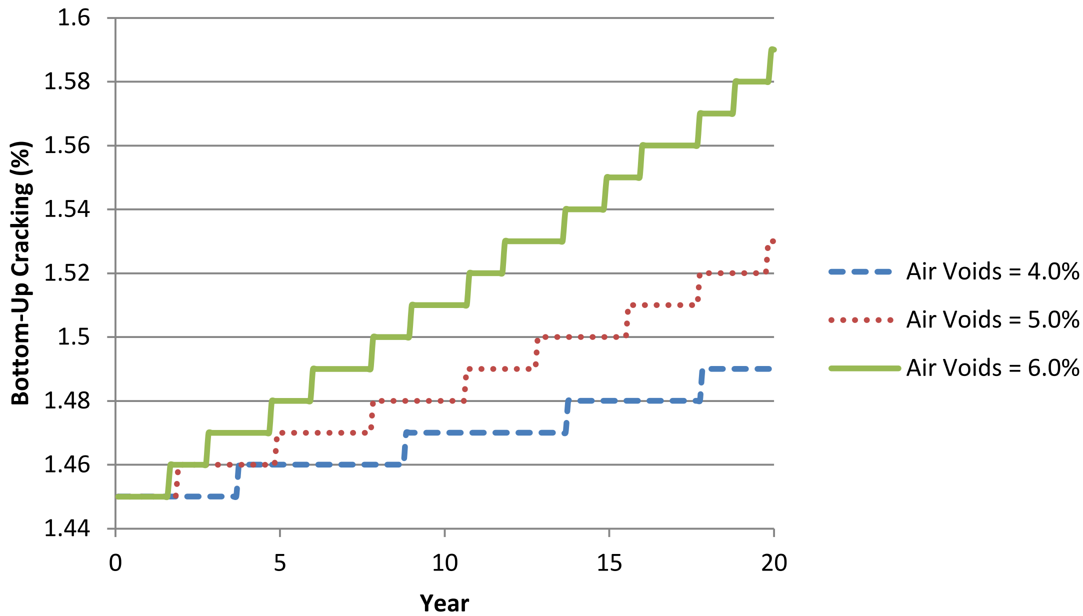

Figure 165 Effect of Air Void Content on Bottom-Up Cracking – Vermont

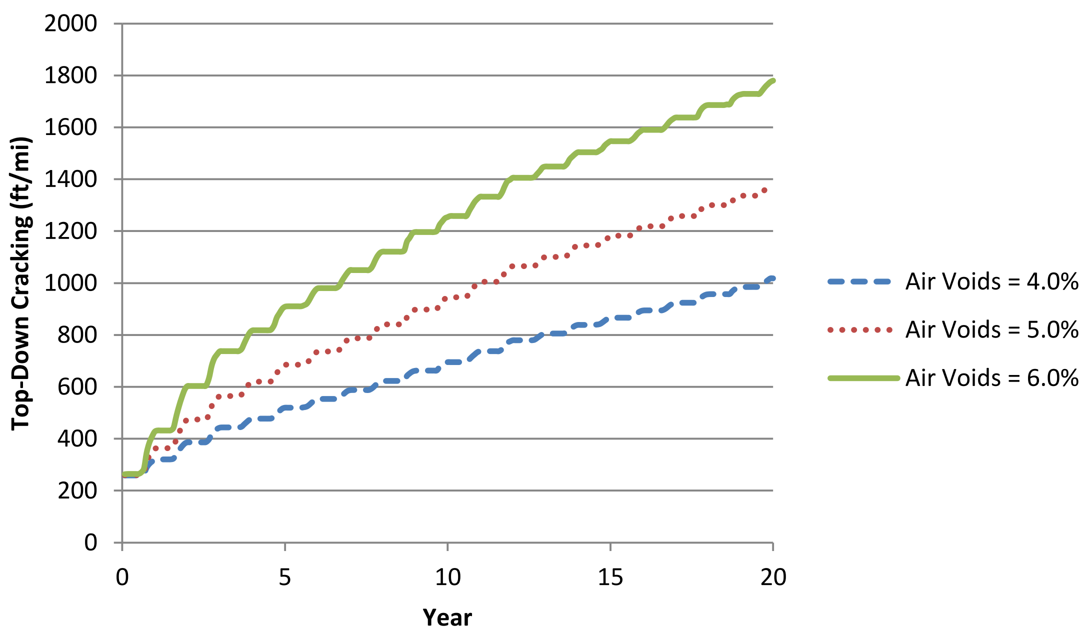

Figure 166 Effect of Air Void Content on Top-Down Cracking – Vermont

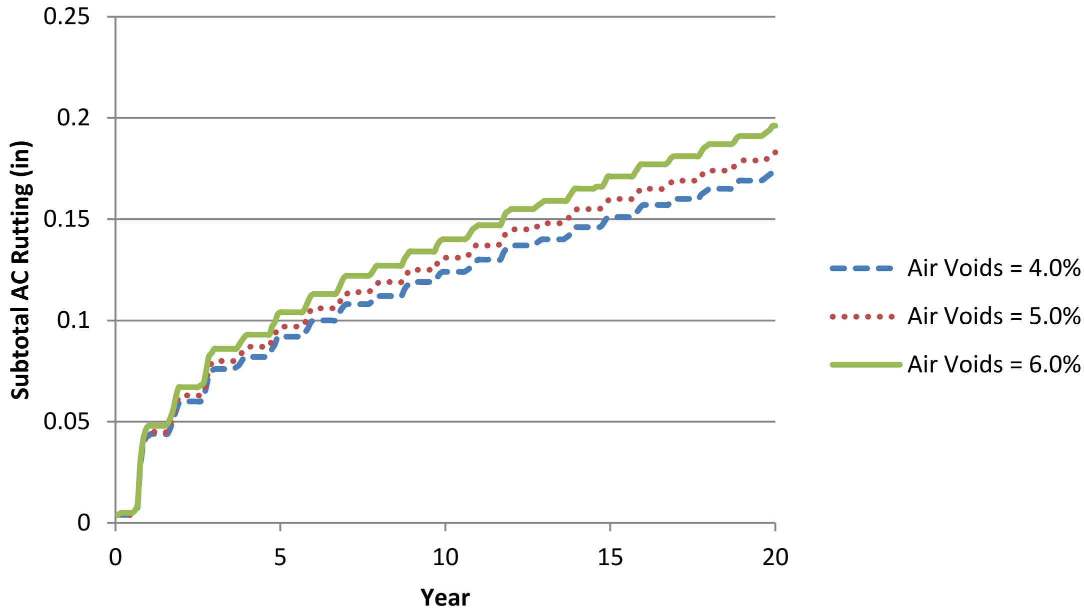

Figure 167 Effect of Air Void Content on Subtotal AC Rutting – Vermont

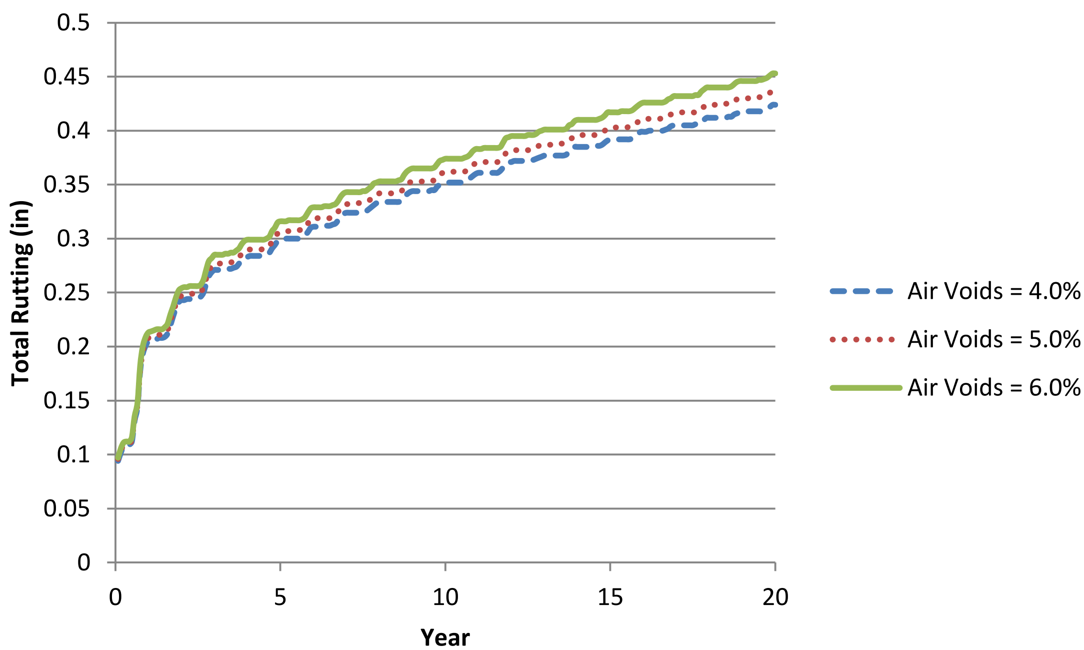

Figure 168 Effect of Air Void Content on Total Rutting – Vermont

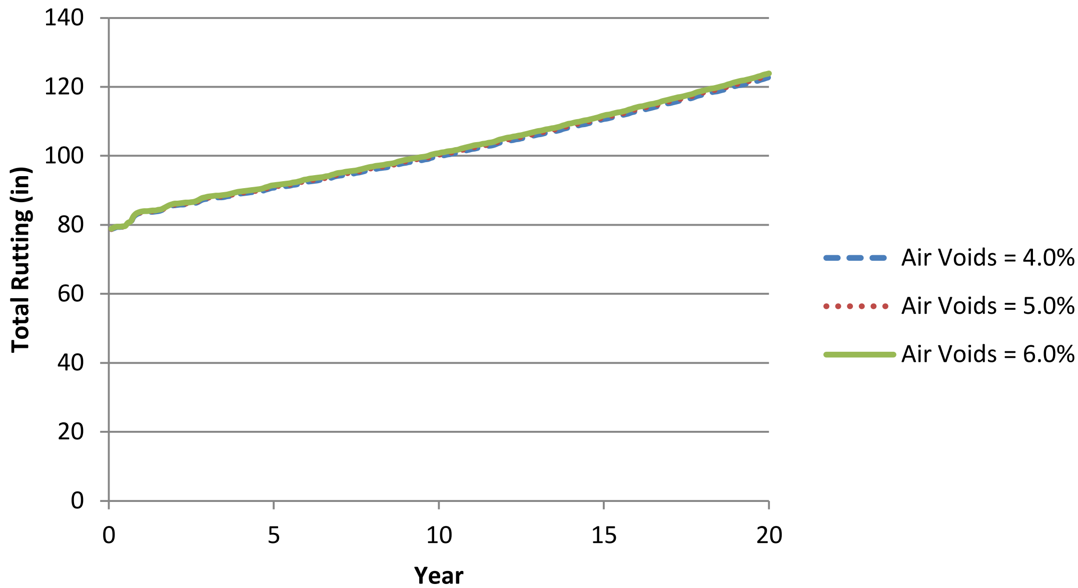

Figure 169 Effect of Air Void Content on IRI – Vermont

Effective Binder Content

Effective binder content of the asphalt concrete mix is required as input for design by the MEPDG. State design specifications provide tolerances for the actual binder content percentage by weight of the asphalt concrete mix; hence there exists a need for a change of specifications to accommodate for tolerances in the effective binder content.

The performance prediction curves for different effective binder contents for all the states are shown for Level 3 design in the following graphs. The graphs show a comparison between Level 2 and Level 3 results. An interesting observation that can be made from the plots at both levels of design is that there is no difference in the sensitivity, whereas the magnitude of distresses is much lower for Level 2 design. Therefore, the tolerances of material properties need not differ for the level of design inputs chosen, as similar pattern is observed for all the various input parameters studied.

Inference: The trend of performance prediction curves does not vary with the level of inputs chosen for design using the MEPDG. Therefore, same tolerances can be applied for both Level 2 and Level 3 design inputs.

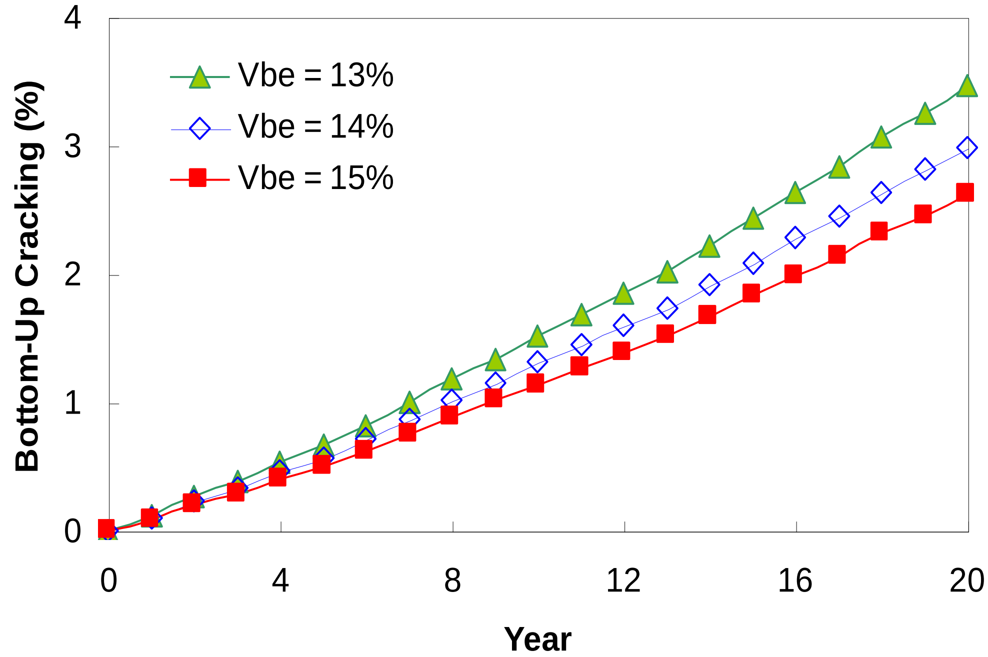

Figure 170 Effect of Effective Binder Content on Bottom-Up Cracking – NH Level 3

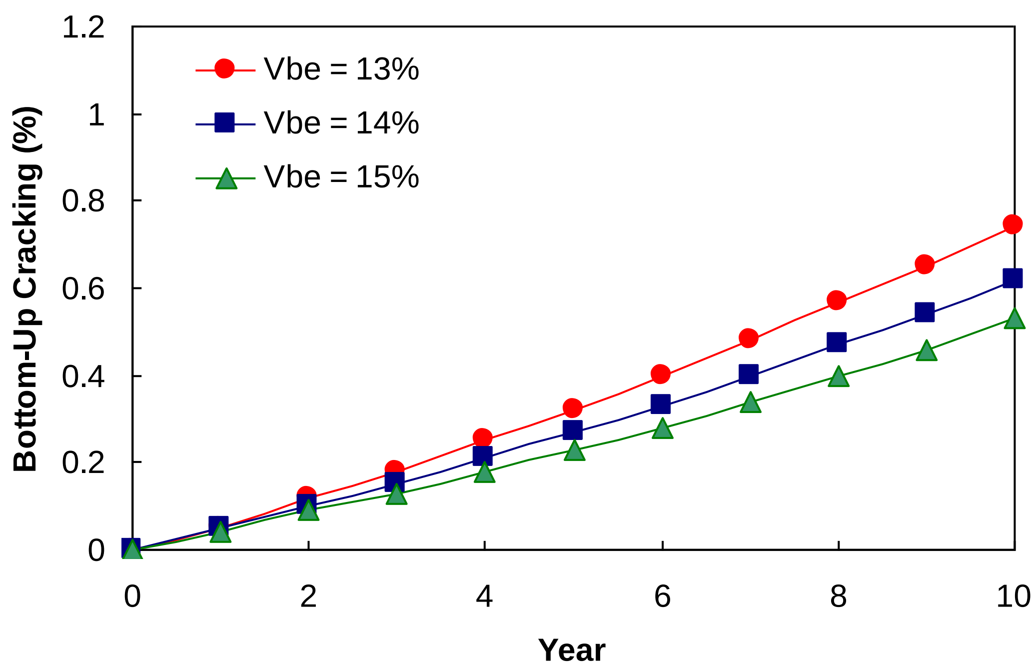

Figure 171 Effect of Effective Binder Content on Bottom-Up Cracking – NH Level 2

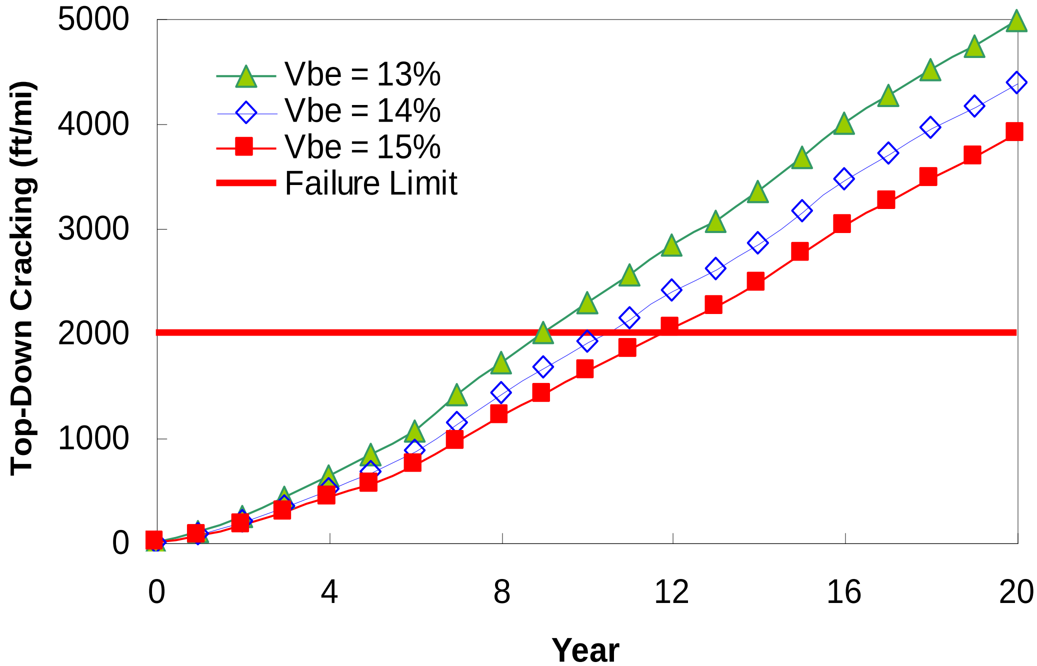

Figure 172 Effect of Effective Binder Content on Top-Down Cracking – NH Level 3

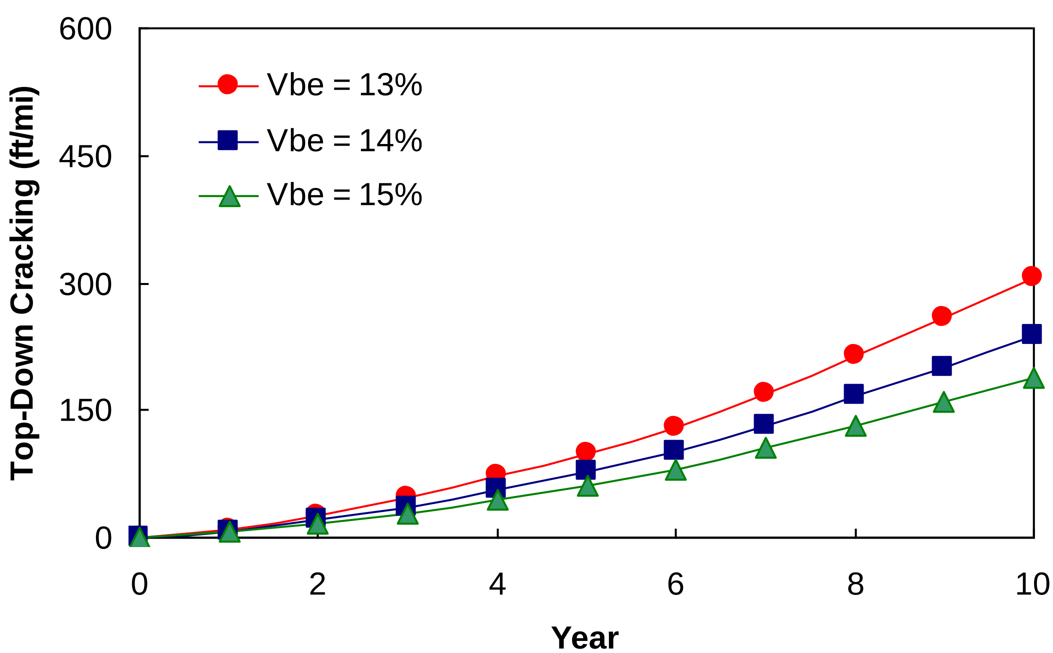

Figure 173 Effect of Effective Binder Content on Top-Down Cracking – NH Level 2

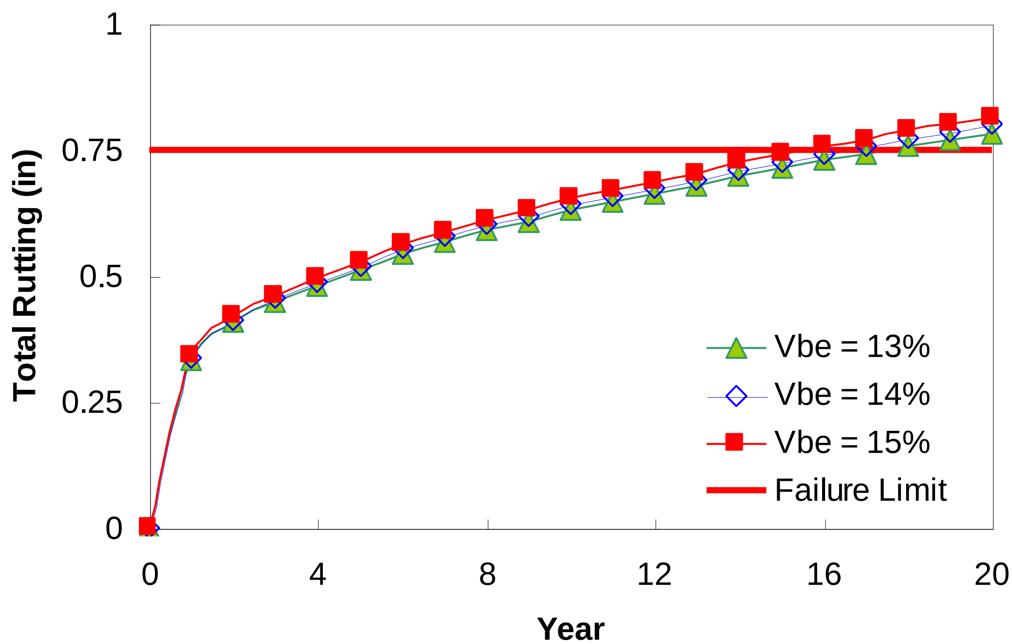

Figure 174 Effective of Effective Binder Content on Total Rutting – NH Level 3

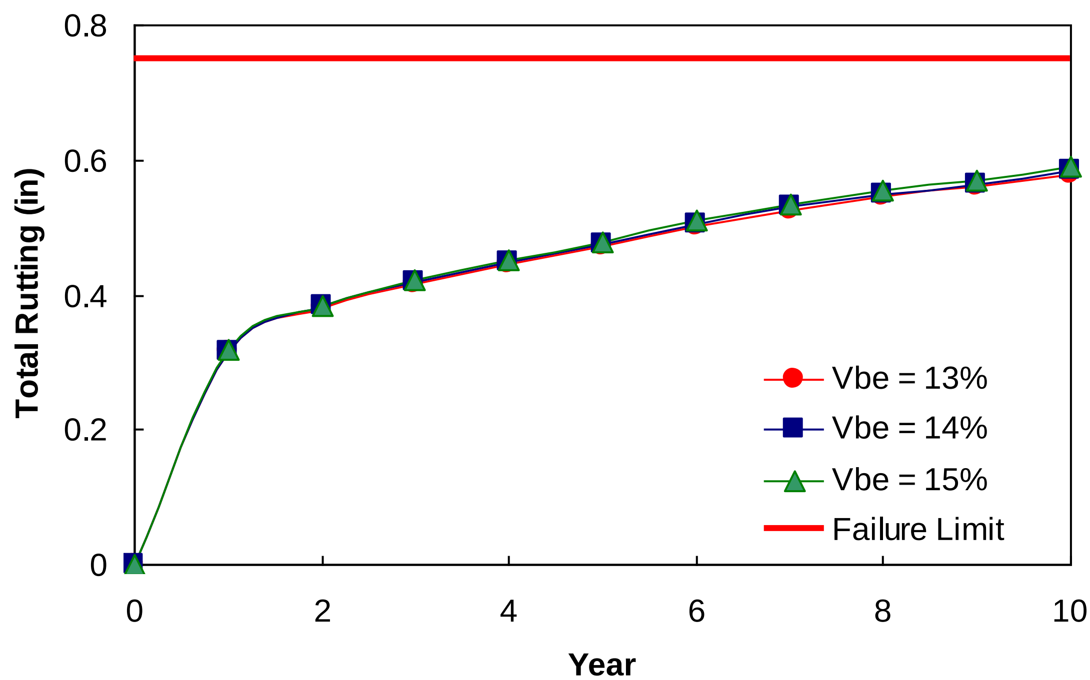

Figure 175 Effect of Effective Binder Content on Total Rutting – NH Level 2

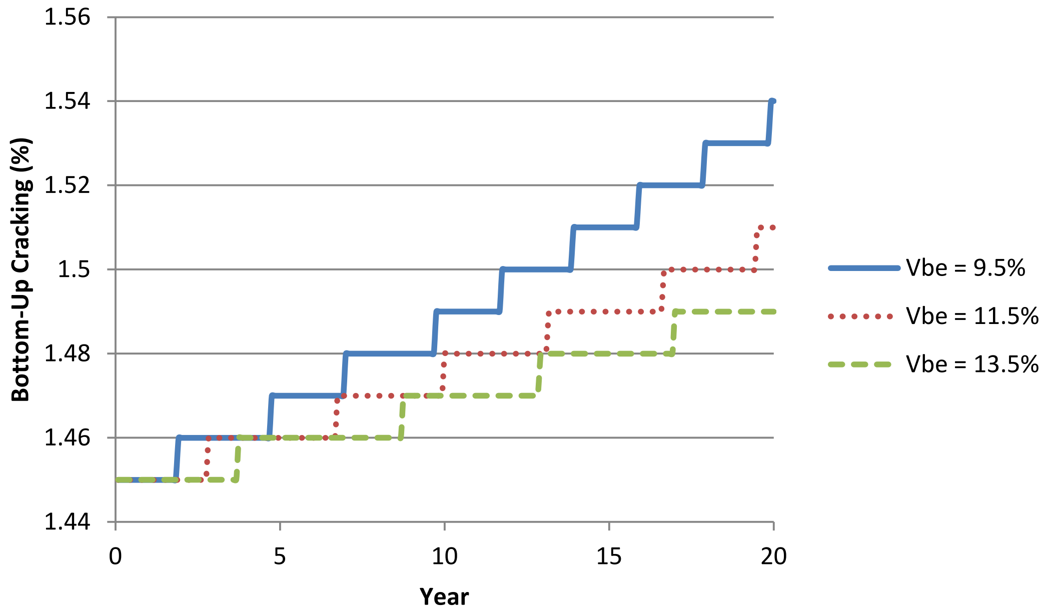

Figure 176 Effect of Effective Binder Content on Bottom-Up Cracking – VT Level 3

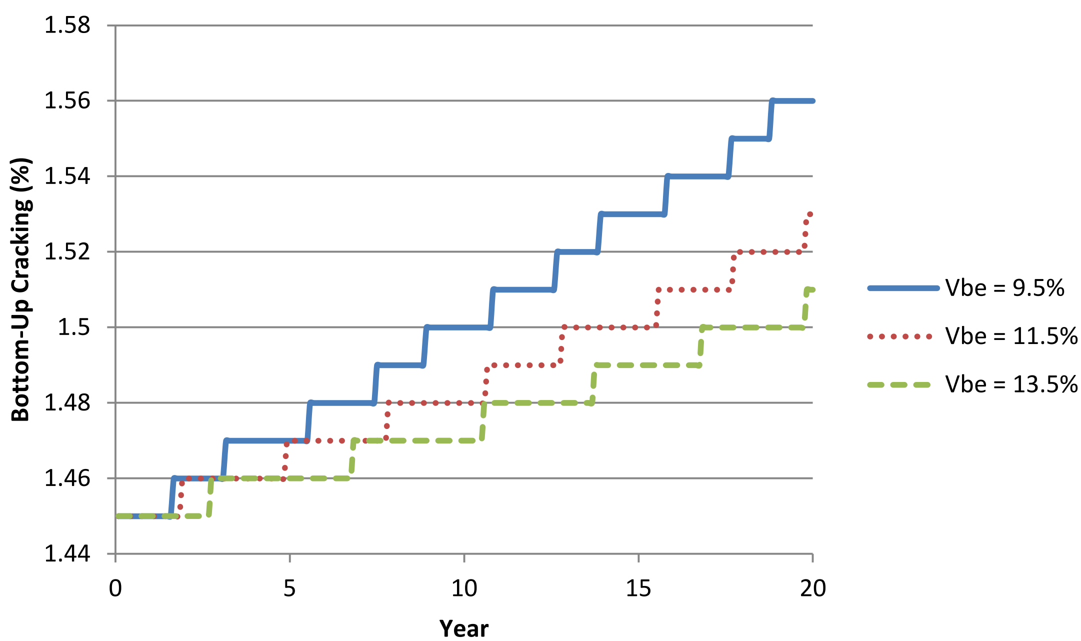

Figure 177 Effect of Effective Binder Content on Bottom-Up Cracking – VT Level 2

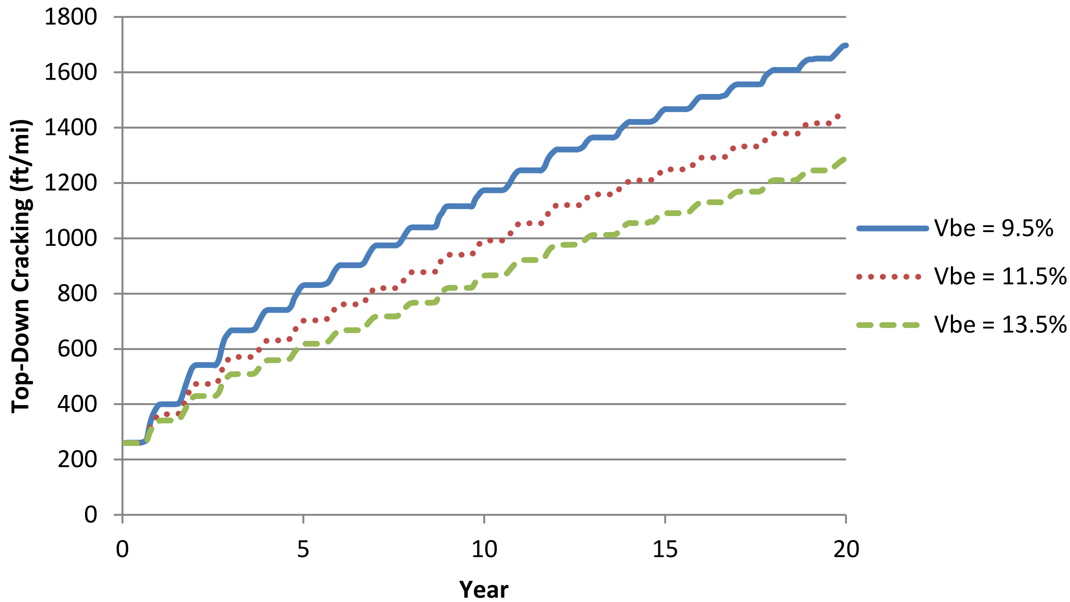

Figure 178 Effect of Effective Binder Content on Top-Down Cracking – VT Level 3

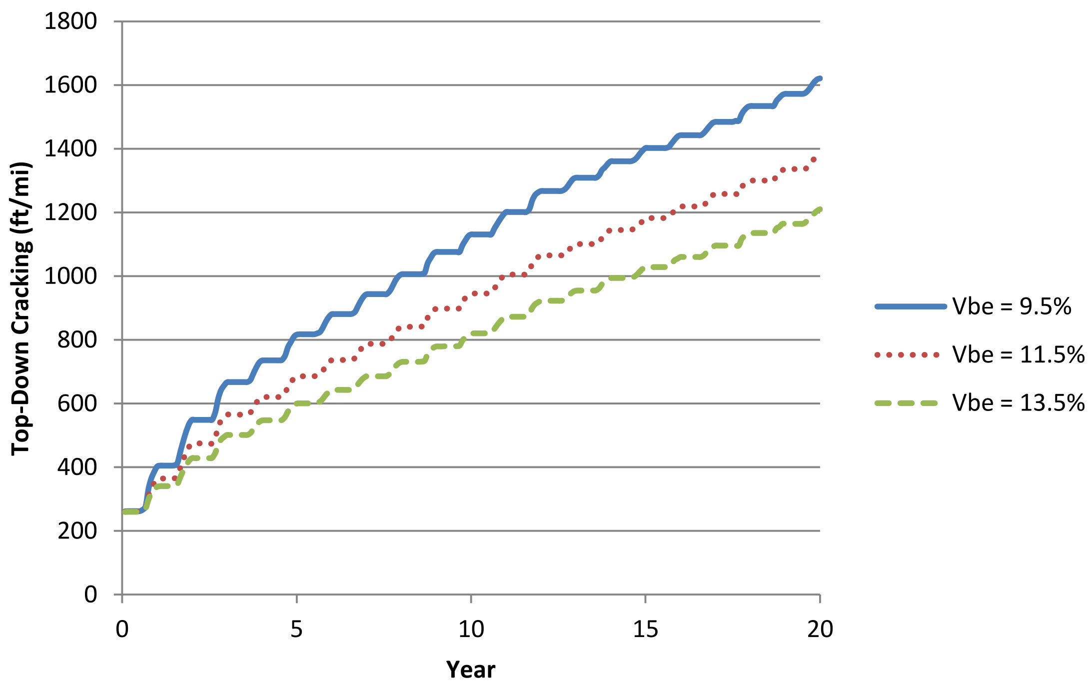

Figure 179 Effect of Effective Binder Content on Top-Down Cracking – VT Level 2

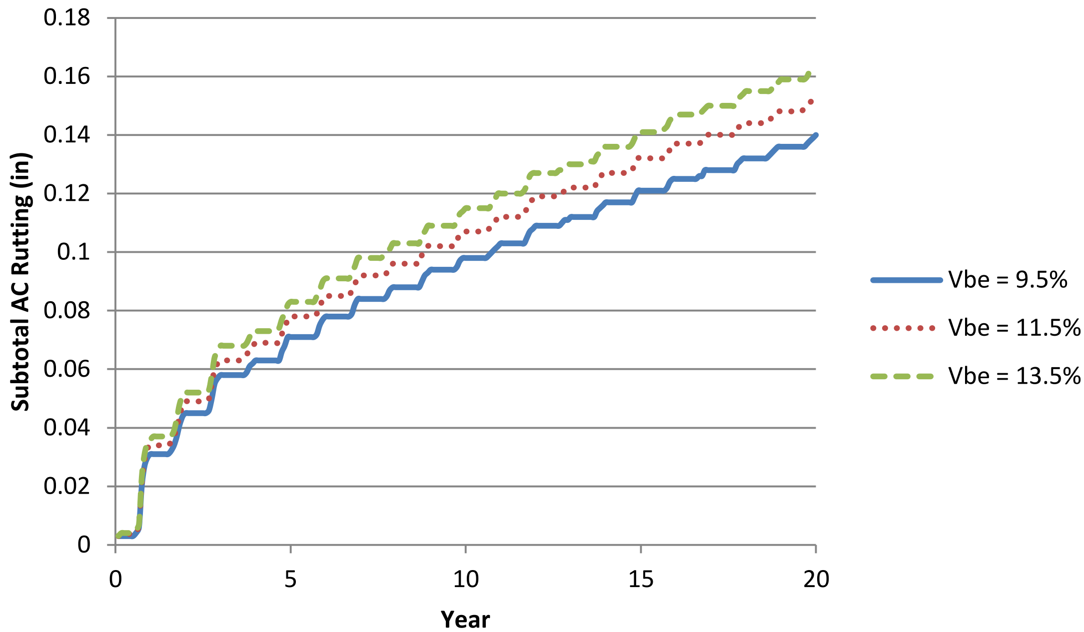

Figure 180 Effect of Effective Binder Content on Subtotal AC Rutting – VT Level 3

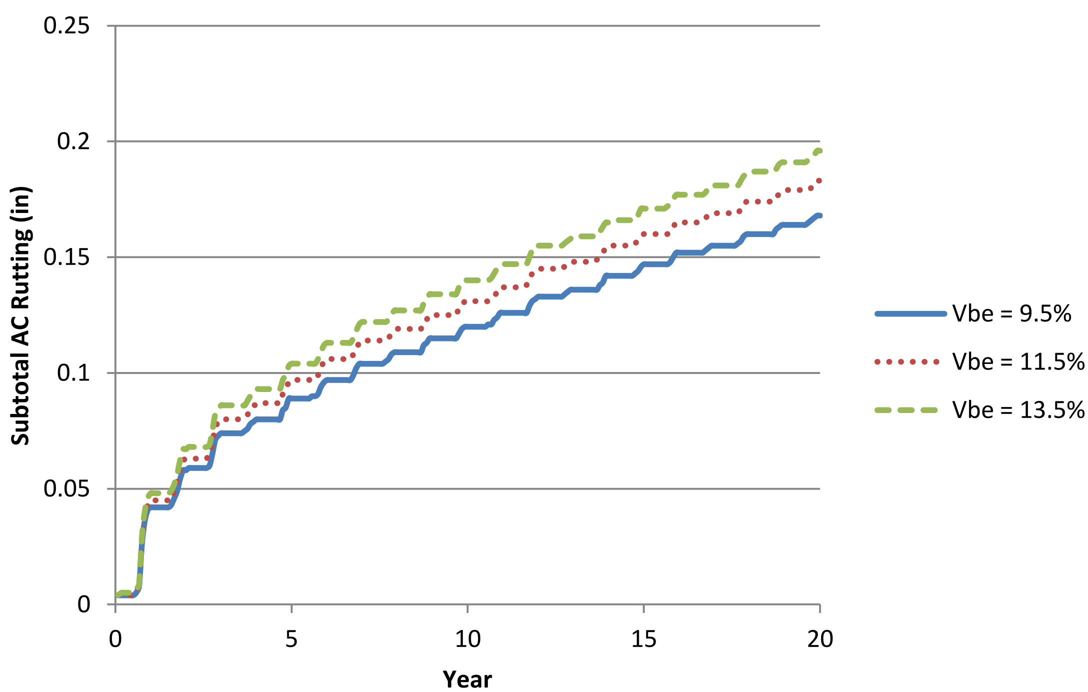

Figure 181 Effect of Effective Binder Content on Subtotal AC Rutting – VT Level 2

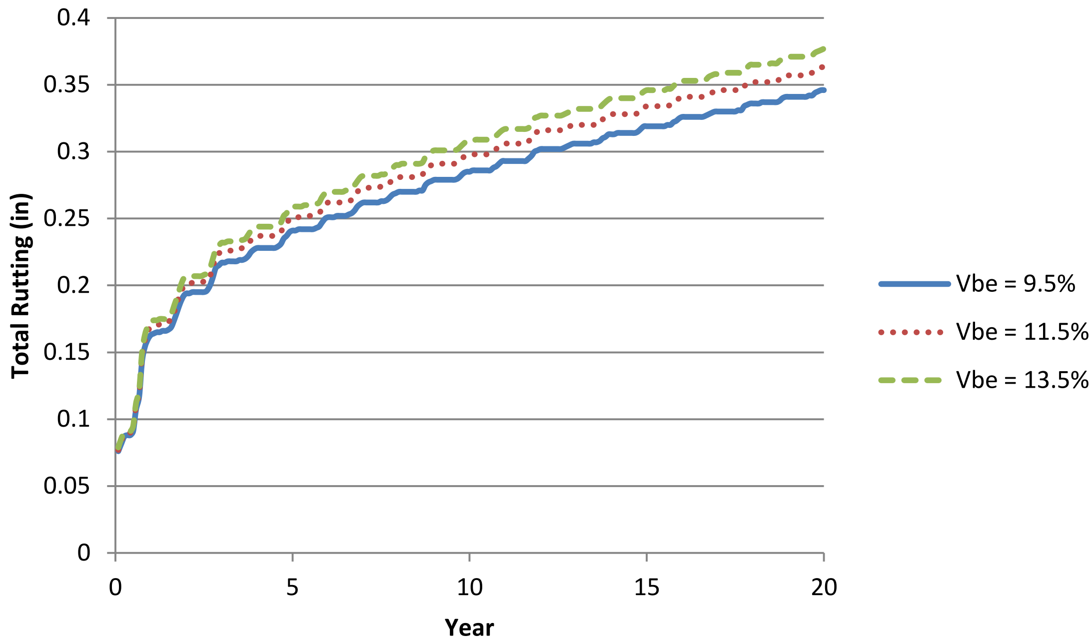

Figure 182 Effective of Effective Binder Content on Total Rutting – VT Level 3

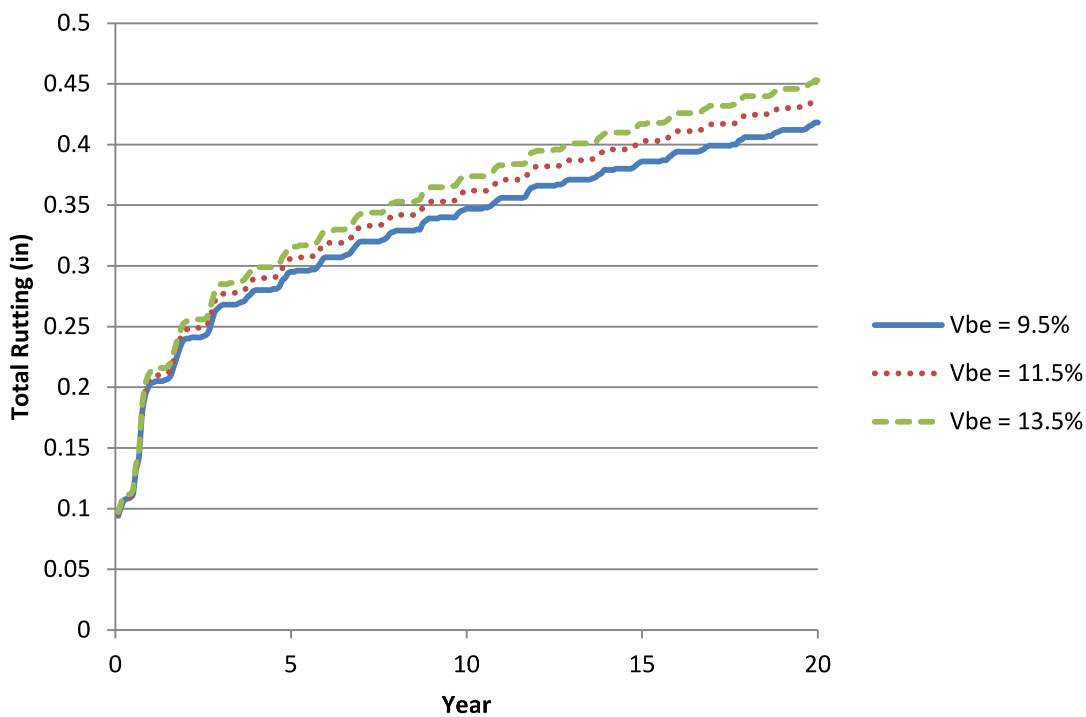

Figure 183 Effect of Effective Binder Content on Total Rutting – VT Level 2

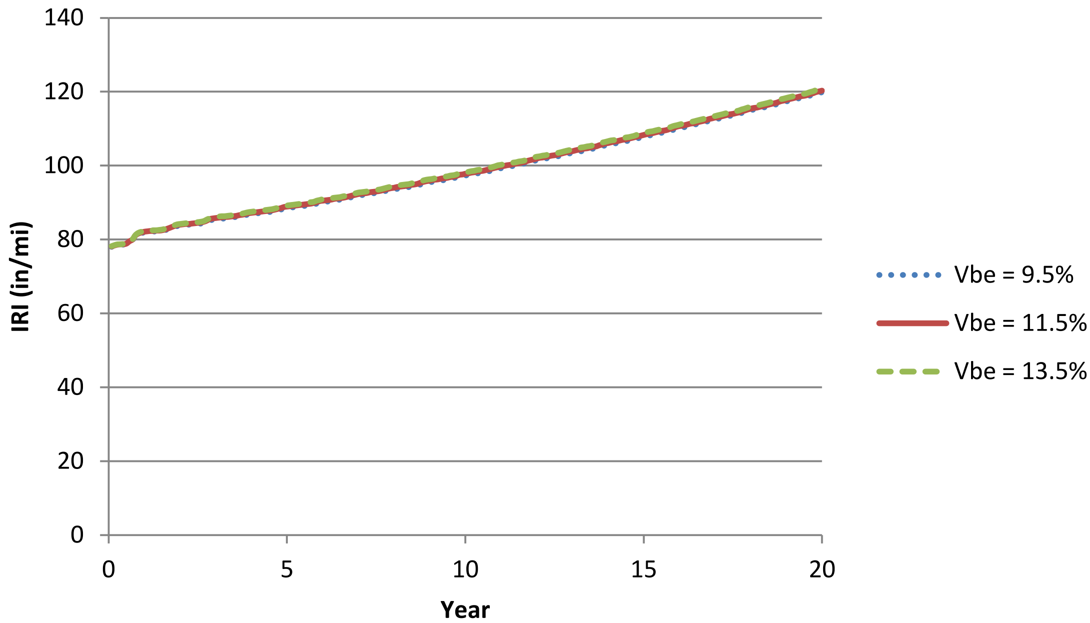

Figure 184 Effect of Effective Binder Content on IRI – VT Level 3

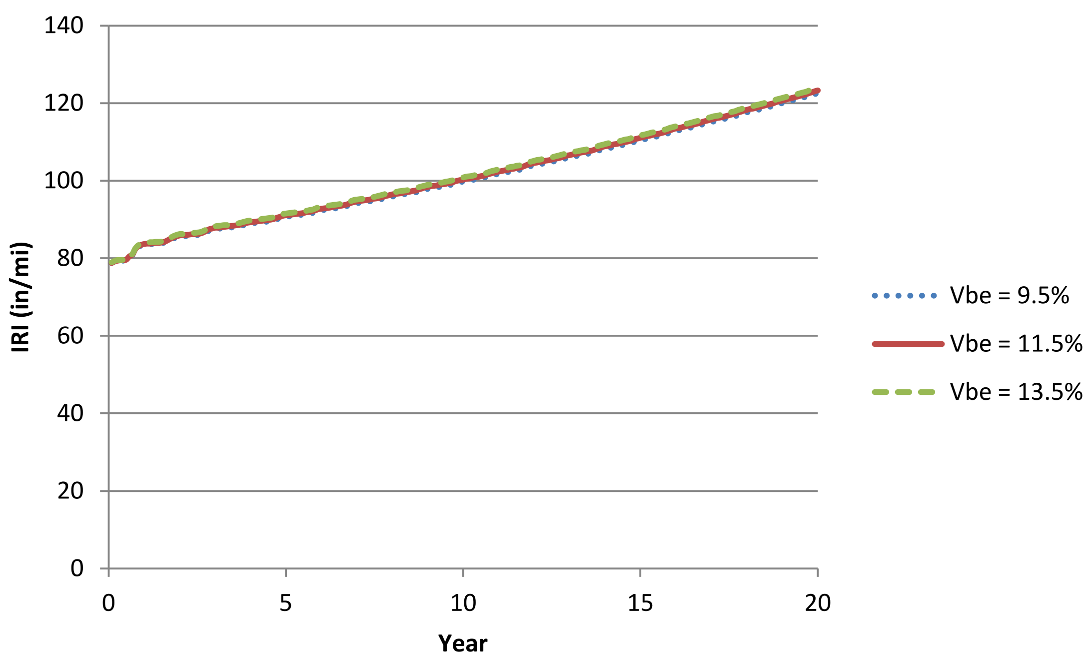

Figure 185 Effect of Effective Binder Content on IRI – VT Level 2

The following graphs (Figures 186 through 195) show performance prediction curves for different effective binder contests for only level 3 sensitivity analyses in the state of New York and Massachusetts.

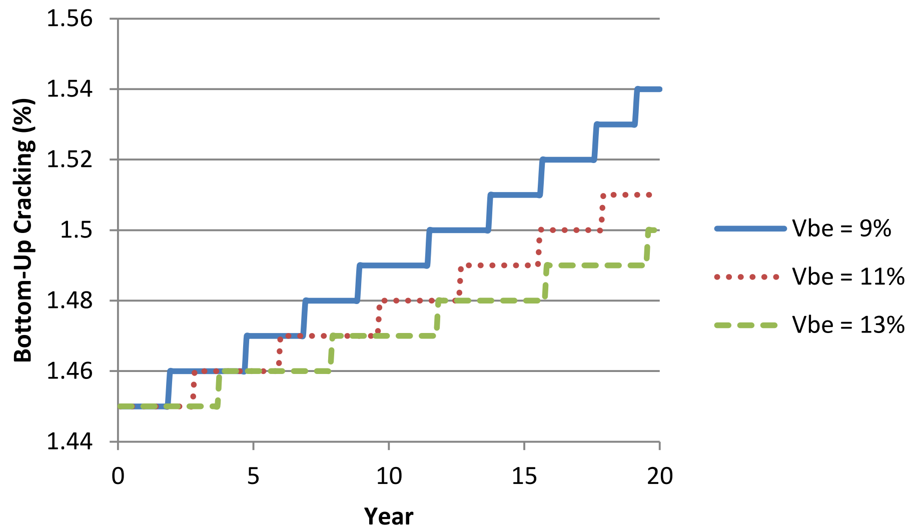

Figure 186 Effect of Effective Binder Content on Bottom-Up Cracking – NY Level 3

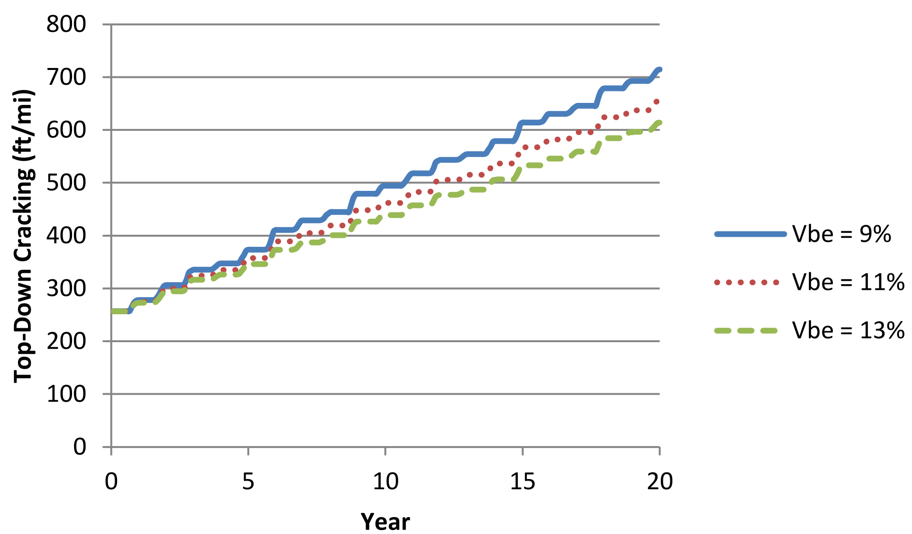

Figure 187 Effect of Effective Binder Content on Top-Down Cracking – NY Level 3

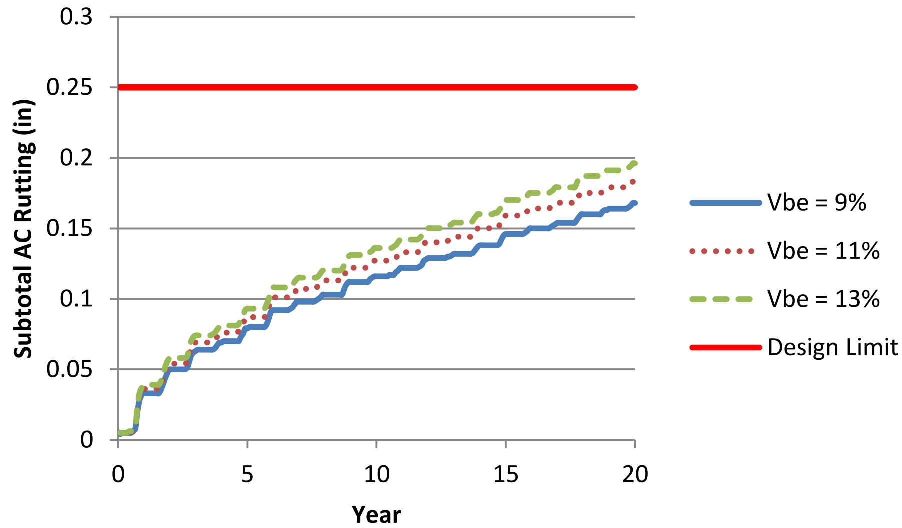

Figure 188 Effect of Effective Binder Content on Subtotal AC Rutting – NY Level 3

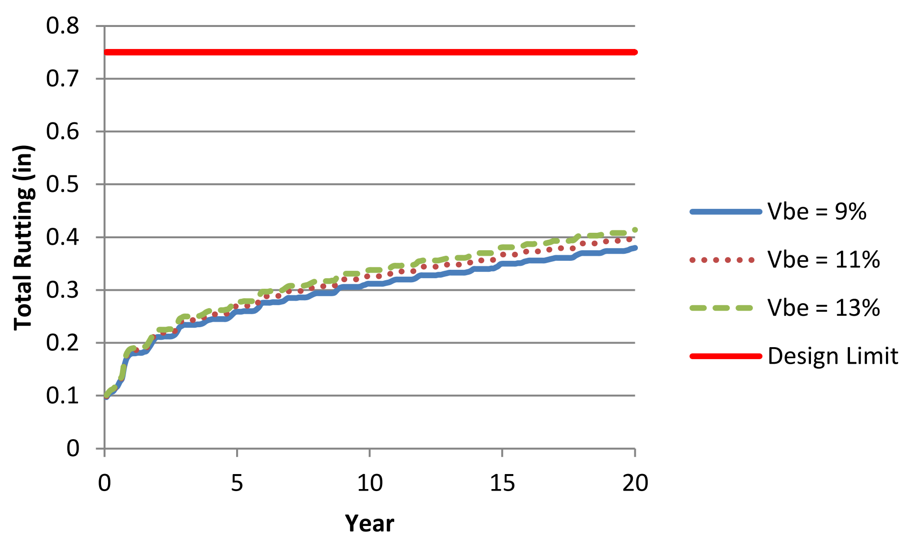

Figure 189 Effect of Effective Binder Content on Total Rutting – NY Level 3

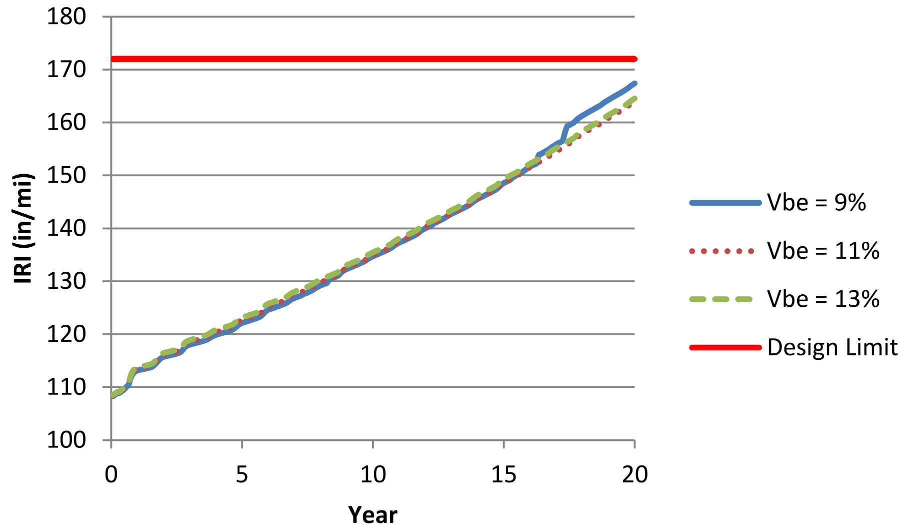

Figure 190 Effect of Effective Binder Content on IRI – NY Level 3

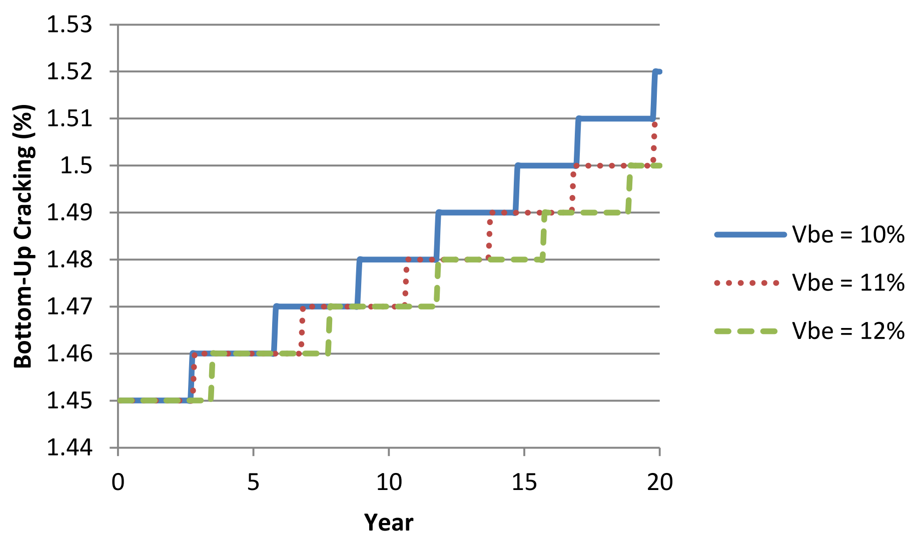

Figure 191 Effect of Effective Binder Content on Bottom-Up Cracking – MA Level 3

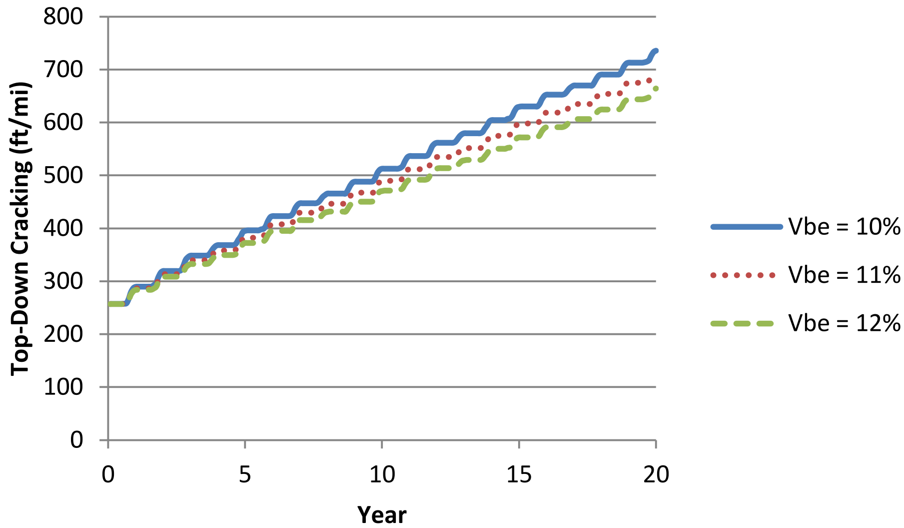

Figure 192 Effect of Effective Binder Content on Top-Down Cracking – MA Level 3

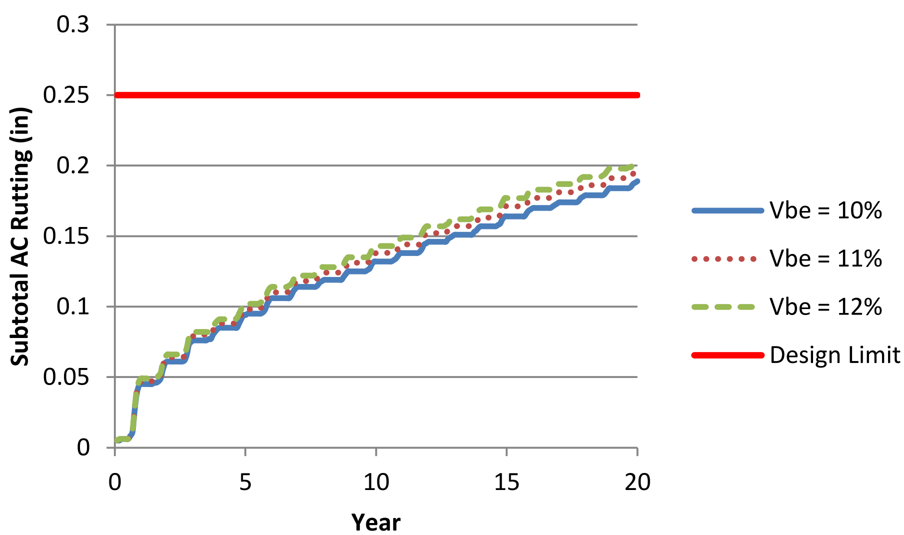

Figure 193 Effect of Effective Binder Content on Subtotal AC Rutting – MA Level 3

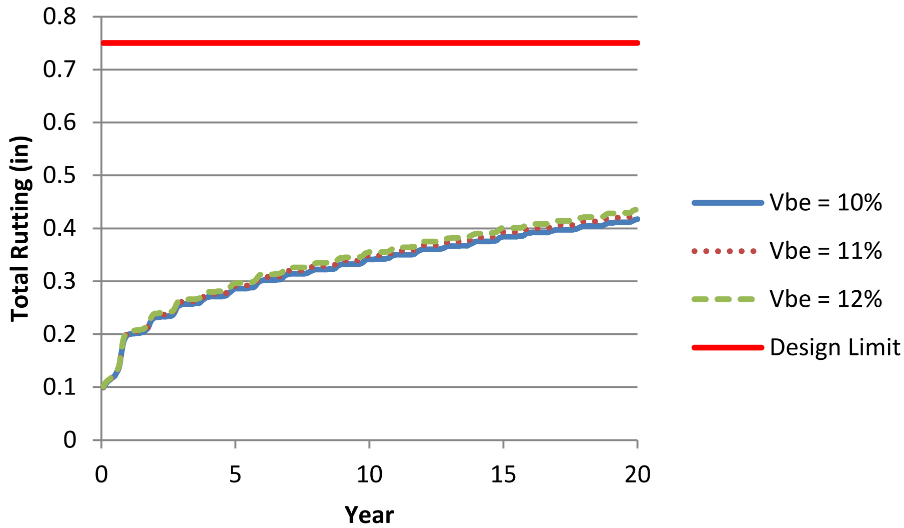

Figure 194 Effect of Effective Binder Content on Total Rutting – MA Level 3

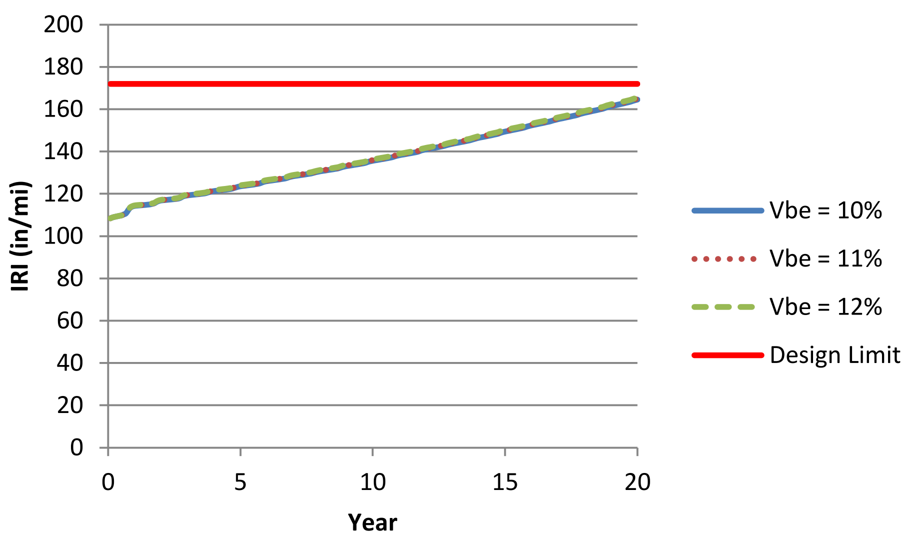

Figure 195 Effect of Effective Binder Content on IRI – MA Level 3

Asphalt Concrete Mix Aggregate Gradation

The aggregate gradation for asphalt concrete mix is a very important mix design parameter as per Superpave mix design specifications as well as Marshall Mix design specifications. Therefore, appropriate tolerances for aggregate gradation for design using the MEPDG should be determined by the state design agencies.

The aggregate gradation values chosen for this study are shown in
Table 24. The mean values for 9.5 mm, 19.0 mm and 25.0 mm asphalt concrete mixes were obtained from (32) as median of the allowable range of values for each sieve size. The coarse and fine aggregate gradations were developed by choosing values close to the boundaries of the range of allowable values to study the effect of variation in mix gradation values.

The predicted pavement distresses were observed to be highest in the case of a coarse mix gradation, and decreased with an increase in the fineness of the mix. The same trend is observed for all three nominal mix sizes studied. Therefore, the inference can be drawn that the percentage of aggregate retained on the sieve sizes required by the MEPDG (namely ¾ in, 3/8 in and #4) and passing #200 can be on the lower side of the mean specified in Superpave design. A suggested method to be followed is to select mix aggregate gradation approaching the mean values for a particular NMAS (nominal maximum aggregate size) such that the actual job mix formula values are lower than the Superpave means.

The significance of the effect of varying aggregate gradation is much higher for Level 3 as compared to Level 2, which shows almost insignificant effect. This is due to the prediction models built into the MEPDG, which utilize the aggregate gradation percentages for asphalt material characterization when Level 3 is selected as the design level (45). For Rhode Island data where the pavement structure is an asphalt concrete overlay on a fractured JPCP, mix gradation did not affect the predicted distresses – rutting and roughness.

The following performance prediction curves show fatigue (bottom-up) cracking and total rutting of the pavement for different aggregate gradations. The plots show that a coarse gradation for a given NMAS fails much earlier than the mean and fine gradations, which almost fail at the same time in rutting. An interesting observation is that a 9.5 mm mix fails much earlier compared to 19.0 mm and 25.0 mm mixes. This is due to lower strength provided by the aggregate skeleton to the asphalt concrete. This also accounts for the decreasing difference in performance of the pavement with varying coarseness of mix for larger NMAS. Similar trends are observed for pavement structures selected for the remaining New England States and the state of New York.

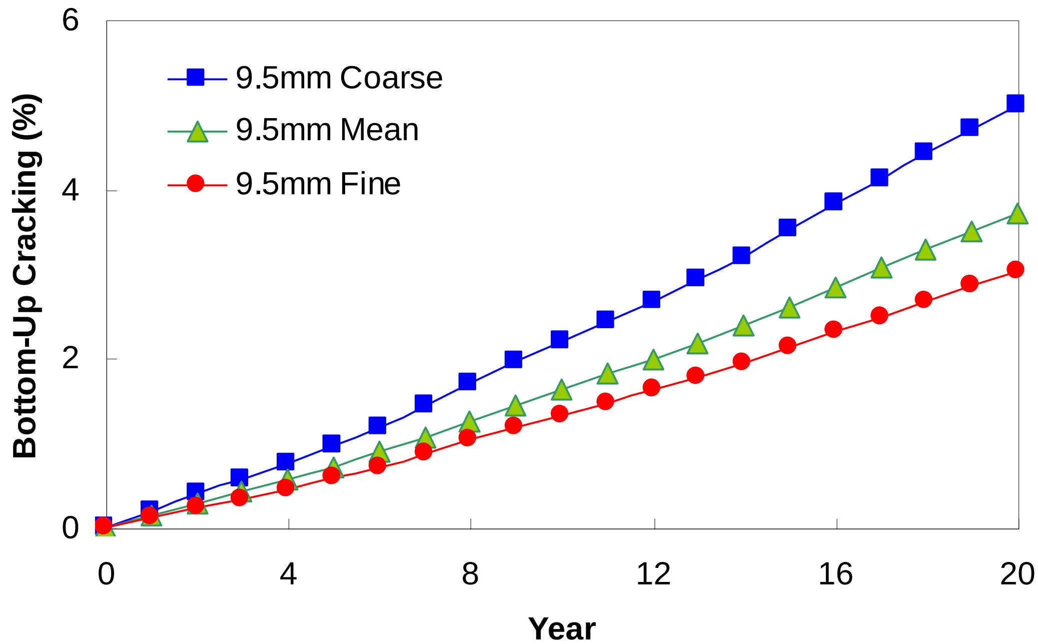

Figure 196 Effect of Gradation of 9.5 mm AC mix on Bottom-Up Cracking – NH

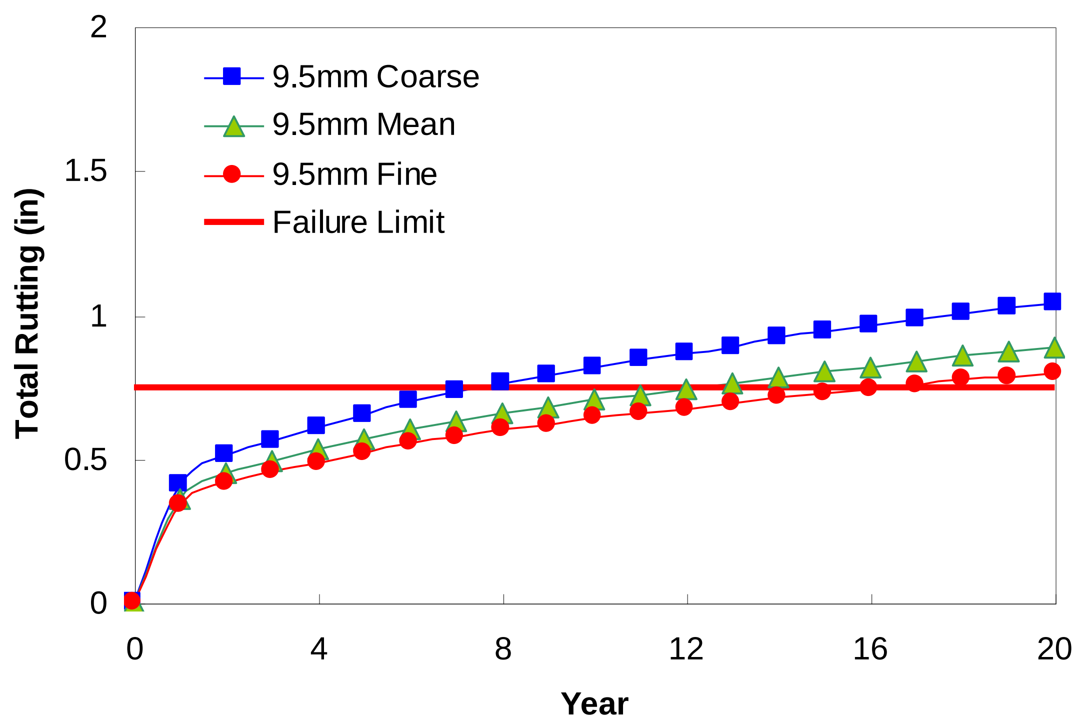

Figure 197 Effect of Aggregate Gradation of 9.5 mm AC mix on Total Rutting – NH

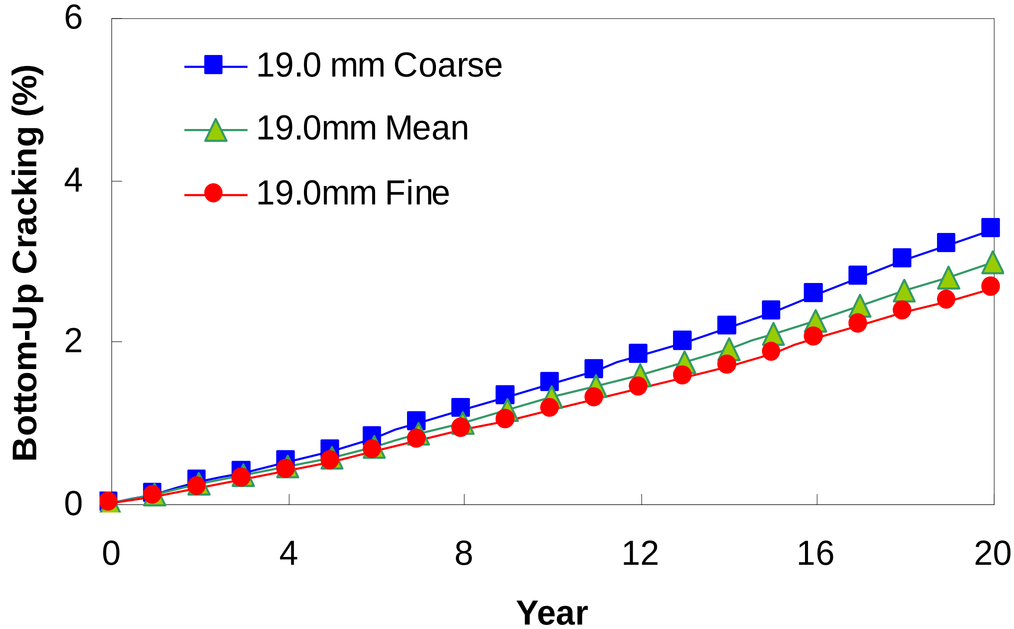

Figure 198 Effect of Aggregate Gradation of 19.0 mm mix on Bottom-Up Cracking – NH

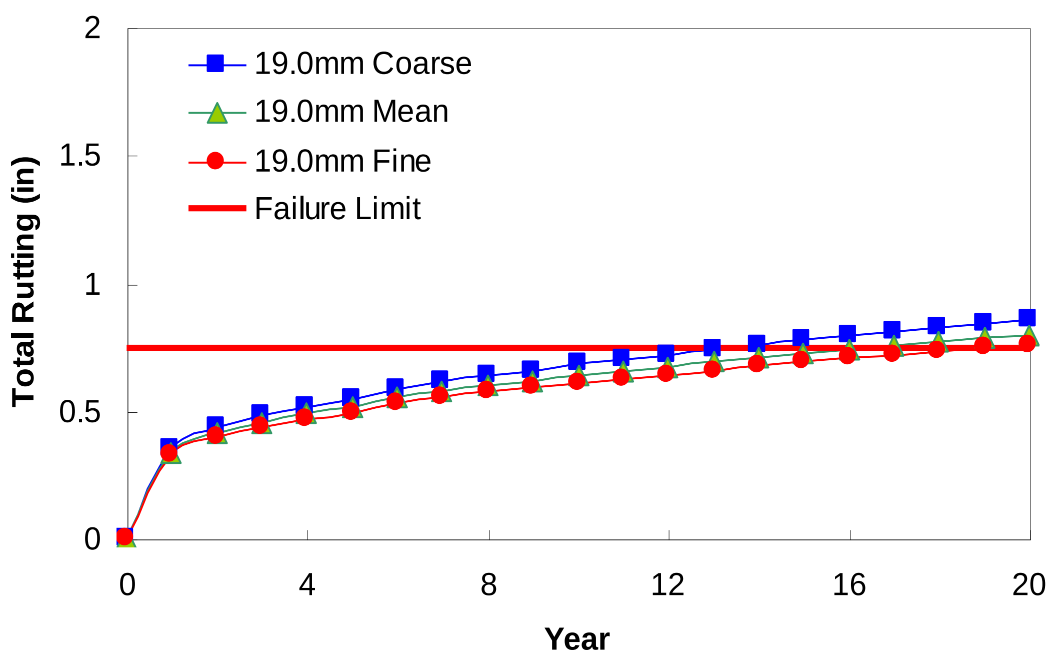

Figure 199 Effect of Aggregate Gradation of 19.0 mm mix on Total Rutting – NH

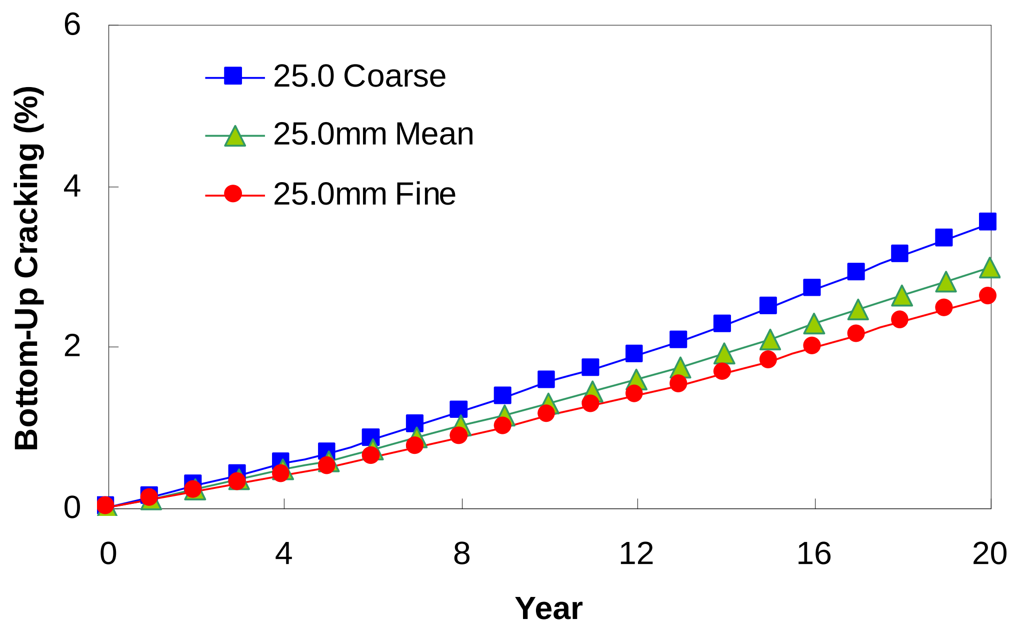

Figure 200 Effect of Aggregate Gradation of 25.0 mm mix on Bottom-Up Cracking – NH

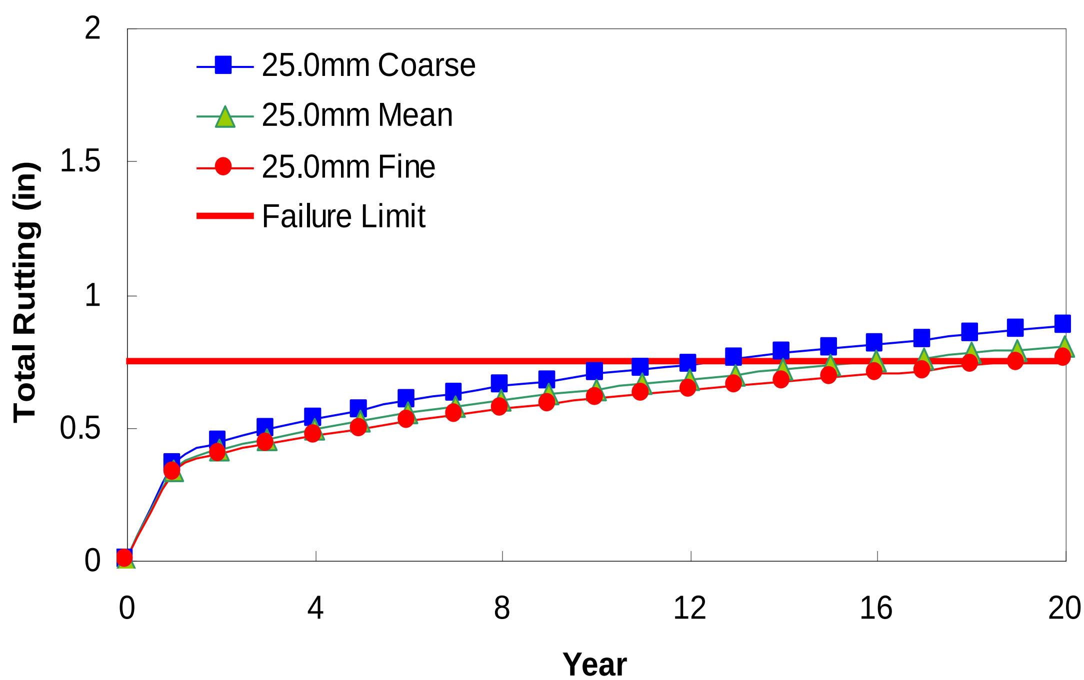

Figure 201 Effect of Aggregate Gradation of 25.0 mm mix on Total Rutting – NH

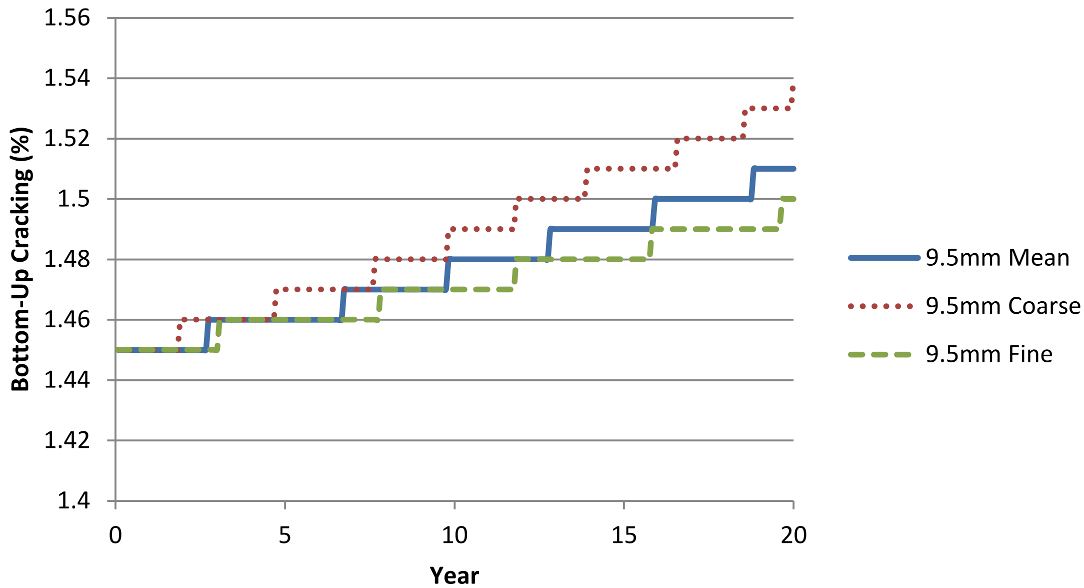

Figure 202 Effect of Aggregate Gradation of 9.5 mm AC mix on Bottom-Up Cracking – Vermont level 3

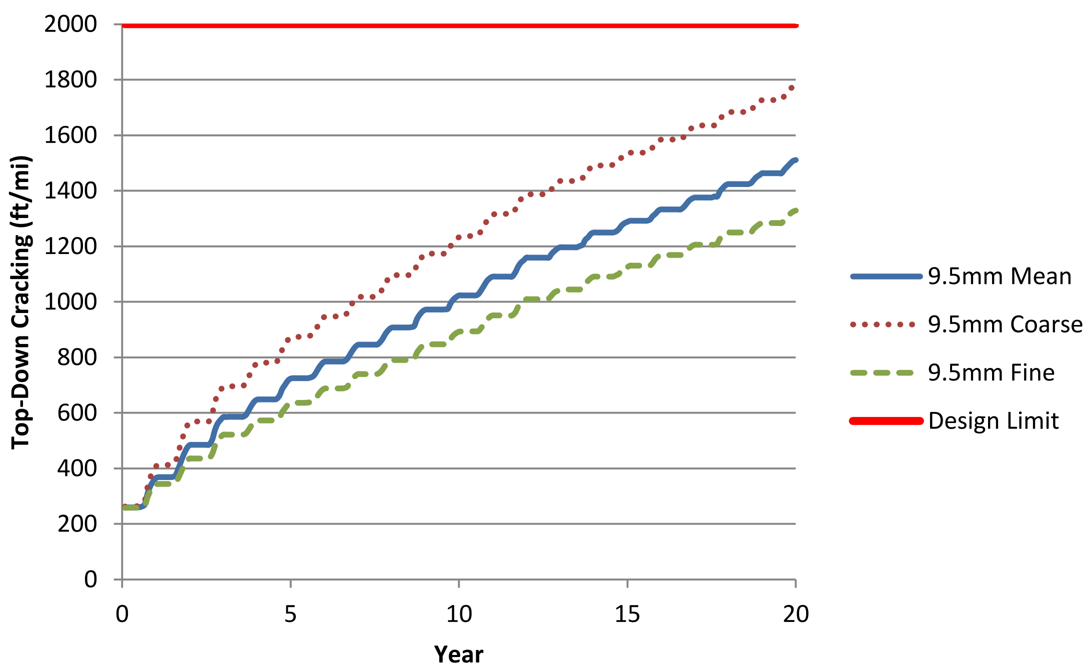

Figure 203 Effect of Aggregate Gradation of 9.5 mm AC mix on Top-Down Cracking – Vermont level 3

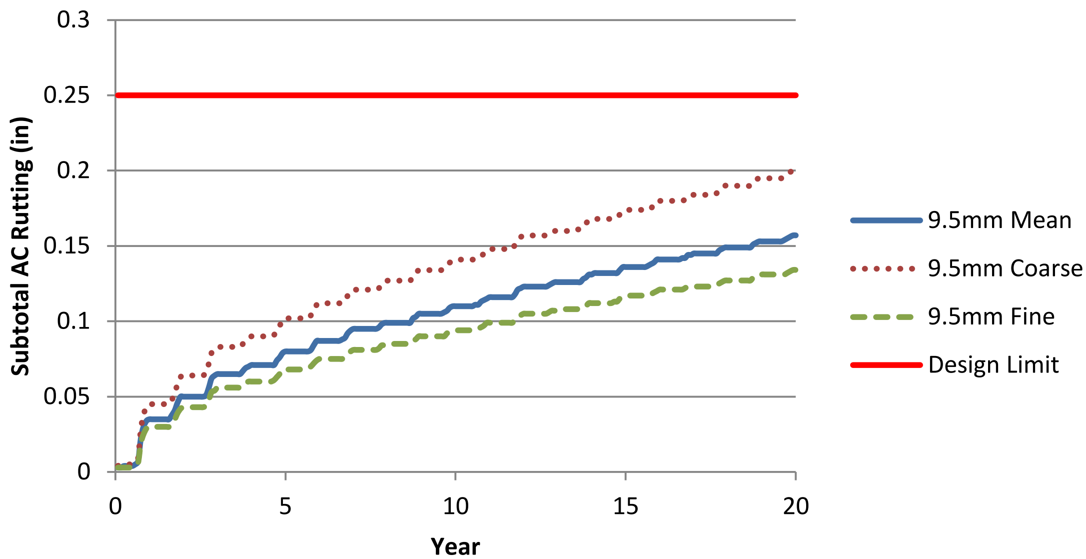

Figure 204 Effect of Aggregate Gradation of 9.5 mm AC mix on Subtotal AC Rutting – Vermont level 3

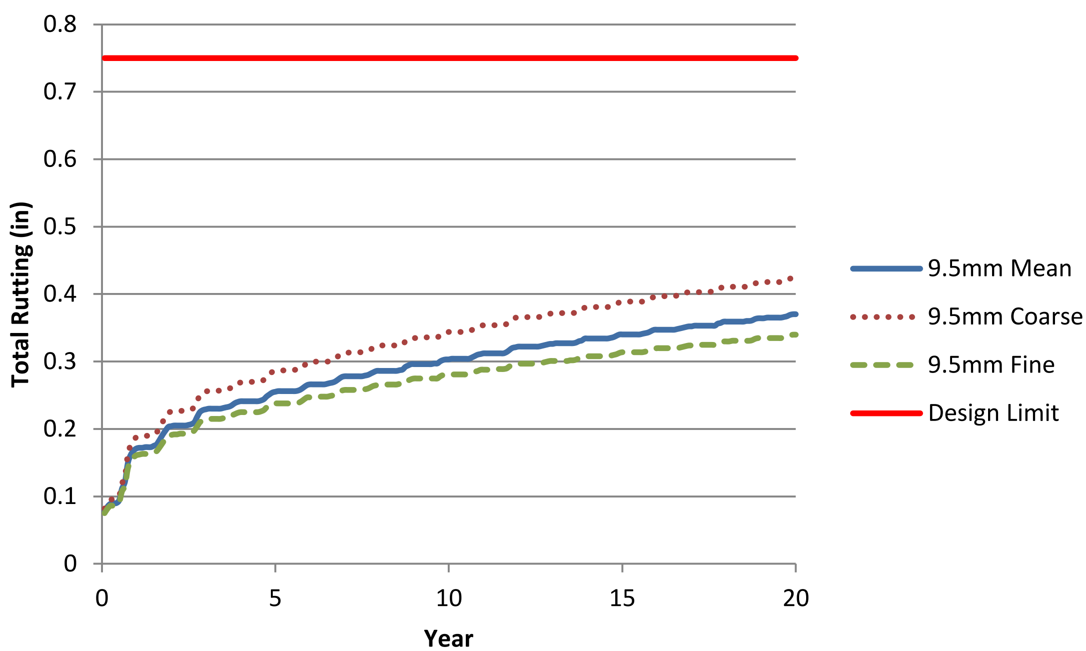

Figure 205 Effect of Aggregate Gradation of 9.5 mm AC mix on Total Rutting – Vermont level 3

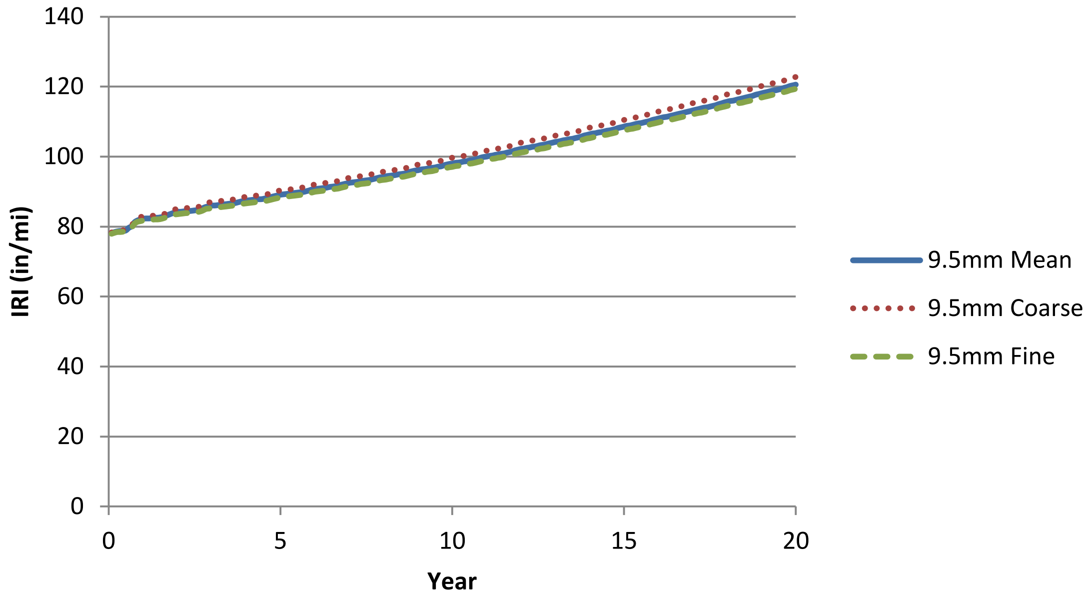

Figure 206 Effect of Aggregate Gradation of 9.5 mm AC mix on IRI – Vermont level 3

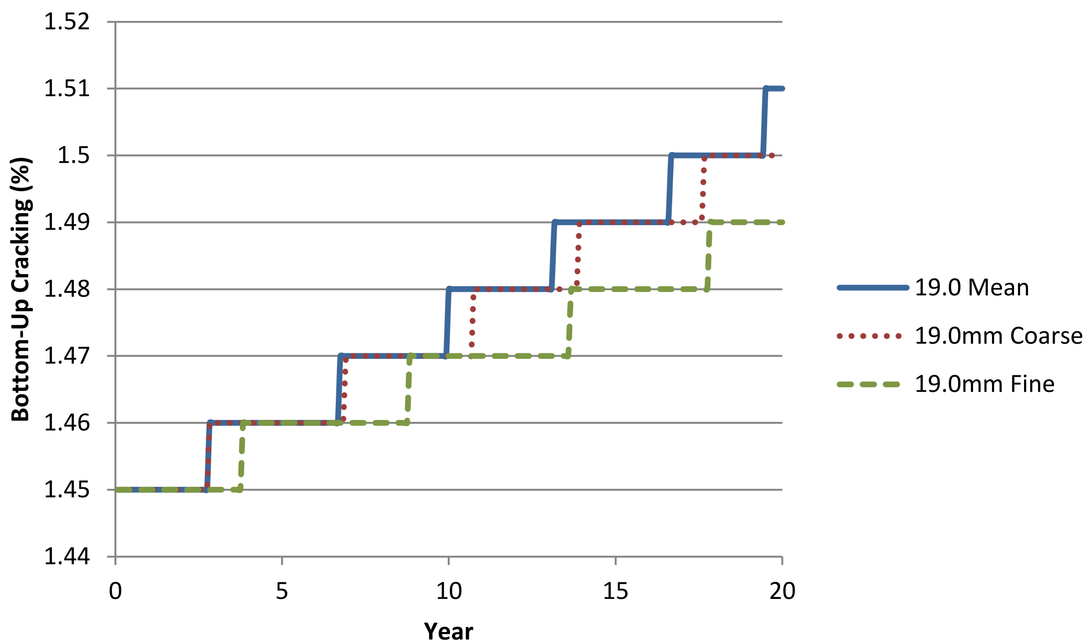

Figure 207 Effect of Aggregate Gradation of 19.0 mm mix on Bottom–Up Cracking – Vermont level 3

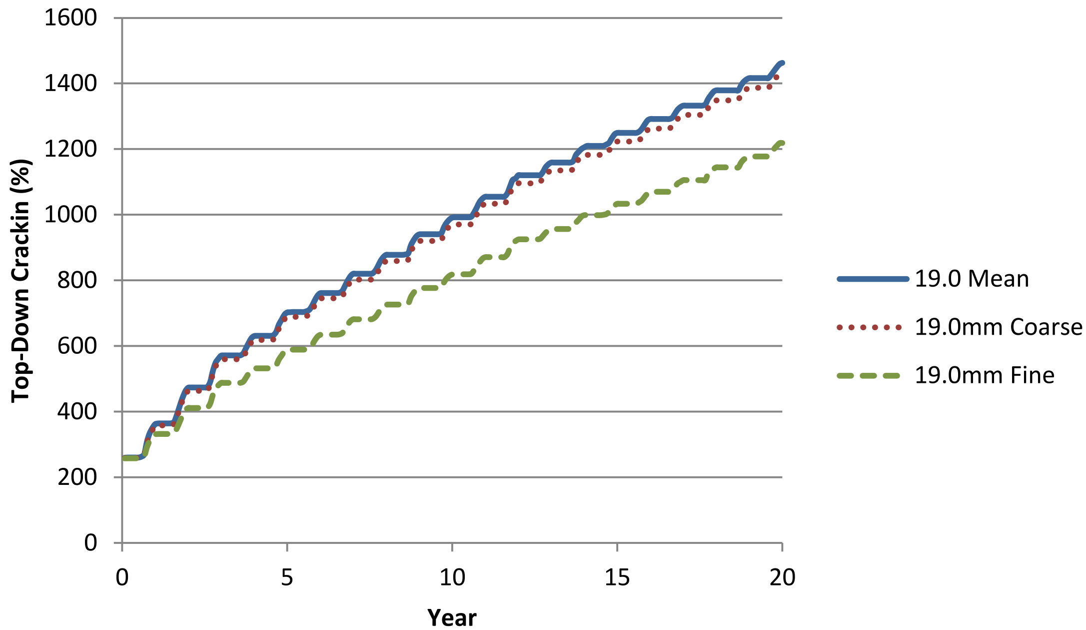

Figure 208 Effect of Aggregate Gradation of 19.0 mm mix on Top-Down Cracking – Vermont level 3

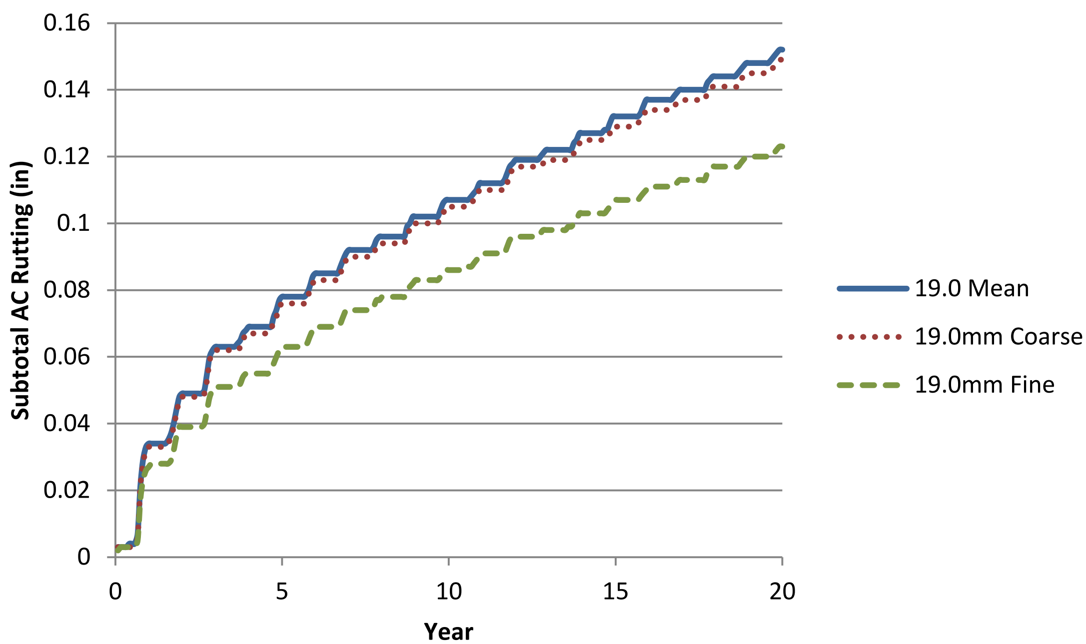

Figure 209 Effect of Aggregate Gradation of 19.0 mm mix on Subtotal AC Rutting – Vermont level 3

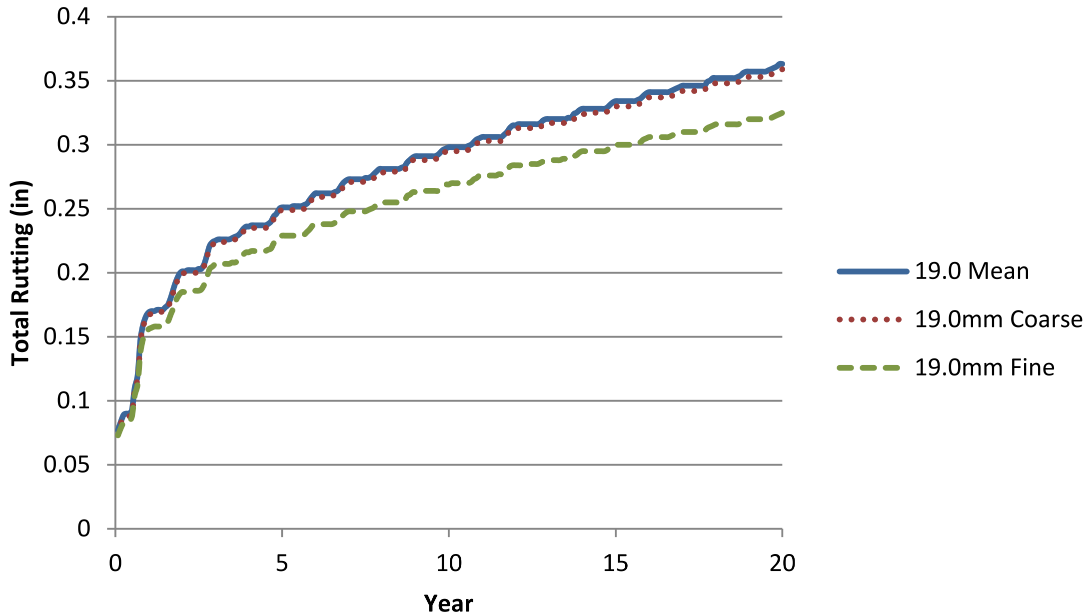

Figure 210 Effect of Aggregate Gradation of 19.0 mm mix on Total Rutting – Vermont level 3

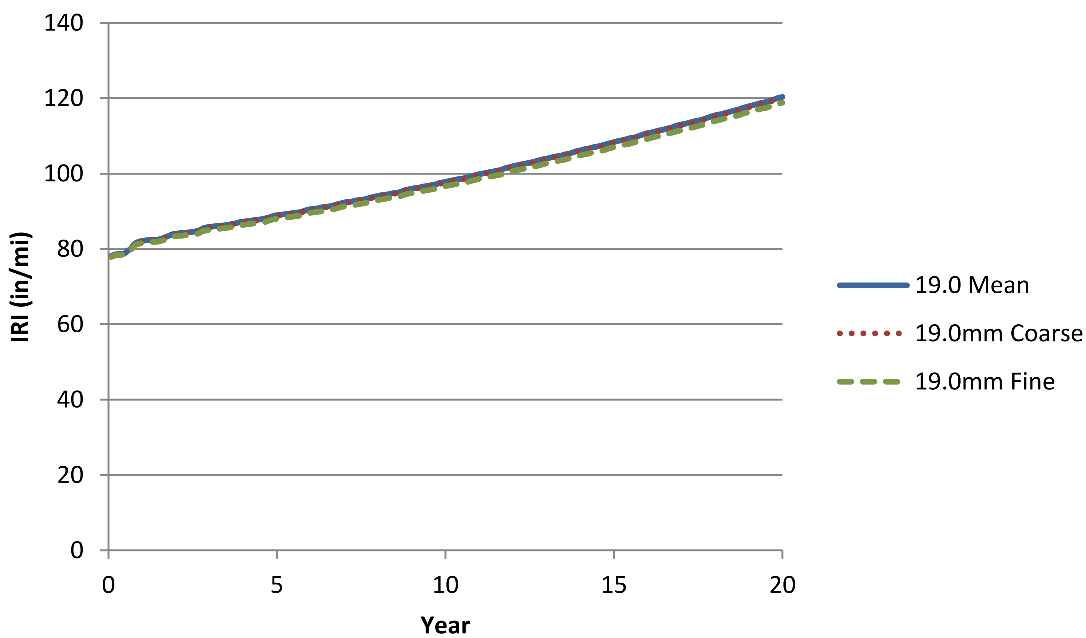

Figure 211 Effect of Aggregate Gradation of 19.0 mm mix on IRI – Vermont level 3

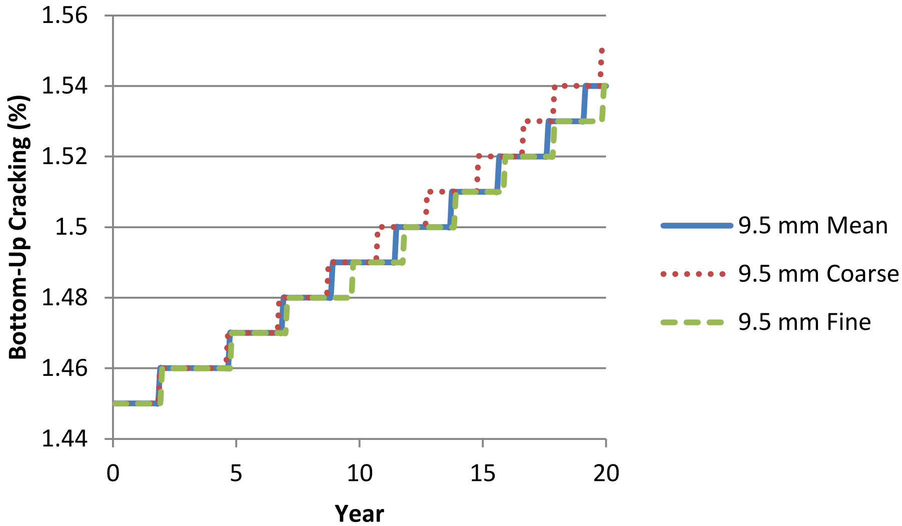

Figure 212 Effect of Aggregate Gradation of 9.5 mm AC mix on Bottom-Up Cracking – New York level 3

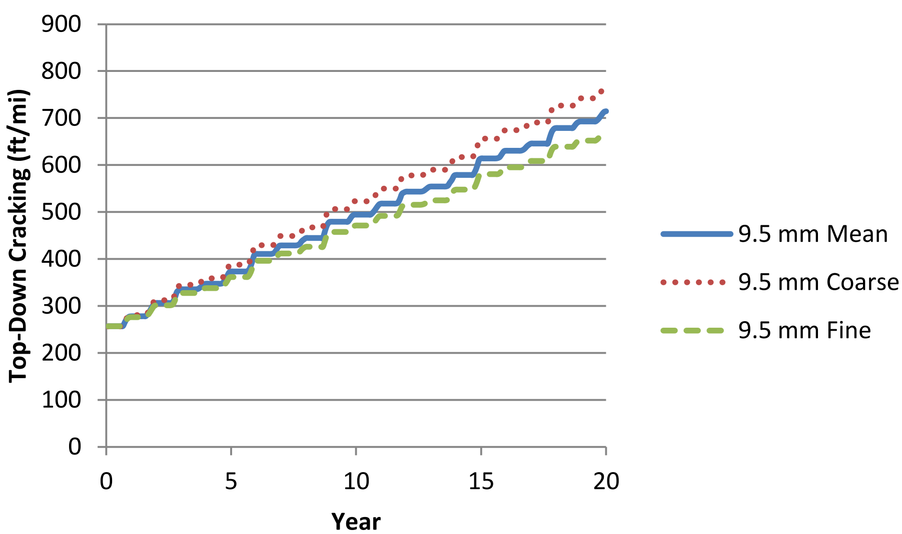

Figure 213 Effect of Aggregate Gradation of 9.5 mm AC mix on Top-Down Cracking – New York level 3

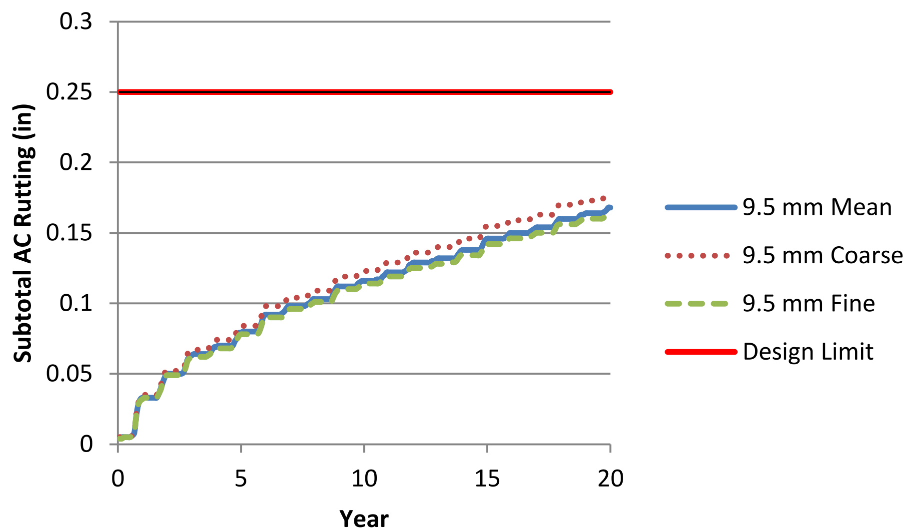

Figure 214 Effect of Aggregate Gradation of 9.5 mm AC mix on Subtotal AC Rutting – New York level 3

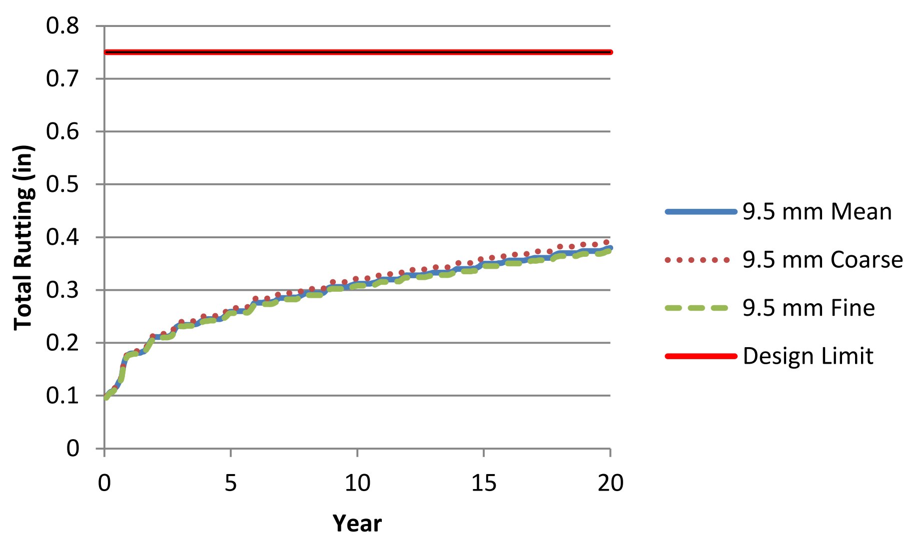

Figure 215 Effect of Aggregate Gradation of 9.5 mm AC mix on Total Rutting – New York level 3

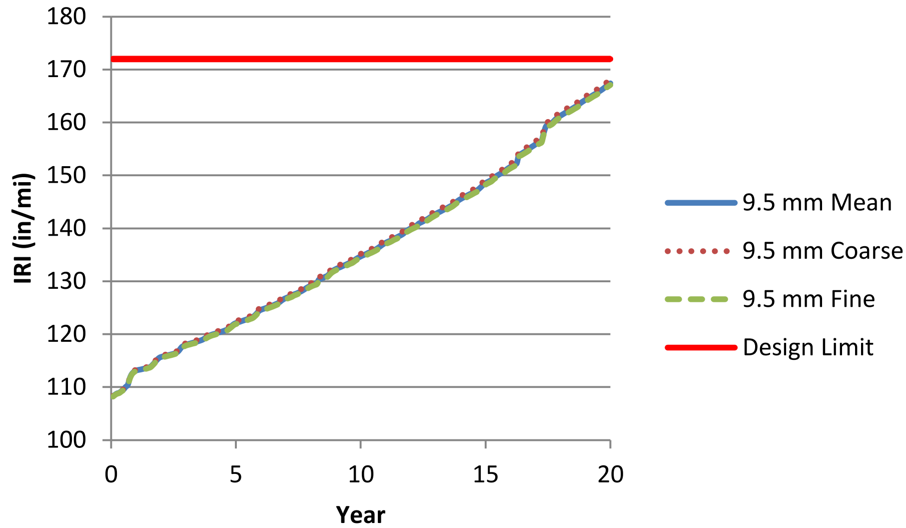

Figure 216 Effect of Aggregate Gradation of 9.5 mm AC mix on IRI – New York level 3

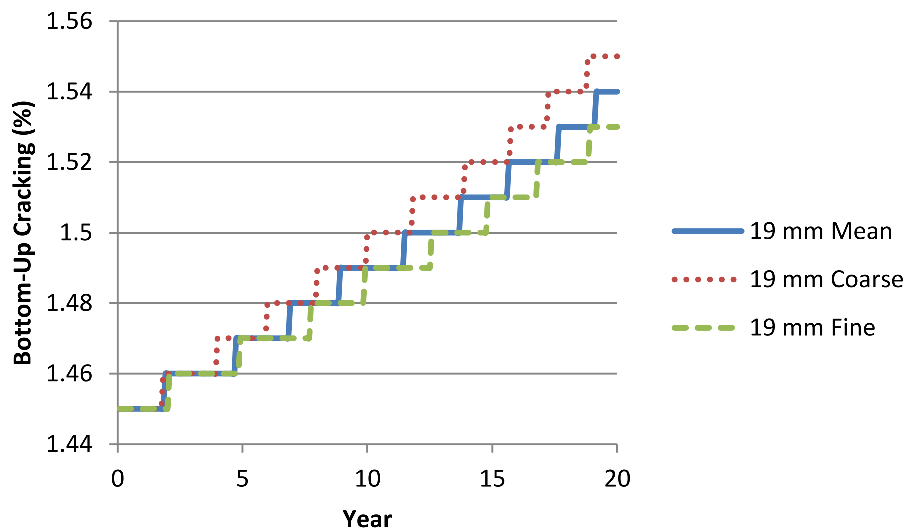

Figure 217 Effect of Aggregate Gradation of 19.0 mm mix on Bottom-Up Cracking – New York level 3

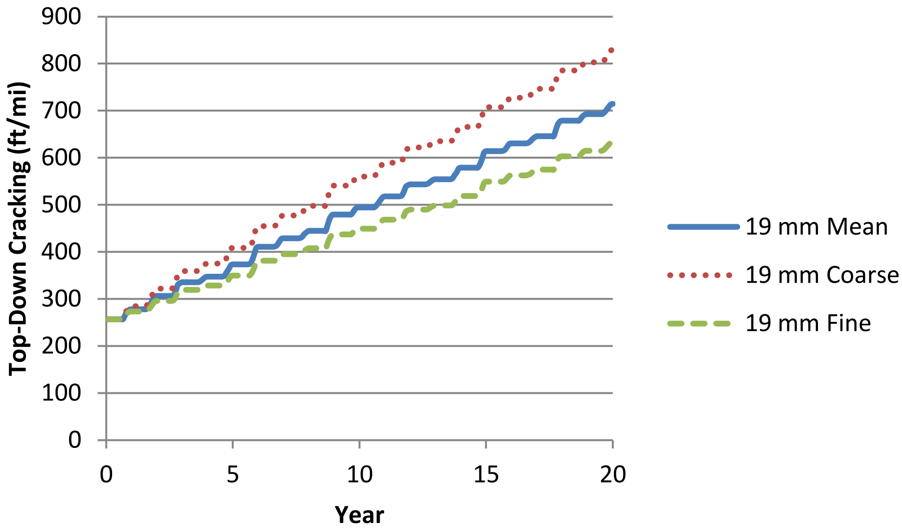

Figure 218 Effect of Aggregate Gradation of 19.0 mm mix on Top-Down Cracking – New York level 3

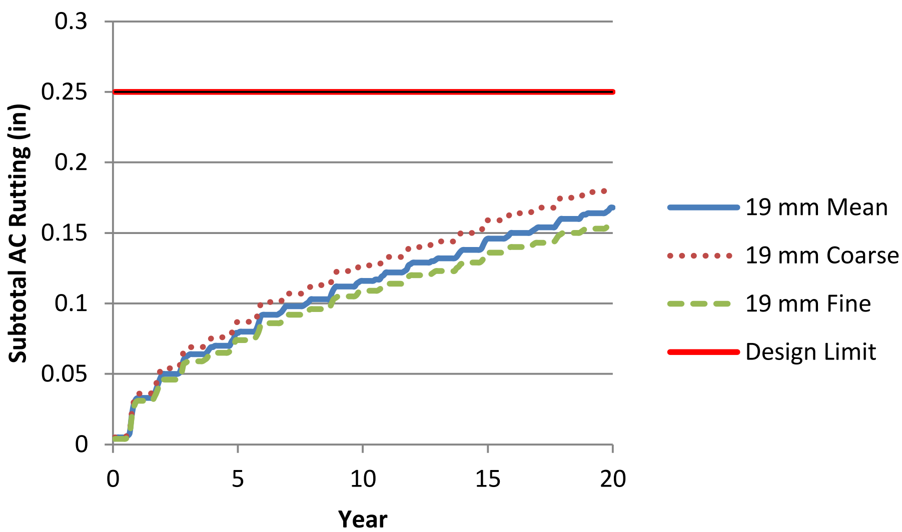

Figure 219 Effect of Aggregate Gradation of 19.0 mm mix on Subtotal AC Rutting – New York level 3

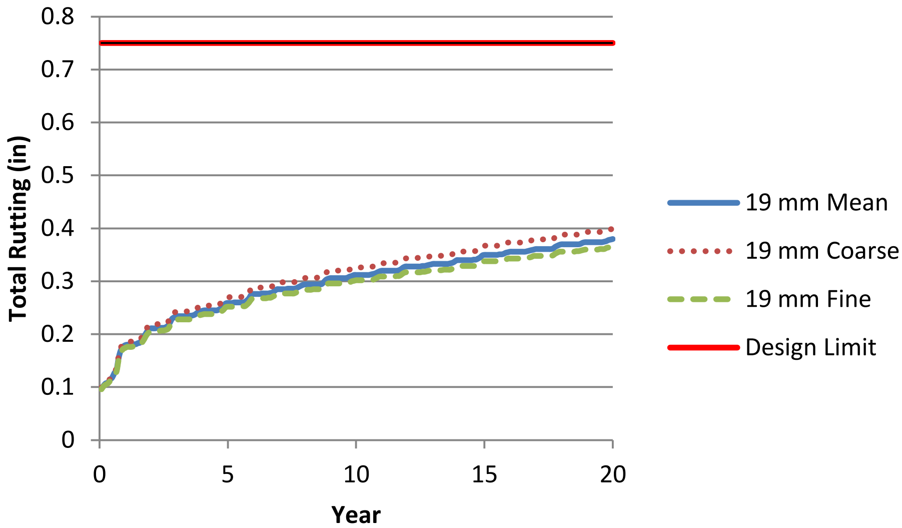

Figure 220 Effect of Aggregate Gradation of 19.0 mm mix on Total Rutting – New York level 3

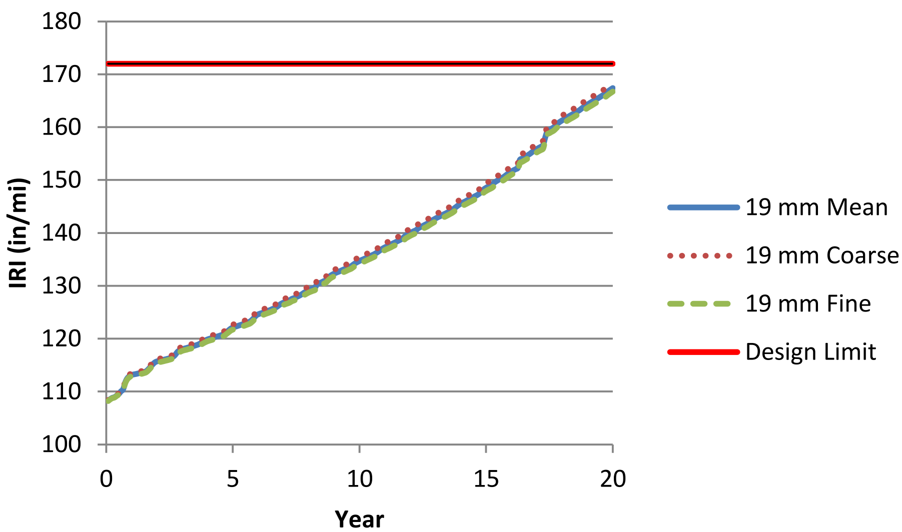

Figure 221 Effect of Aggregate Gradation of 19.0 mm mix on IRI – New York level 3

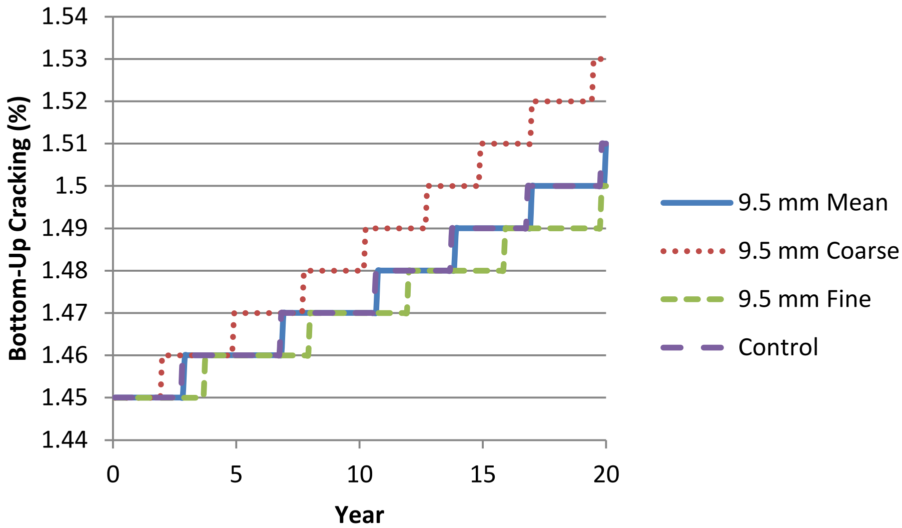

Figure 222 Effect of Aggregate Gradation of 9.5 mm AC mix on Bottom-Up Cracking – Massachusetts level 3

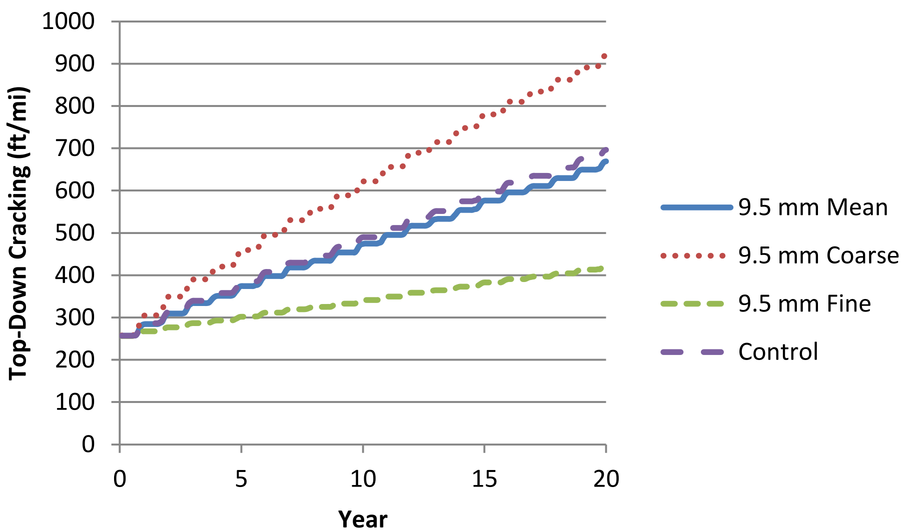

Figure 223 Effect of Aggregate Gradation of 9.5 mm AC mix on Top-Down Cracking – Massachusetts level 3

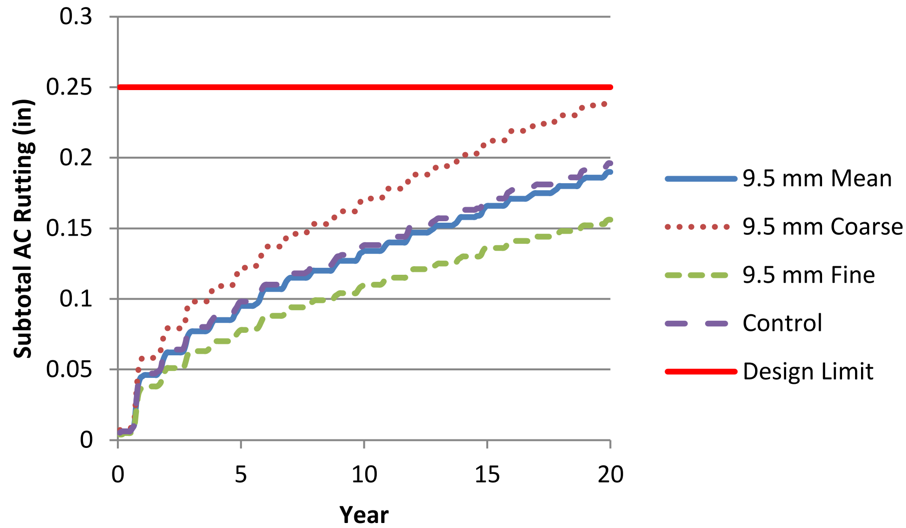

Figure 224 Effect of Aggregate Gradation of 9.5 mm AC mix on Subtotal AC Rutting – Massachusetts level 3

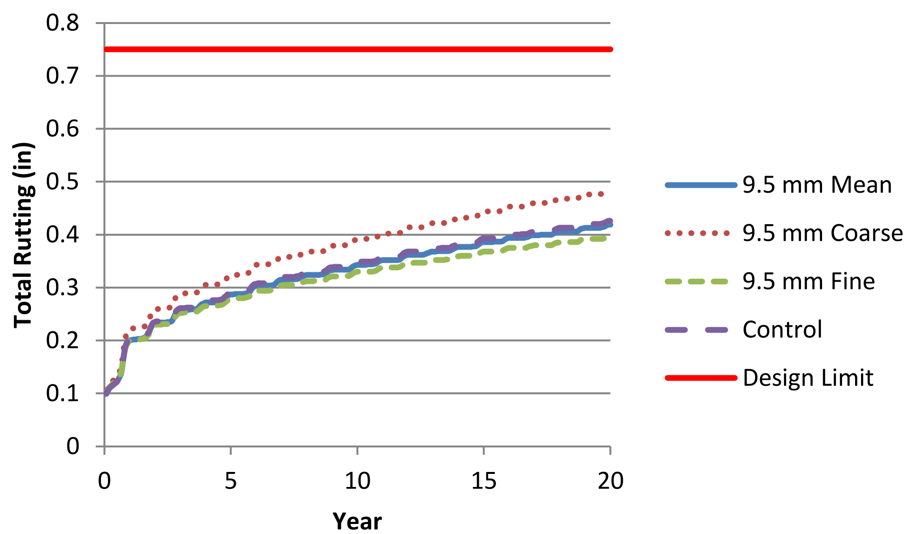

Figure 225 Effect of Aggregate Gradation of 9.5 mm AC mix on Total Rutting – Massachusetts level 3

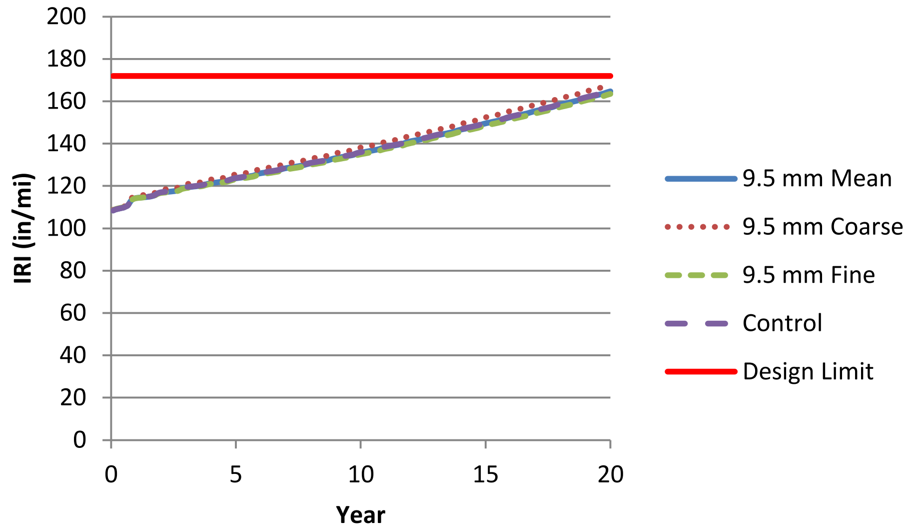

Figure 226 Effect of Aggregate Gradation of 9.5 mm AC mix on IRI – Massachusetts level 3

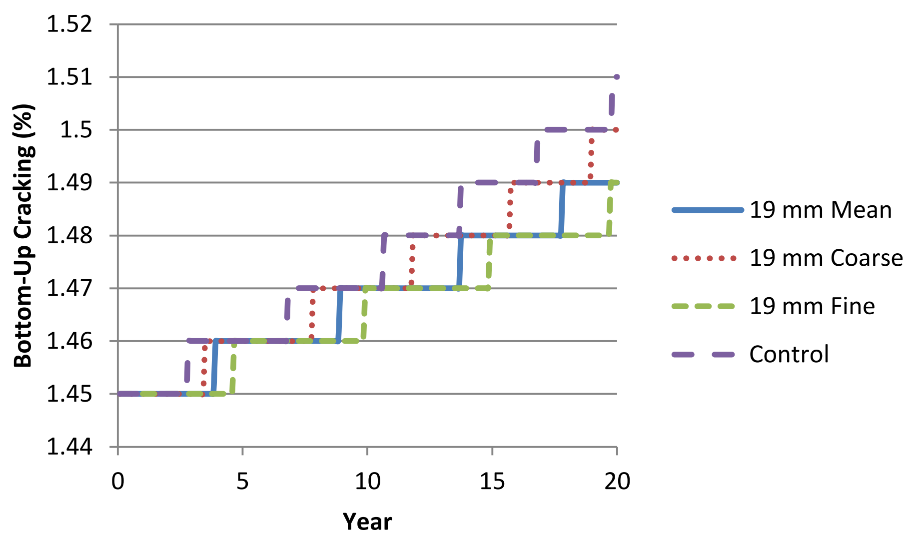

Figure 227 Effect of Aggregate Gradation of 19.0 mm mix on Bottom-Up Cracking – Massachusetts level 3

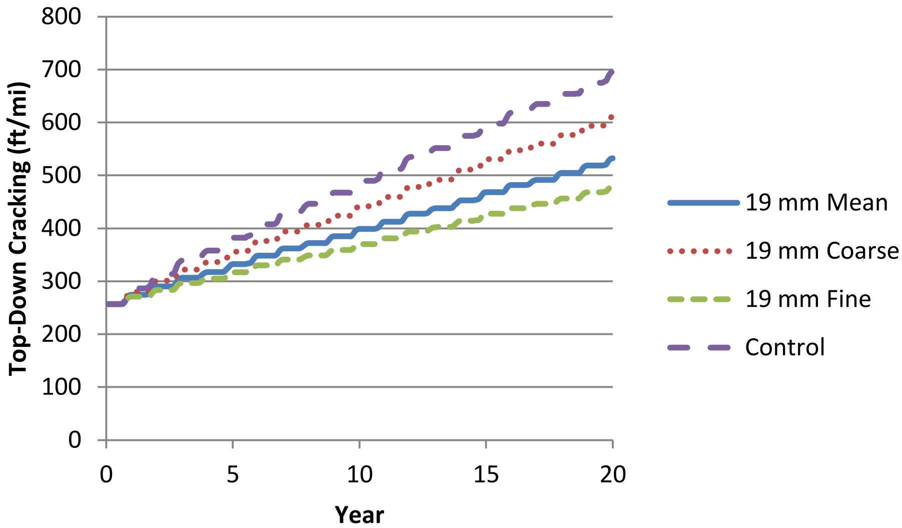

Figure 228 Effect of Aggregate Gradation of 19.0 mm mix on Top-Down Cracking – Massachusetts level 3

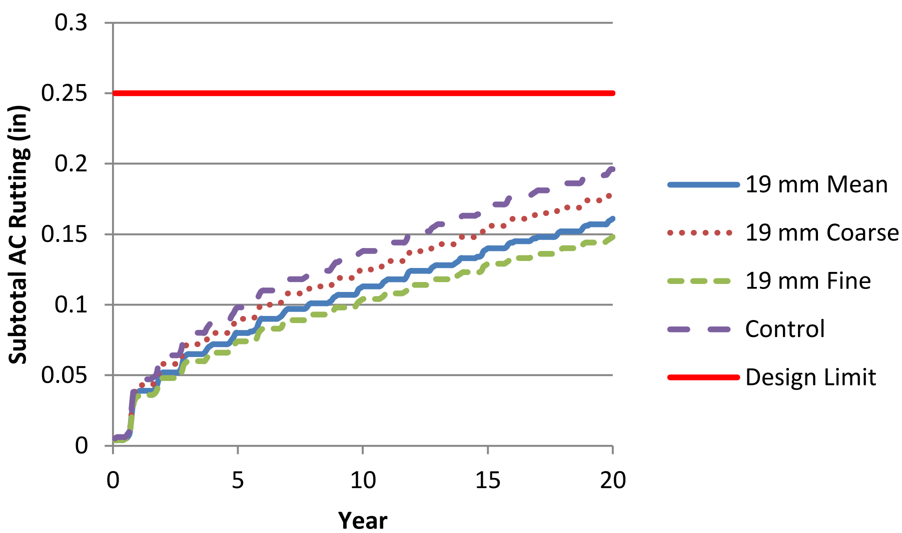

Figure 229 Effect of Aggregate Gradation of 19.0 mm mix on Subtotal AC Rutting – Massachusetts level 3

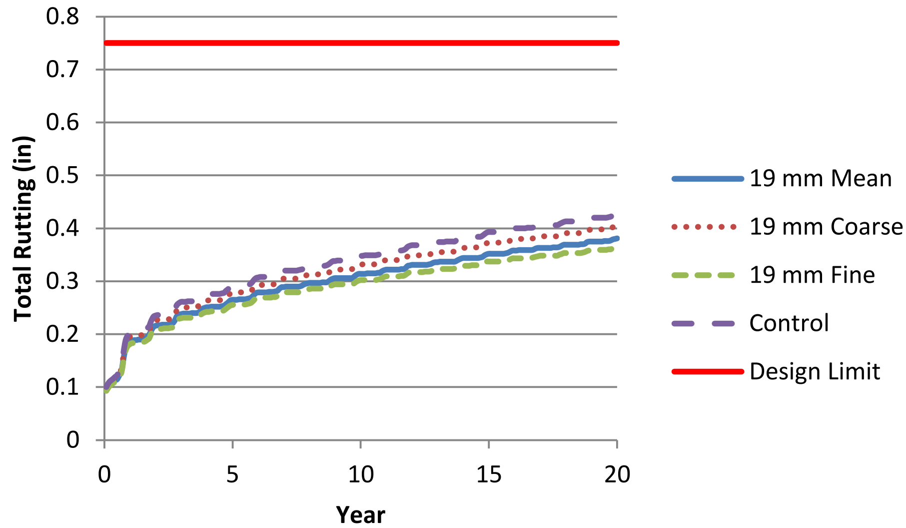

Figure 230 Effect of Aggregate Gradation of 19.0 mm mix on Total Rutting – Massachusetts level 3

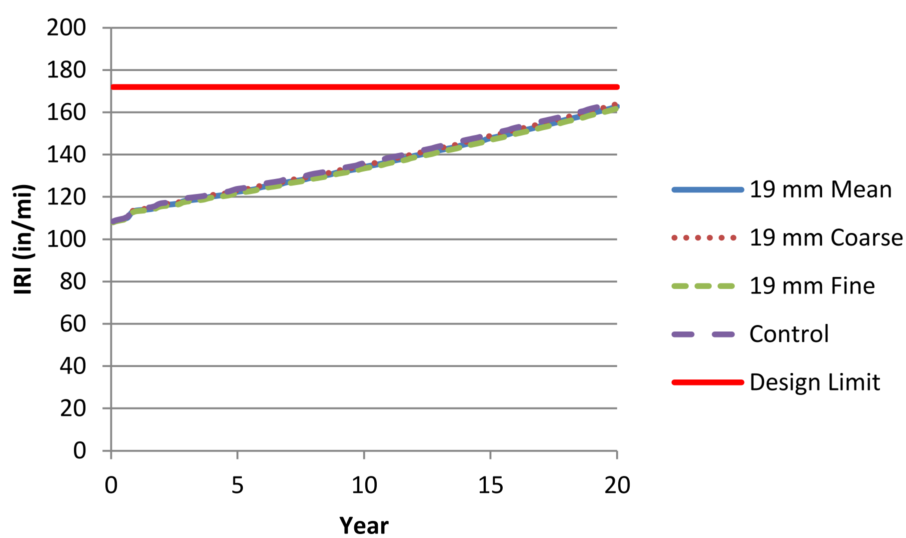

Figure 231 Effect of Aggregate Gradation of 19.0 mm mix on IRI – MA level 3

Asphalt Binder Grade

Asphalt binder grade was selected using the PG binder grading specifications for this study. A list of all permissible binder grades was obtained for each state from LTPP Bind software (46). The median binder grade was selected as the control asphalt binder grade, and the binder grade was varied by one high temperature and one low temperature grade (if the resultant binder grade is allowed for use as directed by the output of LTPP Bind software). Design operational speed on the highway plays a very significant role in determining which binder grade to use for a project. Therefore, the two input parameters were studied by performing a factorial-run experiment.

The results support the recommendation made by Superpave specifications to increase high-temperature binder grade by one grade for slow-moving traffic. Therefore, for design of local roads, depending on the required reliability on predicted distresses and desired design life period, an increase of high-temperature grade of PG of asphalt binder can be considered, but strictly recommended for medium- to high-importance projects for lower operational speeds.

The interaction effect of these two variables is significant on cracking, rutting as well as roughness. The pavement failure is significantly enhanced for low operational speeds. The performance prediction curves shown below show the progressive delay of failure of the pavement in asphalt concrete rutting. A downward shift of the curves for each binder grade indicates that the rutting also decreases with an increase in operational speed.

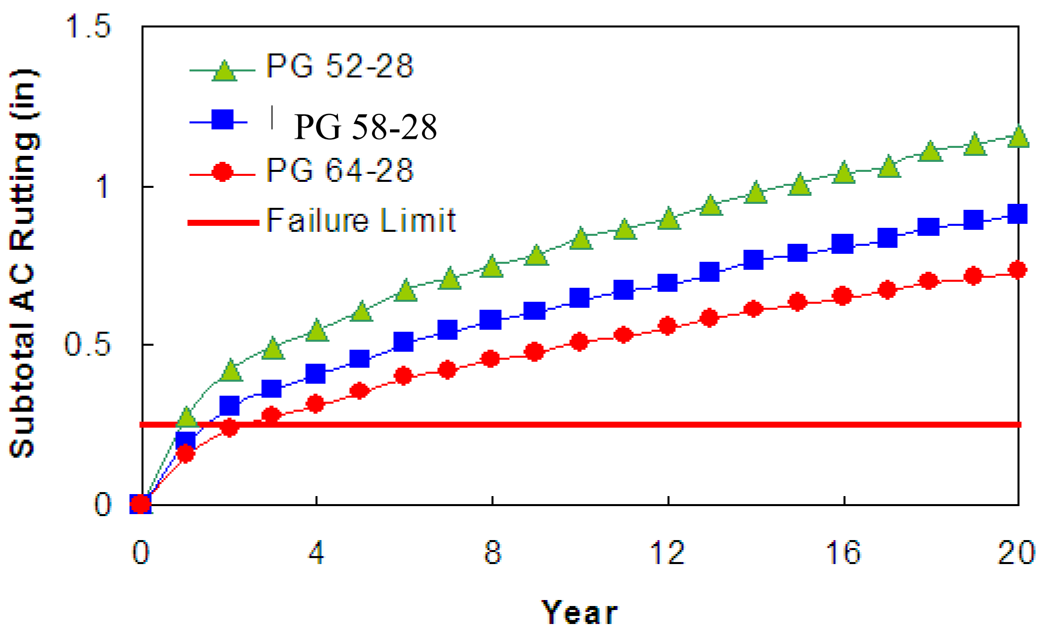

Figure 232 Effect of Binder Grade on Subtotal AC Rutting at Speed 5 mph – NH

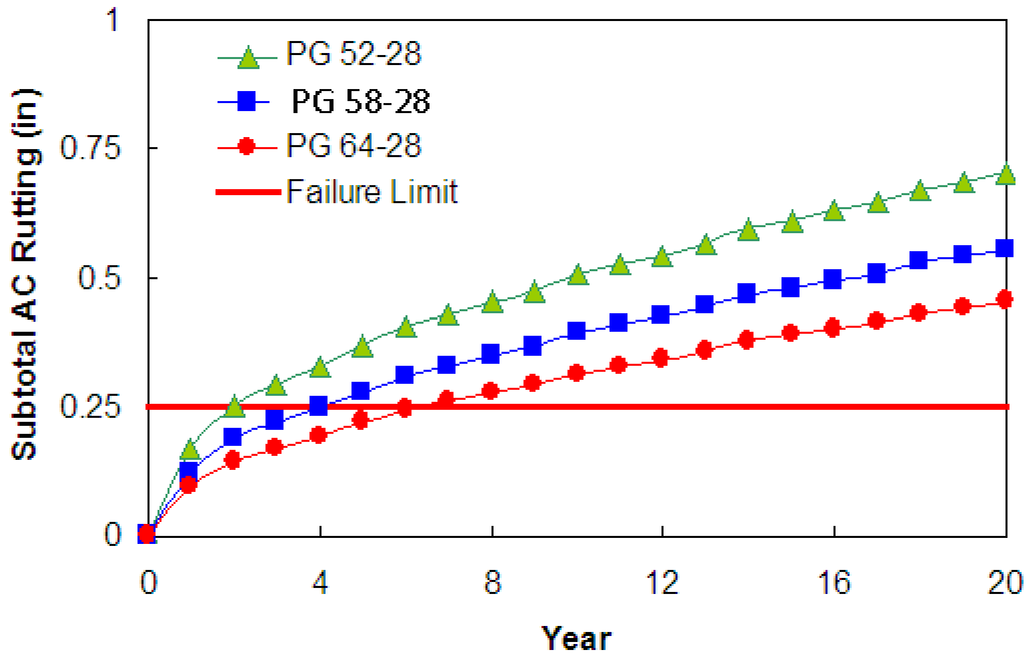

Figure 233 Effect of Binder Grade on Subtotal AC Rutting at Speed 25 mph – NH

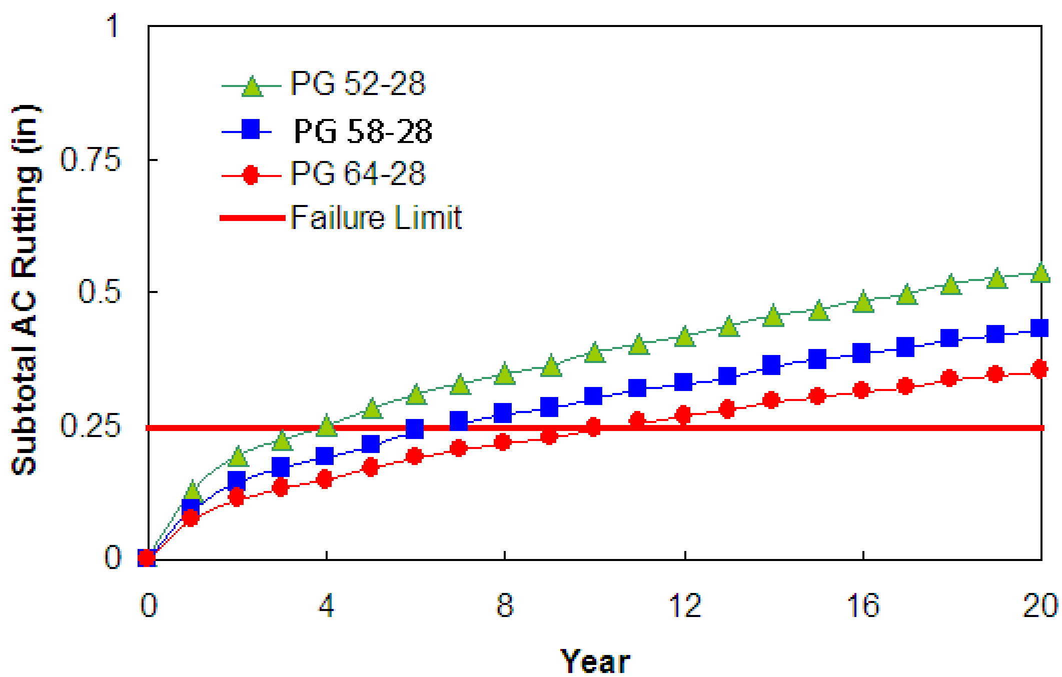

Figure 234 Effect of Binder Grade on Subtotal AC Rutting at Speed 65 mph – NH

For Level 2 analysis, the same binder grades could not be tested due to unavailability of binder testing data from digital shear rheometer – G* and sin δ values are required to characterize asphalt binder in Level 2 design. Therefore, available data was used to assess the sensitivity of predicted distresses for different operational speeds on the highway. This activity provides information on the performance of the highway under different operational conditions of traffic.

Predicted pavement distresses were statistically not sensitive to binder grade and design operational speed for Maine data, and showed no sensitivity to Level 2 input data. This observation can be explained due to a full-depth asphalt concrete pavement structure, where the binder grade of only the 1.2” porous friction course was varied keeping that of the 8.3” asphalt binder course was kept constant. Therefore, the variability in the prediction can be attributed to the effect of changing binder grade of the surface AC layer only.

Binder grade must be selected meticulously after repeated trial runs to minimize the predicted distresses for a new asphalt pavement construction, whereas an overlay of a small thickness (less than or equal to 2 inches) does not require extreme caution in selection of binder grade

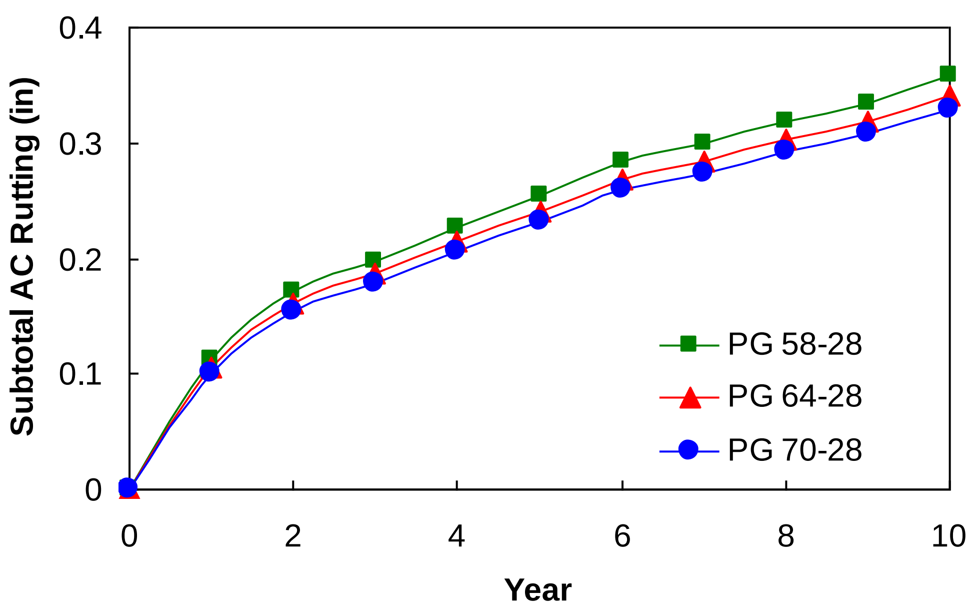

Figure 235 Effect of Binder Grade on AC Rutting: Design Speed 5 mph – Maine Level 3

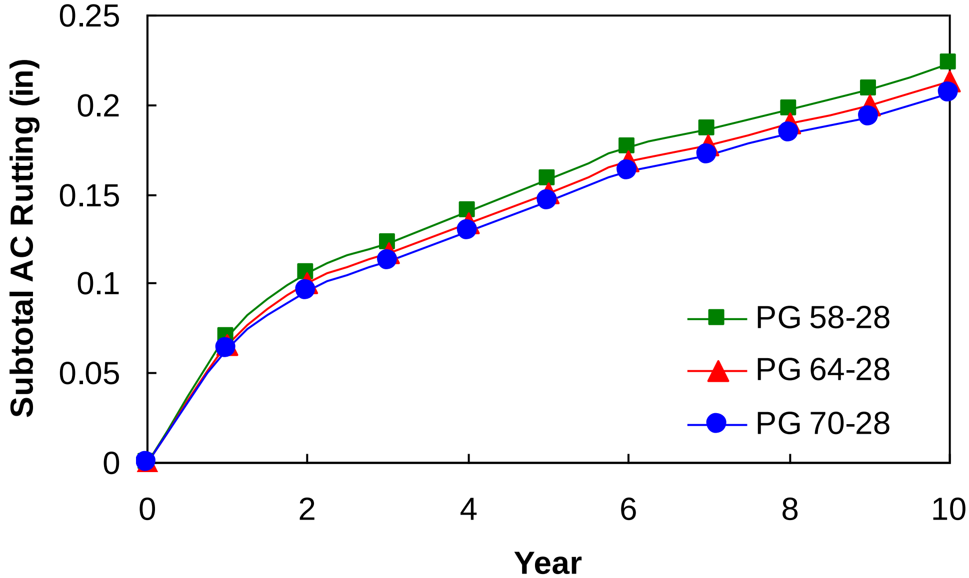

Figure 236 Effect of Binder Grade on AC Rutting: Design Speed 25mph – Maine Level 3

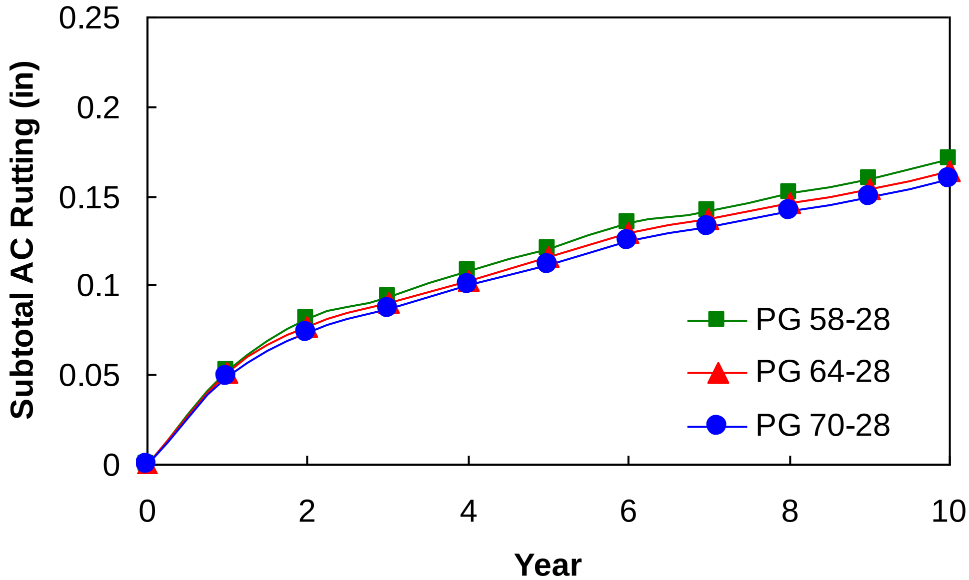

Figure 237 Effect of Binder Grade on AC Rutting: Design Speed 65mph – Maine Level 3

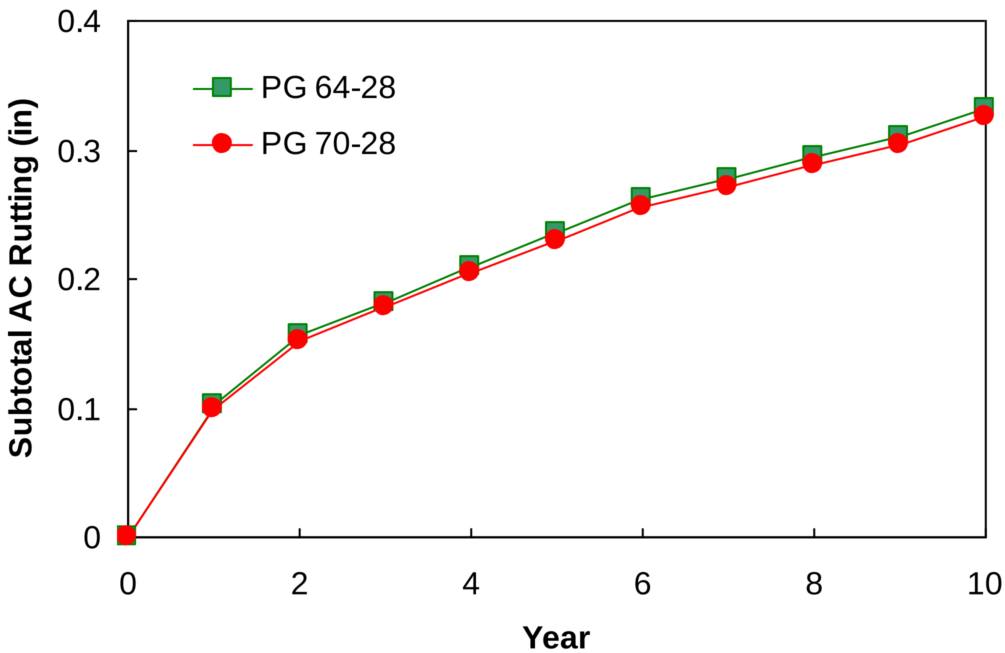

Figure 238 Effect of Binder Grade on AC Rutting: Speed 5 mph – Maine Level 2

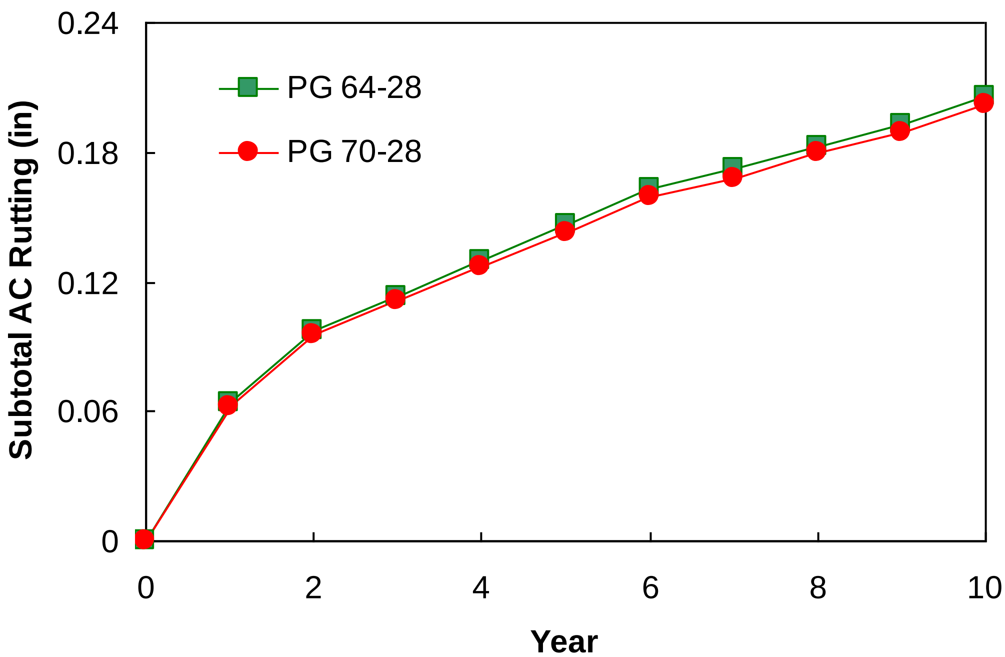

Figure 239 Effect of Binder Grade on AC Rutting: Speed 25 mph – Maine Level 2

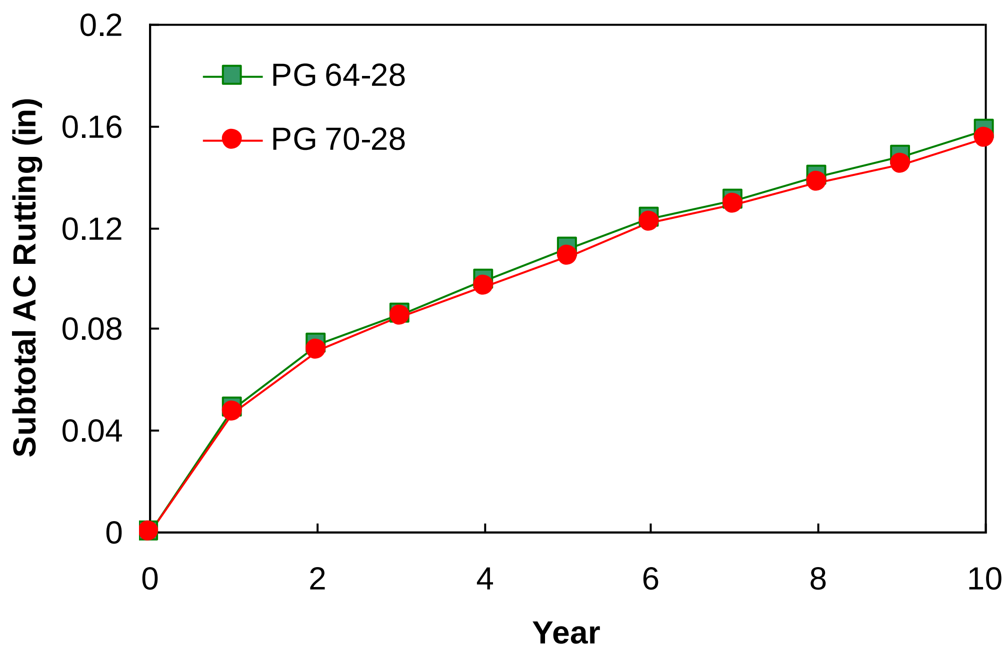

Figure 240 Effect of Binder Grade on AC Rutting: Speed 65 mph – Maine Level 2

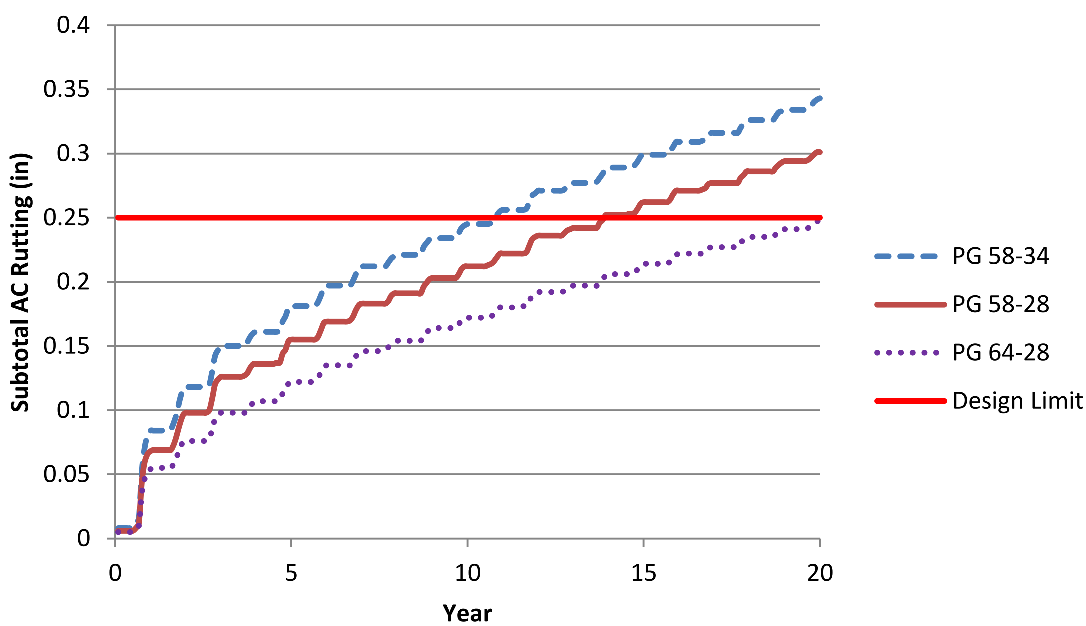

Figure 241 Effect of Binder Grade on Subtotal AC Rutting at Speed 5 mph – Vermont Level 3

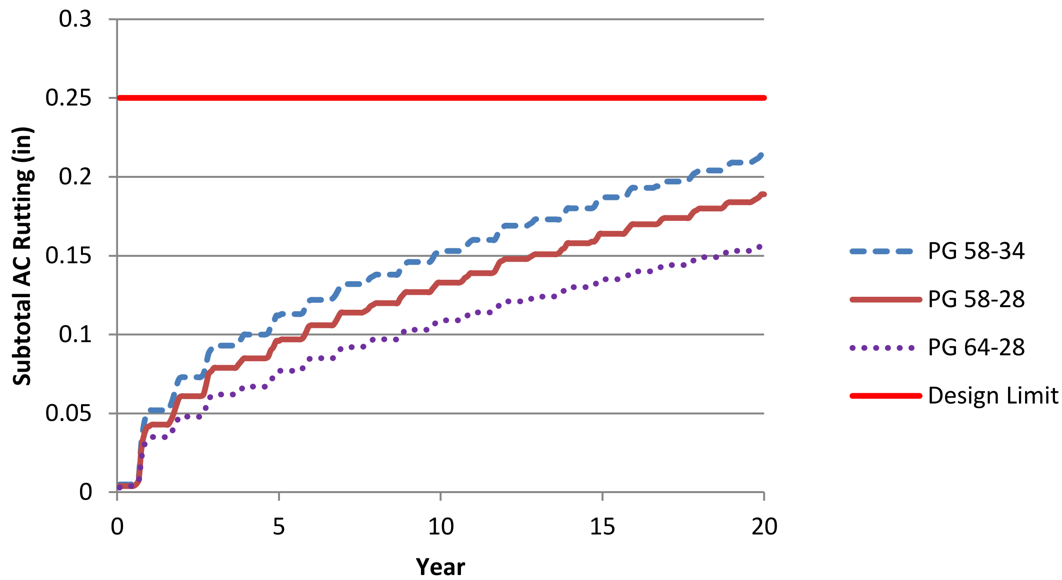

Figure 242 Effect of Binder Grade on Subtotal AC Rutting at Speed 25 mph – Vermont Level 3

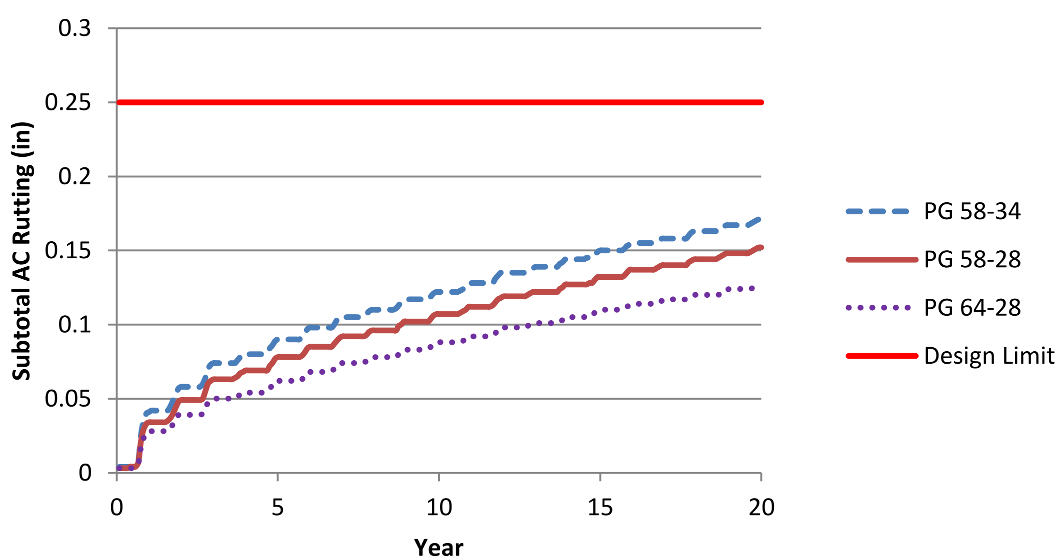

Figure 243 Effect of Binder Grade on Subtotal AC Rutting at Speed 55 mph – Vermont Level 3

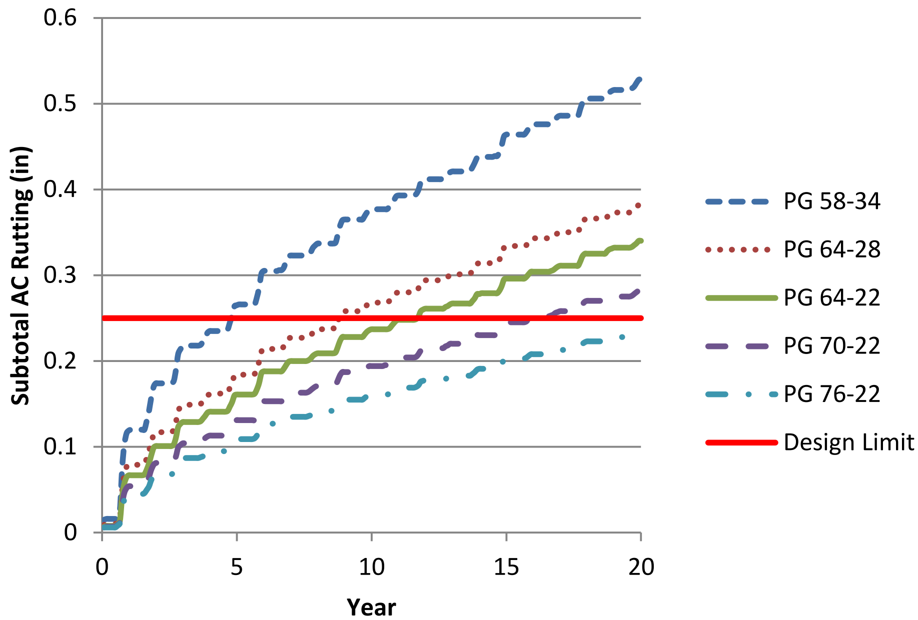

Figure 244 Effect of Binder Grade on Subtotal AC Rutting at Speed 5 mph – New York Level 3

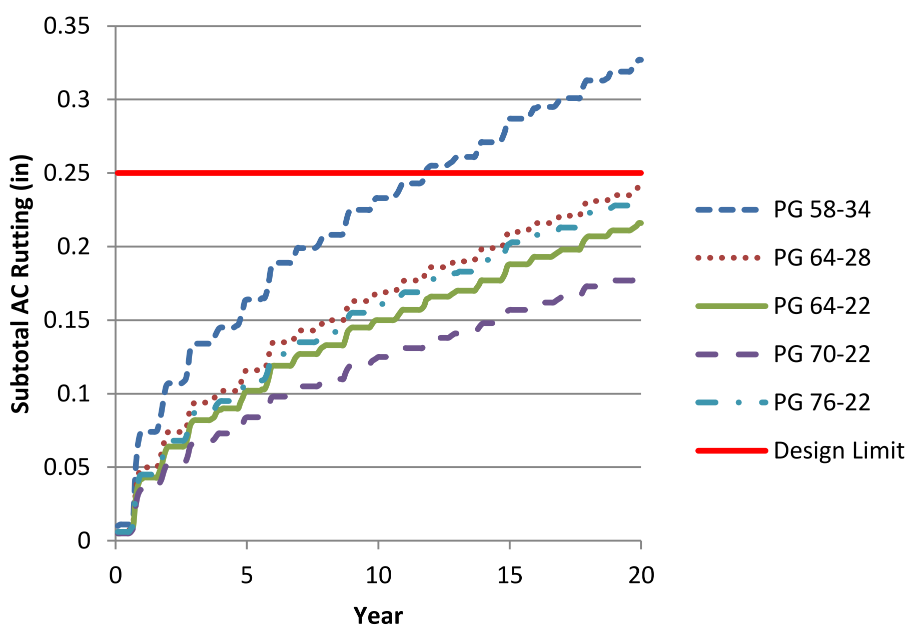

Figure 245 Effect of Binder Grade on Subtotal AC Rutting at Speed 25 mph – New York Level 3

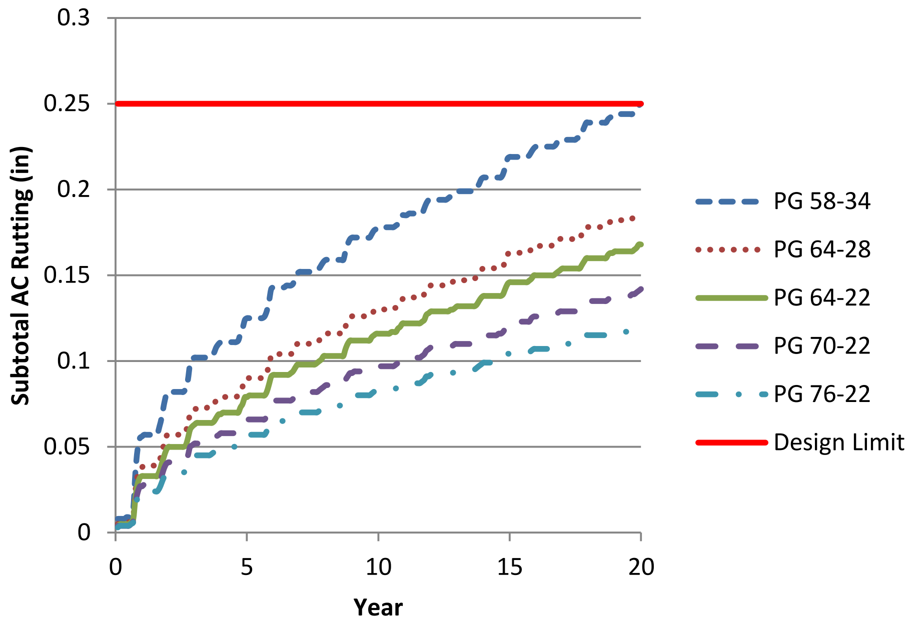

Figure 246 Effect of Binder Grade on Subtotal AC Rutting at Speed 65 mph – New York Level 3

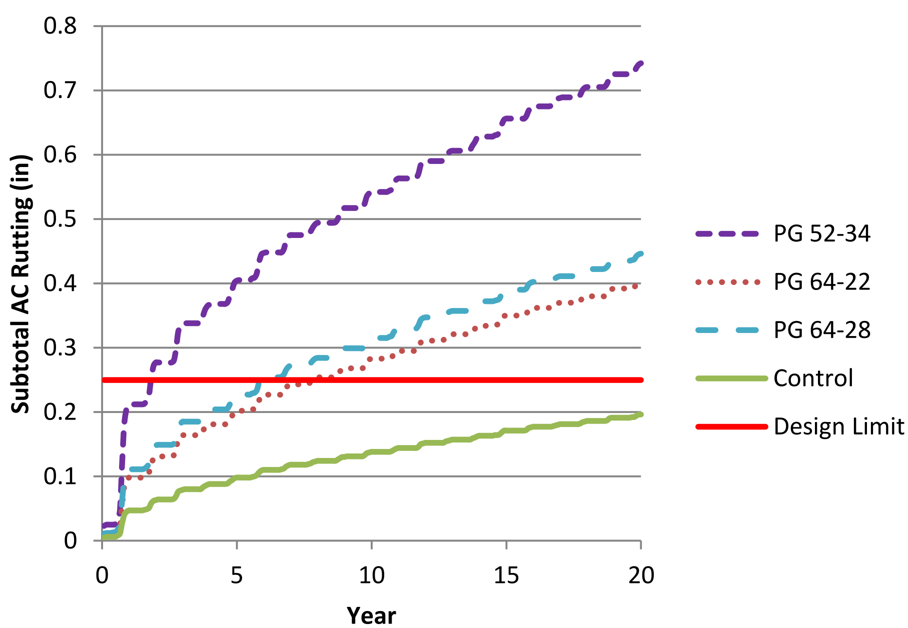

Figure 247 Effect of Binder Grade on Subtotal AC Rutting at Speed 5 mph – Massachusetts Level 3

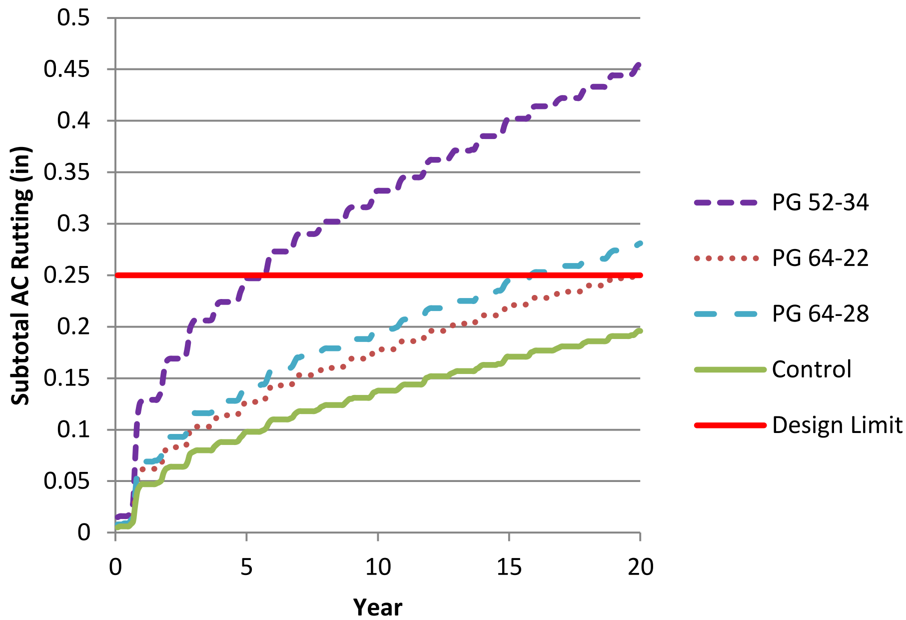

Figure 248 Effect of Binder Grade on Subtotal AC Rutting at Speed 25 mph – Massachusetts Level 3

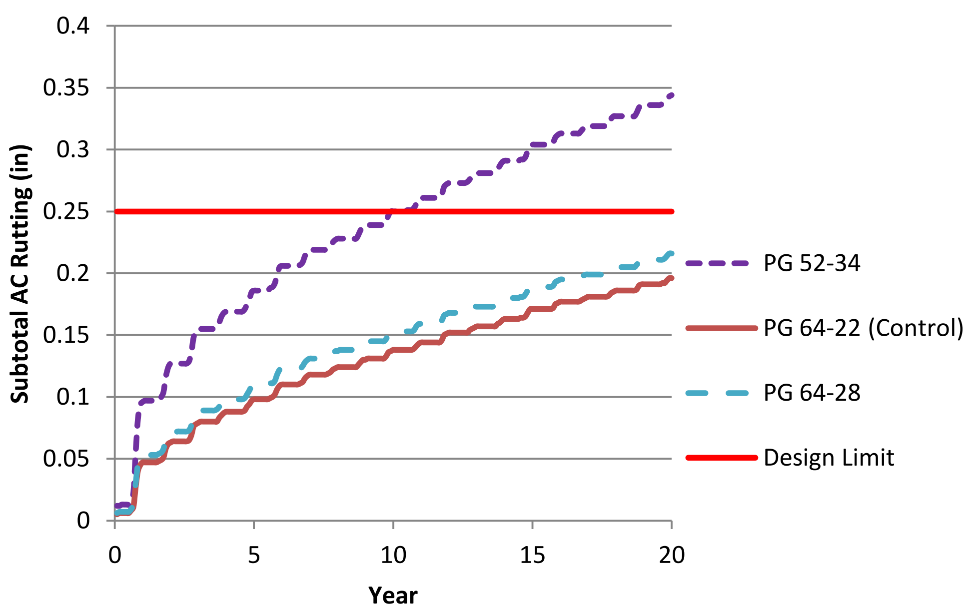

Figure 249 Effect of Binder Grade on Subtotal AC Rutting at Speed 65 mph – Massachusetts Level 3

6.1.4 Effect of Material Inputs on Pavement Distresses – Unbound Materials

Subgrade Properties

Subgrade soil types for all the states studied were obtained from reports published by various researchers, the primary source being the subgrade studies conducted for New England Transportation Consortium, Project 02-3 (33). The effect of varying soil types on predicted distresses explains the adequacy of each type of soil to function under the given traffic, climate and structural conditions. Subgrade type was not found to significantly affect top-down cracking and rutting in the asphalt concrete layer, but affected bottom-up cracking, total rutting and roughness with moderate significance.

The parameters that were entered for subgrade properties are the resilient modulus values. Resilient modulus values for Level 3 were used from the design guide defaults, whereas Level 2 values are obtained from modulus databases compiled after conducting laboratory tests on a large number of specimens (33). Soil gradation values were also extracted by digitization of graphs from the reports used as reference for this purpose.

Since same parameters were entered for Level 2 and Level 3, the performance trends are similar for both levels of design.

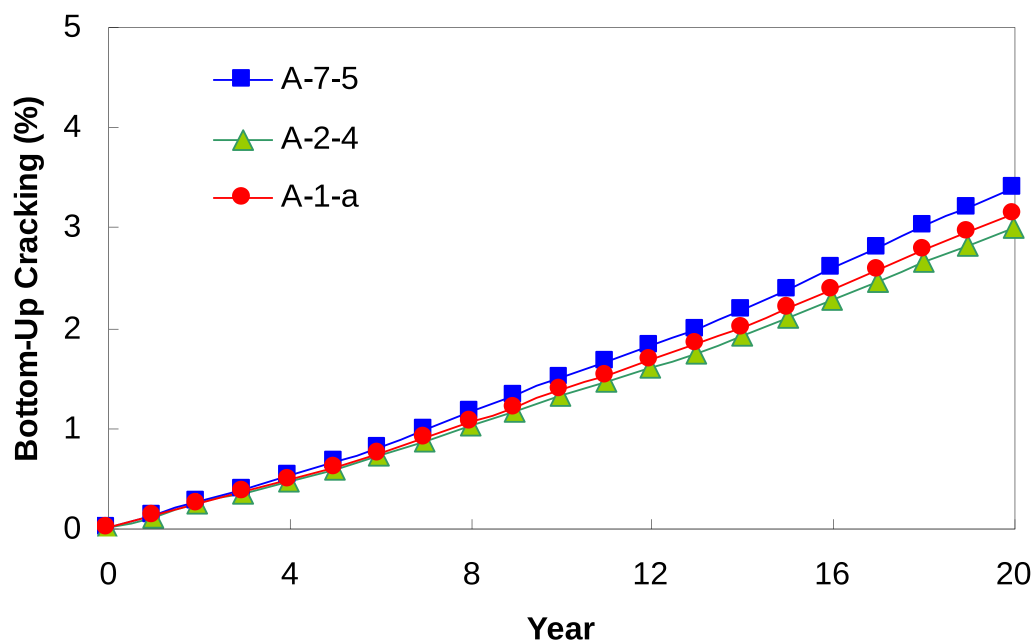

Figure 250 Effect of Subgrade Type on Bottom-Up Cracking – NH Level 3

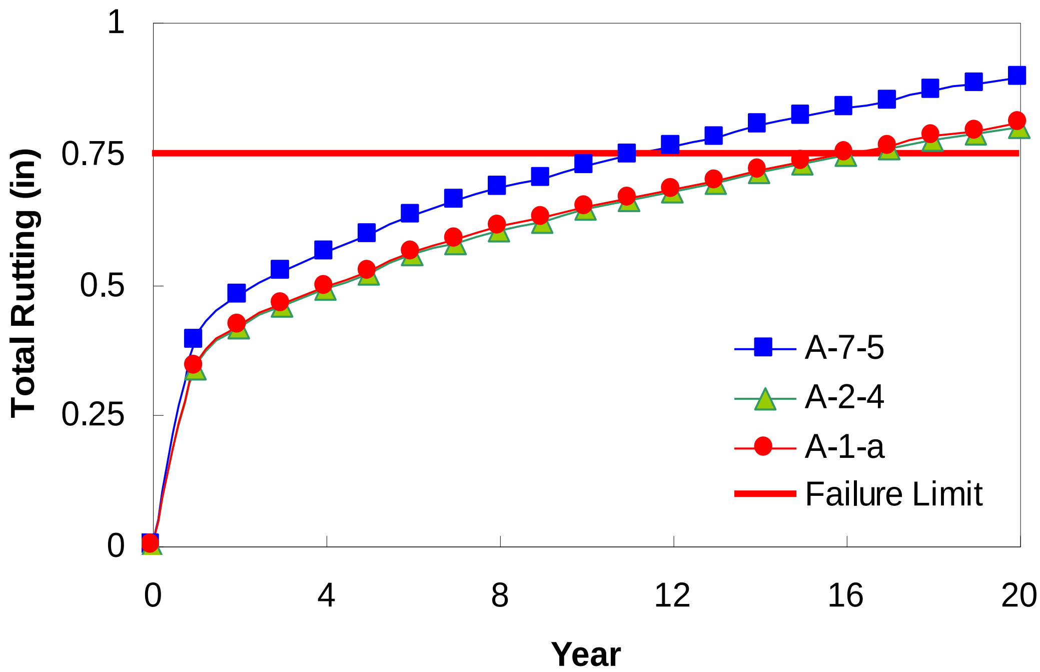

Figure 251 Effect of Subgrade Type on Total Rutting – NH Level 3

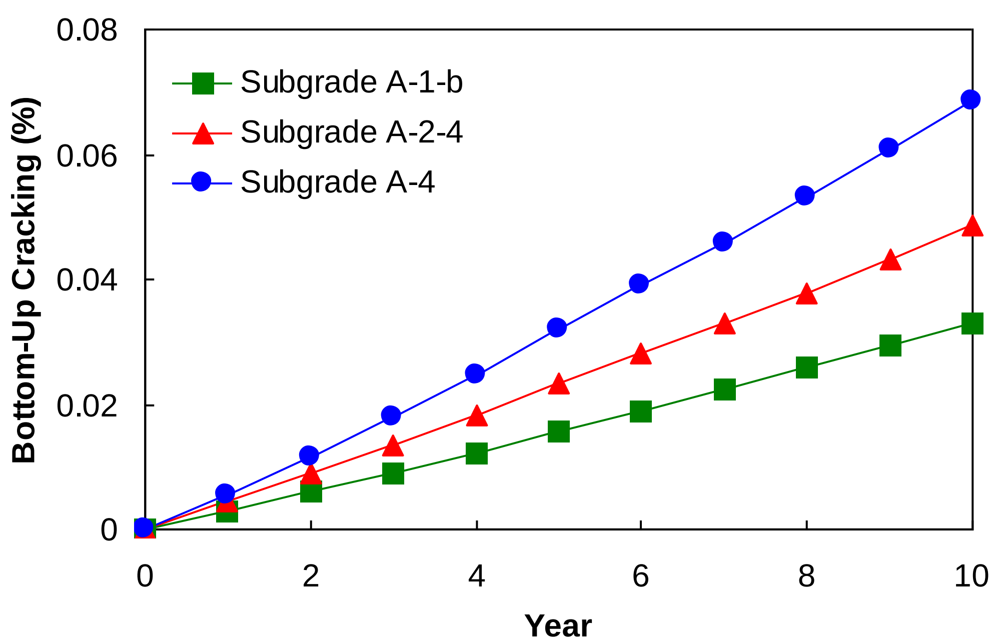

Figure 252 Effect of Subgrade Type on Bottom-Up Cracking – Connecticut Level 2

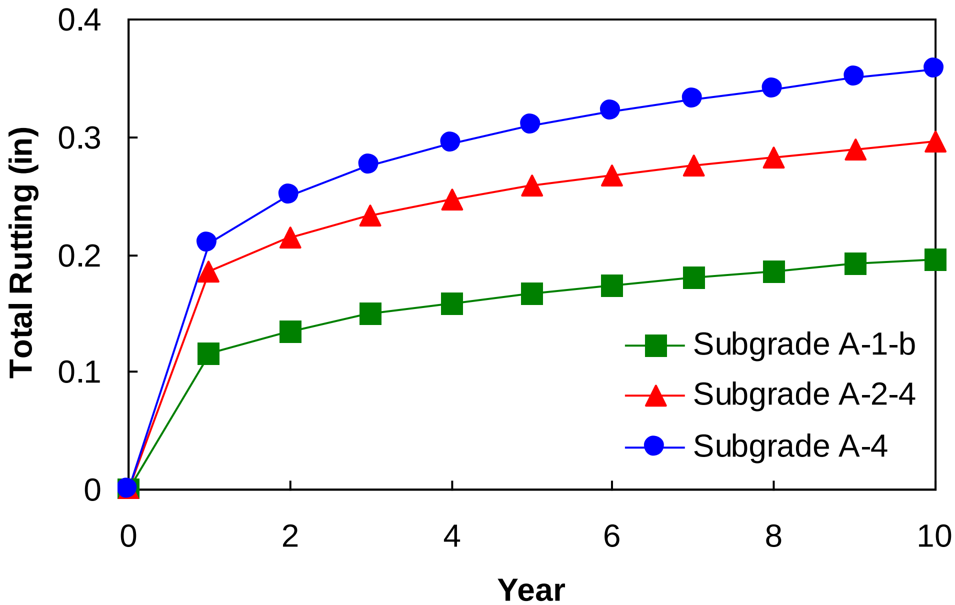

Figure 253 Effect of Subgrade Type on Total Rutting – Connecticut Level 2

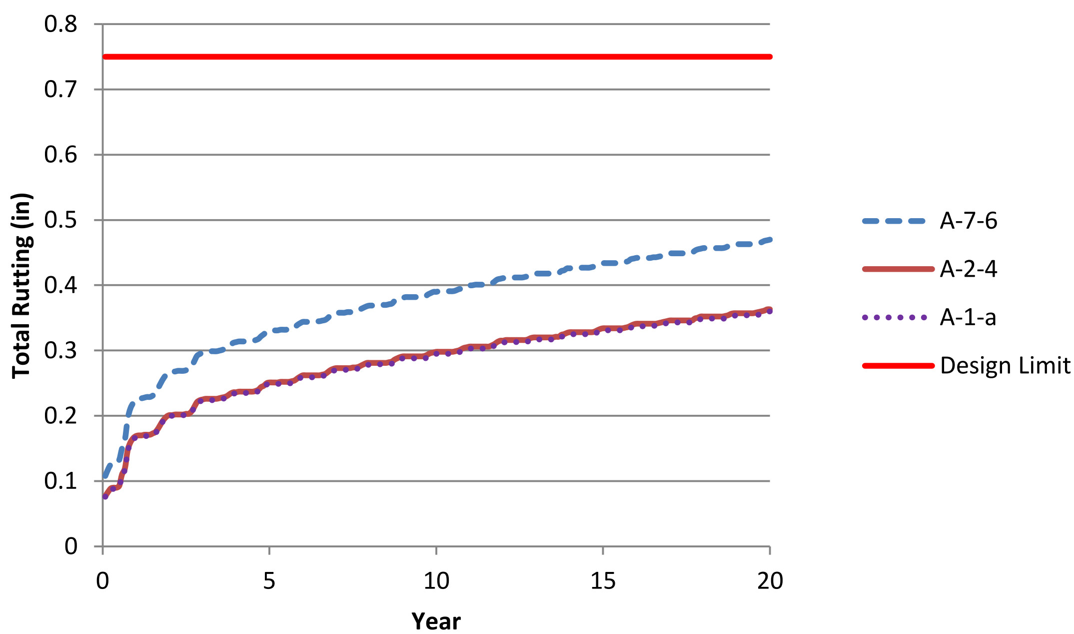

Figure 254 Effect of Subgrade Type on Total Rutting – Vermont Level 3

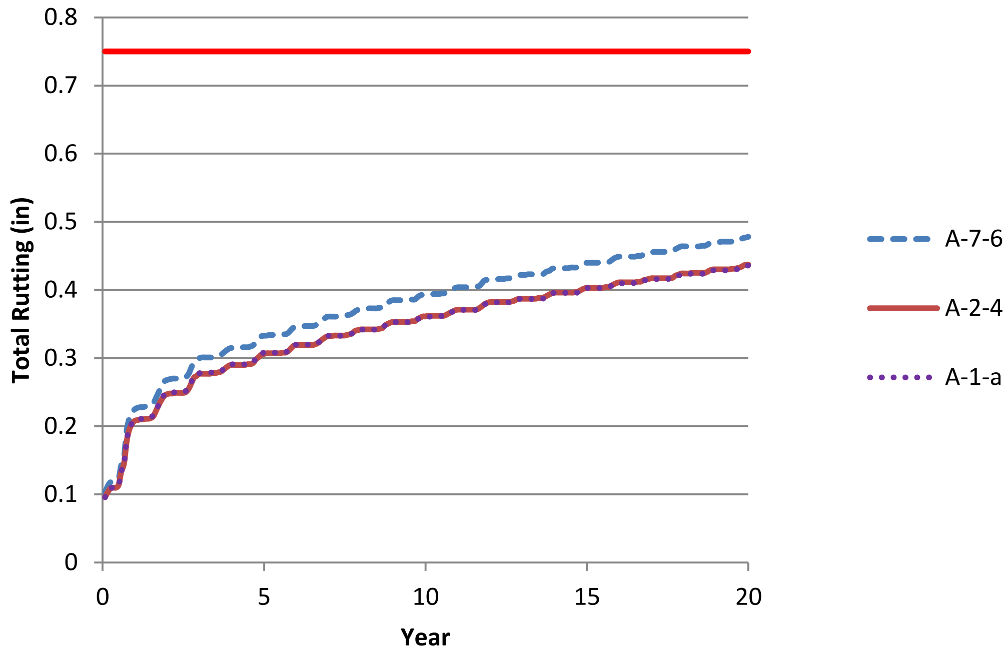

Figure 255 Effect of Subgrade Type on Total Rutting – Vermont Level 2

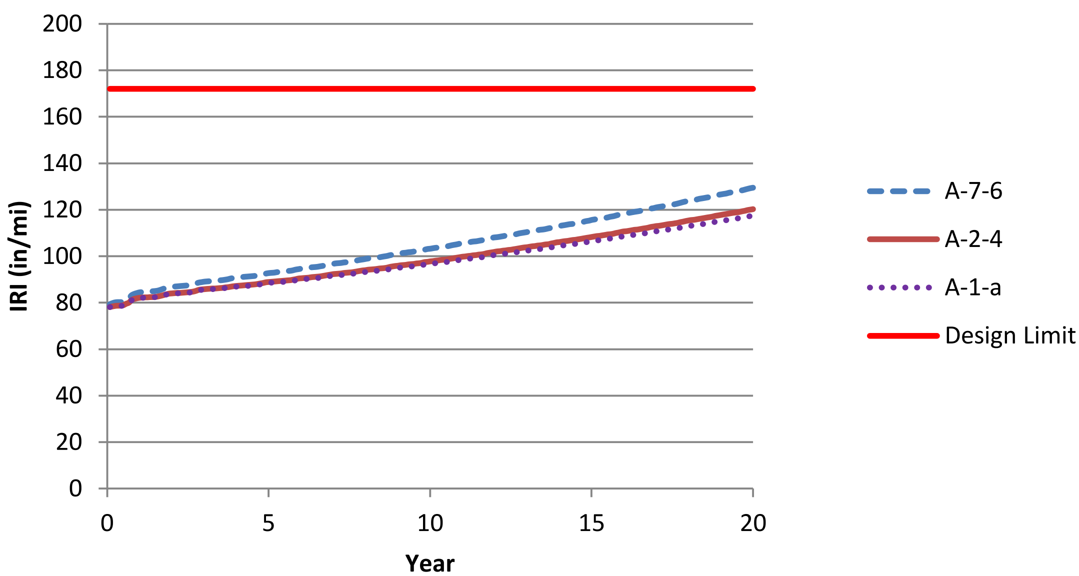

Figure 256 Effect of Subgrade Type on IRI – Vermont Level 3

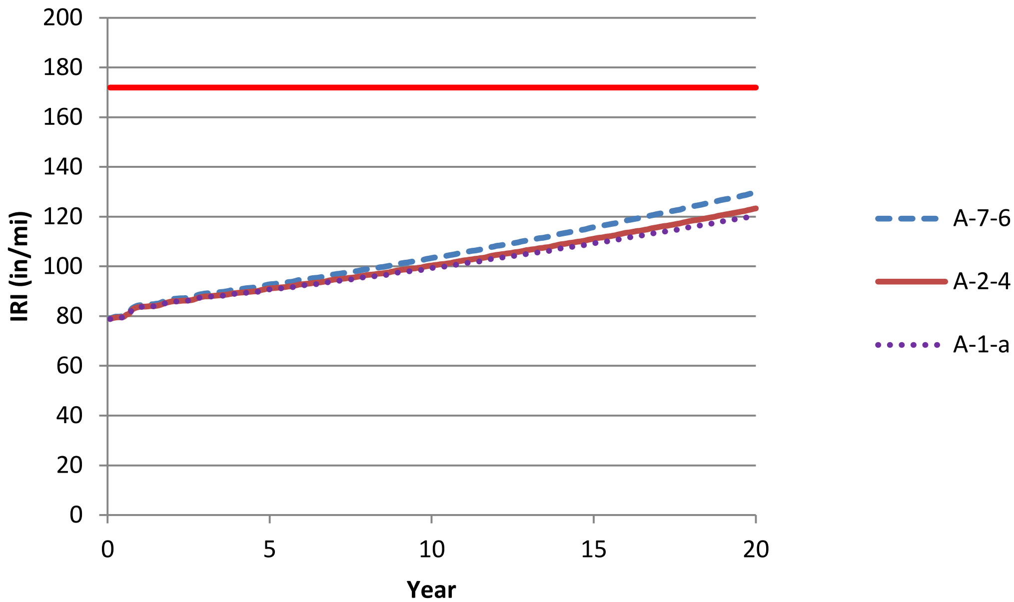

Figure 257 Effect of Subgrade Type on IRI – Vermont Level 2

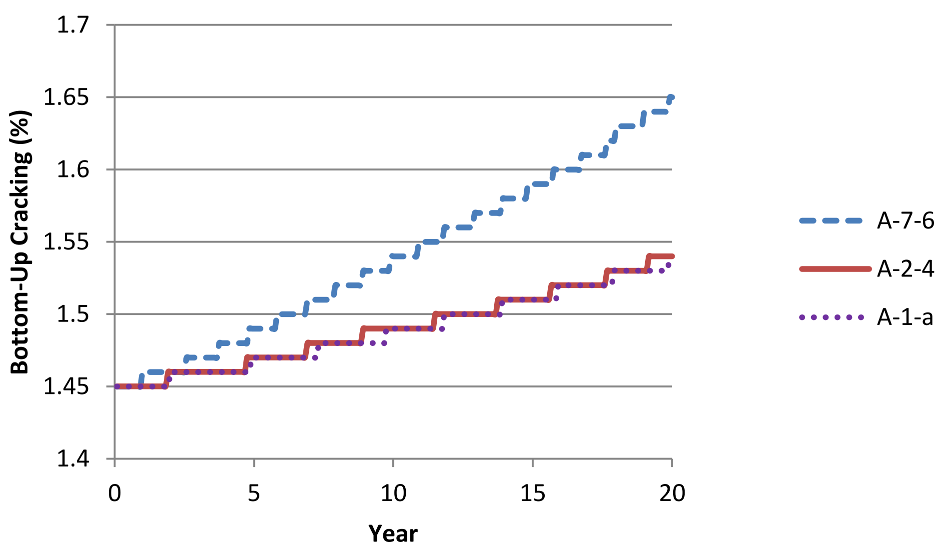

Figure 258 Effect of Subgrade Type on Bottom-Up Cracking – New York Level 3

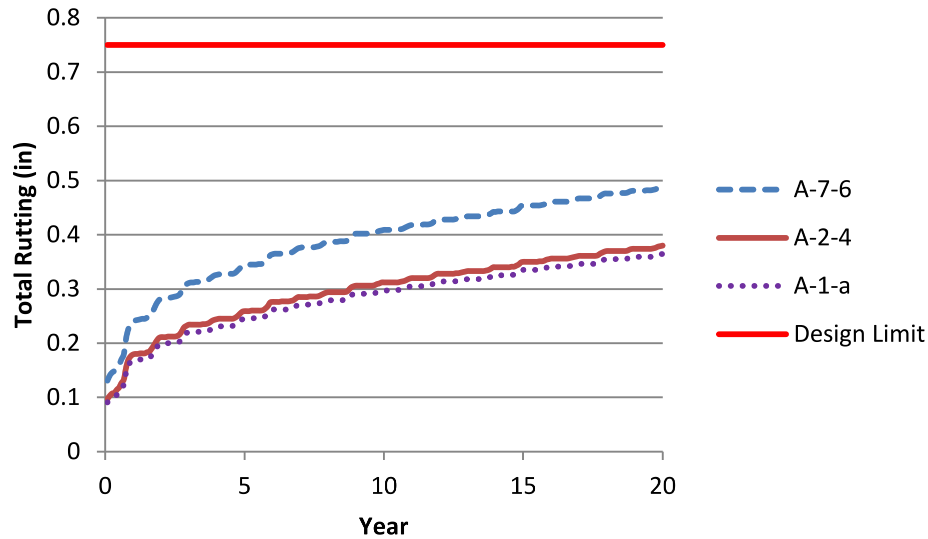

Figure 259 Effect of Subgrade Type on Total Rutting – New York Level 3

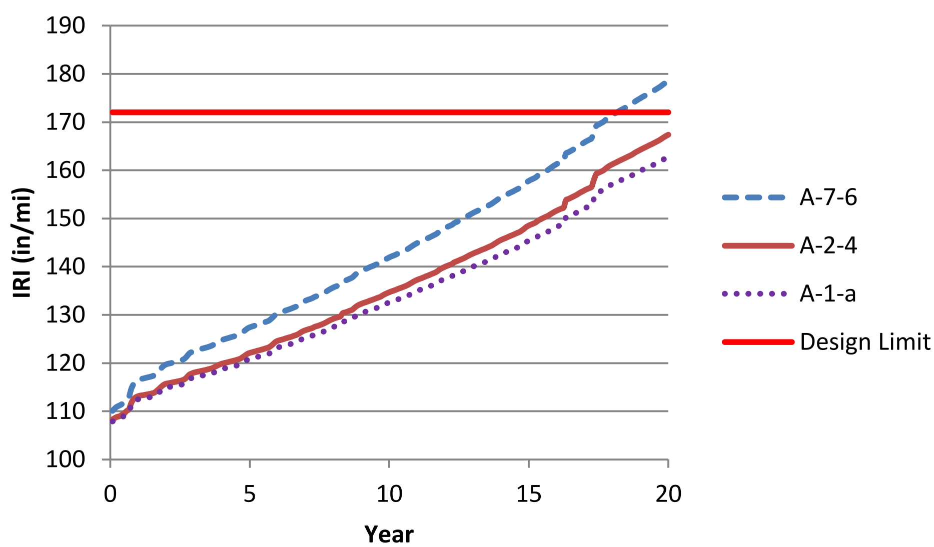

Figure 260 Effect of Subgrade Type on IRI – New York Level 3

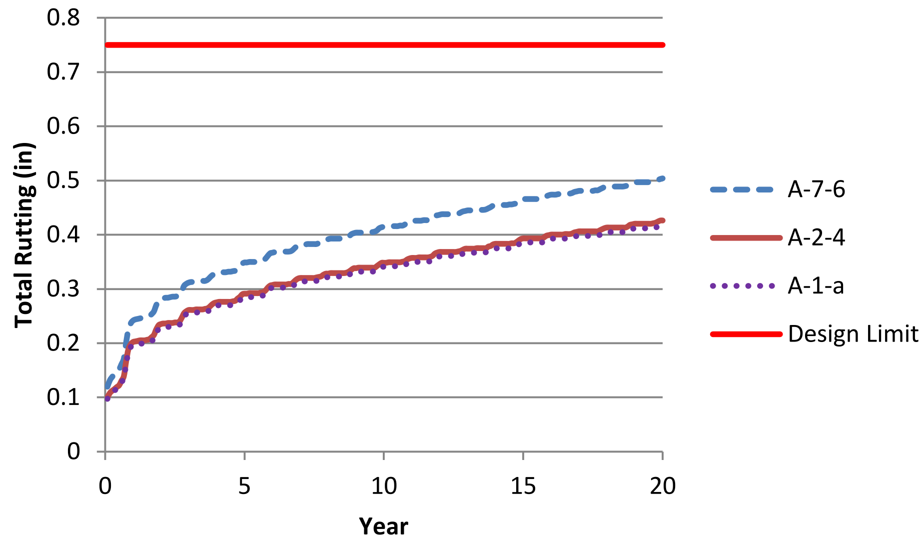

Figure 261 Effect of Subgrade Type on Total Rutting – Massachusetts Level 3

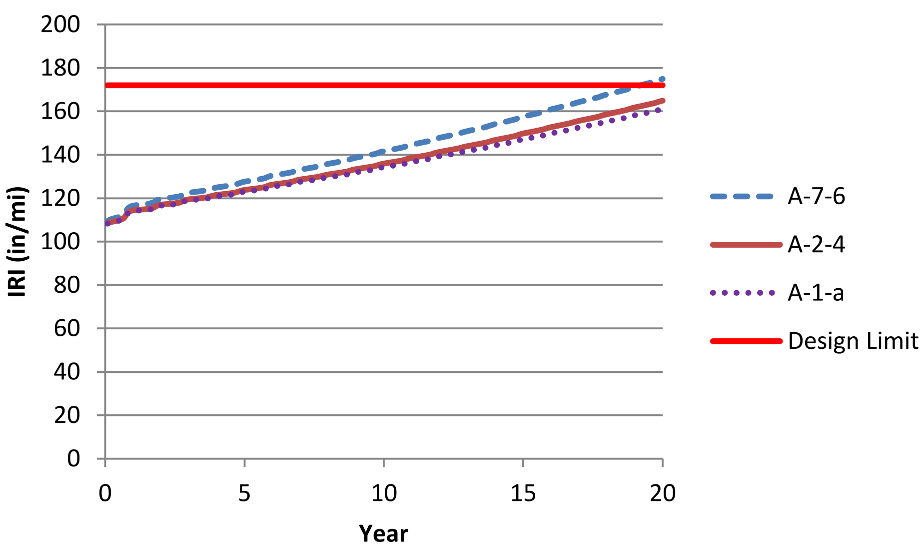

Figure 262 Effect of Subgrade Type on IRI – Massachusetts Level 3

The predicted rutting and fatigue cracking increase with a decrease in the resilient modulus of the subgrade. Therefore, for better prediction of total rutting and fatigue cracking, it is suggested that a database of typical soils found in each state be developed and the values used for design. The relative sensitivity of resilient modulus values provided in the design guide to laboratory measured values from research work is determined statistically using the predicted values at the end of design life.

Base Course Properties

Base course properties are not contained in the material specifications of the state highway agencies' documentation. Therefore, default values provided in the MEPDG were used for Level 3 of design, and for the state of New Hampshire the resilient moduli were obtained from a different source. Base course properties do not significantly affect the pavement distresses. Fatigue cracking alone was found to show sensitivity to change in base course material and strength, both at Level 2 and Level 3 of analysis.

Therefore, depending on the availability of natural resources, appropriate material should be chosen for base course in construction of asphalt pavements on unbound layer. Tests like CBR could be done to determine the material properties, which can also be entered as input for the MEPDG.

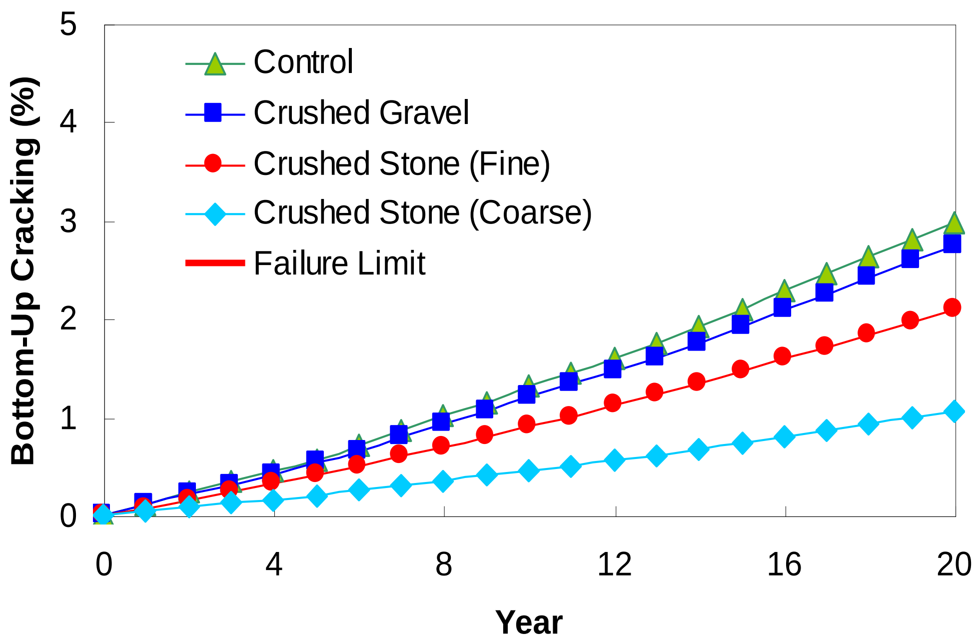

Figure 263 Effect of Base Course Material on Bottom-Up Cracking – New Hampshire

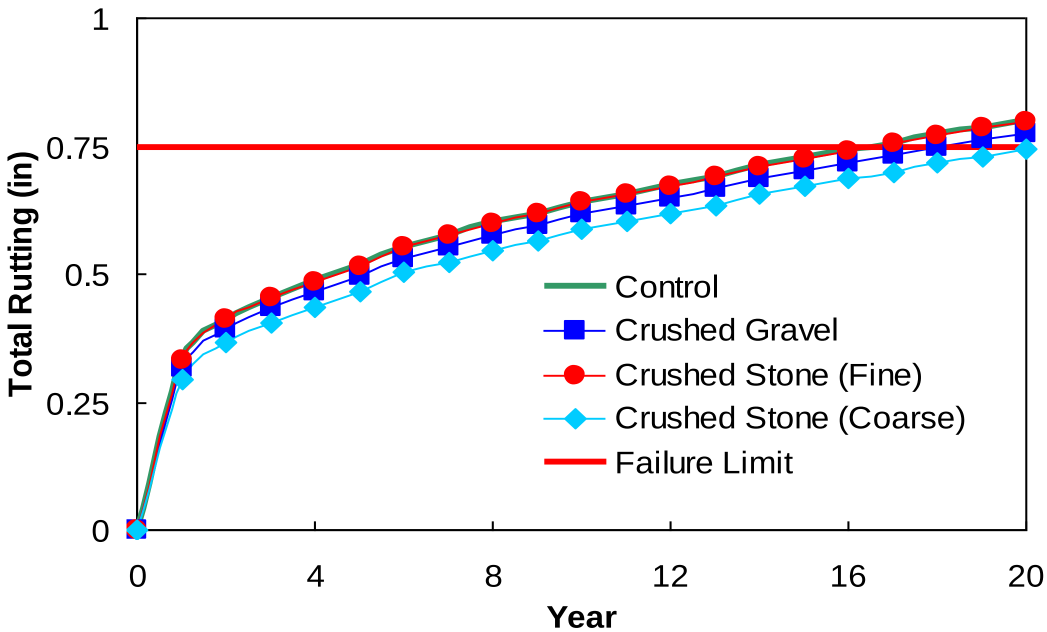

Figure 264 Effect of Base Course Material on Total Rutting – New Hampshire

In this research, the unbound material properties were only characterized by the material types and resilient modulus (measured in psi) values obtained from the state specifications or the LTPP database. It was found that base layer input variables based only on those two values have an insignificant effect on pavement distresses.

Base layer thickness variable was omitted in this study, but it can impact the MEPDG pavement distress predictions as well. Therefore, it is highly recommended to review this topic in the next project.

Figures 265 through 274 show some examples of base course material effect on the different pavement distresses and roughness in Vermont, New York and Massachusetts.

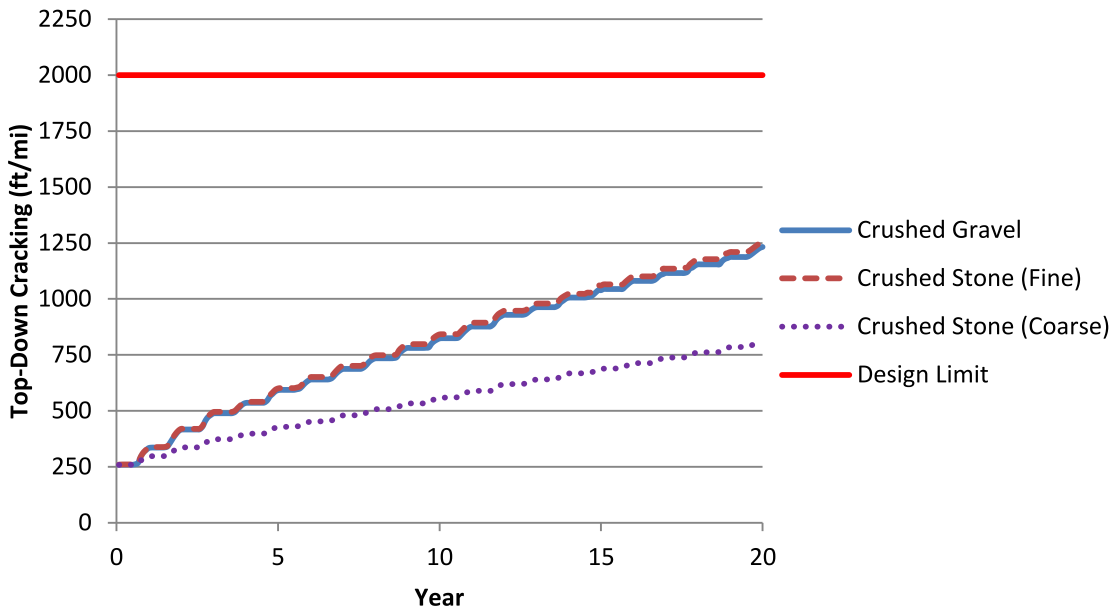

Figure 265 Effect of Base Course Material on Top-Down Cracking – Vermont Level 3

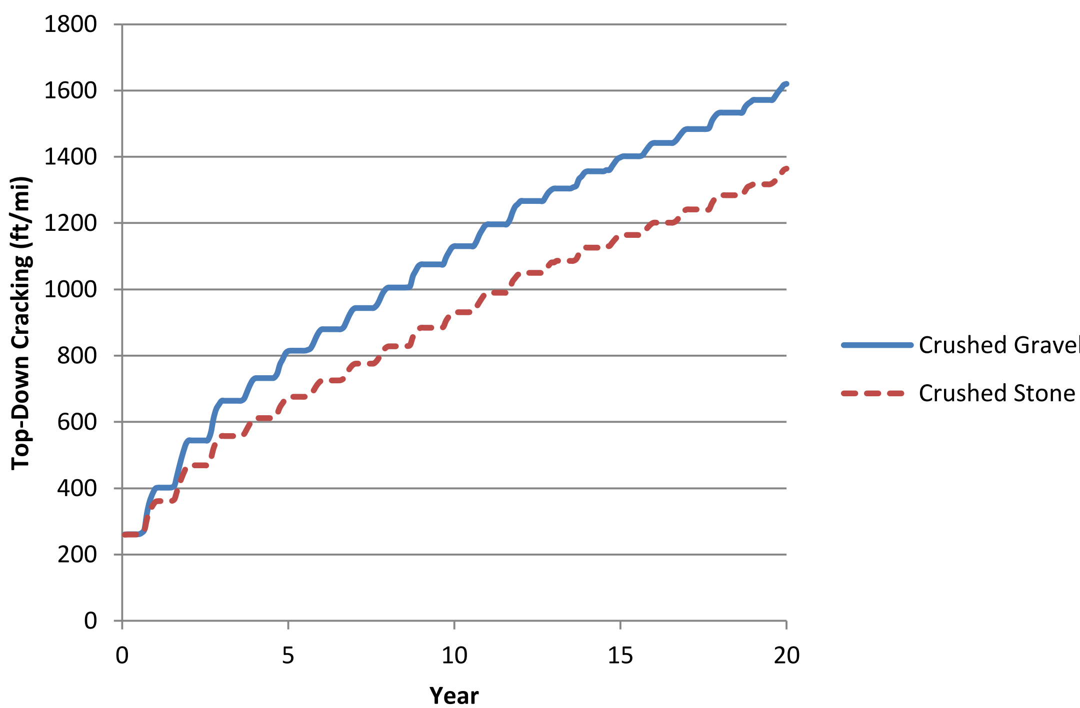

Figure 266 Effect of Base Course Material on Top-Down Cracking – Vermont Level 2

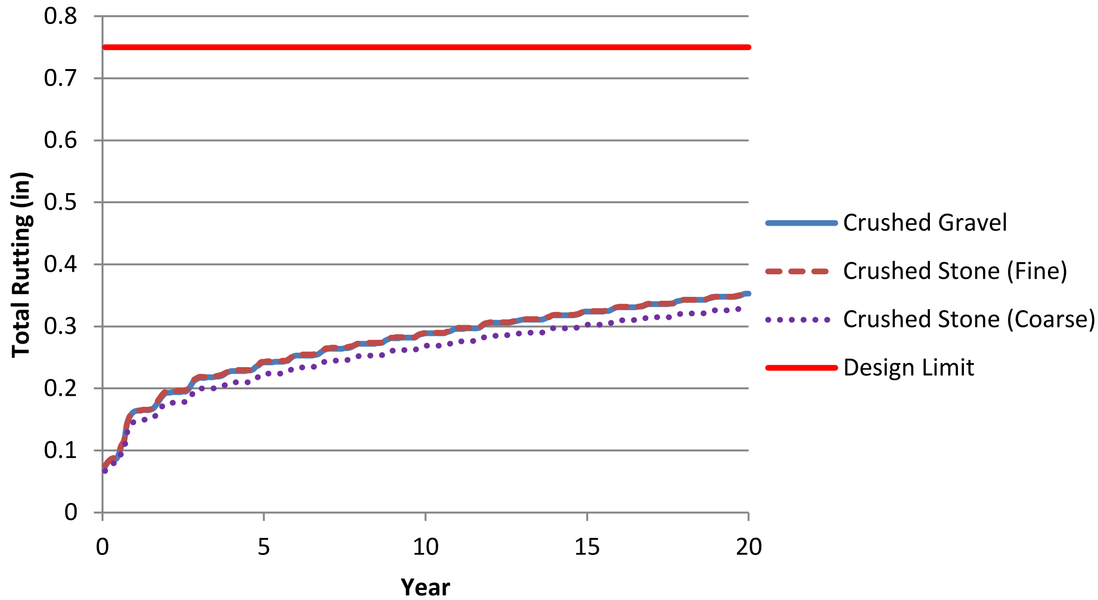

Figure 267 Effect of Base Course Material on Total Rutting – Vermont Level 3

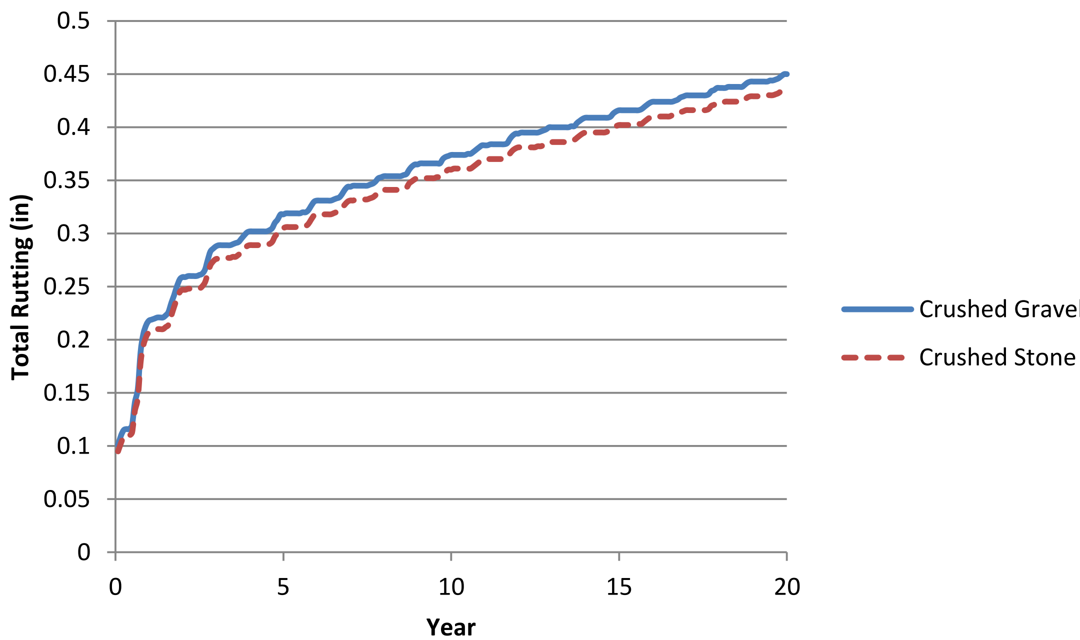

Figure 268 Effect of Base Course Material on Total Rutting – Vermont Level 2

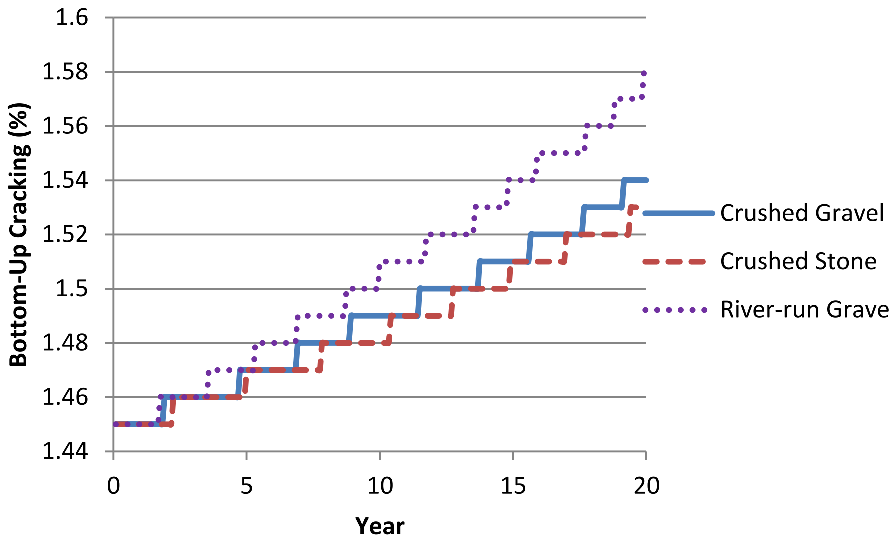

Figure 269 Effect of Base Course Material on Bottom-Up Cracking – NY Level 3

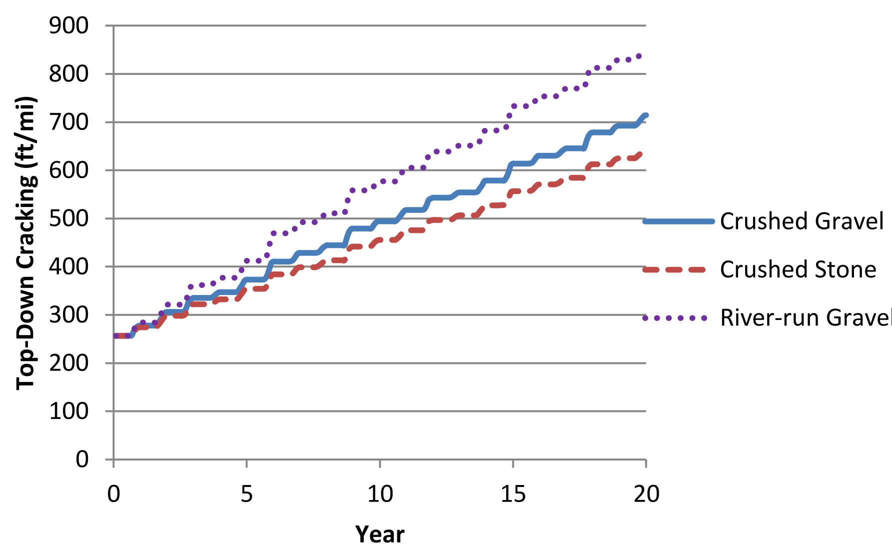

Figure 270 Effect of Base Course Material on Top-Down Cracking – NY Level 3

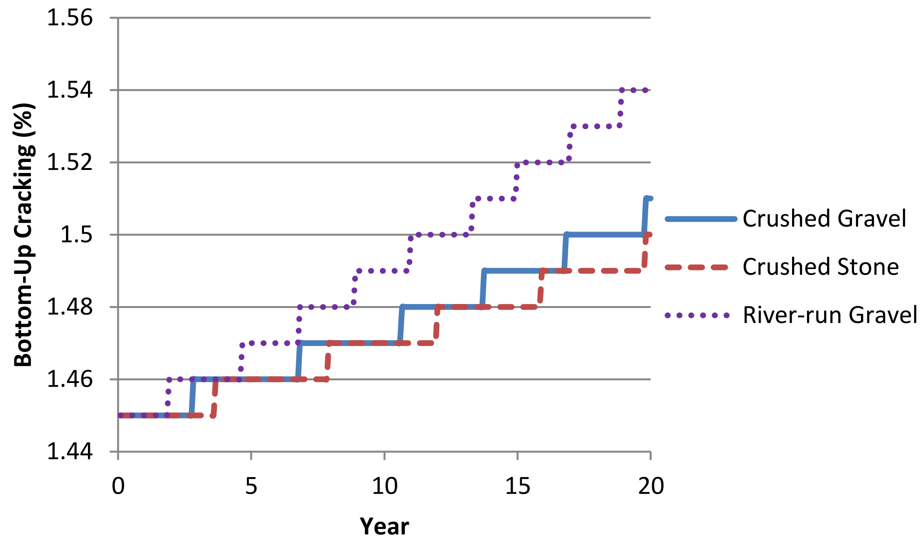

Figure 271 Effect of Base Course Material on Bottom-Up Cracking – MA Level 3

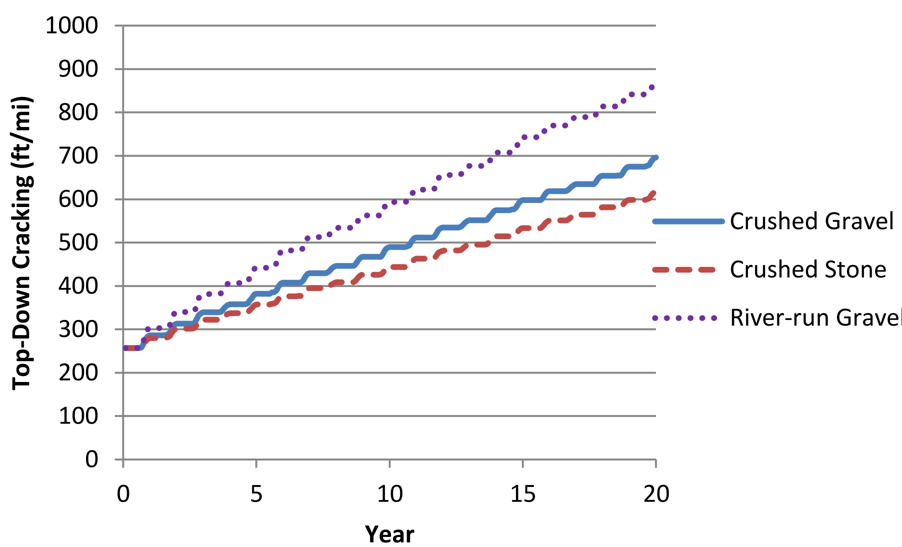

Figure 272 Effect of Base Course Material on Top-Down Cracking – MA Level 3

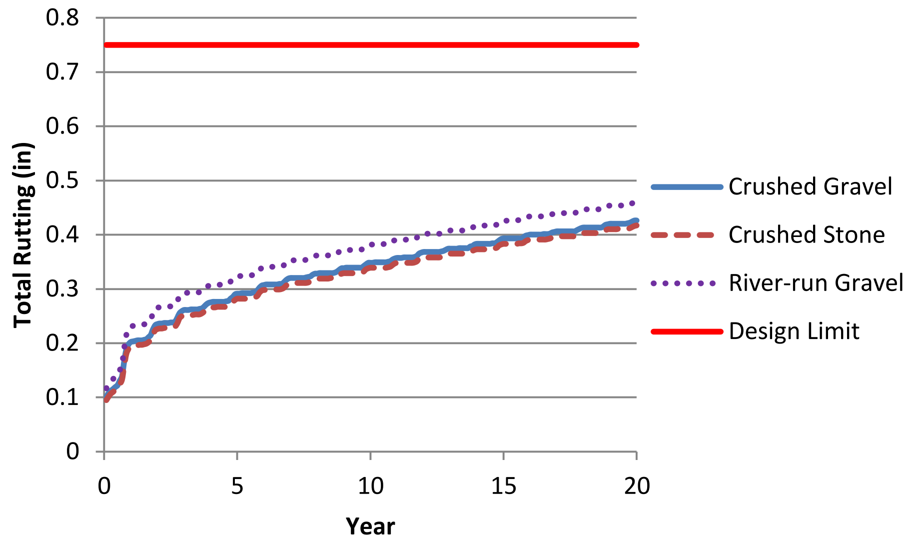

Figure 273 Effect of Base Course Material on Total Rutting – MA Level 3

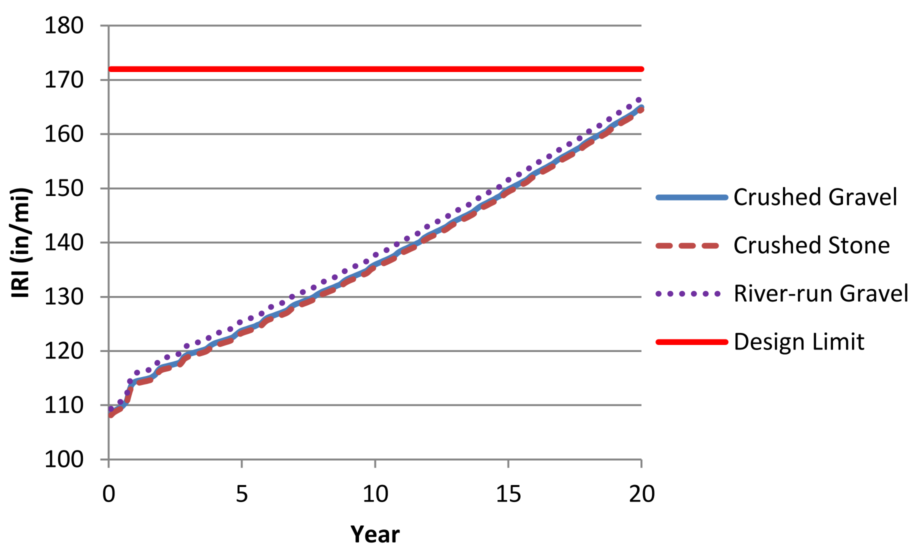

Figure 274 Effect of Base Course Material on IRI – MA Level 3

6.2 Analysis of Data – Graphical Method

6.2.1 Analysis of Data – Vermont Level 3 and 2

Figures 275 through 279 present results for level 3 sensitivity analysis in Vermont.

The "zero" value on the graph indicates, there is no impact of an input on a predicted pavement distress. Figures 275 through 278 show the initial IRI input which has no impact on the predicted pavement distresses such as bottom-up cracking, top-down cracking, AC rutting and total rutting.

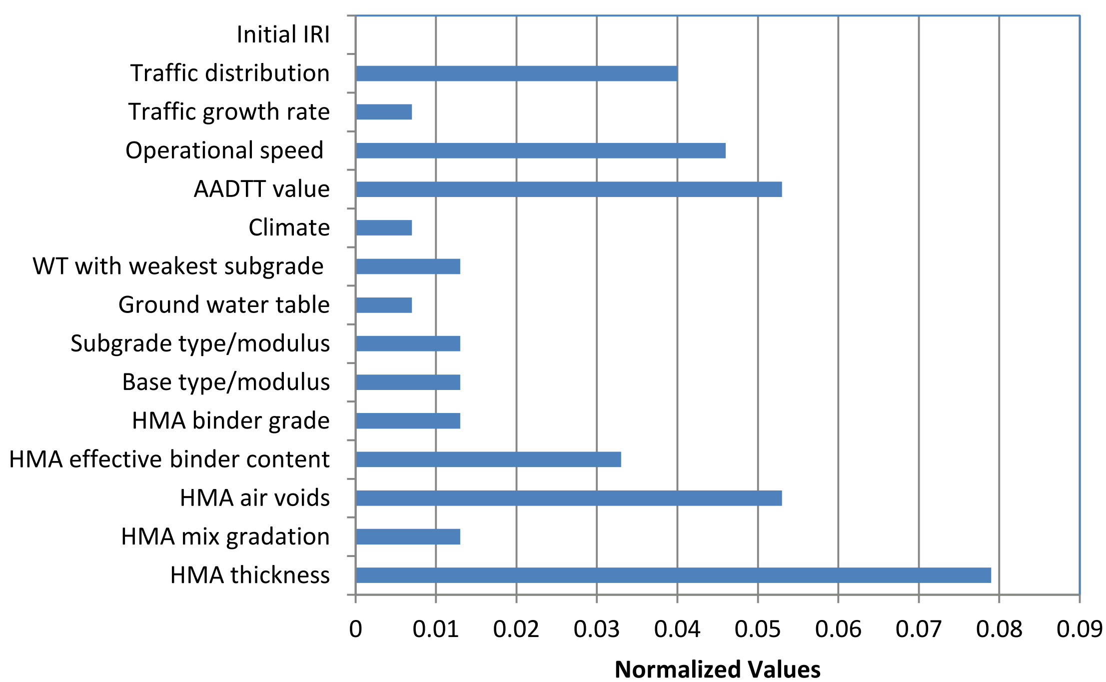

Figure 275 VT Level 3 Significance of Effect of Input Variables on Bottom-Up Cracking

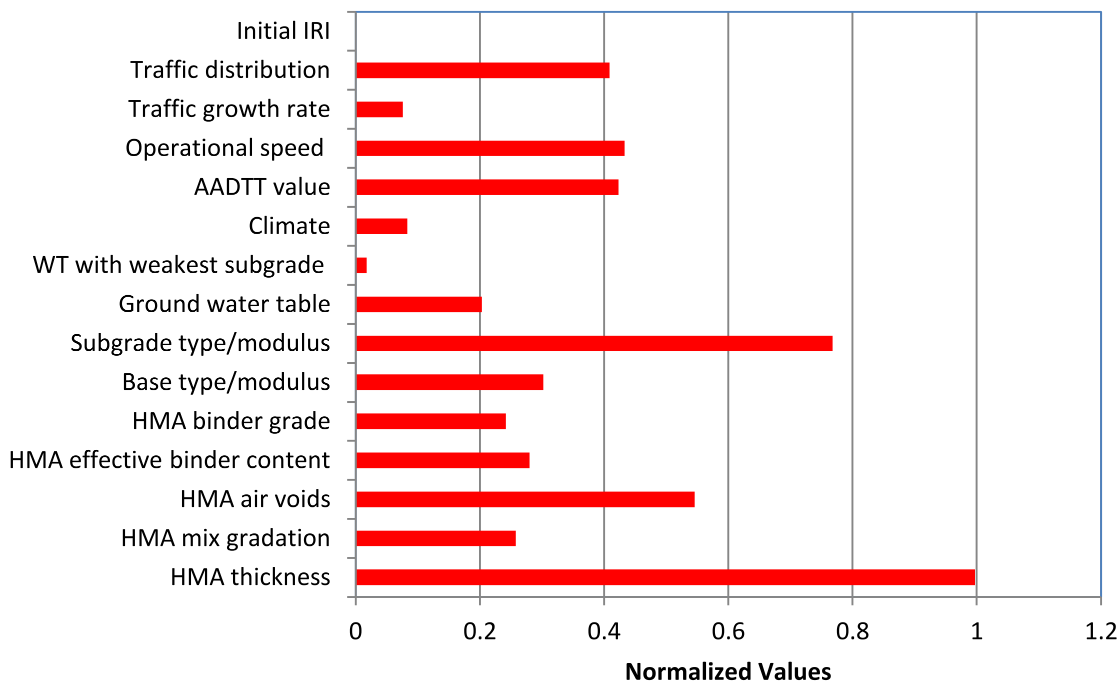

Figure 276 VT Level 3 Significance of Effect of Input Variables on Top-Down Cracking

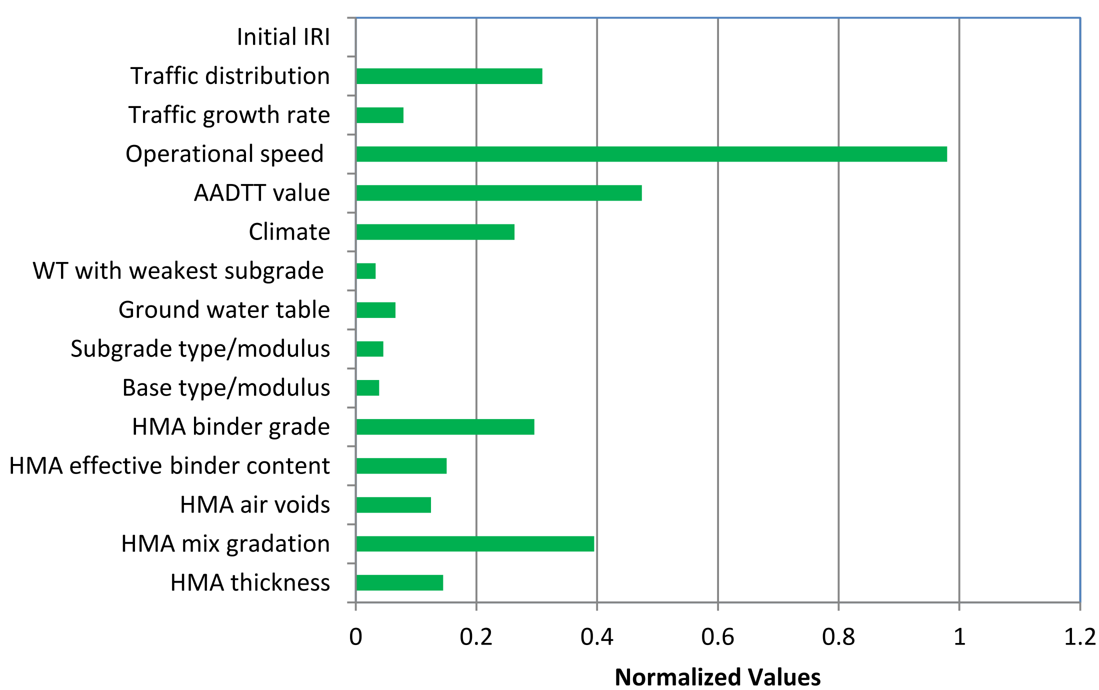

Figure 277 VT Level 3 Significance of Effect of Input Variables on AC Rutting

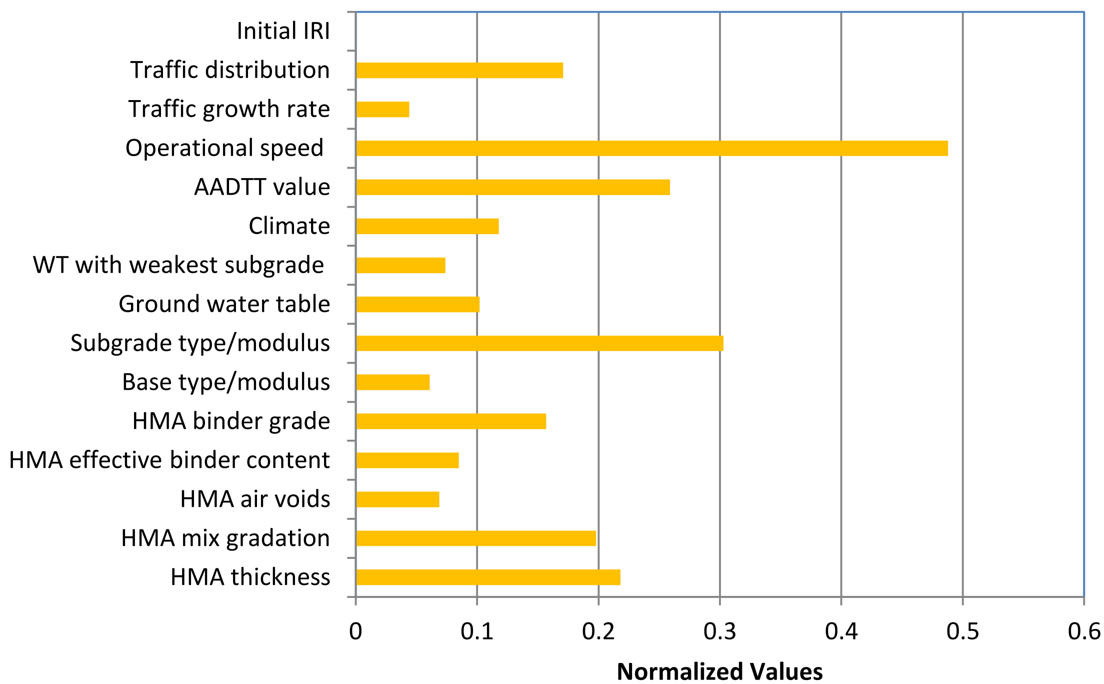

Figure 278 VT Level 3 Significance of Effect of Input Variables on Total Rutting

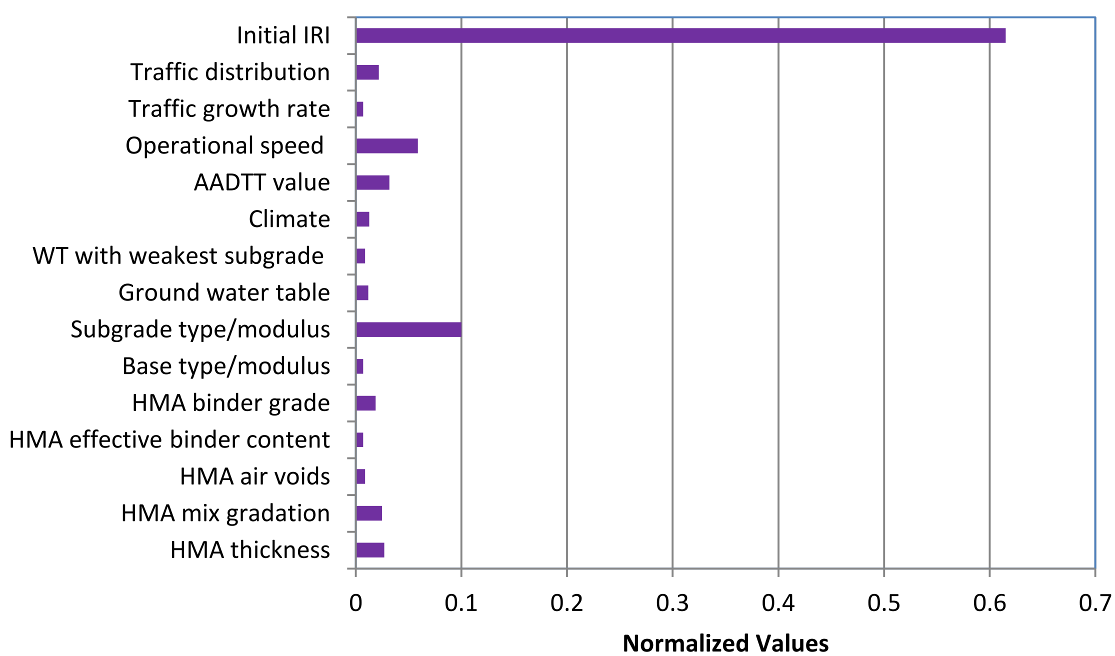

Figure 279 VT Level 3 Significance of Effect of Input Variables on IRI.

HMA thickness had a significant effect on both fatigue cracking distresses (bottom-up and top-down). Both of these distresses increased with the decrease of HMA thickness layer. Longitudinal (top-down) cracking was greatly affected, when the HMA layer thickness was reduced to 7.0". In this example, the failure in pavement compared to the design limit, which occurred after 18 years of service life (VT Report, Figure 51A). The trends observed were reasonable for total rutting and IRI, with the highest distress/IRI for the thinner HMA (VT Report, Figures 53A and 54A).

Traffic composition (i.e., operational speed, AADTT, and vehicle class distribution) are expected to influence the extent of pavement condition deterioration. Based on the literature review, pavement deterioration is significantly increased as the traffic composition is dominated by heavier trucks and axle loads. In Vermont, the AADTT value for the selected LTPP road section has a moderate rate of 10.35%. With the operational speed of 55 mph and the LTPP track distribution, the traffic composition impact was greatest on AC rutting and total rutting (Figures 277 and 278), and a moderate effect on fatigue (bottom-up) alligator cracking (Figure 275). Operational speed had a significant effect on both rutting pavement distresses, with the highest distresses for the lower speed value (VT Report, Figures 70A to 84A). In the overall order of significance ranking the high position of the operational speed was surprising. This research did not investigate how realistic ranking of vehicle speed is as a variable for pavement performance predictions. It is up to the state agency to decide if the change of vehicle speed and its range could really affect the pavement performance.

The effect of subgrade type on pavement performance was determined by comparing distress and IRI over time with subgrade types (Appendix A – AASHTO Classification). Three soil types were chosen (A-1-a, A-2-4, and A-7-6) along with typical default inputs recommended for use in the MEPDG and shown in VT Report, Table 32A. Figures 90A, 93A, and 94A (VT Report) present the effect of subgrade soil type on predicted distresses and roughness. In general, the lower the subgrade type/modulus the higher alligator fatigue cracking, rutting and IRI.

Changes in HMA parameters such as air voids or effective binder content were expected to have an effect on pavement distresses. Based on this research, an increase of air void content in the HMA layer results in a large increase in fatigue alligator and longitudinal cracking (VT Report, Figures 45A and 46A). There were no observed effects on the remaining pavement distresses and IRI with changes in air voids (VT Report, Figures 47A through 49A). The moderate effect of change in the effective binder content was only observed for fatigue alligator (bottom-up) cracking and longitudinal (top-down) cracking (VT Report, Figure 40A through 44A). In general, the increase of binder content reduces alligator and longitudinal cracking and increases rutting (AC and total). There is no impact of change in the effective binder content to the pavement roughness IRI.

The effect of climate on the predicted distress and IRI was determined by selecting three representative weather stations for Vermont and three ground water table depths (2 ft , 5 ft, and 8 ft), and using the representative data to simulate climate condition across the state (VT Report, Figures 30A through 39A). Table 127 presents the moderate effect of climate change only for AC rutting.

Table 127 Ranking of Input Variable Significance for VT Level 3 Sensitivity Analysis

	Bottom-Up Cracking	Top-Down Cracking	AC Rutting	Total Rutting	IRI
Most Significant Variable	HMA Thickness	HMA Thickness	Operational Speed	Operational Speed	Initial IRI
↓	HMA Air Voids	Subgrade Type/ Modulus	AADTT	Subgrade Type/ Modulus	Subgrade Type/ Modulus
	AADTT	HMA Air Voids	HMA Mix Gradation	AADTT	Operational Speed
	Operational Speed	Operational Speed	Traffic Distribution	HMA Thickness	HMA Thickness
	Traffic Distribution	AADTT	HMA Binder Grade	HMA Mix Gradation	AADTT
Least Significant Variable	HMA Effective Binder Content	Traffic Distribution	Climate	Traffic Distribution	HMA Mix Gradation

In general, higher pavement distresses were observed in the southern part of the state due to warmer temperatures (VT Report, Figures 30A through 33A). The effect of ground water table level change was insignificant for all of the predicted pavement distresses. The ground water table effect is not reasonable to the current pavement design knowledge, and it needs to be reevaluated with the new MEPDG version.

The moderate effect of HMA mix grading was observed mostly for AC rutting and total rutting (Table 127). In general, the coarse aggregates used for the production of HMA pavements, exhibited a higher level of all pavement distresses and IRI (VT Report, Figures 65A through 69A).

The effect of a binder grade selection was observed on AC rutting pavement distress. The binder grade selection is presented in VT Report Table 26A, and the effects on the predicted pavement performance in Figures 70A through 84A (VT Report). It was observed, that the lower HMA binder grades (PG 58) exhibited a higher level of all distresses and IRI, when compared to the higher binder grades (PG 64).

Figures 280 through 284 present results for level 2 sensitivity analysis in Vermont.

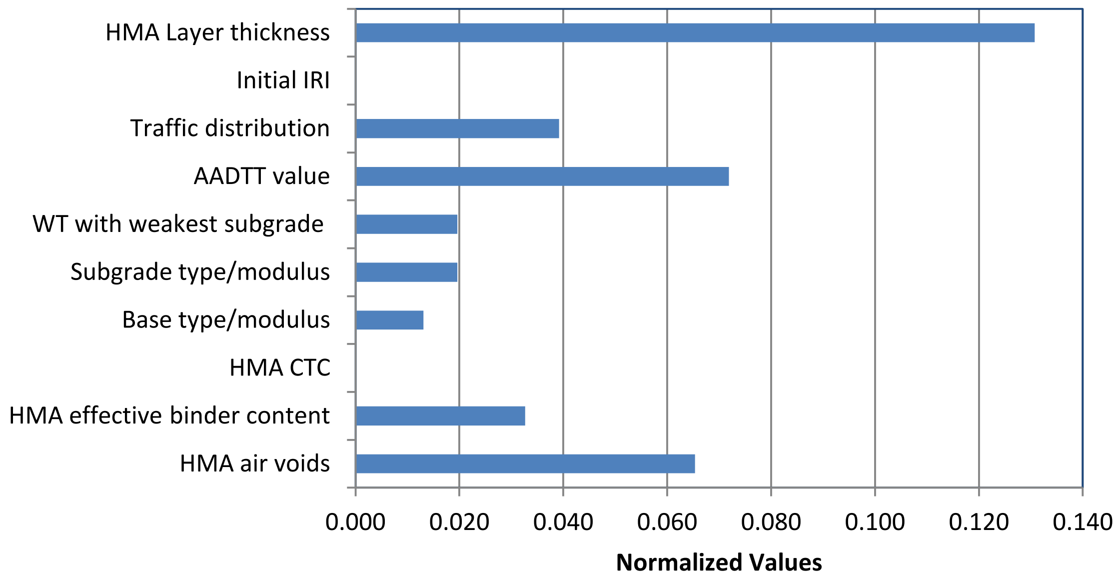

Figure 280 VT Level 2 Significance of Effect of Input Variables on Bottom-Up Cracking.

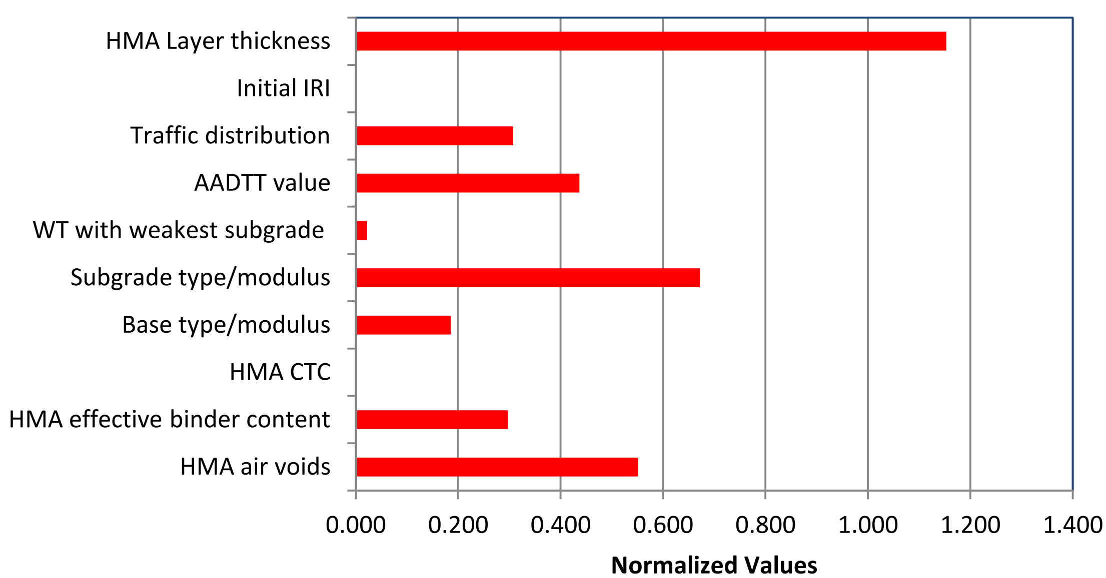

Figure 281 VT Level 2 Significance of Effect of Input Variables on Top-Down Cracking.

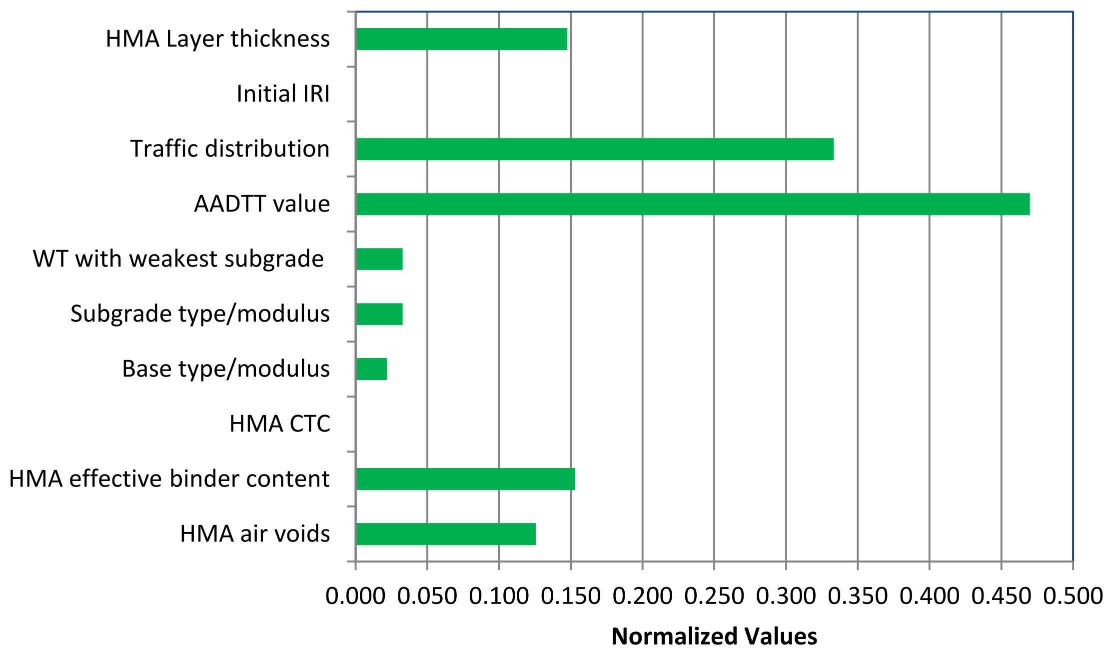

Figure 282 VT Level 2 Significance of Effect of Input Variables on AC Rutting.

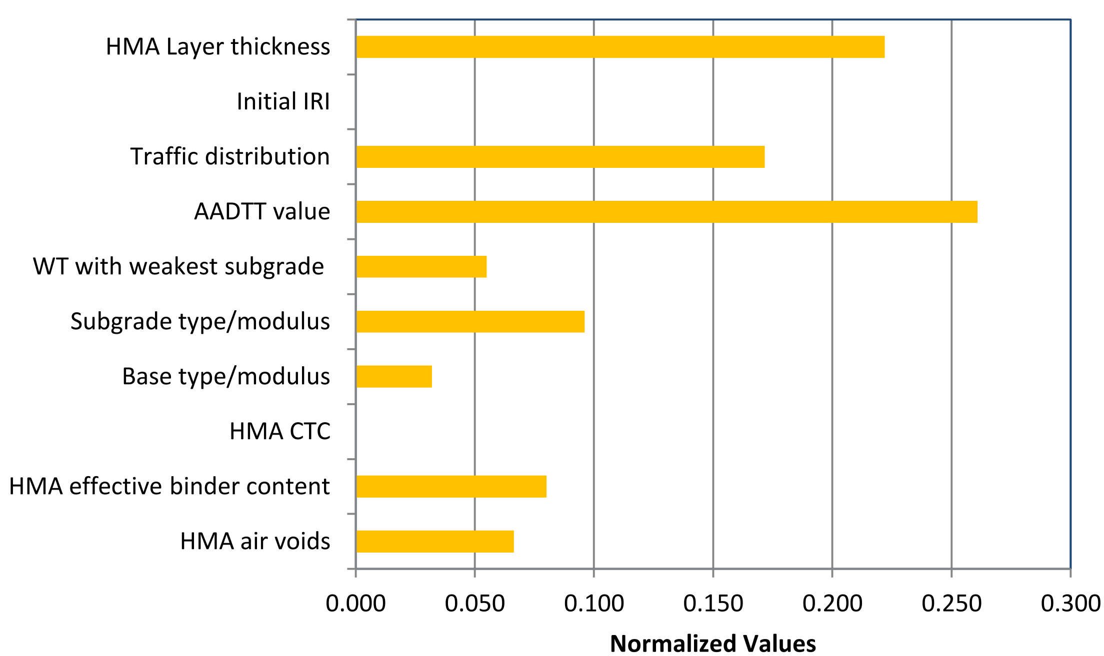

Figure 283 VT Level 2 Significance of Effect of Input Variables on Total Rutting.

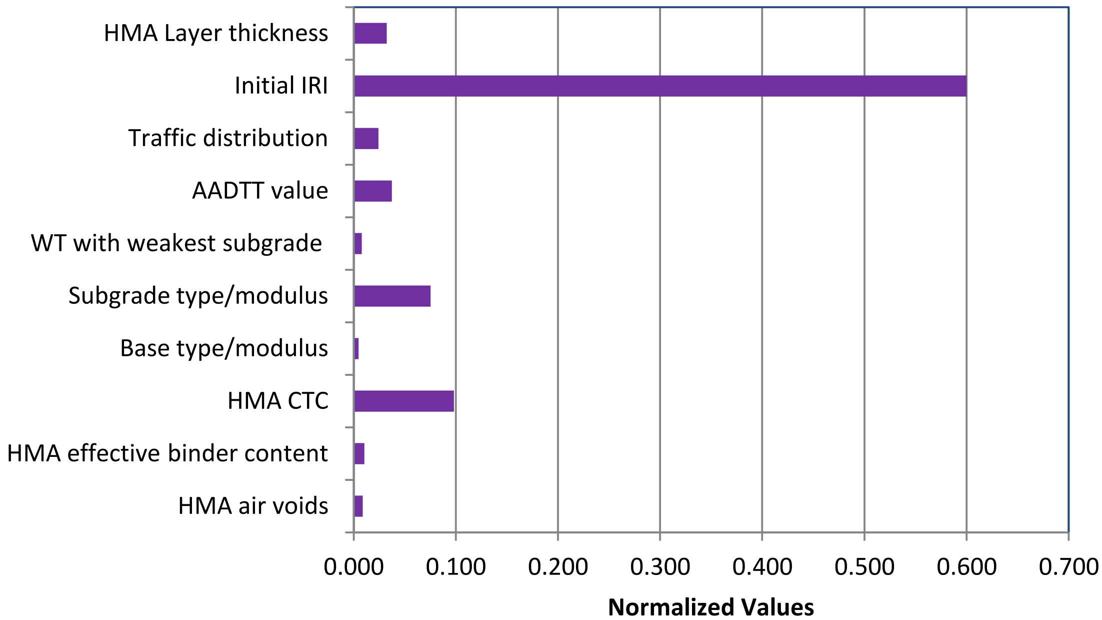

Figure 284 VT Level 2 Significance of Effect of Input Variables on IRI.

The “zero” value on the graph indicates, there is no impact of an input on a predicted pavement distress. As an example, Figures 280 through 283 present the initial IRI and the HMA CTC inputs which have no impact on the predicted pavement distresses such as: bottom-up cracking, top-down cracking, AC rutting and total rutting.

The predicted distresses and trends were observed to be similar with Level 3 sensitivity analysis, with slightly higher values predicted for Level 2 (Figures 275 through 284).

The effect of a new variable (mix coefficient of thermal contraction CTC) in this level of sensitivity analysis was insignificant for all of pavement distresses (zero value in Figures 280 through 283), and had only small effect on the roughness IRI prediction (Figure 284).

6.2.2 Analysis of Data - New York Level 3

Figures 285 through 289 present results for level 3 sensitivity analysis in New York.

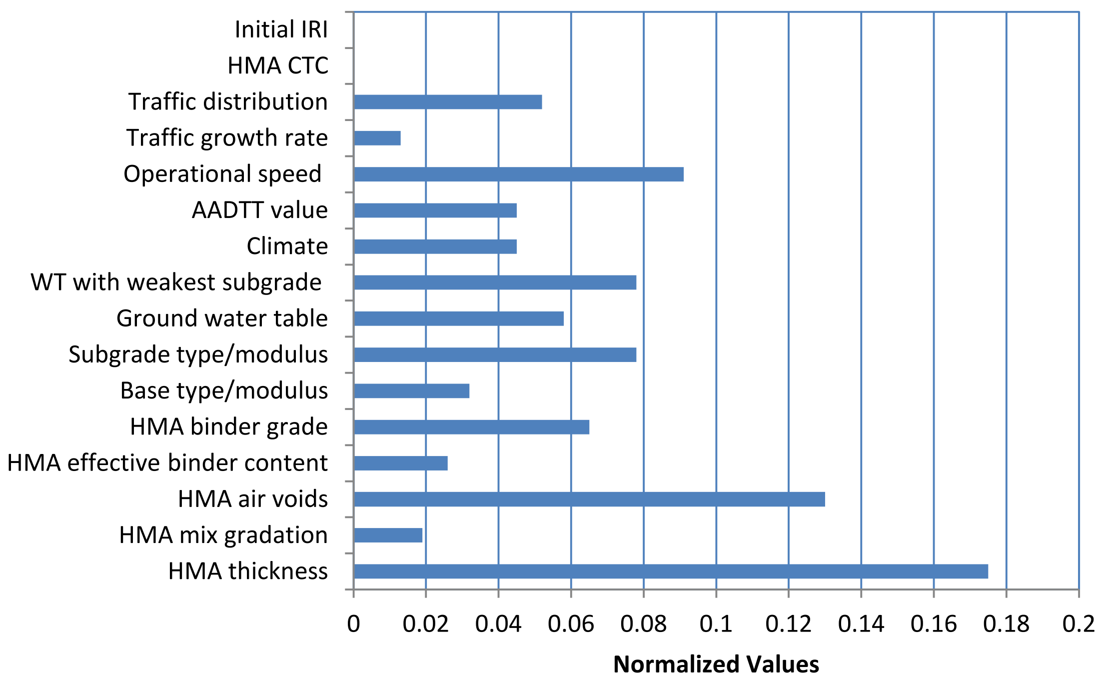

Figure 285 NY Level 3 Significance of Effect of Input Variables on Bottom-Up Cracking

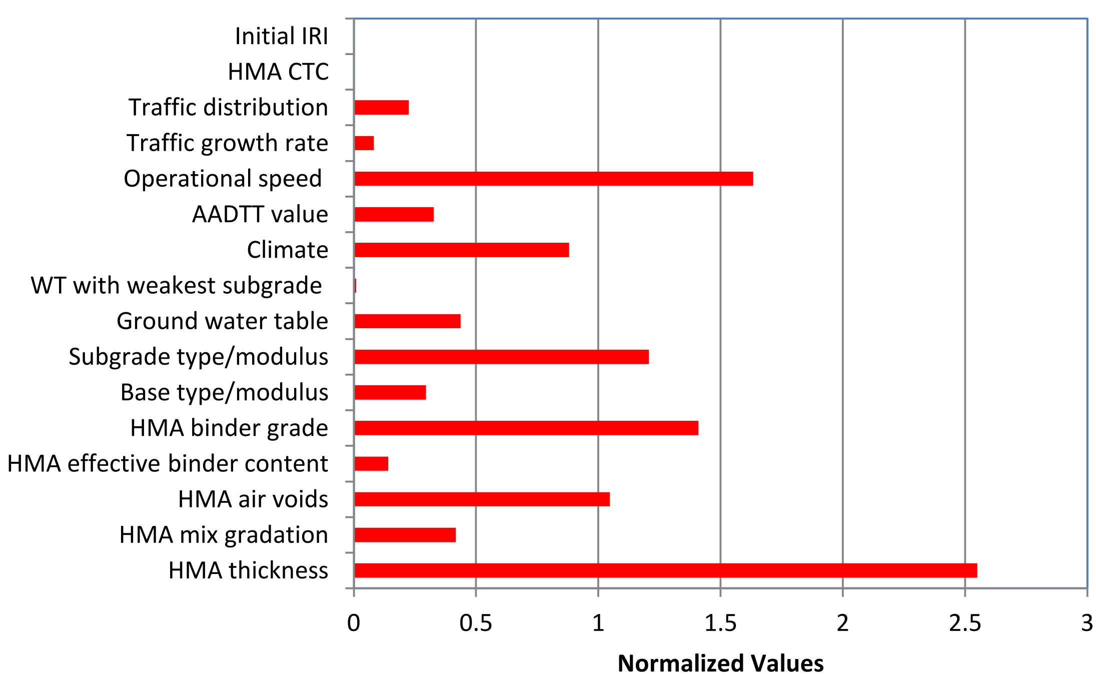

Figure 286 NY Level 3 Significance of Effect of Input Variables on Top-Down Cracking

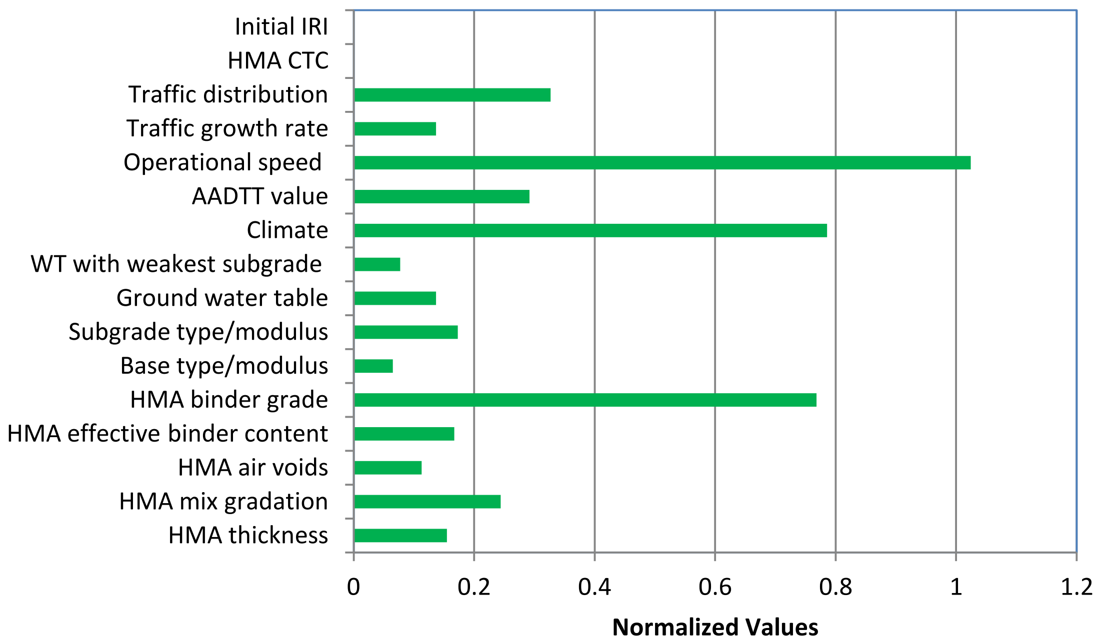

Figure 287 NY Level 3 Significance of Effect of Input Variables on AC Rutting

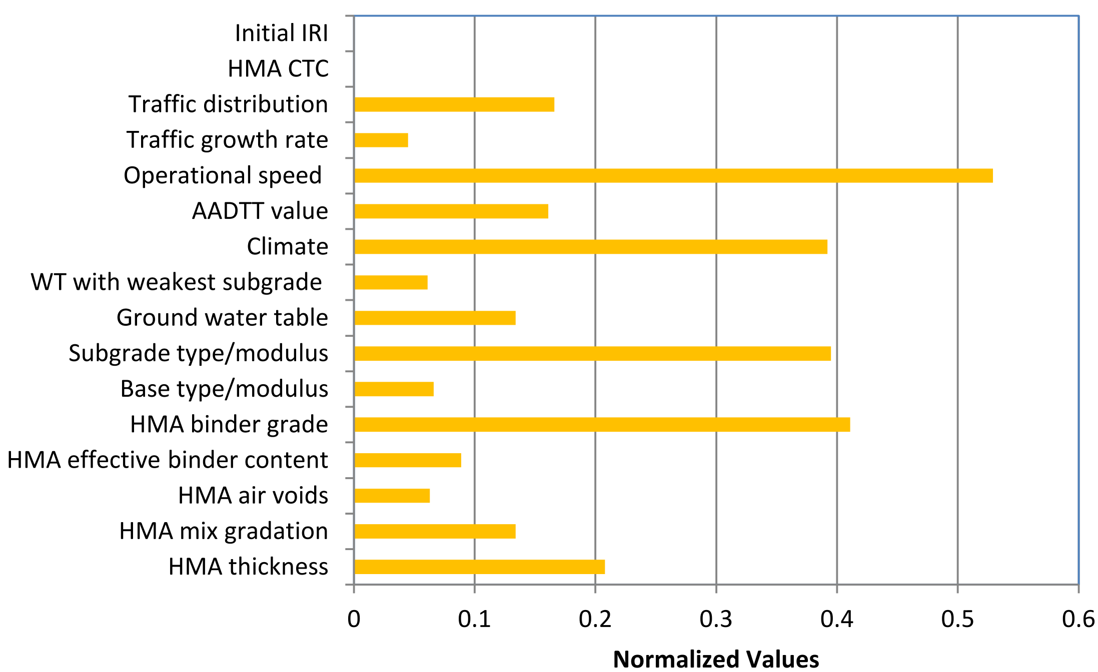

Figure 288 NY Level 3 Significance of Effect of Input Variables on Total Rutting

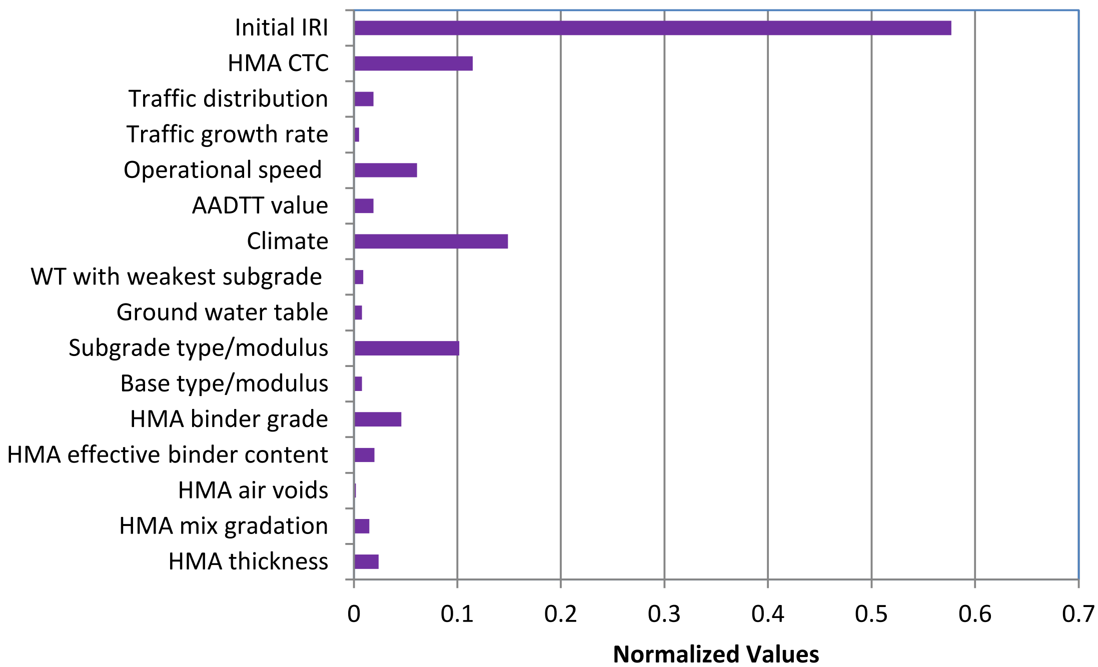

Figure 289 NY Level 3 Significance of Effect of Input Variables on IRI

The "zero" value on the graph indicates, that there is no impact of an input on a predicted pavement distress. As an example, Figures 285 through 288 present the initial IRI and the HMA CTC inputs which have no impact on the predicted pavement distresses such as: bottom-up cracking, top-down cracking, AC rutting and total rutting.

In New York, HMA thickness had a significant effect on bottom-up and top down fatigue cracking distresses. Both of these increased with the decrease of HMA thickness (NY Report, Figures 57B and 58B). The most significant effect of fatigue top-down cracking was especially visible when the HMA layer thickness was reduced to 8.0" (NY Report, Figure 58B). In general, all pavement distresses and roughness IRI were increased with the decrease of the total HMA thickness (NY Report, Figures 57B through 62B).

Traffic variables such as operational speed, AADTT, and vehicle class distribution had an expected influence on the predicted pavement distresses and roughness IRI (Figures 285 through 289). Operational speed was the most significant variable with a large impact on AC rutting and total rutting (NY Report, Figures 26B through 30B). In general, for all pavement distresses and roughness IRI, values increased with the decrease of the operational speed. In the overall order of significance ranking the high position of the operational speed was surprising. This research did not investigate how realistic ranking of vehicle speed is as a variable for pavement performance predictions. It is up to the

state agency to decide if the change of vehicle speed and its range could really affect the pavement performance.
For the AADTT and the vehicle class distribution (axle loads) as was expected, with the increase of these two variables the predicted pavement distresses and IRI increased as well. This study had confirmed this prediction as well (NY Report, Figures 31B – 35B, and Figures 16B – 20B).

The effect of binder grade selection was observed in New York State for all types of predicted pavement distresses and roughness IRI. The selected binder grades were analyzed in conjunction with three different operational speeds. The selected binder grades are listed in Table 22B (NY Report). The significant effect of a selected binder grade was observed on fatigue top-down cracking, and both rutting distresses (AC and total). The small effect was visible on the fatigue (bottom-up) cracking distress and roughness IRI. In both examples, the lower selected pavement grade exhibited a higher distress level and a higher roughness IRI value (NY Report, Figures 77B through 91B).

The New York climate had a significant effect on fatigue top-down cracking and AC rutting, and moderate effects on total rutting and roughness IRI. The influence of climate in NY is very important due to the size of the state, geographic characteristics and local temperature variations. In general, higher predicted pavement distresses in southern state locations were observed (NY Report, Figures 36B through 39B). The opposite effects of binder grades on roughness and thermal cracking were observed in Figures 40B and 41B (NY Report). In those two examples, the state's northern location exhibited a higher thermal cracking distress and a higher roughness IRI value.

Changes in HMA parameters such as air voids (%) or effective binder content (%) were expected to have an influence on pavement distresses. This expectation was only confirmed for the air voids content and its influence on fatigue bottom-up and top-down cracking. Increased HMA air voids content caused a large increase of fatigue alligator and longitudinal cracking distresses (NY Report, Figures 52B and 53B). The effective binder content variations within the state tolerances did not influence any of the predicted pavement distresses or roughness IRI.

The effect of subgrade type (Appendix A - AASHTO Classification) on performance was determined by comparing distress and IRI prediction over time with selected subgrade types (NY Report, Figures 97B to 101B). Figure 98B and 99B (NY Report) showed unexpected results for the weaker subgrade type (A-7-6), where there was no influence on fatigue (top-down) cracking, and an opposite than expected effect on subtotal rutting. In general, the lower the subgrade type/modulus, there could be expected higher pavement distresses and IRI.

The effect of the mix coefficient of thermal contraction (CTC) in this level of sensitivity analysis was insignificant for all of pavement distresses (zero value in Figures 285 through 288), and the mix CTC had only moderate effect on the roughness IRI prediction

(Figure 289). The increase of the mix CTC value affected the increase in roughness IRI (NY Report, Figure 111B).

6.2.3 Analysis of Data - Massachusetts Level 3

Figures 290 through 294 present results for level 3 sensitivity analysis in Massachusetts.

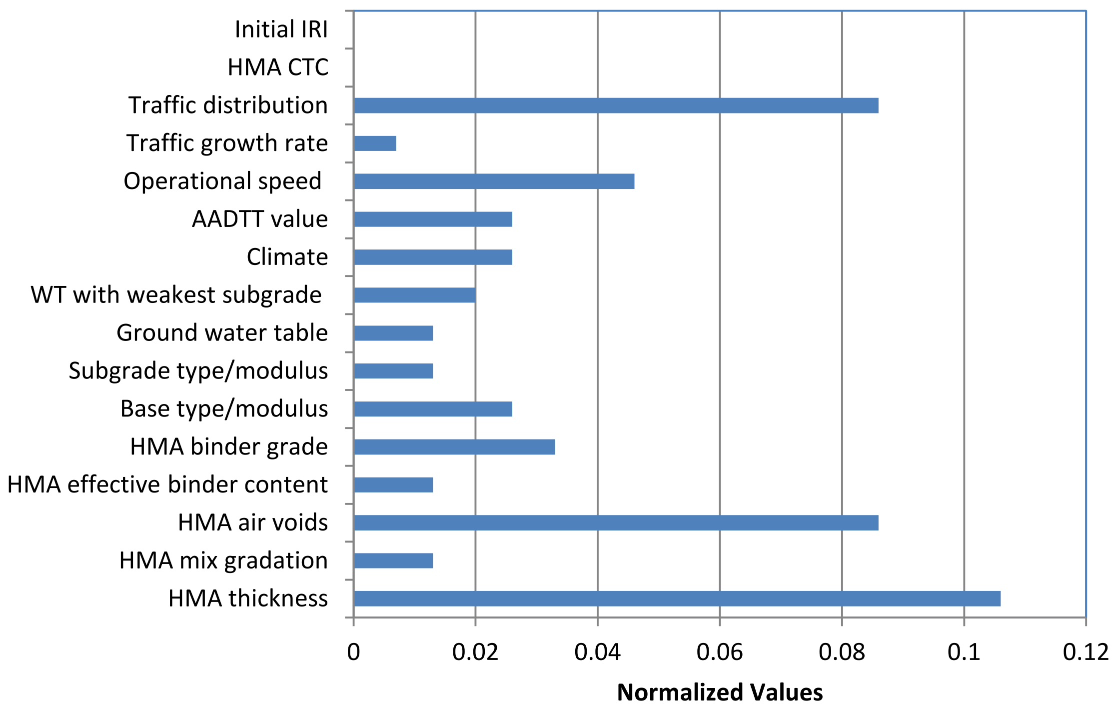

Figure 290 MA Level 3 Significance of Effect of Input Variables on Bottom-Up Cracking.

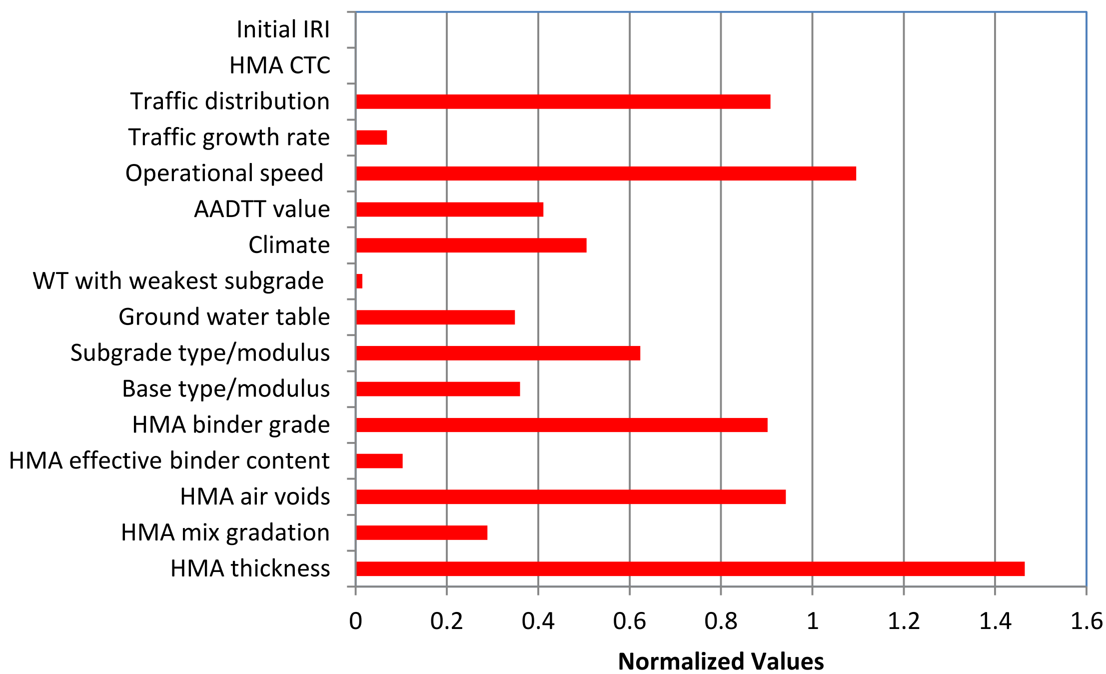

Figure 291 MA Level 3 Significance of Effect of Input Variables on Top-Down Cracking.

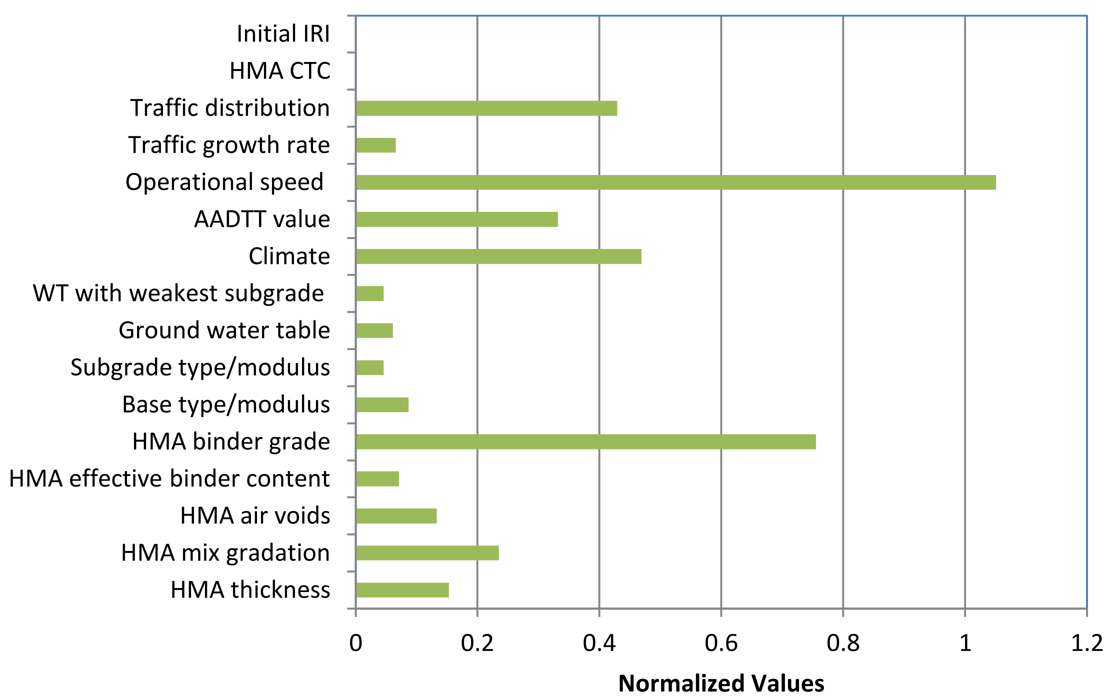

Figure 292 MA Level 3 Significance of Effect of Input Variables on AC Rutting.

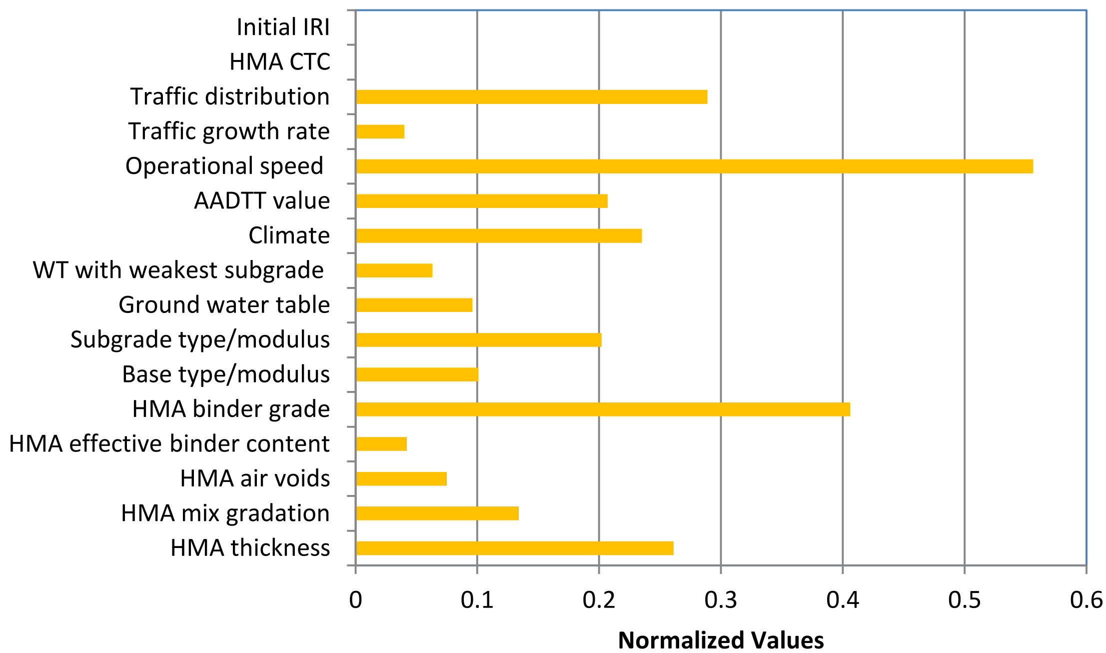

Figure 293 MA Level 3 Significance of Effect of Input Variables on Total Rutting.

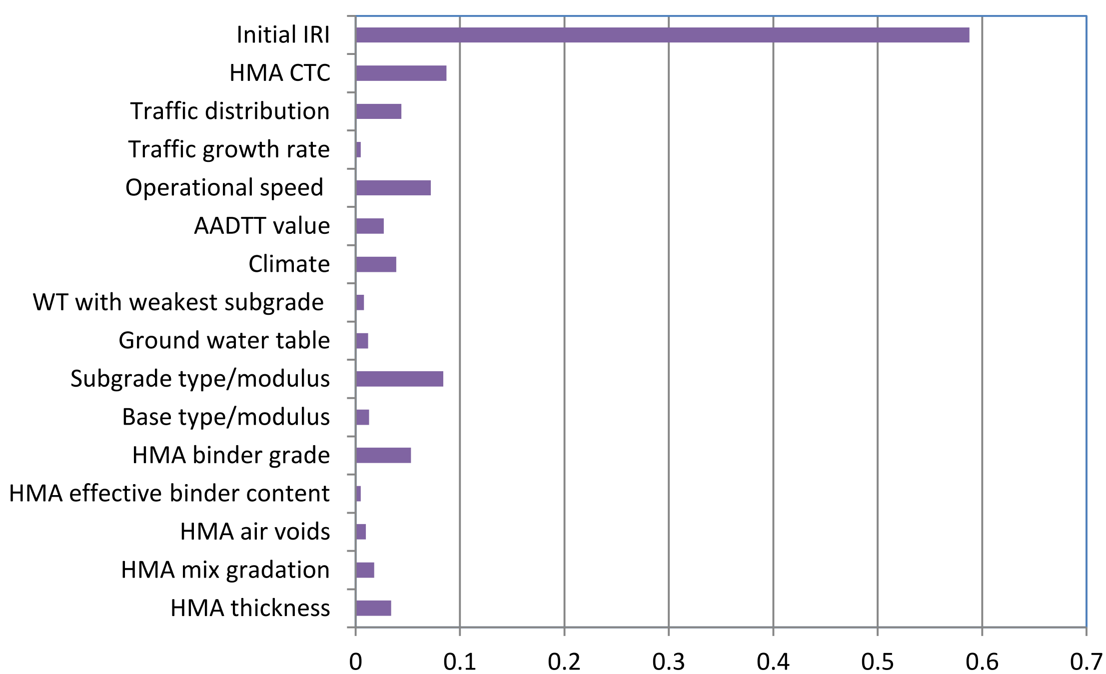

Figure 294 MA Level 3 Significance of Effect of Input Variables on IRI.

The “zero” value on the graph indicates that there is no impact of an input on a predicted pavement distress. As an example, Figures 286 through 289 present the initial IRI and the HMA CTC inputs which have no impact on the predicted pavement distresses such as: bottom-up cracking, top-down cracking, AC rutting and total rutting.

HMA thickness had a significant effect on both fatigue cracking distresses (bottom-up and top-down). Both of these pavement predicted distresses increased with the decrease of HMA thickness (MA Report, Figures 59C – 60C). The moderate effect of HMA thickness was observed for total rutting, and a small effect was observed for AC rutting in Figures 62C and 61C (MA Report). As was expected for the thinner HMA layers, higher pavement distresses and IRI were observed.

Traffic variables such as operational speed, AADTT, and vehicle class distribution had an expected influence on the predicted pavement distresses and roughness IRI (Figures 290 through 294). Operational speed was the most significant variable with the greatest impact on AC rutting and total rutting (MA Report, Figures 29C through 33C). In general, for all pavement distresses and roughness IRI, the decrease of the operational speed increased distresses and IRI values. In the overall order of significance ranking the high position of the operational speed was surprising. This research did not investigate how realistic ranking of vehicle speed is as a variable for pavement performance predictions. It is up to the state agency to decide if the change of vehicle speed and its range could really affect the pavement performance.
For the AADTT and the vehicle class distribution (axle loads), as was expected, with the increase of the track traffic and axle load values, the predicted pavement distresses and IRI increased as well. This study had confirmed this prediction as well (MA Report, Figures 34C – 38C and Figures 19C – 23C).

The effect of binder grade selection was observed in Massachusetts for all types of predicted pavement distresses and roughness IRI. The selected binder grades were analyzed in the conjunction with three different operational speeds (5, 25 and 65 mph). The selected binder grades are listed in Table 23C (MA Report). The significant effect of a selected binder grade was observed on fatigue top-down cracking, and both of rutting distresses (AC and total). The small effect was visible on the fatigue (bottom-up) cracking distress and roughness IRI. In both examples, the lower selected pavement grade exhibited a higher distress level and a higher roughness IRI value (MA Report, Figures 79C through 93C).

Changes in HMA parameters such as air voids (%) or effective binder content (%) were expected to have an influence on predicted pavement distresses in Massachusetts. This expectation was only confirmed for the air voids content and its influence on fatigue bottom-up and top-down cracking distresses. Increased HMA air voids content caused a large increase of fatigue alligator and longitudinal cracking pavement distresses (MA Report, Figures 54C and 55C). The effective binder content variations within the MA

DOT tolerance limits did not influence any of the predicted pavement distresses or roughness IRI.

The Massachusetts climate effects were observed in Figures 39C through 43C (MA Report). Four climatic weather stations and three ground water table levels were selected. The influence on a predicted pavement performance was only observed for the weather station variables, with moderate effects on AC and total rutting, and on fatigue top-down cracking distress. In general, the southern state locations had a higher predicted distress level, with the exception of roughness IRI value prediction, whereas the northern parts of the state exhibited higher values. The ground water table level variable was insignificant for all of the predictions (Table 128).

Table 128 Ranking of Input Variable Significance for MA Level 3 Sensitivity Analysis

	Bottom-Up Cracking	**Top-Down Cracking**	**AC Rutting**	**Total Rutting**	**IRI**
Most Significant Variable	HMA thickness	HMA thickness	Operational speed	Operational speed	Initial IRI
	HMA air voids	Operational speed	HMA binder grade	HMA binder grade	HMA CTC
	Traffic distribution	HMA air voids	Climate	Traffic distribution	Subgrade type/modulus
	Operational speed	Traffic distribution	Traffic distribution	HMA thickness	Operational speed
	HMA binder grade	HMA binder grade	AADTT value	Climate	HMA binder grade
Least Significant Variable	AADTT value	Subgrade type/modulus	HMA mix gradation	AADTT value	Traffic distribution

The ground water table effect is not consistent to current pavement design knowledge, and it needs to be reevaluated with the new MEPDG version.

The effect of subgrade type (Appendix A - AASHTO Classification) on performance was determined by comparing distress and IRI prediction over time with selected subgrade types (MA Report, Figures 99C to 103C). Figure 100C and 101C (MA Report) showed unexpected results for the weaker subgrade type (A-7-6), where there was almost no influence on fatigue (top-down) cracking, and an opposite then expected effect on AC rutting (a weaker subgrade type effected pavement distress less than a stronger subgrade). In general, the lower the subgrade type/modulus the higher the pavement distresses and IRI would be expected.

The effect of mix coefficient of thermal contraction (CTC) in this level of sensitivity analysis was insignificant for all of pavement distresses (zero value in Figures 290 through 293), and had only small effect on the roughness IRI prediction (Figure 294).

6.3 Interpretation of Results – Statistical Method

The predicted pavement distresses at the end of design life were tabulated and the results were used in a statistical model to quantitatively measure the significance of each input parameter on the five output parameters. Thermal cracking was studied by fitting a separate model because of the different input parameters that affect it and version 0.91 used to obtain thermal cracking prediction.

Literature findings on analysis conducted on MEPDG pavement performance prediction data revealed that previous projects purely based their results and recommendations based on a graphical analysis of the prediction data. The implementation report for South Dakota (23) used the general linear model – an analysis of variance (ANOVA) tool to statistically explain the significance of each input parameter on various distress types. The referenced work used the F-ratio to rank the variables in order of significance of their effect. A description of the statistical method used in the report is given in Section 5.1.

Measures of effect size are an important statistical tool that is applicable to the current research (47). Measures of effect size in ANOVA are measures of the degree of association between and effect (e.g., a main effect, an interaction, or a linear contrast) and the dependent variable. They can be thought of as the correlation between an effect and the dependent variable. If the value of the measure of association is squared it can be interpreted as the proportion of variance in the dependent variable that is attributable to each effect. Four of the commonly used measures of effect size in ANOVA are: Eta squared (η^2), partial Eta squared (η_p^2), omega squared (ω^2), and the Intraclass correlation (ρ_I). Eta squared and partial Eta-squared are estimates of the degree of association for the sample.

Eta-squared term is used to quantify the effect size of variables on the predicted distresses for this study. It can be described as the proportion of the total variance that is attributed to one input variable. It is described as the ratio of the variance due to the effect (SS_{Effect}) to the total variance (SS_{Total}), where SS represents the sum of squares calculated by the model.

$\eta^2 = SS_{Effect} / SS_{Total}$

Pie charts are used to graphically display proportion of total variance that is attributable to each effect. The entire circle represents the (corrected) total sums of squares. Each slice of the pie is an effect or the SS for error. The percent of the pie represented by each slice is the effect size, η^2. In a balanced design with equal number of observations for each level of independent (input) variable, the sum of the η^2 for the effects is the total amount of variance in the dependent variable that is predictable from the independent variables.

Partial eta-squared term is not used as a statistical measure because the source of data returns deterministic values; therefore the error term is not a major source of variation. Omega squared is a population-related statistic; therefore it is not applicable to cases where the data is a sample set obtained by selecting the levels of input variables.

General linear model requires coded values for categorical variables (variables that cannot be measured quantitatively, such as binder grade). Hence, variables were coded as

- 1 for low level of the variable,
- 2 for mean and
- 3 for high level of the variable

The variables were coded accordingly if more than three levels were selected for an input variable. The general linear model fitted for the input variables for each pavement structure and the results are explained in this section.

Table 130 is provided as an example to demonstrate how factor levels were decided and coded variables assigned to different input variables for the purpose of fitting a general linear model. The activity was performed for all the obtained sets of prediction data. Table 131 shows the coded variable layout that Minitab Software – software that is capable of performing statistical analysis, accepts to fit a General Linear Model (GLM).

Sensitivity levels based on the percentage variation contributed by each input variable are descriptively shown in tables based on the following criteria:

Table 129 Sensitivity Level Determination

Percentage Variation Explained by the Input Variable	**Sensitivity Level**
Less than 1%	Insensitive
1% - 10%	Low Sensitivity
10% - 25%	Medium Sensitivity
25% - 50%	High Sensitivity
Greater than 50%	Very High Sensitivity

Table 130 Input Values and Factor Levels for New Hampshire – Level 3

Variable	Values		Level	Bottom Up	Top Down	AC Rutting	Total Rutting	IRI
AADTT (X1)	3362		1	2.98	4390	0.43	0.8	164.4
	3655		2	3.26	4720	0.448	0.821	165.4
	6092		3	5.55	6690	0.569	0.963	172.3
TTC (X2)	Default Level 3		1	2.98	4390	0.43	0.8	164.4
	LTPP Derived		2	2.17	2850	0.385	0.724	160.9
	Low High-Class		3	1.62	2000	0.324	0.676	158.7
	High High-Class		4	3.55	3500	0.466	0.834	166.1
Growth Rate (X3)	2%		1	2.8	4160	0.418	0.785	163.7
	2.80%		2	2.98	4390	0.43	0.8	164.4
	4%		3	3.27	4730	0.448	0.821	165.4
Binder Grade (X4) Design Operational Speed (X5)	PG 52-28	5 mph	1,1	7.22	6070	1.161	1.602	198.6
		25 mph	1,2	6.26	5900	0.91	1.335	187.6
		65 mph	1,3	5.3	5680	0.734	1.145	179.5
	PG 58-28	5 mph	2,1	4.75	5360	0.701	1.104	177.5
		25 mph	2,2	3.93	5020	0.555	0.943	170.7
		65 mph	2,3	3.27	4630	0.454	0.829	165.7
	PG 64-28	5 mph	3,1	3.69	4840	0.54	0.924	169.8
		25 mph	3,2	2.98	4390	0.43	0.8	164.4
		65 mph	3,3	2.48	3920	0.354	0.71	160.6
WT Depth (X6)	4 ft		1	3.12	3660	0.421	0.821	165.3
	8 ft		2	2.98	4390	0.43	0.8	164.4
	12 ft		3	2.98	4390	0.43	0.8	164.4
Thickness of AC Layer (X7)	2" 9.5mm Mix		1	16.81	696	0.674	1.492	206.3
	4" 9.5mm Mix		2	18.3	6790	0.551	1.063	185
	4" 19.0mm Mix		2	17.2	6600	0.53	1.036	183.2
	5"		3	6.94	5850	0.465	0.892	170.5
	6"		4	2.98	4390	0.43	0.8	164.4
Air Voids (X8)	4%		1	0.96	1420	0.375	0.734	160.6
	6%		2	2.98	4390	0.43	0.8	164.4
	8%		3	7.8	7660	0.509	0.815	164.8
Effective Binder Content (X9)	13%		1	3.46	4970	0.416	0.783	164
	14%		2	2.98	4390	0.43	0.8	164.4
	15%		3	2.61	3890	0.443	0.815	164.8
NMAS Gradation	9.5 mm Coarse		1,1	4.99	5900	0.637	1.041	175.1
	9.5 mm Mean		1,2	3.72	5070	0.501	0.884	168.2
	9.5 mm Fine		1,3	3.03	4470	0.428	0.798	164.4
	19.0 mm Coarse		2,1	3.38	4730	0.476	0.854	166.8
	19.0 mm Mean		2,2	2.96	4370	0.428	0.797	164.3
	19.0 mm Fine		2,3	2.66	4070	0.394	0.757	162.5
	25.0 mm Coarse		3,1	3.53	4830	0.495	0.876	167.7
	25.0 mm Mean		3,2	2.98	4360	0.431	0.801	164.4
	25.0 mm Fine		3,3	2.61	3990	0.39	0.751	162.3

Variable	Values	Level	Bottom Up	Top Down	AC Rutting	Total Rutting	IRI
Subgrade Modulus	12000 psi	1	3.39	1450	0.419	0.895	172.2
	32000 psi	2	2.98	4390	0.423	0.8	164.4
	40000 psi	3	3.12	4480	0.423	0.805	162
Base Course Modulus	(24370, 30000)		2.78	3830	0.438	0.775	163.3
	(24370, 21150)		2.09	2360	0.426	0.797	163.8
	(33500, 21150)		1.05	642	0.446	0.743	161.1

Table 131 General Linear Model Layout for New Hampshire Level 3 Data

X1	X2	X3	X4	X5	X6	X7	X8	X9	X10	X11	X12	X13	X14	Bottom Up	Top Down	AC Rutting	Total Rutting	IRI
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.98	4390	0.43	0.8	164.4
2	1	2	2	3	2	4	2	2	2	2	1	1	2	3.26	4720	0.448	0.821	165.4
3	1	2	2	3	2	4	2	2	2	2	1	1	2	5.55	6690	0.569	0.963	172.3
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.98	4390	0.43	0.8	164.4
1	2	2	2	3	2	4	2	2	2	2	1	1	2	2.17	2850	0.385	0.724	160.9
1	3	2	2	3	2	4	2	2	2	2	1	1	2	1.62	2000	0.324	0.676	158.7
1	4	2	2	3	2	4	2	2	2	2	1	1	2	3.55	3500	0.466	0.834	166.1
1	1	1	2	3	2	4	2	2	2	2	1	1	2	2.8	4160	0.418	0.785	163.7
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.98	4390	0.43	0.8	164.4
1	1	3	2	3	2	4	2	2	2	2	1	1	2	3.27	4730	0.448	0.821	165.4
1	1	2	1	1	2	4	2	2	2	2	1	1	2	7.22	6070	1.161	1.602	198.6
1	1	2	2	1	2	4	2	2	2	2	1	1	2	6.26	5900	0.91	1.335	187.6
1	1	2	3	1	2	4	2	2	2	2	1	1	2	5.3	5680	0.734	1.145	179.5
1	1	2	1	2	2	4	2	2	2	2	1	1	2	4.75	5360	0.701	1.104	177.5
1	1	2	2	2	2	4	2	2	2	2	1	1	2	3.93	5020	0.555	0.943	170.7
1	1	2	3	2	2	4	2	2	2	2	1	1	2	3.27	4630	0.454	0.829	165.7
1	1	2	1	3	2	4	2	2	2	2	1	1	2	3.69	4840	0.54	0.924	169.8
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.98	4390	0.43	0.8	164.4
1	1	2	3	3	2	4	2	2	2	2	1	1	2	2.48	3920	0.354	0.71	160.6
1	1	2	2	3	1	4	2	2	2	2	1	1	2	3.12	3660	0.421	0.821	165.3
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.98	4390	0.43	0.8	164.4
1	1	2	2	3	3	4	2	2	2	2	1	1	2	2.98	4390	0.43	0.8	164.4
1	1	2	2	3	2	1	2	2	1	2	1	1	2	16.81	696	0.674	1.492	206.3
1	1	2	2	3	2	2	2	2	1	2	1	1	2	18.3	6790	0.551	1.063	185
1	1	2	2	3	2	2	2	2	2	2	1	1	2	17.2	6600	0.53	1.036	183.2
1	1	2	2	3	2	3	2	2	2	2	1	1	2	6.94	5850	0.465	0.892	170.5
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.98	4390	0.43	0.8	164.4

1	1	2	2	3	2	4	1	2	2	2	1	1	2	0.96	1420	0.375	0.734	160.6
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.98	4390	0.43	0.8	164.4
1	1	2	2	3	2	4	3	2	2	2	1	1	2	7.8	7660	0.509	0.815	164.8
1	1	2	2	3	2	4	2	1	2	2	1	1	2	3.46	4970	0.416	0.783	164
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.98	4390	0.43	0.8	164.4
1	1	2	2	3	2	4	2	3	2	2	1	1	2	2.61	3890	0.443	0.815	164.8
1	1	2	2	3	2	4	2	2	1	1	1	1	2	4.99	5900	0.637	1.041	175.1
1	1	2	2	3	2	4	2	2	1	2	1	1	2	3.72	5070	0.501	0.884	168.2
1	1	2	2	3	2	4	2	2	1	3	1	1	2	3.03	4470	0.428	0.798	164.4
1	1	2	2	3	2	4	2	2	2	1	1	1	2	3.38	4730	0.476	0.854	166.8
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.96	4370	0.428	0.797	164.3
1	1	2	2	3	2	4	2	2	2	3	1	1	2	2.66	4070	0.394	0.757	162.5
1	1	2	2	3	2	4	2	2	3	1	1	1	2	3.53	4830	0.495	0.876	167.7
1	1	2	2	3	2	4	2	2	3	2	1	1	2	2.98	4360	0.431	0.801	164.4
1	1	2	2	3	2	4	2	2	3	3	1	1	2	2.61	3990	0.39	0.751	162.3
1	1	2	2	3	2	4	2	2	2	2	1	1	1	3.39	1450	0.419	0.895	172.2
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.98	4390	0.423	0.8	164.4
1	1	2	2	3	2	4	2	2	2	2	1	1	3	3.12	4480	0.423	0.805	162
1	1	2	2	3	2	4	2	2	2	2	1	2	2	2.78	3830	0.438	0.775	163.3
1	1	2	2	3	2	4	2	2	2	2	1	1	2	2.09	2360	0.426	0.797	163.8
1	1	2	2	3	2	4	2	2	2	2	2	1	2	1.05	642	0.446	0.743	161.1

The results of the general linear model were used to calculate the estimates of effect sizes (the eta-squared coefficient) for each variable involved in the model. The advantage of this model is that the single factor effects can be estimated without a large error. A drawback of the model is that it fails to provide interaction effects between input variables. Therefore, the following assumptions have been made to draw conclusions from general linear model outputs:

1. The source of the results returns a deterministic value for a given set of input variables, i.e. for a given set of $\{X_1, X_2\ldots X_n\}$, the same Y values (predicted distresses) are obtained. Therefore, the output of the experiment (MEPDG runs in this case) does not follow a standard normal distribution. This leads to violation of the assumption of a constant variance of the mean for performing a regression analysis. General linear model can be fitted for such data instead of normal linear regression due to its difference in properties from multiple regressions; hence GLM results can be validated for the given set of data values despite the violation of the constant variance assumption.
2. The X-variables are assumed to be independent of each other. Two factorial experiments have been included to study the interaction effects, which are not explicitly distinguished by the general linear model. Therefore, the variation due to the following pairs of input variables are considered to be interchangeable:
   - Design Operational Speed – Binder Grade: Effect estimates of design operational speed can also be considered as effect estimates of binder grade, i.e. importance of selection of binder grade is emphasized by operational speed of the highway being designed
   - NMAS and Aggregate Gradation of the Asphalt Mix: The larger of the effect of the two can be attributed to the extent to which both selection of the correct size of the nominal aggregate for the mix and the gradation is important.
   - Variables like truck traffic class and PG binder grade are difficult to quantify as input variables for a statistical model, because coded variables 1, 2 and 3 do not correspondingly represent changes in actual values of the variables. Hence, truck class distribution which was found to have significant effect on pavement distresses graphically did not result in a large effect sizes statistically.

The tables and plots presented hereafter are the results of the fitted general linear models for data sets obtained from all the states studied. The pie chart also contains an error estimate, which is very small due to highly correlated data and the deterministic procedure used to generate it.

6.3.1 Statistical Analysis of Data – New Hampshire

The percentage variation caused by all individual input parameters on the predicted pavement distresses is presented in Table 134. The significance of the effect of each parameter on a particular distress is measured by the percentage variation it causes in the predicted values.

The graphical representation of the effect sizes are shown below in Figure 295 through Figure 299 for Level 3 design.

The graphs of effect sizes of input variables can be interpreted to quantify the significance of each input variable. Fatigue cracking is very sensitive to change in asphalt concrete layer thickness, which accounts for more than 80% of variation in the predicted cracking values as seen from Figure 295. Asphalt concrete layer thickness also is the parameter that top-down cracking is most sensitive to. Rutting in the asphalt concrete layer is most sensitive to asphalt binder grade, whereas total rutting of the pavement is most sensitive to binder grade as well as the asphalt layer thickness. IRI is most sensitive to asphalt layer thickness, followed by binder grade.

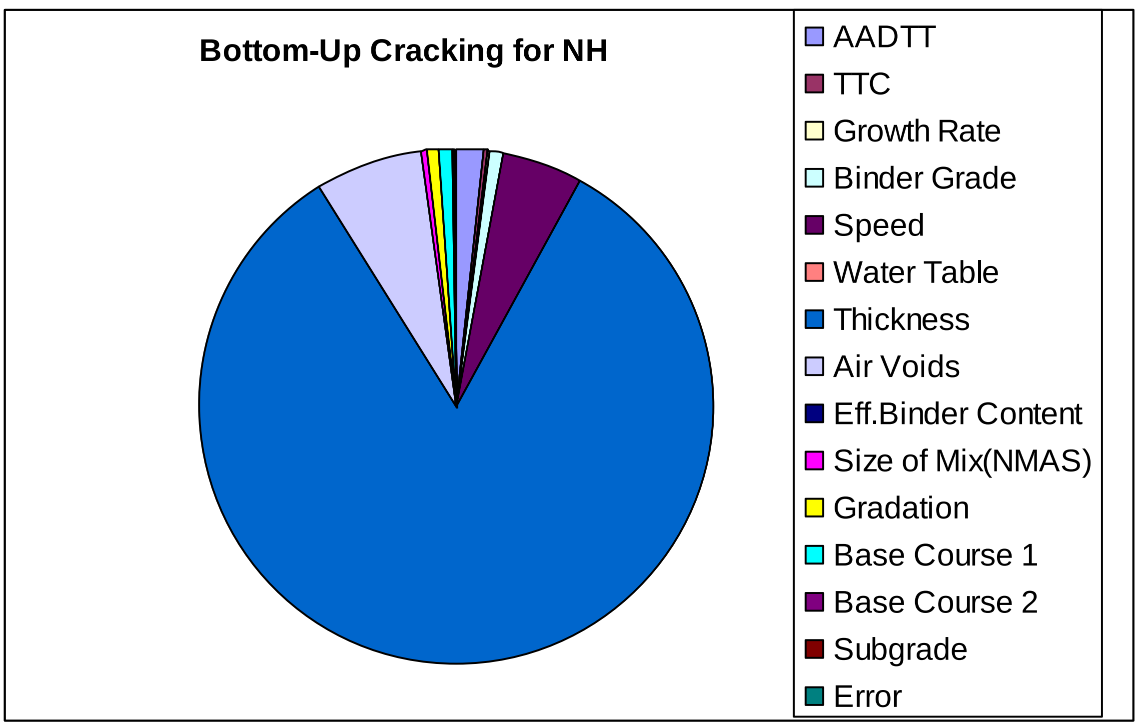

Figure 295 New Hampshire Level 3 – Effect Sizes for Fatigue Cracking

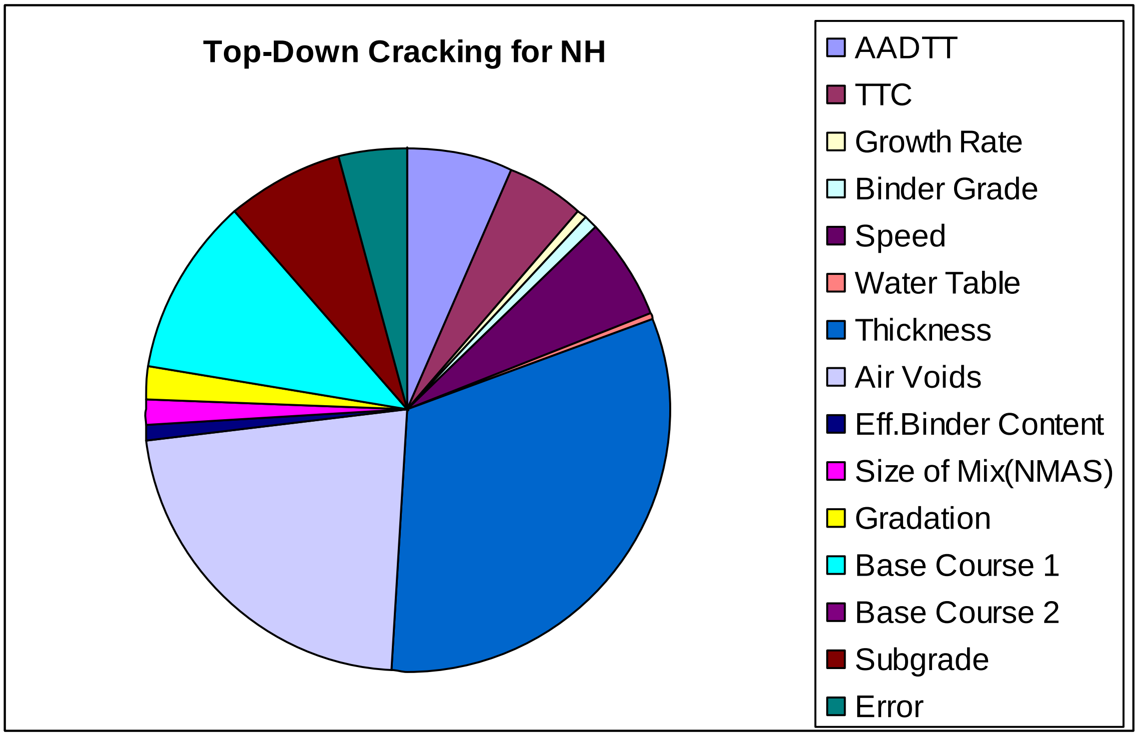

Figure 296 New Hampshire Level 3 – Effect Sizes for Top-Down Cracking

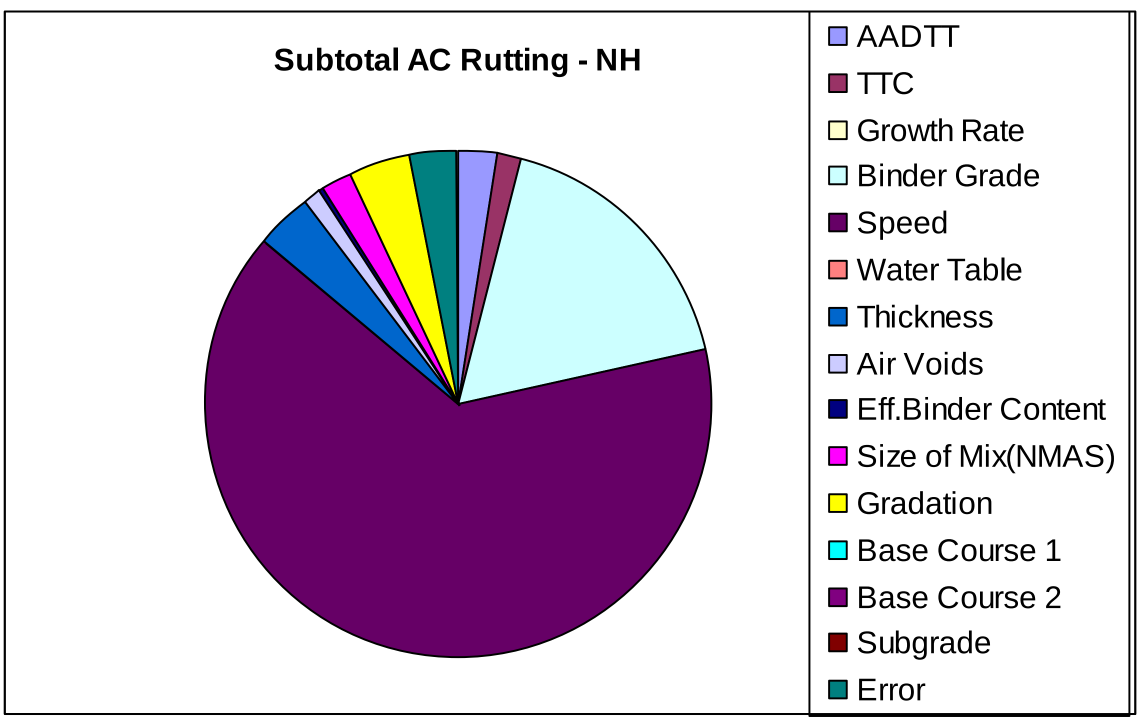

Figure 297 New Hampshire Level 3 – Effect Sizes for Subtotal AC Rutting

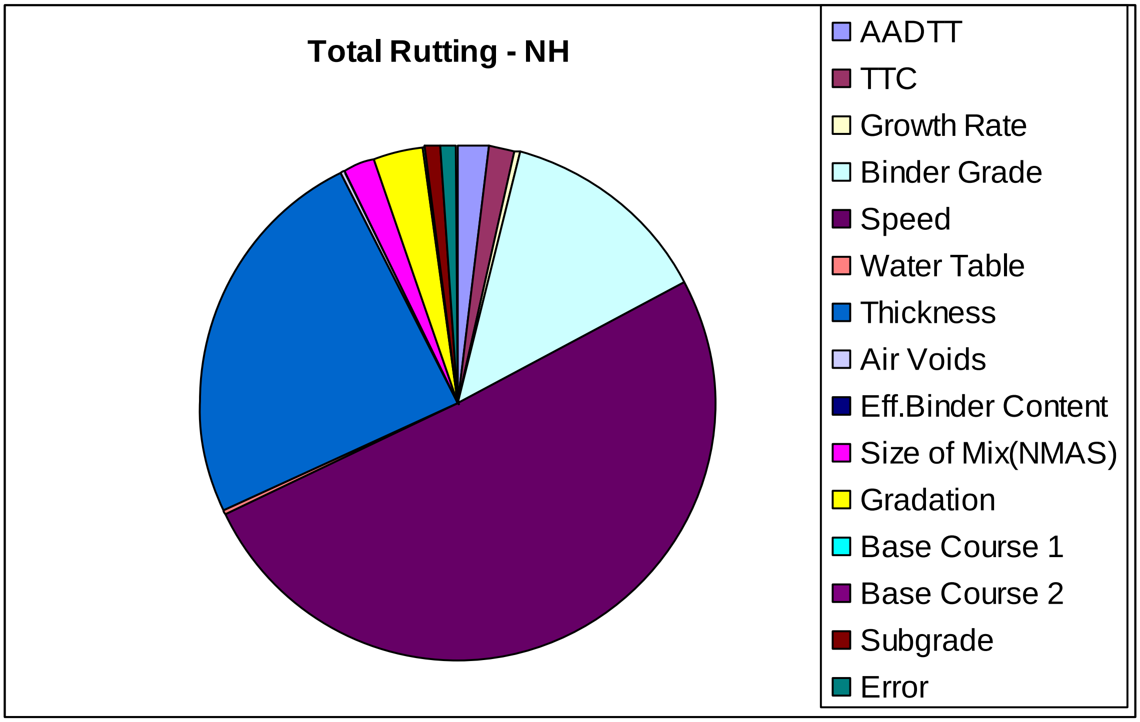

Figure 298 New Hampshire Level 3 – Effect Sizes for Total Rutting

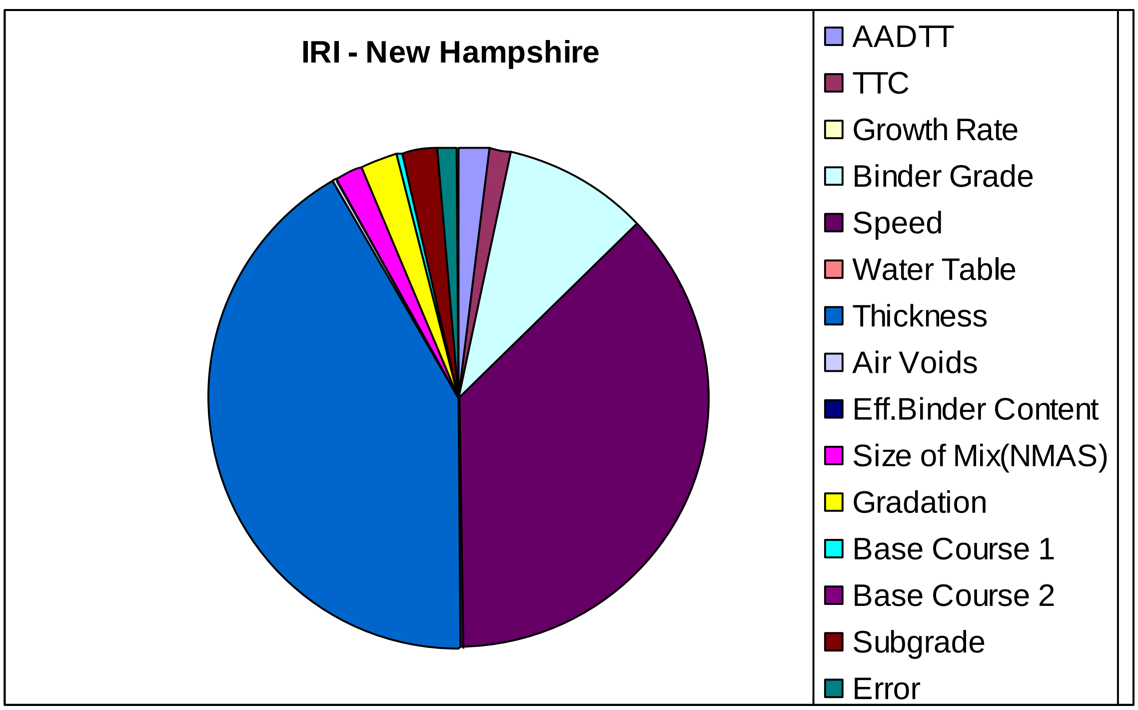

Figure 299 New Hampshire Level 3 – Effect Sizes for IRI

The input variables selected for New Hampshire data were ranked based on their individual contribution to the variation in the output (predicted distresses). The percentages followed by the ranking of the variables are given in Table 132.

Table 132 Significance of Effect of Input Variables – New Hampshire Level 3

Pavement Distress	Bottom-Up		Top-Down		AC Rutting		Total Rutting		IRI	
Input Variable	%	Rank	%	Rank	%	Rank	%	Rank	%	Rank
AADTT	1.55	4	6.73	5	2.41	6	2.07	5	1.92	6
TTC	0.53	9	4.79	7	1.74	8	1.65	7	1.37	8
Growth Rate	0.05	12	0.44	12	0.07	10	0.06	11	0.06	11
Vehicle Speed	5.06	3	6.27	6	64.59	1	50.75	1	37.0	2
Binder Grade	0.87	5	0.89	11	17.36	2	13.36	3	9.39	3
Water Table	0.03	13	0.42	13	0.01	12	0.04	13	0.06	12
AC Layer Thickness	83.14	1	31.30	1	3.44	4	24.45	2	41.79	1
Air Voids	6.51	2	22.19	2	1.30	9	0.36	9	0.42	9
Effective Binder Content	0.11	10	0.89	10	0.05	11	0.05	12	0.03	13
Nominal Aggregate Size	0.61	7	1.74	9	2.06	7	1.81	6	1.52	7
Aggregate Gradation	0.55	8	1.89	8	3.78	3	3.30	4	2.48	4
Base Course Modulus	0.63	6	11.03	3	0.05	11	0.22	10	0.29	10
Subgrade Modulus	0.08	11	7.08	4	0.01	13	0.72	8	2.23	5

Table 133 Sensitivity Descriptions – New Hampshire Level 3

Pavement Distress	**Bottom-Up**	**Top-Down**	**AC Rutting**	**Total Rutting**	**IRI**
Input Variable	**Sensitivity Level**				
AADTT	Low	Low	Low	Low	Low
TTC	Insensitive	Low	Low	Low	Low
Growth Rate	Insensitive	Insensitive	Insensitive	Insensitive	Insensitive
Vehicle Speed	Low	Low	Very High	Very High	High
Binder Grade	Insensitive	Insensitive	Medium	Medium	Low
Water Table	Insensitive	Insensitive	Insensitive	Insensitive	Insensitive
AC Layer Thickness	Very High	High	Low	Medium	High
Air Voids	Low	Medium	Low	Insensitive	Insensitive
Effective Binder Content	Insensitive	Insensitive	Insensitive	Insensitive	Insensitive
Nominal Aggregate Size	Insensitive	Low	Low	Low	Low
Aggregate Gradation	Insensitive	Low	Low	Low	Low
Base Course Modulus	Insensitive	Medium	Insensitive	Insensitive	Insensitive
Subgrade Modulus	Insensitive	Low	Insensitive	Insensitive	Low

Table 134 Estimates of Effect Sizes – New Hampshire Level 3

Bottom – Up Cracking			**Top – Down Cracking**			**Subtotal AC Rutting**			**Total Rutting**			**IRI**		
Name	Adj SS	%	Name	Adj SS	%	Name	Adj SS	%	Name	Adj SS	%	Name	Adj SS	%
Thickness	346.18	83.14	Thickness	27970873	31.30	Speed	0.4647	64.59	Speed	0.5727	50.75	Thickness	1206.25	41.79
Air Voids	27.10	6.51	Air Voids	19827767	22.19	Binder Grade	0.1249	17.36	Thickness	0.2759	24.45	Speed	1068.14	37.00
Speed	21.09	5.06	Base Course 1	9881794	11.06	Gradation	0.0272	3.78	Binder Grade	0.1508	13.36	Binder Grade	271.08	9.39
AADTT	6.44	1.55	Subgrade	6330900	7.08	Thickness	0.0247	3.44	Gradation	0.0372	3.30	Gradation	71.68	2.48
Binder Grade	3.64	0.87	AADTT	6013361	6.73	Error	0.0225	3.12	AADTT	0.0233	2.07	Subgrade	64.3	2.23
Base Course 1	2.61	0.63	Speed	5603044	6.27	AADTT	0.0173	2.41	NMAS	0.0204	1.81	AADTT	55.57	1.92
NMAS	2.53	0.61	TTC	4284515	4.79	NMAS	0.0148	2.06	TTC	0.0186	1.65	NMAS	43.86	1.52
Gradation	2.27	0.55	Error	3808925	4.26	TTC	0.0125	1.74	Error	0.0128	1.13	Error	42.14	1.46
TTC	2.22	0.53	Gradation	1690710	1.89	Air Voids	0.0093	1.30	Subgrade	0.0081	0.72	TTC	39.52	1.37
Error	1.24	0.30	NMAS	1556853	1.74	Growth Rate	0.0005	0.07	Air Voids	0.0041	0.36	Air Voids	12.26	0.42
Effective Binder Content	0.44	0.11	Effective Binder Content	799569	0.89	Effective Binder Content	0.0004	0.05	Base Course 1	0.0025	0.22	Base Course 1	8.31	0.29
Subgrade	0.34	0.08	Binder Grade	792239	0.89	Base Course 1	0.0003	0.05	Growth Rate	0.0007	0.06	Growth Rate	1.6	0.06
Growth Rate	0.19	0.05	Growth Rate	396150	0.44	Base Course 2	0.0001	0.02	Effective Binder Content	0.0005	0.05	Water Table	0.99	0.03
Water Table	0.11	0.03	Water Table	374412	0.42	Water Table	0.0001	0.01	Water Table	0.0005	0.04	Base Course 2	0.73	0.03
Base Course 2	0.00	0.00	Base Course 2	40520	0.05	Subgrade	0.0001	0.01	Base Course 2	0.0004	0.04	Effective Binder Content	0.37	0.01

6.3.2 Statistical Analysis of Data – Connecticut

The graphical representation of the effect sizes are shown below in Figure 300 through Figure 304 for Level 3 design.

Fatigue cracking is very sensitive to change in asphalt concrete layer thickness and asphalt binder grade as seen in Figure 300. Top-down cracking is affected by a set of input parameters like asphalt binder grade, asphalt layer thickness, air void content of asphalt concrete and truck traffic volume (AADTT). Rutting in the asphalt concrete layer is most sensitive to asphalt binder grade, whereas total rutting of the pavement is most sensitive to binder grade as well as the subgrade resilient modulus. IRI is most sensitive to asphalt binder grade, as well as the subgrade resilient modulus.

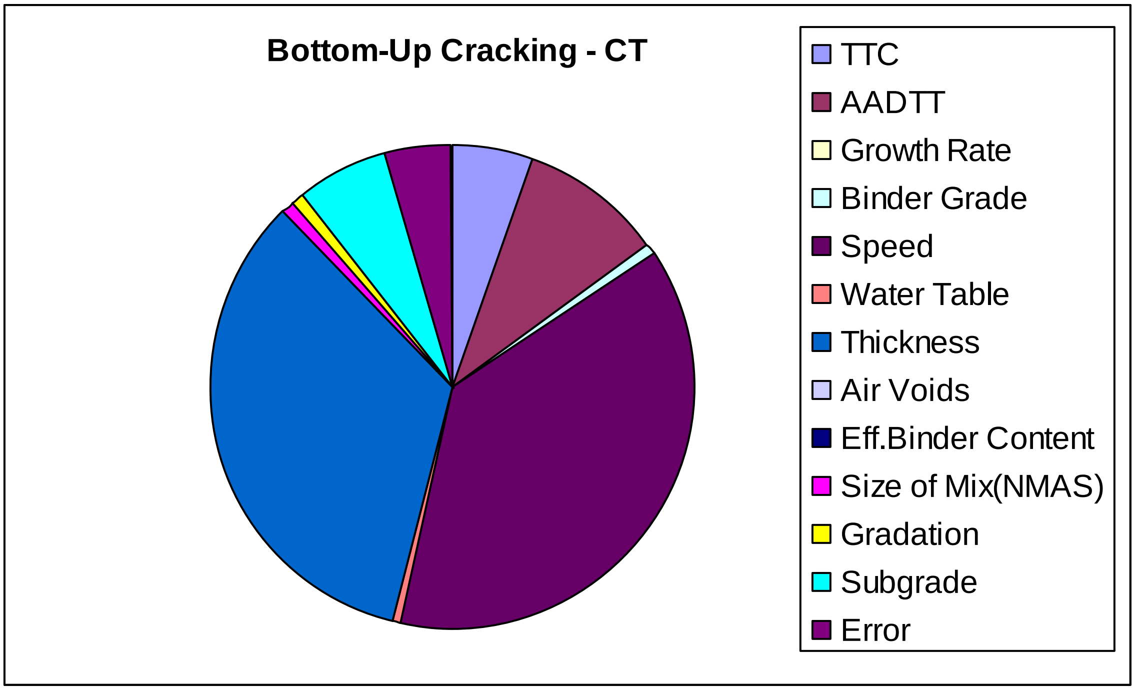

Figure 300 Connecticut Level 3 – Effect Sizes for Fatigue Cracking

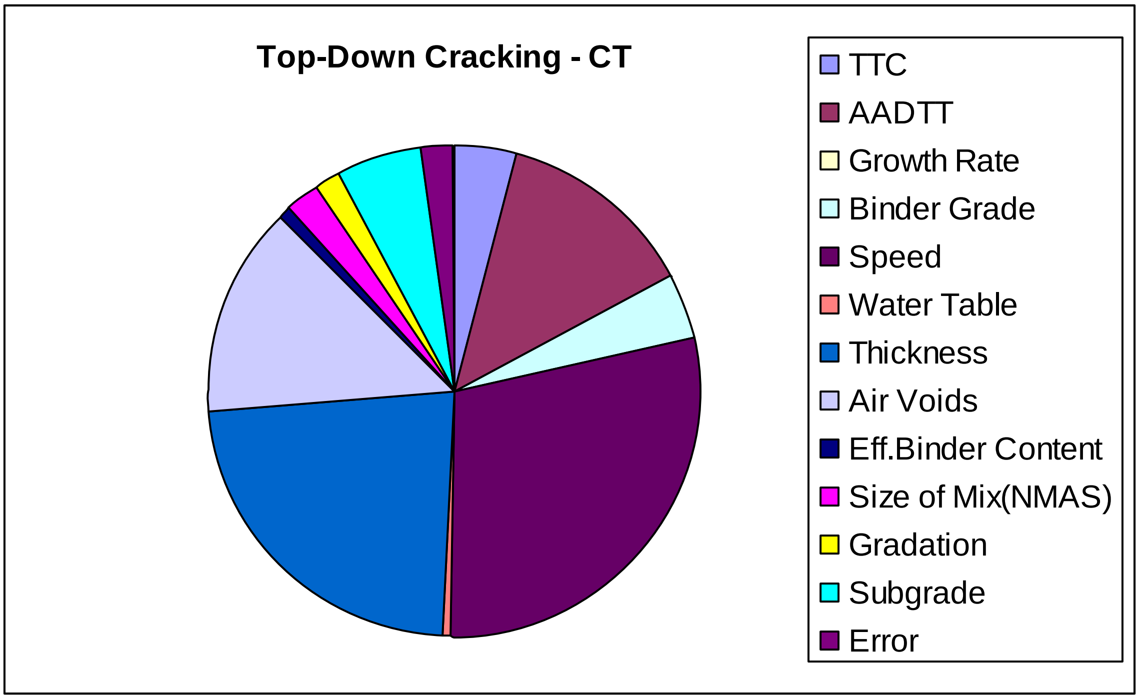

Figure 301 Connecticut Level 3 – Effect Sizes for Top-Down Cracking

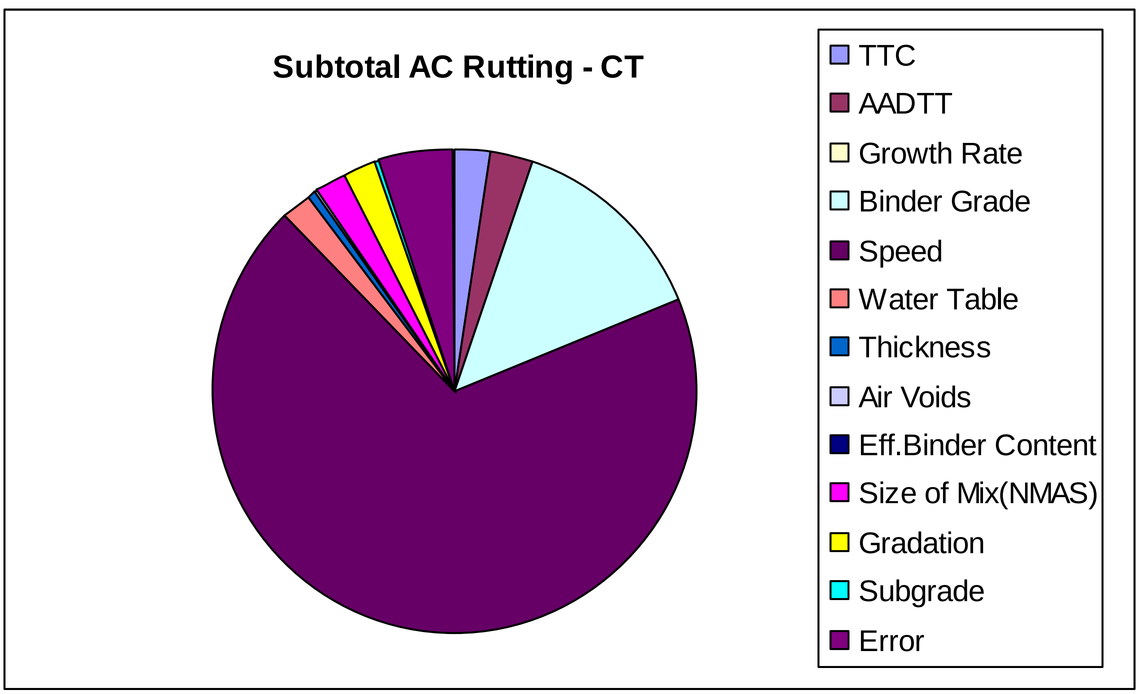

Figure 302 Connecticut Level 3 – Effect Sizes for Rutting in Asphalt Layer

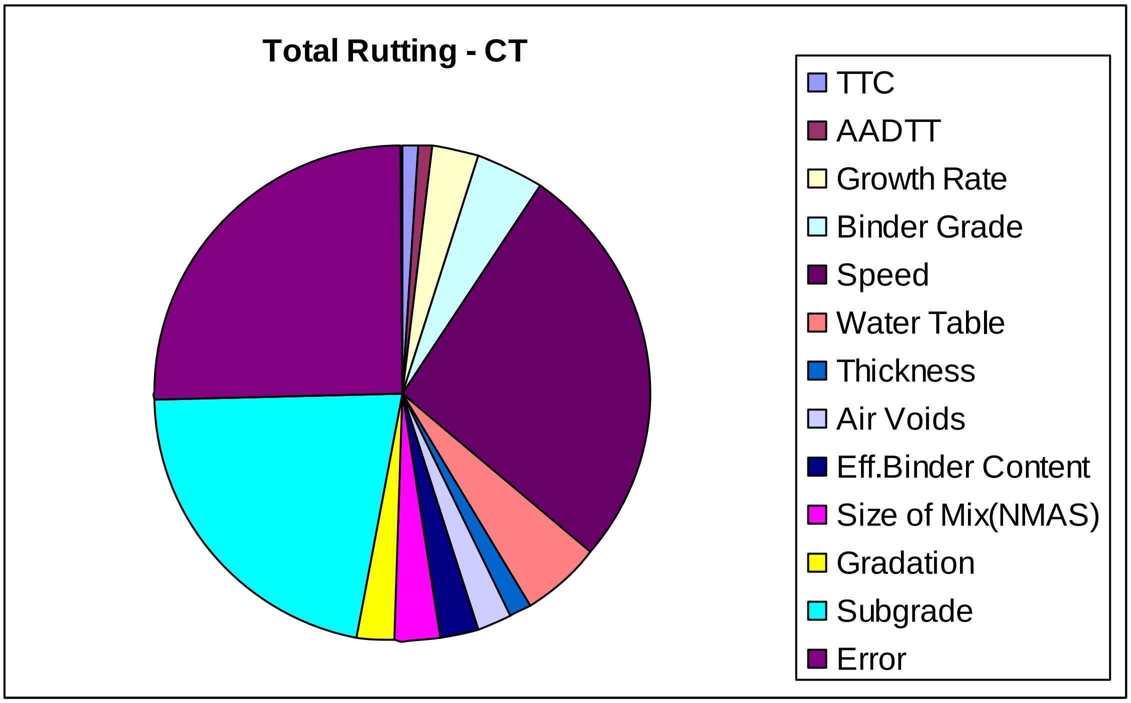

Figure 303 Connecticut Level 3 – Effect Sizes for Total Rutting

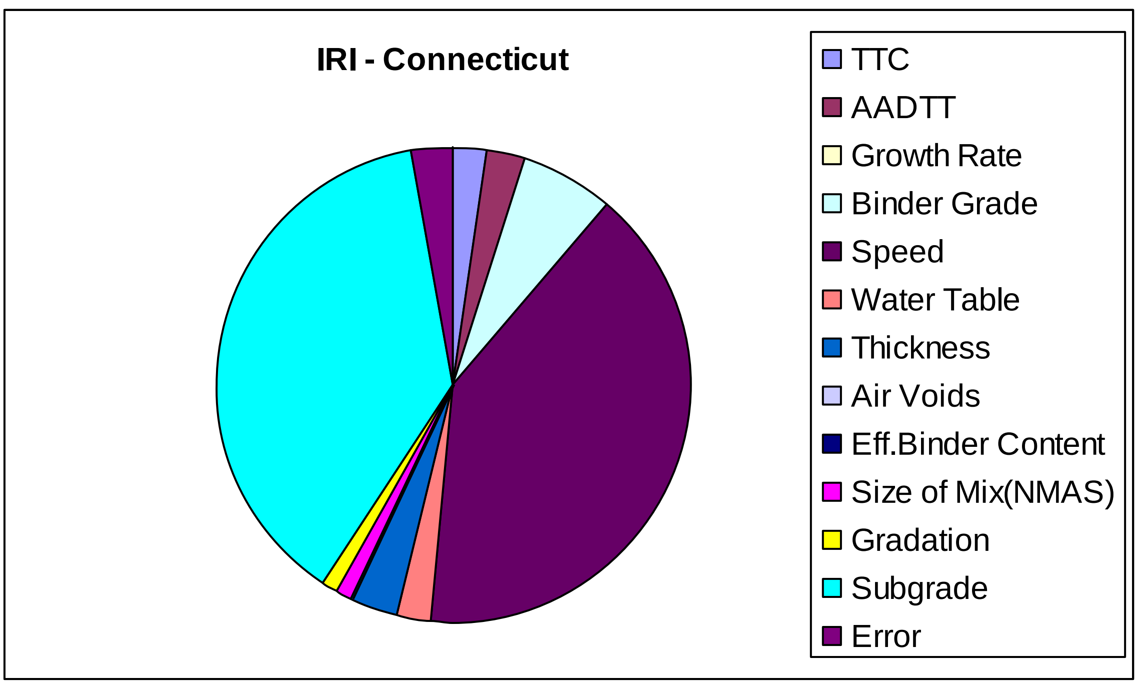

Figure 304 Connecticut Level 3 – Effect Sizes for IRI

The input variables selected for Connecticut data were ranked based on their individual contribution to the variation in the output (predicted distresses). The percentages followed by the ranking of the variables are given in Table 135.

Table 135 Significance of Effect of Input Variables – Connecticut Level 3

Pavement Distress	Bottom-Up		Top-Down		AC Rutting		Total Rutting		IRI	
Input Variable	%	Rank	%	Rank	%	Rank	%	Rank	%	Rank
AADTT	9.42	3	13.15	4	2.74	3	0.88	12	2.34	6
TTC	5.56	5	4.19	7	2.25	4	1.11	11	2.58	5
Growth Rate	0.02	11	0.01	12	0	12	3.12	6	0	12
Vehicle Speed	37.40	1	28.79	1	68.95	1	26.43	1	40.20	1
Binder Grade	0.92	6	4.23	6	13.53	2	4.47	4	6.34	3
Water Table	0.46	9	0.52	11	1.83	7	5.34	3	2.19	7
AC Layer Thickness	33.90	2	22.81	2	0.73	8	1.33	10	3.21	4
Air Voids	0.13	10	13.94	3	0.12	10	2.42	7	0.07	10
Effective Binder Content	0.01	12	0.64	10	0.03	11	2.39	8	0.02	11
Nominal Aggregate Size	0.90	7	2.18	8	2.08	6	3.19	5	1.13	9
Aggregate Gradation	0.63	8	1.81	9	2.28	5	2.10	9	1.18	8
Subgrade Modulus	6.16	4	5.59	5	0.17	9	21.65	2	38.09	2

Table 136 Sensitivity Descriptions – Connecticut Level 3

Pavement Distress	**Bottom-Up**	**Top-Down**	**AC Rutting**	**Total Rutting**	**IRI**
Input Variable	**Sensitivity Level**				
AADTT	Low	Medium	Low	Insensitive	Low
TTC	Low	Low	Low	Low	Low
Growth Rate	Insensitive	Insensitive	Insensitive	Low	Insensitive
Vehicle Speed	High	High	Very High	High	High
Binder Grade	Insensitive	Low	Medium	Low	Low
Water Table	Insensitive	Insensitive	Low	Low	Low
AC Layer Thickness	High	Medium	Insensitive	Low	Low
Air Voids	Insensitive	Medium	Insensitive	Low	Insensitive
Effective Binder Content	Insensitive	Insensitive	Insensitive	Low	Insensitive
Nominal Aggregate Size	Insensitive	Low	Low	Low	Low
Aggregate Gradation	Insensitive	Low	Low	Low	Low
Subgrade Modulus	Low	Low	Insensitive	Insensitive	Low

Table 137 Estimates of Effect Sizes – Connecticut Level 3

Bottom – Up Cracking			**Top – Down Cracking**			**Subtotal AC Rutting**			**Total Rutting**			**IRI**		
Name	Adj SS	%	Name	Adj SS	%	Name	Adj SS	%	Name	Adj SS	%	Name	Adj SS	%
Speed	0.0170	37.40	Speed	238.704	28.79	Speed	0.0182	68.95	Speed	0.0426	26.43	Speed	44.677	40.20
Thickness	0.0154	33.90	Thickness	189.067	22.81	Binder Grade	0.0036	13.53	Error	0.0412	25.56	Subgrade	42.329	38.09
AADTT	0.0043	9.42	Air Voids	115.555	13.94	Error	0.0013	5.01	Subgrade	0.0349	21.65	Binder Grade	7.060	6.35
Subgrade	0.0028	6.16	AADTT	108.982	13.15	AADTT	0.0007	2.74	Water Table	0.0086	5.34	Thickness	3.563	3.21
TTC	0.0025	5.56	Subgrade	46.340	5.59	TTC	0.0007	2.55	Binder Grade	0.0072	4.47	Error	2.937	2.64
Error	0.0020	4.49	Binder Grade	35.079	4.23	Gradation	0.0006	2.28	NMAS	0.0051	3.19	TTC	2.867	2.58
Binder Grade	0.0004	0.92	TTC	34.709	4.19	NMAS	0.0005	2.08	Growth Rate	0.0050	3.12	AADTT	2.603	2.34
NMAS	0.0004	0.90	NMAS	18.075	2.18	Water Table	0.0005	1.83	Air Voids	0.0039	2.42	Water Table	2.435	2.19
Gradation	0.0003	0.63	Error	17.891	2.16	Thickness	0.0002	0.70	Effective Binder Content	0.0039	2.39	Gradation	1.307	1.18
Water Table	0.0002	0.46	Gradation	15.019	1.81	Subgrade	0.0000	0.17	Gradation	0.0034	2.10	NMAS	1.254	1.13
Air Voids	0.0001	0.13	Effective Binder Content	5.285	0.64	Air Voids	0.0000	0.12	Thickness	0.0021	1.33	Air Voids	0.081	0.07
Growth Rate	0.0000	0.02	Water Table	4.286	0.52	Effective Binder Content	0.0000	0.03	TTC	0.0018	1.11	Effective Binder Content	0.021	0.02
Effective Binder Content	0.0000	0.01	Growth Rate	0.067	0.01	Growth Rate	0.0000	0.00	AADTT	0.0014	0.88	Growth Rate	0.002	0.00

6.3.3 Statistical Analysis of Results - Maine

The graphical representation of the effect sizes are shown below in Figure 305 through Figure 309 for Level 3 design.

The pavement layer structure is a 1.2” asphalt concrete porous friction course over an 8.3” asphalt concrete binder course. Since the properties of only the 1.2” asphalt concrete layer were varied, the analysis resulted in the design operational speed on the highway being the single most important factor that influences predicted distresses. Bottom-up cracking is the only distress that showed sensitivity to variation in other parameters like asphalt layer thickness, average daily truck traffic, truck class distribution and subgrade type. Therefore, for overlay design of a thin asphalt concrete layer over an existing asphalt pavement (thickness not exceeding 2 inches), the binder grade selection is the most important parameter with respect to the design operational speed among other factors.

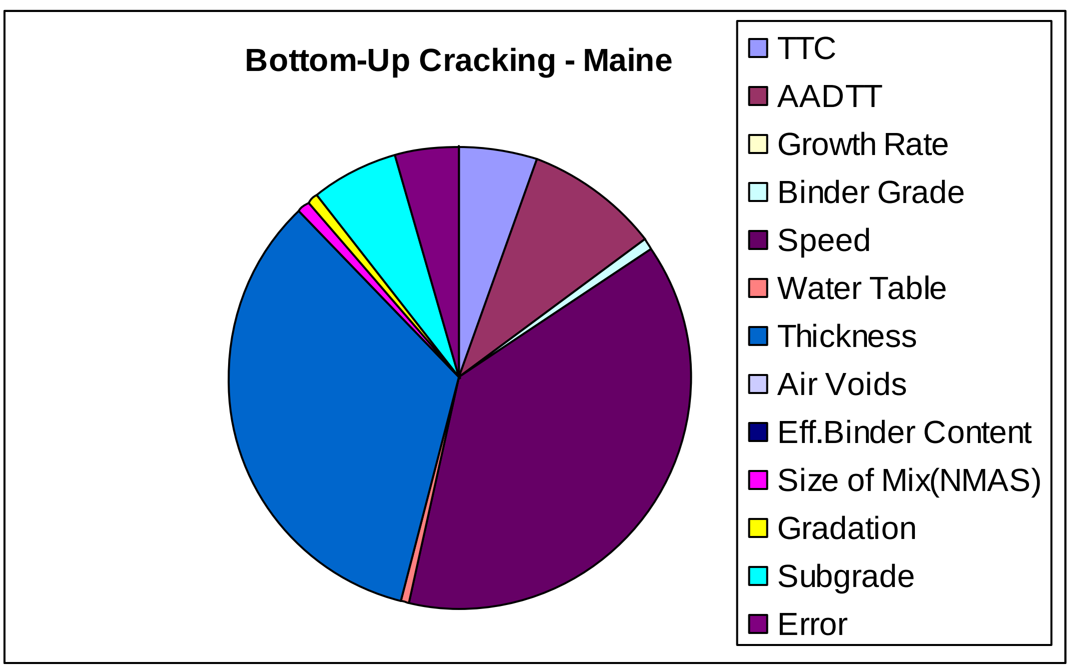

Figure 305 Maine Level 3 – Effect Sizes for Fatigue Cracking

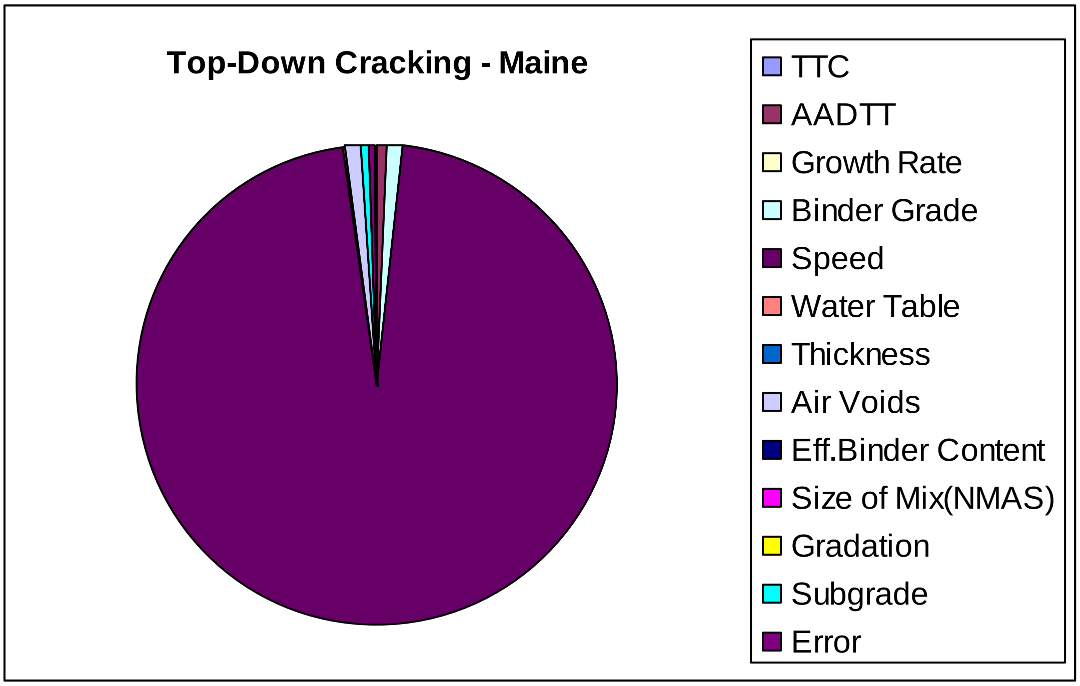

Figure 306 Maine Level 3 – Effect Sizes for Top-Down Cracking

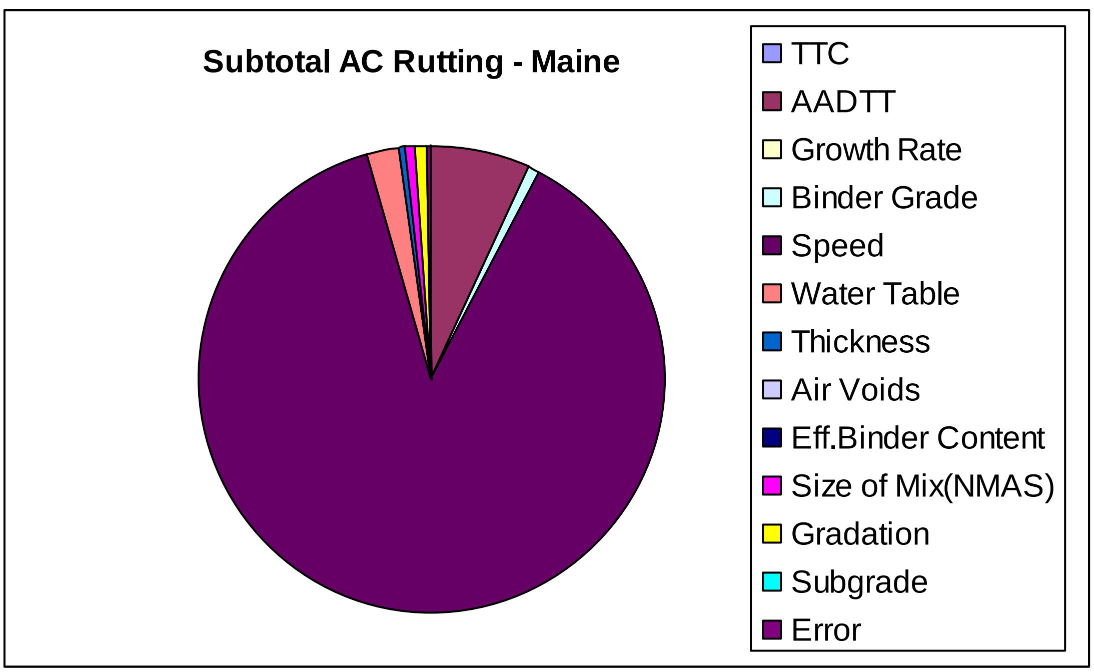

Figure 307 Maine Level 3 – Effect Sizes for Rutting in Asphalt Layer

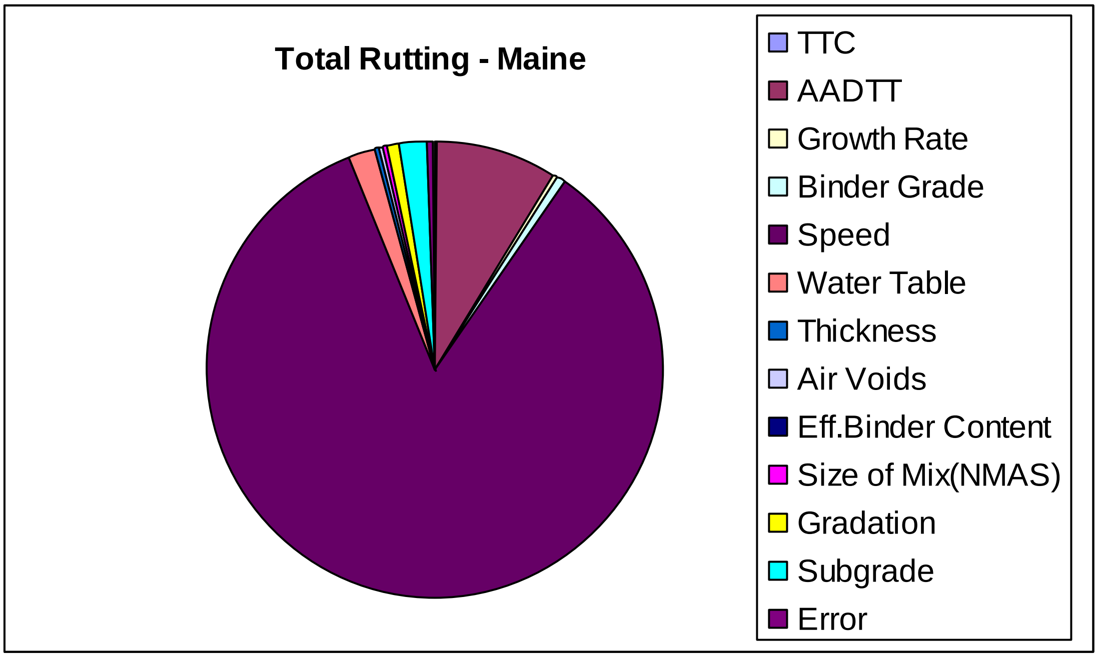

Figure 308 Maine Level 3 – Effect Sizes for Total Rutting

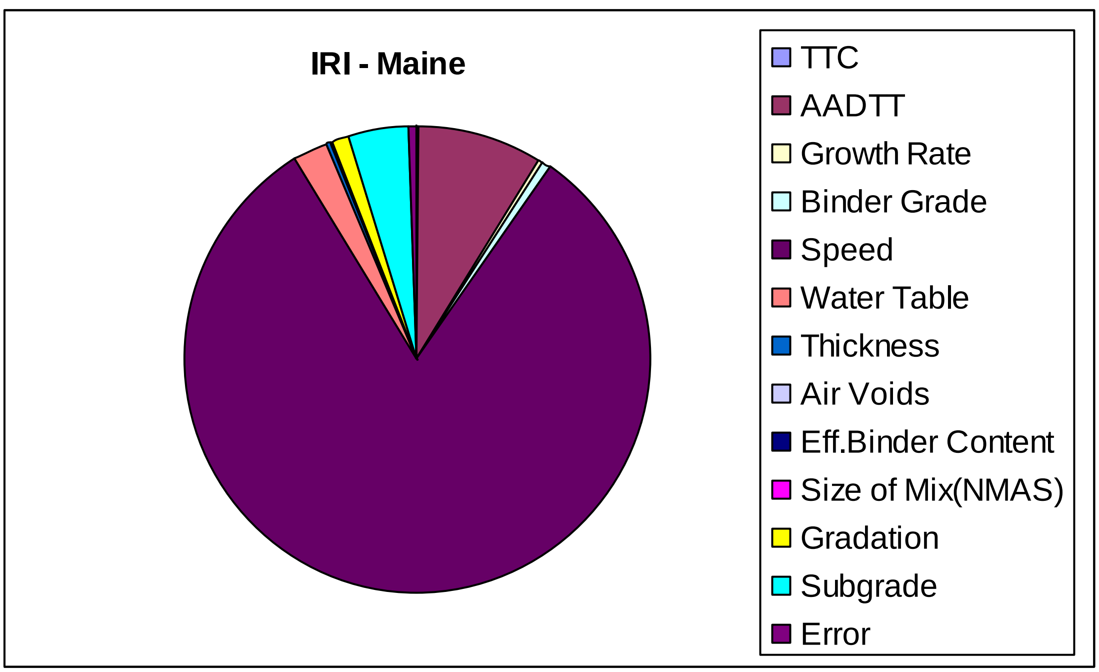

Figure 309 Maine Level 3 – Effect Sizes for IRI

Table 138 Significance of Effect of Input Variables – Maine Level 3

Pavement Distress	Bottom-Up		Top-Down		AC Rutting		Total Rutting		IRI	
Input Variable	%	Rank	%	Rank	%	Rank	%	Rank	%	Rank
TTC	5.56	5	0.02	10	0.05	8	0.36	8	0.34	8
AADTT	9.42	3	0.89	2	6.87	2	8.64	2	8.65	2
Growth Rate	0.02	11	0.00	12	0.04	9	0.04	10	0.05	10
Binder Grade	0.92	6	0.74	4	0.85	5	0.74	6	0.69	6
Speed	37.40	1	96.12	1	87.86	1	84.13	1	81.79	1
Water Table	0.46	9	0.29	6	2.18	3	2.02	3	2.00	4
Thickness	33.90	2	0.01	11	0.44	6	0.30	9	0.31	9
Air Voids	0.13	10	0.79	3	0.02	10	0.02	11	0.03	11
Effective Binder Content	0.01	12	0.07	7	0.00	11	0.00	12	0.01	12
Size of Mix(NMAS)	0.90	7	0.04	9	0.43	7	0.42	7	0.39	7
Gradation	0.63	8	0.05	8	0.90	4	0.89	5	0.88	5
Subgrade	6.16	4	0.43	5	0.00	12	1.92	4	4.34	3

Table 139 Sensitivity Descriptions – Maine Level 3

Pavement Distress	**Bottom-Up**	**Top-Down**	**AC Rutting**	**Total Rutting**	**IRI**
Input Variable	**Sensitivity Level**				
TTC	Low	Insensitive	Insensitive	Insensitive	Insensitive
AADTT	Low	Insensitive	Low	Low	Low
Growth Rate	Insensitive	Insensitive	Insensitive	Insensitive	Insensitive
Binder Grade	Insensitive	Insensitive	Insensitive	Insensitive	Insensitive
Speed	High	Extremely Sensitive (greater than 80% for all distresses)			
Water Table	Insensitive	Insensitive	Low	Low	Low
Thickness	High	Insensitive	Insensitive	Insensitive	Insensitive
Air Voids	Insensitive	Insensitive	Insensitive	Insensitive	Insensitive
Effective Binder Content	Insensitive	Insensitive	Insensitive	Insensitive	Insensitive
Size of Mix(NMAS)	Insensitive	Insensitive	Insensitive	Insensitive	Insensitive
Gradation	Insensitive	Insensitive	Insensitive	Insensitive	Insensitive
Subgrade	Low	Insensitive	Insensitive	Low	Low

Table 140 Estimates of Effect Sizes – Maine Level 3

Bottom – Up Cracking			**Top – Down Cracking**			**Subtotal AC Rutting**			**Total Rutting**			**IRI**		
Name	Adj SS	%	Name	Adj SS	%	Name	Adj SS	%	Name	Adj SS	%	Name	Adj SS	%
Speed	0.0170	37.40	Speed	56655.6	96.12	Speed	0.0548	87.86	Speed	0.0838	84.13	Speed	135.567	81.79
Thickness	0.0154	33.90	AADTT	526.5	0.89	AADTT	0.0043	6.87	AADTT	0.0086	8.64	AADTT	14.337	8.65
AADTT	0.0043	9.42	Air Voids	462.7	0.79	Water Table	0.0014	2.18	Water Table	0.0020	2.02	Subgrade	7.187	4.34
Subgrade	0.0028	6.16	Binder Grade	438.7	0.74	Gradation	0.0006	0.90	Subgrade	0.0019	1.92	Water Table	3.315	2.00
TTC	0.0025	5.56	Error	323.2	0.55	Binder Grade	0.0005	0.85	Gradation	0.0009	0.89	Gradation	1.453	0.88
Error	0.0020	4.49	Subgrade	252.0	0.43	Thickness	0.0003	0.44	Binder Grade	0.0007	0.74	Binder Grade	1.136	0.69
Binder Grade	0.0004	0.92	Water Table	172.8	0.29	NMAS	0.0003	0.43	Error	0.0005	0.51	Error	0.869	0.52
NMAS	0.0004	0.90	Effective Binder Content	41.2	0.07	Error	0.0002	0.34	NMAS	0.0004	0.42	NMAS	0.641	0.39
Gradation	0.0003	0.63	Gradation	30.5	0.05	TTC	0.0000	0.05	TTC	0.0004	0.36	TTC	0.572	0.34
Water Table	0.0002	0.46	NMAS	20.9	0.04	Growth Rate	0.0000	0.04	Thickness	0.0003	0.30	Thickness	0.518	0.31
Air Voids	0.0001	0.13	TTC	10.2	0.02	Air Voids	0.0000	0.02	Growth Rate	0.0000	0.04	Growth Rate	0.081	0.05
Growth Rate	0.0000	0.02	Thickness	3.0	0.01	Effective Binder Content	0.0000	0.00	Air Voids	0.0000	0.02	Air Voids	0.045	0.03
Effective Binder Content	0.0000	0.01	Growth Rate	2.9	0.00	Subgrade	0.0000	0.00	Effective Binder Content	0.0000	0.00	Effective Binder Content	0.021	0.01

6.4 Sensitivity Analysis – Thermal Cracking

Thermal cracking analysis is conducted by performing runs on MEPDG Version 0.91. The parameters that were identified to be critical to the prediction of thermal cracking of a pavement are enlisted below. Climate, asphalt layer thickness and asphalt material properties are the primary factors that affect thermal cracking.

- Asphalt Concrete Layer Thickness
- Air Voids of the asphalt concrete layer
- Climate
- Average Tensile Strength of asphalt concrete mix
- PG Binder grade of asphalt cement
- Coefficient of Thermal Contraction of asphalt concrete
- Surface Shortwave Absorptivity

Statistical model was also fitted on the thermal crack length data as response variable and the above input variables as predictors, and a 100% R-squared value was obtained for the model, indicating a perfect fit of data. The general linear model data showed that all the input variables included in the model were significant predictors of the response, and hence thermal cracking results are explained graphically to illustrate the variation in the predicted thermal crack length (measured in feet per mile) of asphalt pavement with change in each input variable.

MEPDG allows the user to input the coefficient of thermal contraction of the aggregate or the coefficient of thermal contraction of the asphalt concrete mix. A number of trial runs were performed using a wide range of values for the CTC of the aggregate, and it was found that thermal cracking prediction is not sensitive to changes in the CTC of the aggregate. Hence, it is important to determine the CTC of the asphalt concrete mix and not the aggregate used.

Thermal cracking is not a load-associated phenomenon, hence traffic is not considered a variable in studying the effect of MEPDG input variables. The tolerances on some of these input parameters were also obtained from literature, and input values of other parameters were obtained from related research work.

Thermal cracking analysis was conducted using version 0.91 of the software, as no results were obtained from using the newer version used for the remaining runs. This is a widely known problem that the thermal cracking model in version 1.0 is not functional and hence, this method was adopted. For Level 3 design, creep compliance which is a property of the asphalt concrete mix, and the average tensile strength of the mix is obtained by the design guide software for the selected binder grade. The creep compliance data for Level 3 for each asphalt binder contents is documented in the design guide (48).

Thermal cracking is a major pavement distress of concern, especially in regions that experience extremely low temperatures and frequent cyclic changes in temperature. The New England states are subjected to sub-freezing temperatures in winter, supplemented with repetitive rise and fall in temperature due to diurnal changes and weather phenomena such as precipitation in the form of snow. Therefore, thermal cracking is a significant pavement distress in the region for which this study is conducted.

The predicted thermal crack length obtained from the results of runs conducted on the MEPDG is presented in Tables 141-143.

Table 141 New Hampshire Results – Level 3 Thermal Cracking

VARIABLE		THERMAL CRACKING Thermal Crack Length @ 10 years (feet/mile)	Failure Year
Climate	Concord	1580.2	8
	Lebanon	2112	4
	Berlin	2112	3
Air Voids	4%	1608.4	8
	6%	1580.2	8
	8%	1781.4	7
Coefficient of Thermal Contraction	1.0E-03	1155.5	16
	1.3E-03	1580.2	8
	2.0E-03	1984.4	7
Surface Shortwave Absorptivity	0.80	1585.4	8
	0.85	1580.2	8
	0.90	1560.4	7
	0.95	1540.4	7
AC Layer Thickness	2 inches	2112	6
	4 inches	1993.7	7
	5 inches	1726.7	7
	6 inches	1580.2	8
PG Binder Grade	PG 52-28	1240.6	14
	PG 58-22	2112	4
	PG 58-28	1580.2	8
	PG 64-22	2112	4
	PG 64-28	1601.4	8

Table 142 Connecticut Results – Level 3 Thermal Cracking

VARIABLE		THERMAL CRACKING Thermal Crack Length @ 10 years (feet/mile)	Failure Year
Climate	Groton, New London	788.8	
	Bridgeport	95.3	
	Bradley	1535.5	7
Air Voids	3%	947.8	
	4%	788.8	
	5%	274.8	
Coefficient of Thermal Contraction	1.0E-05	79.5	
	1.5E-05	788.8	
	2.0E-05	1390.9	8
Surface Shortwave Absorptivity	0.80	789.9	
	0.85	788.8	
	0.90	504.6	
	0.95	776.8	
AC Layer Thickness	2” + 4.3”	565.9	
	3”+4.3”	788.8	
	4”+4.3”	714.5	
PG Binder Grade	PG 58-22	788.8	
	PG 58-28	0.3	
	PG 64-22	397.3	
	PG 64-28	0.3	

Table 143 Maine Results – Level 3 Thermal Cracking

VARIABLE		THERMAL CRACKING Thermal Crack Length @ 10 years (feet/mile)	Failure Year
Climate	Portland	493.4	
	Millinocket	1803.3	5
	Frenchville	2112	2
Air Voids	4%	753.3	
	5%	493.4	
	6%	959.7	
Coefficient of Thermal Contraction	1.0E-05	44.8	
	1.5E-05	493.4	
	2.0E-05	1387.8	6
Surface Shortwave Absorptivity	0.80	495.3	
	0.85	493.4	
	0.90	497.2	
	0.95	479.7	
AC Layer Thickness	1.2"+8.3"	493.4	
	2"+7.5"	229.2	
	3"+6.5"	231.3	
	4"+5.5"	Software error – no output obtained	
PG Binder Grade	PG 58-22	1729.7	6
	PG 58-28	779.8	
	PG 64-22	1700.6	6
	PG 64-28	493.4	
	PG 70-28	245.3	

6.4.1 Effect of Asphalt Concrete Layer Thickness

Thermal cracking of asphalt pavement decreased with an increase in the thickness of asphalt concrete layer. The presence of an asphalt binder layer below the asphalt surface wearing course significantly affects the thermal crack propagation in the pavement. With an increase of thickness of the asphalt concrete surface layer, thermal cracking decreased significantly. Therefore, sufficient thickness must be provided for the AC surface layer when designing the pavement layer structure such that the pavement does not prematurely fail in thermal cracking.

6.4.2 Effect of Air Void Content

Thermal cracking of an asphalt pavement increases with an increase in air void content. However, different trends were obtained for each state, which is assumed to be due to an error in running the files on the design guide software. The set of input parameters were verified and were found to be correct, however the predicted values of thermal cracking did not follow the expected trend. Tolerances need not be adjusted to accommodate resistance of the pavement to thermal cracking, but might be compensated for by increasing the thickness.

6.4.3 Effect of Coefficient of Thermal Contraction & Average Tensile Strength

Coefficient of thermal contraction drastically affects the predicted thermal crack length. The CTC of an asphalt concrete mix is representative of the response of the asphalt material to temperature variation. Therefore, an AC mix with higher coefficient of thermal contraction undergoes greater dimensional reduction for a particular drop in temperature and hence has greater probability of cracking due to development of thermal stresses. Thermal stresses vary very significantly with changes in CTC; hence CTC must be determined very accurately for the AC mixture. The interaction of CTC and average tensile strength was emphasized in other research work (13). Hence, a factorial design of runs was conducted to study the significance of the interaction of CTC and tensile strength. Since tensile strength is a measure of the resistance of AC mix to thermal cracking, a higher strength mixture would lead to prediction of lower thermal cracking. The average tensile strength of the mixture is calculated by the MEPDG for a given binder grade and volumetric properties. The value of the average tensile strength was varied by $\pm$ 100 psi for each set of control data, and the interaction effect was studied by analyzing the data in statistical software, Minitab®.

The data for the states of Connecticut and Maine is presented to illustrate the interaction effect. The data was analyzed using a two-factor interaction experiment, with each factor at three levels (low, medium and high levels). The results of the analysis of experimental data returned a zero error, indicating perfect interaction between the two variables. The plots shown below explain the variation of thermal cracking length with change in the two X-variables selected, i.e. CTC and average tensile strength.

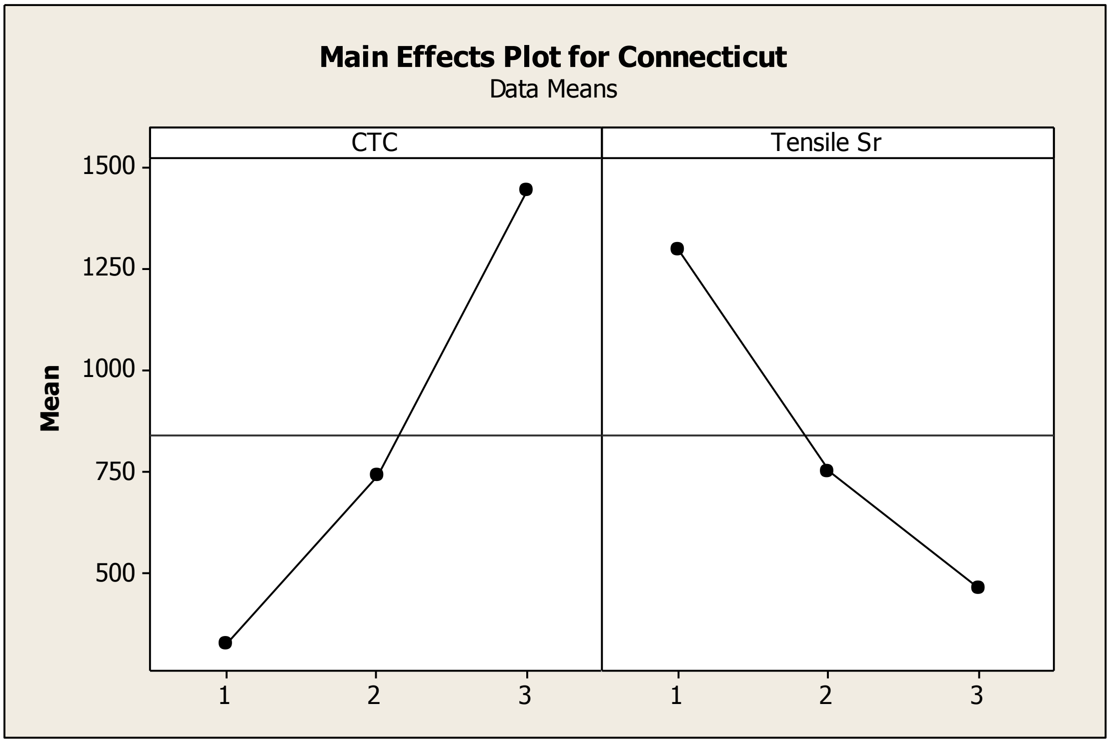

Figure 310 Effects of Individual Factors on Thermal Cracking – Connecticut

The main effects plots show that the predicted thermal cracking length significantly increases with an increase in coefficient of thermal contraction of the mix, and significantly decreases with an increase in average tensile strength of the mix.

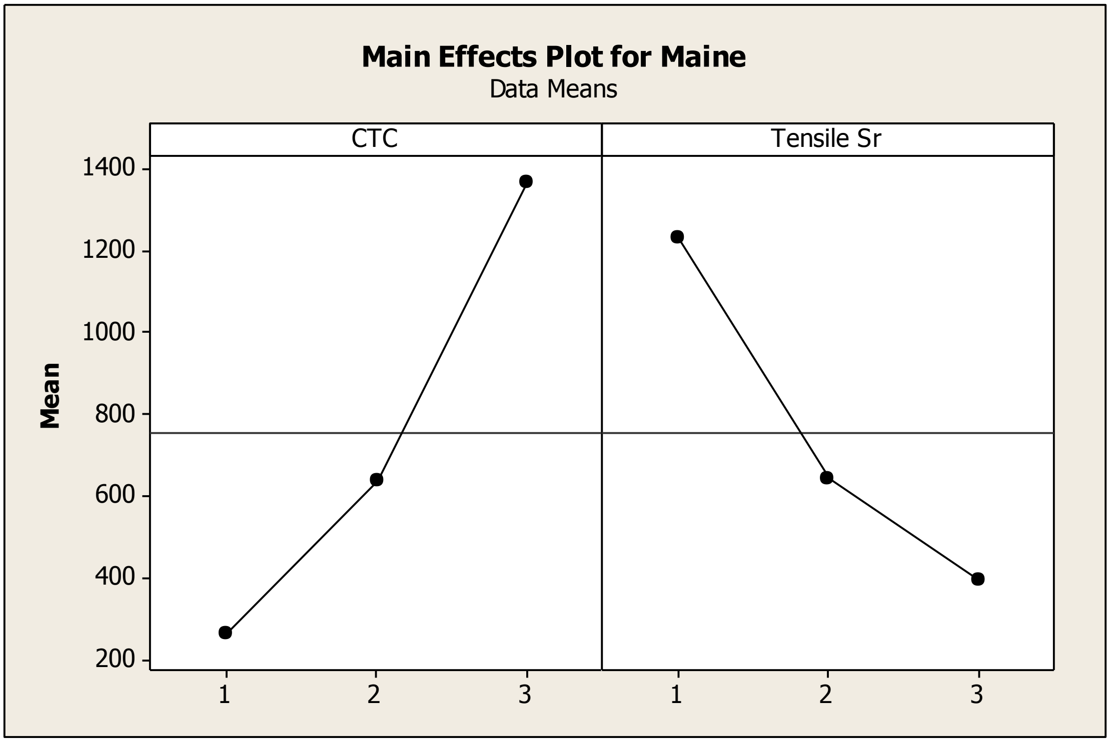

Figure 311 Effects of Individual Factors on Thermal Cracking – Maine

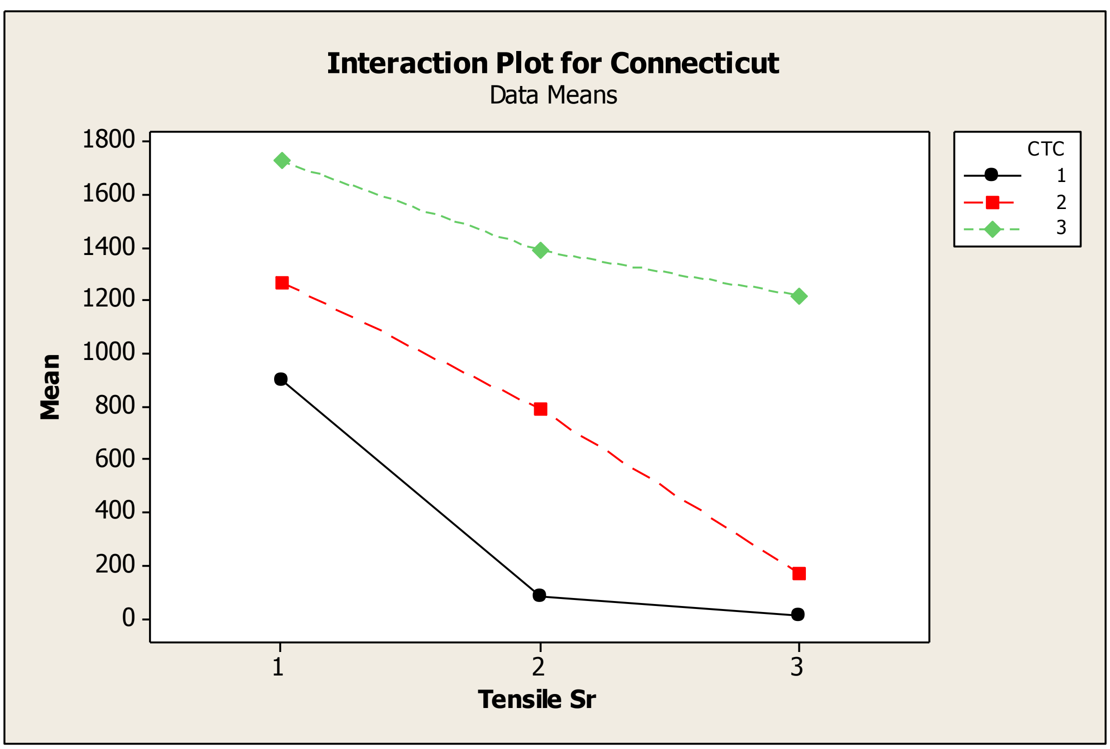

Figure 312 Interaction Plot – Connecticut Thermal Cracking Prediction

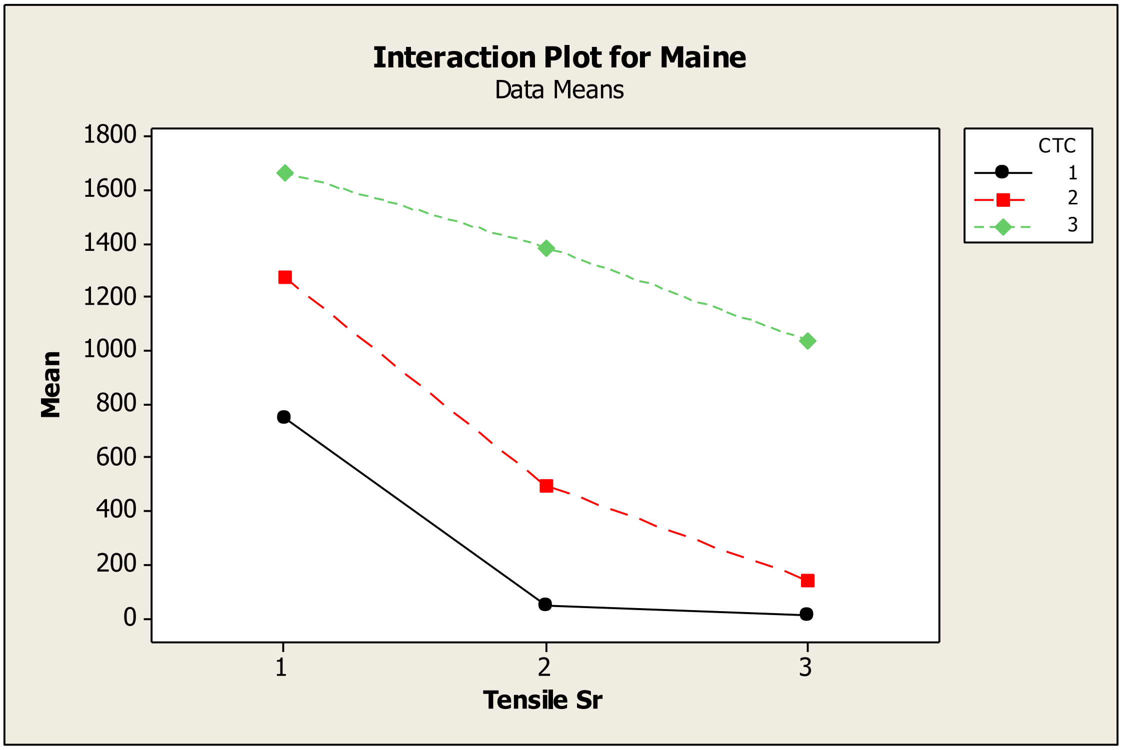

Figure 313 Interaction Plot – Maine Thermal Cracking Prediction

The interaction plots show that for a low value of CTC, the magnitude of decrease in predicted thermal cracking with increase in tensile strength is greater than for a higher CTC. Therefore, tensile strength must be much higher for a mix whose CTC is high for the mix to resist thermal cracking, than for a mix with lower CTC.

6.4.4 Effect of Climate

Climate of a region significantly influences the amount of thermal cracking. Colder regions usually result in prediction of a greater magnitude of thermal crack length, because thermal cracking increases with a decrease in the mean annual air temperature (48). In addition, the predicted thermal cracking was also found to be related to the average frequency of hours with pavement surface temperature less than 15 ^{0}F. The results of this study were found to be in accordance with the findings of the MEPDG research team, where the thermal crack length increased significantly as latitude of the climate station increased.

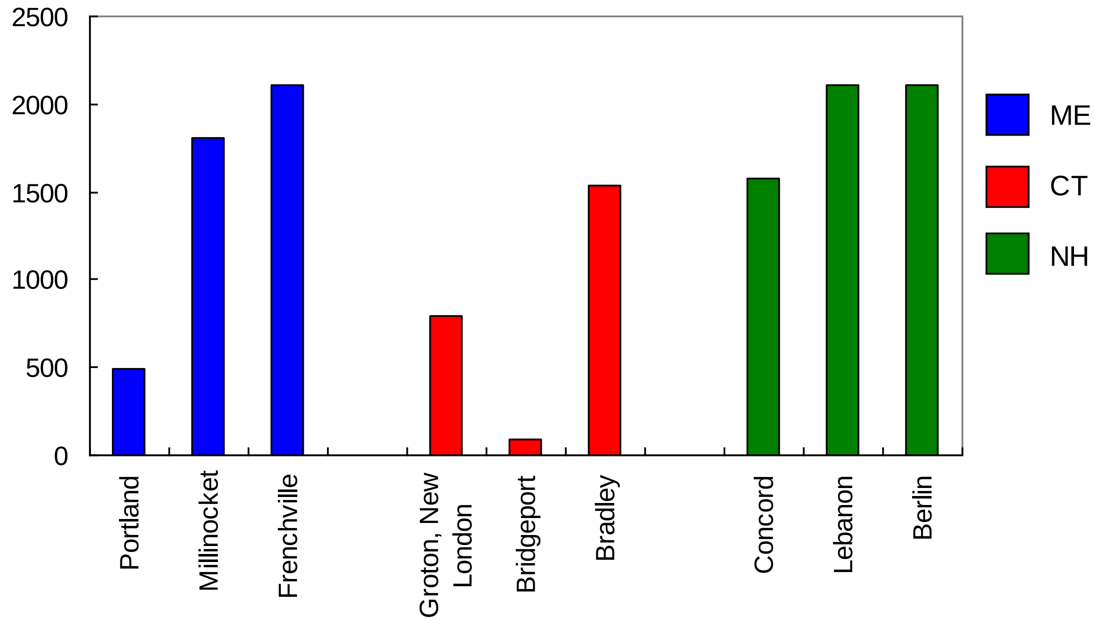

Figure 314 Thermal Crack Length @ 10 years – Effect of Climate

6.4.5 PG Binder Grade

Thermal cracking depends to a great extent on the low-temperature properties of the asphalt binder. The low temperature binder properties are represented by the low-temperature PG binder grade. For all the states, a low PG grade of -28 resulted in significantly lower distresses. Therefore, from the results it can be inferred that low temperature PG grade should be carefully selected for designing flexible pavements depending on the climate location as well as the importance of the road.

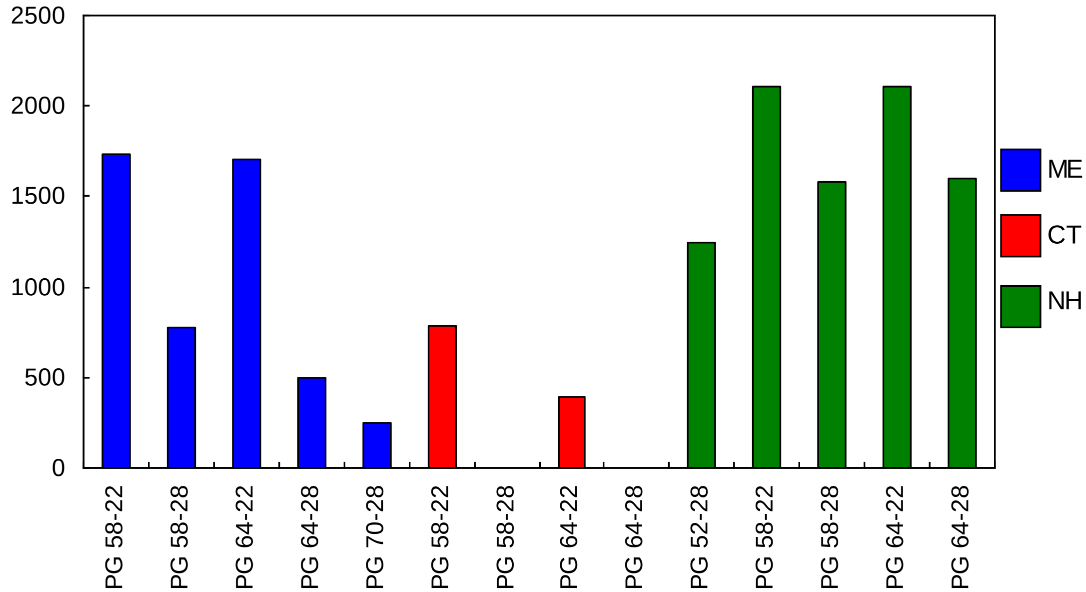

Figure 315 Thermal Crack Length @ 10 years – Effect of Binder Grade

7. CONCLUSIONS AND RECOMMENDATIONS

The research team arrived at the following conclusions from the results of the study. The conclusions and recommendations are applicable to all the states except for additional recommendations, which are made based on state-specific results. The recommendations are formulated on how to obtain data for different input parameters that were used in the study, and what input parameters to assign Level 3 default values. A basis to select Level 3 default values for certain input parameters are also documented in this section. Conclusions and recommendations are made for the major categories of input parameters, namely

- General Inputs
- Traffic Inputs
- Climate Inputs
- Pavement Layer Structure Inputs
- Asphalt Material Properties
- Unbound Layer Material Properties

7.1 General Pavement Design Inputs

General pavement design inputs need to be entered by the user for design of all types of pavements and at all levels of input data. The following data must be collected by the user for the general information section:

- Design Life of the Pavement (in years)
- Base/Subgrade Construction Month – Month & Year
- Pavement Construction Month – Month & Year
- Traffic Open Month – Month & Year

The importance of the pavement construction and opening dates is to ensure the following:

1. Environmental module should generate climate data in accordance with activity dates
2. Climate module should be correctly synchronized with the opening of traffic on the pavement, which affects the prediction of pavement distresses

Activity dates are difficult to predict much ahead of the actual construction schedule; therefore they should be approximately determined from typical construction histories.

The type of pavement design should also be specified in the general information section. The reliability input (default value in the guide is 90%) should be ignored during initial implementation of the MEPDG. ME Design Guide documentation states that the reliability level should be chosen depending on the functional classification of the road if the user chooses a probabilistic model for performance prediction. Since the MEPDG uses a deterministic model and probabilistic model is still not incorporated into the software, reliability level can be safely ignored.

Site/Project Identification information is useful for identification and tracking of the input and output summary files. The information that is required to be entered by the user is:

- Location
- Project ID
- Section ID
- Date of generation of the input file (loaded by default by the MEPDG at the time of creation of the input file)

Station / milepost data and traffic travel direction can also be added as an additional identification measure by the user and for documentation purposes.

There is no relationship between the data entered in the identification section and the prediction of pavement distresses, and is solely for the purpose of identification of the input and output data, to aid in documentation.

Analysis parameters are provided by the MEPDG when an input file is created. The values should be changed by the state highway agency using the MEPDG for pavement design such that the failure criteria are set according to the state performance specifications. If the state pavement design and maintenance documentation does not contain information on the permitted levels of pavement distresses predicted by the MEPDG, the values provided in the MEPDG by default can be used. The limits corresponding to medium level distress levels accrued by a pavement. The values used in this project are shown in Table 144.

Table 144 Performance Criteria for Flexible Pavement – Default Limits

Performance Criterion	**Failure Limit**
Terminal IRI (inches/mile)	172
AC Surface Down (Longitudinal) Cracking (feet/mile)	2000
AC Bottom-Up (Fatigue) Cracking (% area of lane)	25
AC Thermal Fracture – Crack Length (feet/mi)	1000
Permanent Deformation – Total Pavement (inches)	0.75
Permanent Deformation – AC Only (inches)	0.25

7.2 Traffic Inputs

The traffic input parameters for which the user must enter values from various data sources are

- Annual Average Daily Truck Traffic (AADTT)
- Number of lanes in design direction
- Percentage of trucks in design direction
- Percentage of trucks in the design lane
- Design Operational Speed (mph)
- Traffic Growth Factor
- Vehicle Class Distribution (if available), else can be imported from national averages available in MEPDG for different road classes
- Design Lane Width

The other traffic input parameters can be assigned default values, or entered by the values if appropriate data exists for the purpose. The monthly and hourly traffic distribution factors do not affect pavement distresses; hence they can be assigned default values.

AADTT for the pavement section to be designed must be obtained from the history of traffic volume counts. If vehicle-type and vehicle-class specific volume counts exist, analysis can be performed on the data to obtain the truck traffic class distribution also to be entered in the Vehicle Class Distribution field. If data does not exist for different vehicle types and truck classes, the annual average daily traffic data (AADT) can be obtained from the relevant sources – Department of Transportation for each state has traffic volume count stations installed which collects and document AADT data. This data can be extrapolated for the year of construction which is used as the base AADT for design. The traffic extrapolation activities done in this research resulted in excellent linear regression fits, indicating that traffic data can be linearly extrapolated without large error. For a new pavement construction, traffic data can be assumed from a similar section with similar functionality, class of highway, traffic conditions and pavement structure.

For Level 3 AC design, default distribution can be used as it provides a more conservative design (higher distress prediction values leading to a design that exceeds expected performance on actual highway) than measured truck class percentages. If truck class percentage data exists, it can be used for obtaining a design whose predicted performance is closer to actual highway performance. LTPP data can be used for interstate sections with similar functionality, class of highway, traffic conditions and pavement structure.

Default traffic load spectrum can be used for Level 2 and 3 design, as installation of weigh-in-motion for each highway class and traffic conditions leads to a very high cost of implementation. Therefore, axle load distribution factors can be set at default values.

The conclusion of the study shows AADTT values must be calculated with greater accuracy for roads with low truck volumes. This conclusion contrasts the observation that is expected generally that high AADTT values should be measured with greater accuracy due to high magnitude of predicted distresses. This is compensated by very low values of prediction distresses, which eliminates the requirement of high precision measurements. Therefore, a final conclusion can be reached that AADTT measurement does not require a high accuracy level.

Traffic growth rate was found to be insignificant to the prediction of pavement distresses. Graphical as well as statistical analysis showed that pavement distresses did not vary significantly with change in traffic growth rate. Therefore, the slope of the regression line of traffic volume (AADT or AADTT on Y-axis) versus year (on X-axis) can be used as the growth rate factor in design. If appropriate data is not available for this purpose, growth factors can be assumed from sections with similar functionality, class of highway, traffic conditions and pavement structure. Truck traffic distribution can be loaded from the available default values present in the MEPDG through the vehicle class distribution screen.

Traffic inputs have a low sensitivity effect on the prediction of pavement distresses. For an asphalt pavement, material properties affect distress prediction to a larger extent than traffic variables. Therefore, a reasonable variability or error is allowable in collection of traffic input for a design project using the MEPDG. The following table shows the range of sensitivity variation for different pavement structures and traffic ranges studied in this project:

Table 145 Range of Sensitivity of Traffic Inputs on Pavement Distresses

Pavement Distress	**Traffic Input**		
	AADTT	Growth Rate	Truck Class Distribution
Fatigue Cracking	Low	Insensitive	Insensitive-Low
Longitudinal Cracking	Insensitive-Medium		Insensitive-Low
AC Layer Rutting	Low		Insensitive-Low
Total Rutting	Insensitive-Low		Insensitive-Low
IRI	Low		Insensitive-Low

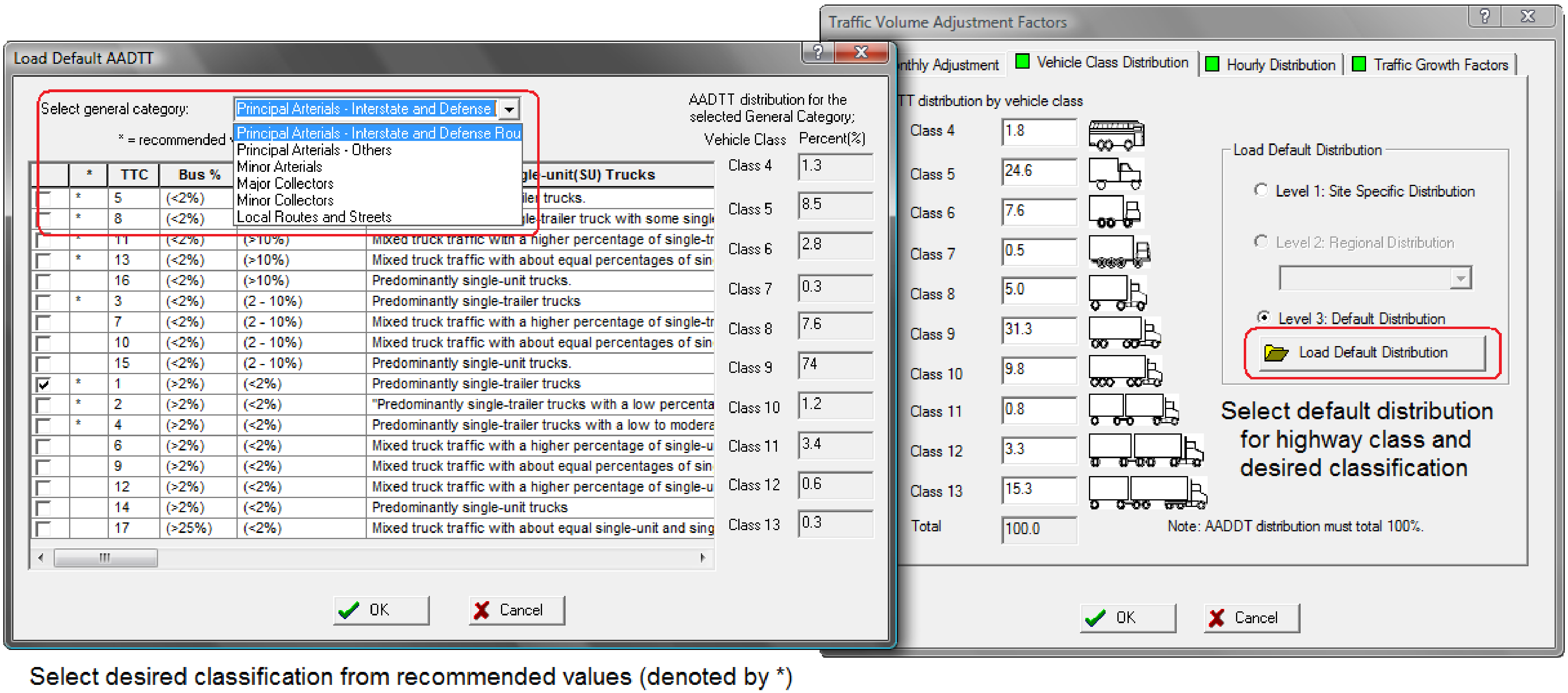

Figure 316 Truck Traffic Class Distributions – Default Values in MEPDG

Design speed of operation of vehicles on the highway is a very important factor that needs to be entered by the user. Highways can have multiple speed limits for different types of vehicles that travel on them – trucks usually have lower speed limits on highways in residential areas and selected sections on interstate highways and other principal arterial roads. Since speed significantly affects the pavement distresses, the pavement section must be designed with appropriate design speed.

Design Operational Speed is a very significant factor in predicting pavement distresses. If a pavement section has different speed limits, it is recommended to design the pavement by dividing it into sub-sections with corresponding maximum allowable speed for correct prediction of distresses.

The range of variation of sensitivity of operational speed on pavement distresses for all the pavement structures studied in this project is shown in the table below.

Table 146 Range of Sensitivity of Design Operational Speed to Pavement Distresses

Pavement Distress	Sensitivity Range - Speed
Fatigue Cracking	Low – High
Longitudinal Cracking	Low – Very High
AC Layer Rutting	Very High
Total Rutting	High – Very High
IRI	High – Very High

Percentage of trucks in the design direction and percentage in design lane can be obtained from traffic data, or default values can be retained for design. The default values represent the generally expected distribution of truck traffic in the lanes of a highway. The default values provided by the design guide are

- Percentage of trucks in travel direction - 50%
- Percentage of trucks in travel lane – 95%

Lane width should be entered by the user for the pavement section to be designed. Other traffic inputs can be retained as default values that are present when an input file is created.

7.3 Climate Inputs

The climate inputs in the MEPDG consist of climate data, which the software either obtains from an inbuilt data file for existing climate stations or interpolates based on the geographical details of the location, and the water table depth. Climate station data was not found to significantly influence the predicted distresses computed by Version 1.0 of the software. Hence, existing climate data is sufficient for design. However, the New England states have very few climate stations built into the MEPDG climate data module. Therefore, there is a need for interpolation of climate data for a large number of geographical locations in these states. A study was conducted on the sensitivity of distresses to interpolation, as compared to the actual station data. The method of triangulation was applied to interpolate climate data for the three actual stations used. The details of interpolation and stations selected have been discussed in the climate sensitivity section.

Thermal cracking is highly sensitive to climate data. Therefore, material selection should be properly done according to the climate of the location where the pavement is to be constructed. Since the geographical area of New Hampshire, Connecticut, Maine and Rhode Island is not very large, there was no significant effect of climate on cracking, rutting and roughness of pavement. Therefore, climate data effects can be ignored if fatigue cracking, rutting and roughness of the road are the primary issues of concern.

Water table depth was selected as a parameter whose sensitivity is tested. The pavement distresses affected by water table depth are total rutting (due to debilitation of subgrade, and freeze-thaw influence) and roughness. Fatigue and longitudinal cracking, as well as rutting in the asphalt layer were found to be not affected by change in water table. A number of runs were conducted with water table values greater than 12 ft, and the distresses were found to remain unchanged. The following table shows the range of sensitivity of water table for different pavement structures studied in this project.

Table 147 Range of Sensitivity of Water Table Depth on Pavement Distresses

Pavement Distress	**Sensitivity Range - Speed**
Fatigue Cracking	Insensitive
Longitudinal Cracking	Insensitive
AC Layer Rutting	Insensitive – Low
Total Rutting	Insensitive – Low
IRI	Insensitive – Low

Therefore, average water table depth must be entered by the user for the construction location. Very low water table depths (less than 8 feet) must be entered with higher accuracy after conducting field tests, whereas average test values or values interpolated from surrounding GWT stations can be used for greater depths. Water table data can be obtained from (5), which contains GWT monthly values documented on its website.

7.4 Pavement Layer Inputs

The pavement layer inputs that were varied in the study are the thicknesses and type of different layers in the pavement structure. Asphalt layer type and asphalt concrete material was selected for each asphalt concrete layer for the study. Base course and subgrade materials were varied within the typical material types found in the states and their properties were identified from literature review (33). Thickness was varied for asphalt concrete layers and material type was varied for base and subgrade layers.

The range of sensitivity of asphalt layer thickness to different pavement distresses is shown in Table 148.

Table 148 Range of Sensitivity of Asphalt Layer Thickness on Pavement Distresses

Pavement Distress	Sensitivity Range - Speed
Fatigue Cracking	High – Very High
Longitudinal Cracking	Insensitive – High
AC Layer Rutting	Insensitive – Low
Total Rutting	Insensitive – Medium
IRI	Insensitive – High

Rutting is not highly sensitive to changes in pavement thickness, and is dependent greatly on the properties of the asphalt surface layer rather than on its thickness. Cracking of the pavement is highly sensitive to thickness variation. Therefore, various design alternatives should be considered for AC overlays involving milling of an existing layer and construction of an overlay. The minimum overlay thickness should be sufficient to prevent early failure of the pavement in fatigue cracking, longitudinal cracking and roughness.

Quality control must be ensured by measurement of thicknesses on a sufficient sample of field cores and distress prediction values must be obtained for extreme values obtained from tests. This can be used for performance evaluation of the pavement constructed and should help contractors negotiate the variability in as-constructed pavement thickness. The results of the study lead to a conclusion that existing tolerances on layer thicknesses are sufficient for a two-layered structure (which can be extended to overlay design), whereas strict tolerances (variation of + 0.5 inches) should be imposed on a single asphalt concrete layer on unbound material. Thickness of asphalt concrete overlay on concrete pavement can be assigned the tolerances existing in state specifications. No tolerance suggestions are made for base course thicknesses, as base course variation does not significantly affect pavement distresses.

Surface shortwave absorptivity is a property of asphalt and concrete pavements, and is a measure of the amount of solar energy absorbed by the pavement surface (8). The lighter and more reflective the surface, the lower is the surface shortwave absorptivity. The suggested ranges for this value are:

- Aged PCC layer: 0.70-0.90
- Weathered asphalt (gray): 0.80-0.90
- Fresh asphalt (black): 0.90-0.98

Surface shortwave absorptivity is a parameter that affects thermal cracking of the pavement, and is also a term incorporated into the predictive equation for thermal crack length. Therefore, a value of 0.90 is suggested for new asphalt concrete layer design instead of 0.85 provided as default value by the design guide.

7.5 Material Inputs – Asphalt Material Inputs

Critical asphalt material inputs that are required for Level 2 and Level 3 design were identified from literature review. The following inputs were varied within the tolerances based on specifications for flexible pavement design for each state:

- Asphalt concrete layer thickness (studied in section above)
- Asphalt concrete mix – aggregate gradation
- Asphalt binder grade / binder properties – viscosity
- Air void content of asphalt concrete mixture
- Effective binder content of asphalt concrete mixture
- Coefficient of thermal contraction of AC mixture
- Average Tensile Strength of asphalt concrete mixture

Aggregate gradation for the mix for Superpave mix design as well as Marshal Design should be close to the mean values or finer than mean gradation. Gradation does not have a significant effect on prediction of pavement distresses; therefore the gradation provided in the job mix formula must conform to existing specifications for each state. A coarser mix leads to an insignificant increase in the predicted distresses, hence for each nominal size of the AC mix the gradation should be between the mean values and lower limits of the range for the sieve sizes required by the MEPDG.

The following sieve size percentages are required for Level 2 and Level 3 design:

- Percentage retained on 3/8 inch sieve
- Percentage retained on ¾ inch sieve
- Percentage retained on #4 sieve
- Percentage passing #200 sieve

Asphalt binder grade/ viscosity determination is subjected to the climate, traffic volume level and operational speed of the highway. In this study, only interaction between binder grade and operational speed was studied. PG grade determination for different climate and traffic conditions should be made as per the recommended value provided by LTPP Bind software (46). Recommendations made by Superpave specifications for asphalt pavement design (32) have been validated in this study, and are strongly suggested to be used for pavement design.

The low-temperature PG grade of asphalt binder is very important for prediction of thermal cracking in pavements.

New Hampshire: New Hampshire pavement structure resulted in the prediction of very high thermal cracking with pavement failure occurring at only 4 years for PG XX-22 grade and at 8 years for PG XX-28 grade. Therefore, a minimum low temperature PG grade of -28 is recommended for use for New Hampshire climate.

Connecticut: For Connecticut climate, almost no cracking was predicted for PG XX-28 binder grade and moderate level of thermal cracking predicted for PG XX-22. Therefore, it is recommended to use a -28 low temperature PG grade for projects of high importance and -22 is sufficient for low-importance roads.

Maine: Use of PG XX-22 binder grade resulted in failure of the pavement in thermal cracking, whereas the crack length remained under failure limit for PG XX-28 binder. Hence, a low temperature PG grade of -28 is recommended for design of asphalt concrete surface layer for the state of Maine.

Rhode Island: The state design specifications specify that the PG grade of asphalt binder should be minimum PG 64-28. This binder grade specification is sufficient for designing the pavement to resist thermal cracking.

The range of significance of effect of binder grade on different pavement distresses is shown in Table 149.

Table 149 Range of Sensitivity of Asphalt Binder Grade on Pavement Distresses

Pavement Distress	Sensitivity Range - Speed
Fatigue Cracking	Insensitive
Longitudinal Cracking	Insensitive – Low
AC Layer Rutting	Insensitive – Medium
Total Rutting	Insensitive – Medium
IRI	Insensitive – Low

Since asphalt binder grade and operational speed together affect pavement distresses very significantly, appropriate binder grade should be selected according to the operational speed on the highway.

Air void content of the asphalt concrete layer significantly affects thermal cracking, and significance of its effect on other pavement distresses is low. The range of sensitivity of effect of air voids on different distresses is shown in Table 150.

Table 150 Range of Sensitivity of Air Void Content on Pavement Distresses

Pavement Distress	Sensitivity Range - Speed
Fatigue Cracking	Insensitive – Low
Longitudinal Cracking	Insensitive – Medium
AC Layer Rutting	Insensitive – Low
Total Rutting	Insensitive – Low
IRI	Insensitive

Therefore, the existing tolerances on air voids of asphalt pavement are sufficient to obtain correct prediction of pavement distresses.

Effective binder content of the AC mixture is required by the MEPDG for flexible pavement design, instead of the actual binder percentage by weight of the mix. The study results show that pavement distresses are not sensitive to changes in effective binder content; therefore existing tolerance on percentage binder content may be retained and the effective binder content should be calculated from other volumetric parameters to be used in the MEPDG.

Coefficient of thermal contraction and average tensile strength are important parameters that influence thermal cracking of a flexible pavement. Thermal cracking prediction is very highly sensitive to changes in coefficient of thermal contraction and average tensile strength of the asphalt concrete mix. Aggregate coefficient of thermal contraction does not have any effect on predicted distresses, as was found from a number of trial runs for a wide range of values. Therefore, it is recommended that laboratory tests should be performed on the AC mixture to determine these properties.

7.6 Material Properties – Unbound Layer Inputs

Subgrade and base course type and material properties were also used as input parameters in this study. The various types of subgrade materials found in each state were obtained from a study on subgrade soils in New England states (33). Base course type was not found to significantly affect pavement distresses, but subgrade type had effect of low significance in prediction of fatigue cracking and total rutting. The range of variability of significance of effect of subgrade on various pavement distresses is given in Table 151.

Table 151 Range of Sensitivity of Subgrade on Pavement Distresses

Pavement Distress	Sensitivity Range - Speed
Fatigue Cracking	Insensitive – Low
Longitudinal Cracking	Insensitive – Low
AC Layer Rutting	Insensitive
Total Rutting	Insensitive – Low
IRI	Insensitive – Low

Base course materials found in each state should be documented in a database along with the strength properties – the allowable properties for design using MEPDG are:

- Resilient Modulus (measured in psi)
- CBR value
- R – value
- AASHTO layer Coefficient – ai
- Penetration value (from dynamic cone penetration test)
- Gradation and plasticity index of the subgrade

Table 152 provides a summary of correlations the Design Guide adopts to estimate modulus from other material properties that can be input in level 2 (8).

Table 152 Subgrade Modulus Predictive Equations

Strength/Index Property	Model	Comments	Test Standard
CBR	$M_r = 2555(CBR)^{0.64}$	CBR = California Bearing Ratio, percent	AASHTO T193—The California Bearing Ratio
R-value	$M_r = 1155 + 555R$	R = R-value	AASHTO T190—Resistance R-Value and Expansion Pressure of Compacted Soils
AASHTO layer coefficient	$M_r = 30000\left(\frac{a_i}{0.14}\right)$	a_i = AASHTO layer coefficient	AASHTO Guide for the Design of Pavement Structures (1993)
PI and gradation*	$CBR = \frac{75}{1+0.728(wPI)}$	wPI = P200*PI P200= percent passing No. 200 sieve size PI = plasticity index, percent	AASHTO T27—Sieve Analysis of Coarse and Fine Aggregates AASHTO T90—Determining the Plastic Limit and Plasticity Index of Soils
DCP*	$CBR = \frac{292}{DCP^{1.12}}$	CBR = California Bearing Ratio, percent DCP =DCP index, in/blow	ASTM D6951—Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications

Level 2 values for subgrade and base course layers are recommended for design using MEPDG. The state specifications do not contain tolerances on the allowable range of resilient modulus values for each soil type; hence a database of material properties should be developed for the available soil types in each state. Since the resilient modulus is calculated by the MEPDG from parameters that can be determined from simple tests like CBR, it is recommended to implement Level 2 for subgrade and base course properties.

The various subgrade types present in the New England states based on AASHTO classification are listed below (33).

Table 153 Typical Subgrade Types present in New England states

State	Subgrade Types
New Hampshire	A-1, A-2, A-4
Connecticut	A-2,A-4
Maine	A-1, A-2, A-3,A-4,A-5, A-6
Rhode Island	A-1-b,A-3
Massachusetts	A-1, A-2, A-3,A-4,A-5, A-6
Vermont	A-1, A-2, A-4, A-6,A-7

The referenced report does not contain values directly usable for MEPDG flexible design. The graphical data presented in the report were digitized to obtain weighted average values of resilient modulus of different subgrade soil types and are tabulated below.

Table 154 Typical Subgrade Types – Laboratory Measured Resilient Modulus Values

Subgrade Type	Resilient Modulus (MPa)	Resilient Modulus (psi)
A-1-b	83.9	12168
A-2-4	79.5	11530
A-2-6	85.9	12458
A-3	78.8	11430
A-4	87.25	12655
A-6	97.2	14097
A-7	93.25	13524

7.7 General observations for the MEPDG implementation

Implementation of the MEPDG requires:

a) Time and agency resources (staffing, training, testing facilities and equipment).
b) Establishment of performance criteria against which the design evaluation can be measured.

c) Validation of the MEPDG nationally calibrated pavement distress and smoothness prediction models with current state conditions.
d) Local calibration as may needed.

An example of an implementation plan which can be use by state highway agencies:

1. Form an Implementation Team and develop a communication plan
2. Establish a set of performance criteria against which design evaluations can be measured. These criteria may be stratified to reflect different levels of traffic, different levels of functional class, etc.
3. Set recommend MEPDG input levels, required resources, and obtain necessary testing equipment
4. Conduct sensitivity analysis of MEPDG inputs
5. Develop and populate a central database with required MEPDG input values.
6. Conduct staff training
7. Develop a formal state specific MEPDG-related documentation
8. Resolve differences between the MEPDG predicted distresses and distresses collected in the field
9. Calibrate and validate MEPDG performance prediction models to local conditions
10. Define long-term plan for adopting the MEPDG design procedure
11. Develop a design catalog.

The benefits of implementing MEPDG are:

a) Achieving the more cost effective and reliable pavements designs
b) Lower initial and life cycle cost to the agency
c) Reduced highway user impact due to less lane closures for maintenance and rehabilitation of pavements

7.8 Recommendations for Future Work

Based on the findings of this study, the following recommendations are made for future work:

- Confirm results using the new version of the software (Darwin-ME), with particular attention on the thermal cracking predictions.
- Improve interactions and data sharing between state highway agencies and researchers, (i.e., academia) to benefit future studies (knowledge of states specific issues, implementation plans, founding's, local calibrations, etc.,)
- The MEPDG predicted pavement distresses should be validated against the recorded measurements by each of the state highway agencies covered by this research.
- Reevaluate the ground water table affect on pavement performance predictions, due to suspect findings in this research.

- Investigate the interaction between asphalt binder grades and traffic level.
- Investigate the interactions between asphalt binder grades and climate.
- Investigate unbound layer thickness effect on predicted pavement distresses for base and subbase.
- Compare summary resilient modulus values to average resilient modulus values for unbound layers.
- Compare affect of base and subbase on pavement distress predictions (as an example: rock base/sand subbase).
- Investigate how the MEPDG ground water table values relate to unbound M_r values.
- Investigate how realistic is ranking of vehicle speed as a variable for pavement performance predictions.
- Perform the MEPDG Level 1 sensitivity analysis for the New England States and New York.

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APPENDIX

Subgrade Resilient Modulus: Level 3 Design

The following table provided in the Mechanistic-Empirical Pavement Design Guide contains recommended values for subgrade resilient modulus for Level 3 design.

Material Classification	M_R Range	Typical M_R*
A-1-a	38,500 – 42,000	40,000
A-1-b	35,500 – 40,000	38,000
A-2-4	28,000 – 37,500	32,000
A-2-5	24,000 – 33,000	28,000
A-2-6	21,500 – 31,000	26,000
A-2-7	21,500 – 28,000	24,000
A-3	24,500 – 35,500	29,000
A-4	21,500 – 29,000	24,000
A-5	17,000 – 25,500	20,000
A-6	13,500 – 24,000	17,000
A-7-5	8,000 – 17,500	12,000
A-7-6	5,000 – 13,500	8,000
CH	5,000 – 13,500	8,000
MH	8,000 – 17,500	11,500
CL	13,500 – 24,000	17,000
ML	17,000 – 25,500	20,000
SW	28,000 – 37,500	32,000
SP	24,000 – 33,000	28,000
SW-SC	21,500 – 31,000	25,500
SW-SM	24,000 – 33,000	28,000
SP-SC	21,500 – 31,000	25,500
SP-SM	24,000 – 33,000	28,000
SC	21,500 – 28,000	24,000
SM	28,000 – 37,500	32,000
GW	39,500 – 42,000	41,000
GP	35,500 – 40,000	38,000
GW-GC	28,000 – 40,000	34,500
GW-GM	35,500 – 40,500	38,500
GP-GC	28,000 – 39,000	34,000
GP-GM	31,000 – 40,000	36,000
GC	24,000 – 37,500	31,000
GM	33,000 – 42,000	38,500

NH Traffic Volume Count Data (28)

Year 1981 to 2007, New Hampshire – Section ID 99103

Year	AADT	AADTT	Year	AADT	AADTT
1981	10200	836	1995	32197	2640
1982	11902	976	1996	33612	2756
1983	14282	1171	1997	34467	2826
1984	17280	1417	1998	35773	2933
1985	18581	1524	1999	36767	3015
1986	20732	1700	2000	37793	3099
1987	23046	1890	2001	38477	3155
1988	24149	1980	2002	35674	2928
1989	26102	2140	2003	NA	NA
1990	26773	2195	2004	41050	3366
1991	26566	2178	2005	41000	3362
1992	27332	2241	2006	40709	3338
1993	28937	2373	2007	41000	3362
1994	30161	2473			

Ground Water Table Depth – New Hampshire (5)

COUNTY	BELKNAP	CARROLL			CHESHIRE
Month	BAW 10	OXW 38	ADW 15	ADW 14	KEW 2
	S Barnstead	Parsonsfield	Chocorua		Keene
Jan	2.68	35.52	7.77	5.95	3.50
Feb	2.78	35.61	8.42	6.56	3.17
Mar	2.33	35.51	8.33	6.43	2.08
Apr	2.13	34.48	5.09	3.95	2.59
May	2.31	34.25	6.73	5.06	3.29
Jun	2.85	34.51	7.69	5.88	3.95
Jul	3.00	34.93	8.45	6.51	4.52
Aug	3.21	35.41	8.72	7.01	4.57
Sep	3.15	35.87	9.39	6.92	4.57
Oct	2.82	35.69	7.86	6.17	3.80
Nov	2.74	35.54	7.58	5.79	3.01
Dec	2.55	35.52	7.56	5.85	3.17

COUNTY	COOS			
Month	SJW 2	LCW 1	ETW 1	CTW 73
	Berlin	Lancaster	Errol	Colebrook
Jan	4.38	1.39	13.21	6.39
Feb	4.73	1.34	13.19	6.92
Mar	4.62	0.76	13.15	7.26
Apr	3.57	0.63	12.01	6.70
May	3.87	1.14	11.80	7.22
Jun	4.40	1.85	12.08	7.24
Jul	4.86	2.14	12.41	7.68
Aug	5.14	2.15	12.77	7.64
Sep	5.09	2.21	12.86	7.49
Oct	3.99	1.83	12.87	7.50
Nov	3.88	1.65	12.98	7.43
Dec	4.28	1.51	13.01	6.92

* - Values in feet from ground surface

COUNTY	GRAFTON			
Month	ENW 30	CBW 34	LLW 19	BSW 44
	Enfield	Lisbon	Littleton	Compton Hollow
Jan	5.83	12.55	12.35	-
Feb	5.85	12.84	12.80	-
Mar	5.34	12.51	12.68	-
Apr	2.29	10.92	12.31	-
May	2.72	11.42	13.21	-
Jun	3.79	12.27	13.83	-
Jul	5.37	13.00	14.25	-
Aug	6.62	13.61	14.61	22.04
Sep	7.44	13.58	14.40	20.11
Oct	6.86	12.82	13.63	-
Nov	5.91	12.35	13.50	-
Dec	5.66	12.33	12.87	-

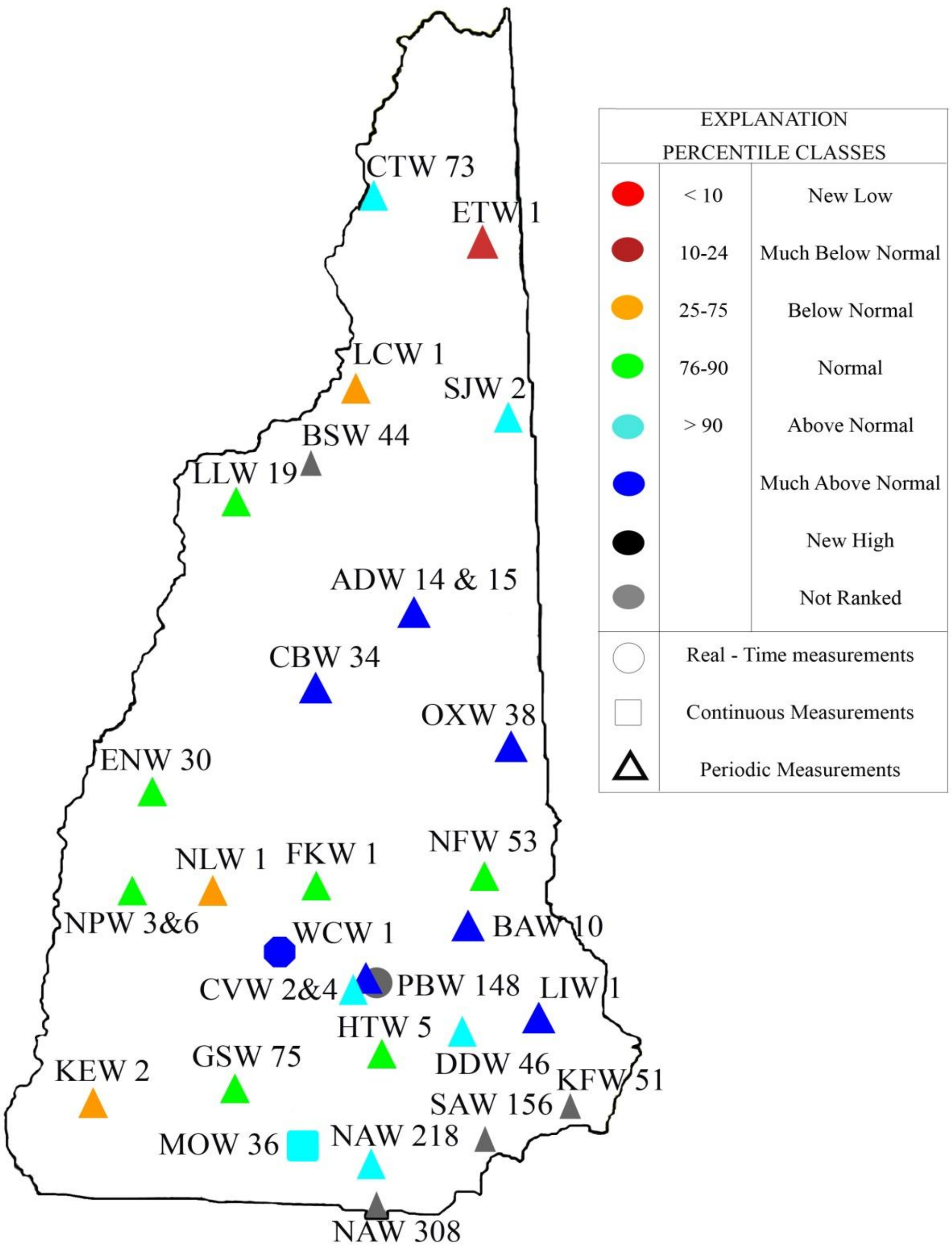
COUNTY	HILLSBOROUGH			
Month	NAW 308	NAW 218	MOW 36	GSW 75
	Sky Meadow Club	Bowers Pond	Milton – West	Greenfield St. Park
Jan	-	28.27	7.62	62.46
Feb	-	28.20	7.46	62.10
Mar	-	27.82	6.97	61.97
Apr	-	27.08	7.03	61.14
May	-	27.15	7.46	59.88
Jun	-	27.52	7.90	60.18
Jul	-	28.05	8.47	60.43
Aug	10.67	28.52	8.85	60.68
Sep	12.00	28.91	8.98	61.45
Oct	-	28.91	8.55	61.84
Nov	-	28.58	7.92	62.18
Dec	-	28.33	7.55	62.15

COUNTY	MERRIMACK						
Month	HTW 5 Merrimack River (W)	CVW 4 Cilley St. Forest	PBW 148 Pembroke	CVW 2 Airport Rd Concord	WCW 1 Warner	NLW 1 Old Coach Road	FKW 1 Webster Lake
Jan	47.65	17.74	8.44	41.14	30.84	9.20	13.12
Feb	47.82	17.71	8.91	41.07	30.68	9.00	12.96
Mar	46.98	17.18	8.13	41.15	30.40	6.79	12.46
Apr	45.80	16.10	7.12	40.73	28.82	4.58	10.82
May	46.33	16.13	7.51	40.80	28.20	6.29	10.35
Jun	46.95	16.65	7.97	40.55	28.72	8.34	10.61
Jul	48.06	17.42	8.91	40.60	29.62	10.36	11.31
Aug	48.71	17.91	9.22	40.77	30.42	11.68	12.18
Sep	49.30	18.32	9.96	40.81	31.16	12.60	12.82
Oct	49.16	18.21	9.13	40.97	31.38	12.14	13.27
Nov	48.89	17.94	9	41.01	31.26	10.82	13.34
Dec	48.01	17.62	8.41	41.04	31.18	9.16	13.05

COUNTY	ROCKINGHAM			STRAFFORD		SULLIVAN	
Month	SAW 156 Shanning Rd*	KFW 51 Kensington	DDW 46 Raymond Rd	LIW 1 Lee	NFW 53 New Durham	NPW 3	NPW 6 Newport
Jan	-	-	38.77	30.98	19.01	5.55	5.57
Feb	-	-	38.62	31.02	18.99	5.64	5.67
Mar	-	-	38.47	30.55	18.83	5.19	5.04
Apr	-	-	37.92	30.56	18.62	4.85	4.22
May	-	-	37.91	30.59	18.71	5.28	4.93
Jun	-	-	38.11	30.87	19.14	5.93	5.50
Jul	18.66	5.70	38.45	31.19	19.37	6.47	6.04
Aug	-	-	38.86	31.34	19.73	7.13	6.65
Sep	-	6.62	39.16	31.51	19.85	7.30	6.81
Oct	-	-	39.15	31.31	19.38	6.62	6.18
Nov	15.33	-	39.02	31.17	19.09	6.23	5.80
Dec	-	-	38.91	31.04	19.05	5.60	5.54

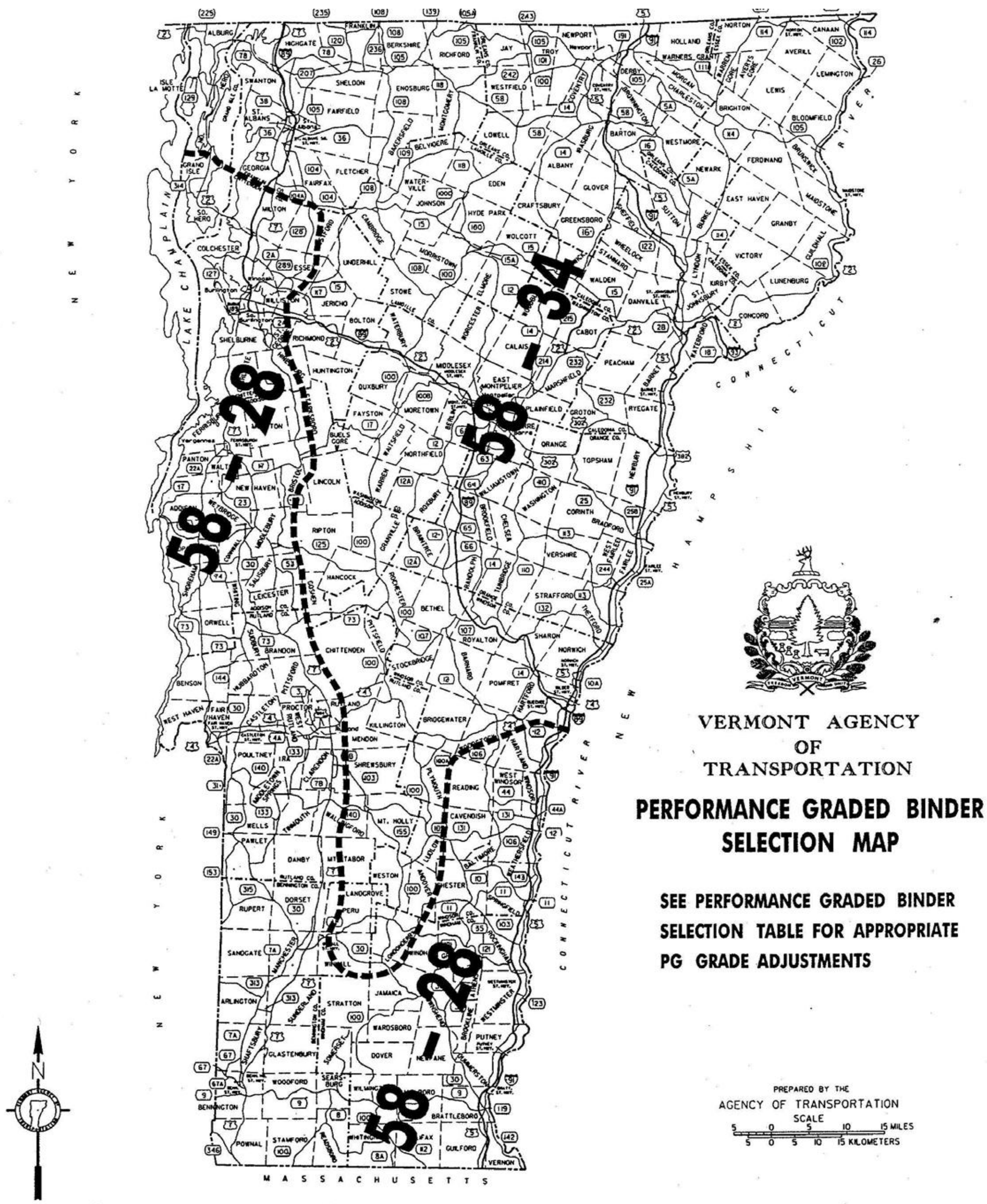
* - near Atkinson Country Club and Resort

New Hampshire Distribution of Water Table Depth



Reference: Edited from Google© Earth Information Map

Vermont Performance Graded Binder Selection Map



Performance Graded Binder Selection Table (Vermont)

Performance Graded Binder Selection Table

Adjusted PG Binder
on the Basis of Traffic Speed and Traffic Level

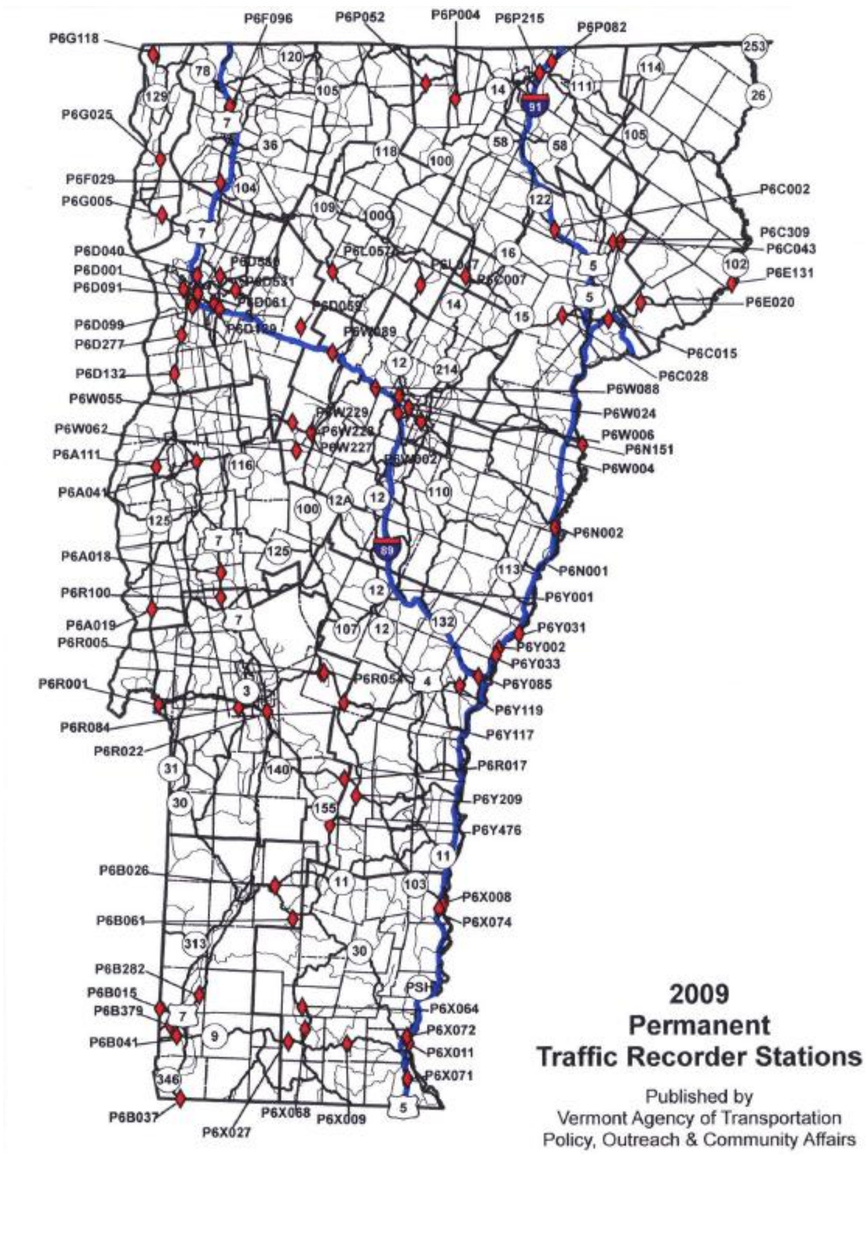
Design ESALs(1) (million)	Adjusted PG Binder Grade		
	Average Traffic Speed		
	< 20 km/h (12 mph)	20 to 70 km/h (12 to 44 mph)	> 70 km/h (44 mph)
< 0.3	PG 58-XX(2)	PG 58-XX	PG 58-XX
0.3 to < 3	PG 64-XX	PG 58-XX	PG 58-XX
3 to < 10	PG 70-28 (3)	PG 64-XX	PG 58-XX
10 to < 30	PG 70-28 (3)	PG 64-XX	PG 64-XX
> 30	PG 70-28 (3)	PG 64-XX	PG 64-XX

(1) Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period, regardless of the actual design life of the roadway.

(2) XX indicates the low temperature of the selected PG Binder determined from the Performance Graded Binder Selection Map, either -28 or -34.

(3) When the high-end temperature is adjusted two grades to a 70, the low-end temperature needs to be changed to a -28 if the selected PG binder is a PG 58-34. If selected PG binder is a PG 58-28, then no change to the low-end temperature is needed when changing the high-end temperature two grades to 70.

Permanent Traffic Recorder Stations (Vermont)



Performance Graded Binder Selection – Standard (New York)

Performance Graded Binder Selection - Standard

Location	Location by Counties	Performance Grade (Spec Number)
Upstate	All Other Counties Not Listed Under Downstate	64-22[1] (702-6422)
Downstate	Orange, Putnam, Rockland, Westchester, Nassau, Suffolk Counties and City of New York	70-22 (702-7022)

1. For high volume roadways in Dutchess County, PG 70-22 or PG 76-22 may be specified with the concurrence of the Regional materials Engineer.

Performance Graded Binder Selection – Polymer Modified (New York)

Performance Graded Binder Selection – Polymer Modified

Conditions for Use	Location	Performance Grade (Spec Number)[1]
Cold temperature data warrants its use with the concurrence of the Regional Materials Engineer. Typically Adirondack Region.	Jefferson, Lewis, St. Lawrence, Franklin, Clinton, Essex, and the Northern Sections of Herkimer, Oswego, Hamilton, Warren, and Washington Counties	58-34 (702-5834)
Multiple course overlays, reconstruction, or new construction where cold temperature data warrants its use with the concurrence of the Regional Materials Engineer.	Upstate	64-28 (702-6428)
Multiple course overlays, reconstruction, new construction or roadway segments containing (a) grades in excess of 4.0% or (b) intersections that have traffic control signals (3 light signal) with the concurrence of the Region Materials Engineer.	Upstate	64-22 (702-6422)
Where the traffic level is greater than 30 million ESALs based on a 20-year design life or the roadway segment contains (a) grades in excess of 4.0% or (b) intersections that have traffic control signals (3 light signal).	Downstate	76-22 (702-7622)

1. Other PG binder grades may be specified in a given location with approval from the Regional Materials Engineer and the Materials Bureau.

Specification for Hot Mix Asphalt (Massachusetts)

Specifications for Hot Mix Asphalt
Percent by Weight Passing Sieve Designation

Sieve Designation and % Binder Content	HMA Base Course	HMA Base/ Intermed. Course - Binder	HMA Intermed. Course Dense Binder	HMA Surface Course - Dense Binder	HMA Surface Course – Standard Top	HMA Surface Course - Modified Top	HMA Dense Mix	HMA Surface Treatment	HMA OGFC
2 inches	100								
1 inch	57 – 87	100	100	100		100			
¾ inch		80 - 100	80 – 100	80 – 100		95 - 100			
5/8 inch					100				
½ inch	40 - 65	55 - 75	65 – 80	65 – 80	95 - 100	79 – 100	100		100
3/8 inch					80 - 100	68 – 88	80 - 100	100	90 –
No. 4	20 - 45	28 - 50	48 – 65	48 – 65	50 - 76	48 – 68	55 - 80	80 - 100	30 – 50
No. 8	15 - 33	20 - 38	37 – 49	37 – 49	37 - 49	33 – 46	48 - 59	64 - 85	5 – 15
No. 16					26 - 40	20 – 40	36 - 49	46 - 68	
No. 30	8 - 17	8 - 22	17 – 30	17 – 30	17 - 29	14 – 30	24 - 38	26 - 50	
No. 50	4 - 12	5 - 15	10 – 22	10 – 22	10 - 21	9 – 21	14 - 27	13 - 31	
No. 100					5 - 16	6 – 16	6 - 18	7 - 17	
No. 200	0 - 4	0 - 5	0 – 6	0 – 6	2 - 7	2 – 6	4 - 8	3 - 8	1 – 3
Binder	4 - 5	4.5 - 5.5	5 – 6	5.1 – 6	5.6 - 7.0	5.1 – 6	7 - 8	7 - 8	6 – 7

Engineering Limits for HMA Aggregate Gradation and PG Binder Content (Massachusetts)

Engineering Limits for HMA Aggregate Gradation and PG Binder Content

Sieve Designation / Binder Content	Engineering Limit for OGFC	Engineering Limit for all other mixes
Passing No. 4 sieve and larger sieve sizes	JMF Target ± 5%	JMF Target ± 7%
Passing No. 8 to No. 100 sieves (inclusive)	JMF Target ± 3%	JMF Target ± 4%
Passing No. 200 sieve	JMF Target ± 1%	JMF Target ± 2%
Binder	JMF Target ± .3%	JMF Target ± 0.4%

MassDOT Hot Mix Asphalt Formulas (Massachusetts)

PLANTS	ID#	Batch/Drum Size		Automatic Controls (X)			PG BINDER TANKS		MIX SILOS, Insul/Heat (X)			
TO BE	___	___Ton/	___Tph	part___	full__,w/	prntr___	____@	__________ gals.	____@	____Ton	,Ins____	/Htd____
USED:	___	___Ton/	___Tph	part___	full__,w/	prntr___	____@	__________ gals.	____@	____Ton	,Ins____	/Htd____
	___	___Ton/	___Tph	part___	full__,w/	prntr___	____@	__________ gals.	____@	____Ton	,Ins____	/Htd____

ALLOWABLE TOLERANCES		
Sieve Designation/ Binder Content	Engineering Limit all mixes	Engineering Limit OGFC
Passing # 4 sieve and larger sieve sizes	JMF Target ± 7 %	JMF Target ± 5 %
Passing # 8 to # 100 sieve sizes (inclusive)	JMF Target ± 4 %	JMF Target ± 3 %
Passing #200	JMF Target ± 2 %	JMF Target ± 1 %
% Binder	JMF Target ± 0.4%	JMF Target ± 0.3 %

COMPONENT MATERIALS

COARSE AGGREGATE	Nom. Sizes		Producer & City	
FINE AGGREGATE BLEND	%	Screenings	Producer & City	
	%	Stone Sand		
	%	Natural Sand		
RAP AMT. FOR:	Base	=______%, ________,w/PG_______ ,modifier	MINERAL FILLER, % and Kind:______	PG BINDER/MODIFIER, Grade & Source
	Intermediate	=______%, ________,w/PG_______ ,modifier		
	Surface	=______%, ________,w/PG_______ ,modifier		
SPECIAL MATL:			ANTI-STRIP:	% of Bit., Kind:
OGFC:	Polymer	% and Kind:	SILICONE:	oz. Per gals.

NOTE LIMITATIONS (Unless Design Data Approved)
Unless authorized by the Engineer, no Job-Mix Formula will be approved which specifies:
* Less than 6% binder for HMA Surface Course-Standard Top
** Less than 5.5% binder for HMA Surface Course-Dense Binder and Modifiied Top for mixes containing RAP

FORMULAS (MR=Master Range of Specifications; JM=Job Mix Formula)
Aggregate percentages below are proportional percentages of total aggregate for the mix.

Sieve	HMA Base Course BASE		HMA Base/ Intermediate Course BINDER		HMA Intermediate Course DENSE BINDER		** HMA Surface Course DENSE BINDER		* HMA Surface Course STANDARD TOP		** HMA Surface Course MODIFIED TOP		HMA DENSE MIX		HMA Surface Treatment		HMA OGFC			
Size	MR	JM	MR	JM	MR	JM	MR	JM	MR	JM	MR	JM	MR	JM	MR	JM	MR	JM	MR	JM
2 "	100		\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\		
1 "	57-87		100		100		100		\\\\\\	\\\\\\	100		\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\		
3/4"	\\\\\\		80-100		80-100		80-100		\\\\\\	\\\\\\	95-100		\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\		
5/8"	\\\\\\		\\\\\\		\\\\\\		\\\\\\		100		\\\\\\		\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\	\\\\\\		
1/2"	40-65		55-75		65-80		65-80		95-100		79-100		100		\\\\\\	\\\\\\	100			
3/8"	\\\\\\		\\\\\\		\\\\\\		\\\\\\		80-100		68-88		80-100		100		90-100			
# 4	20-45		28-50		48-65		48-65		50-76		48-68		55-80		80-100		30-50			
# 8	15-33		20-38		37-49		37-49		37-49		33-46		48-59		64-85		5-15			
# 16	\\\\\\		\\\\\\		\\\\\\		\\\\\\		26-40		20-40		36-49		46-68		\\\\\\			
# 30	8-17		8-22		17-30		17-30		17-29		14-30		24-38		26-50		\\\\\\			
# 50	4-12		5-15		10-22		10-22		10-21		9-21		14-27		13-31		\\\\\\			
# 100	\\\\\\		\\\\\\		\\\\\\		\\\\\\		5-16		6-16		6-18		7-17		\\\\\\			
# 200	0-4		0-5		0-6		0-6		2-7		2-6		4-8		3-8		1-3			
%Bind	4.0-5.0		4.5-5.5		5.0-6.0		5.1-6.0		5.6-7.0		5.1-6.0		7.0-8.0		7.0-8.0		6.0-7.0			
Max.Theo	Sp. Gr.		\\\\\\		\\\\\\		\\\\\\		\\\\\\		\\\\\\		\\\\\\		\\\\\\		\\\\\\			

MEPDG USAGE SUGGESTIONS

This section provides the MEPDG user with useful suggestions and a description of problems and their solutions encountered by the research team while conducting runs on the software. MEPDG is relatively simple software but there are some precautions to be taken while creating input files.

1. The MEPDG software must be installed directly onto the drive where the operating system is installed. If the drive C contains the operating system, then MEPDG software must be installed to the address C:\DG2002 and not C:\Program Files\DG2002. The software will not function properly if it is installed elsewhere on the hard disk drive.
2. Traffic inputs are not impeded by input bugs; hence data can be modified without any precautionary measures.
3. Climate station selection is subjected to data acquisition bug, as difficult was experienced by the research team while conducting runs for different climate files. If the climate data alone is to be altered in an already existing MEPDG input file, simply selecting a new climate station will not change the climate data already acquired by the input file. A new file must be created with the exact set of input data and a new climate file must be selected.
   Example: The control climate file for New Hampshire was created using Concord climate station data. All other parameters were kept constant, and Berlin climate data was loaded in the climate module. The predicted distress data was found to be exactly similar to the Concord design run, indicating that the previous data had been retained and not changed to Lebanon data.
4. Water table depth is a parameter that must be entered at the time of creation of a climate file. Therefore, the same procedure suggested above should be adopted for changing only the water table depth value. A new climate file with the changed water table depth must be created after the creation of a new input file with all other data remaining the same.
5. Use of G* and sin δ values for binder data in the asphalt binder section is necessary for Level 2 design. If thermal cracking is the parameter to be predicted in Version 0.91 of the software, the entry of values G* and sin δ leads to erroneous values of creep compliance in the thermal cracking module fields. It was found that all values of creep compliance, as well as the average tensile strength field are replaced by '#.INF' value. Therefore, the PG grade corresponding to the binder must first be entered at Level 3, such that MEPDG loads the corresponding creep compliance and tensile strength values from its in-built database, and the input file should be saved. The data level for binder grade should then be changed to Level 2 so that the G* and sin δ values may be entered.