UNIVERSITY OF SOUTH CAROLINA

MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE IMPLEMENTATION

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and
The Federal Highway Administration

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The recently introduced Mechanistic-Empirical Pavement Design Guide (MEPDG) and associated computer software provides a state-of-practice mechanistic-empirical highway pavement design methodology. The MEPDG methodology is based on pavement responses computed using detailed traffic loading, material properties, and environmental data. The responses are used to predict incremental damage over time. Design is an iterative process using analysis results based on trial designs postulated by the designer. A trial design is analyzed for adequacy against user input performance criteria. These criteria are established by policy decisions and represent the amount of distress or roughness that would trigger some major rehabilitation or reconstruction activity. The output of the computer software is a prediction of distresses and smoothness against set reliability values. If the predictions do not meet the desired performance criteria at the given reliability, the trial design is revised and the evaluation is repeated. The MEPDG method provides for three hierarchical levels of design inputs to allow the designer to match the quality and level of detail of the design inputs to the level of importance of the project (or to best utilize available input data). In addition to inputs required to quantify a trial pavement structure, the MEPDG requires over 100 inputs to characterize traffic loading, material properties, and environmental factors.

Currently, the South Carolina Department of Transportation (SCDOT) designs flexible and rigid pavement structures using AASHTO regression equation methodology (1972 and later with some modifications). Implementation of the MEPDG will require a substantial effort. This report summarizes an initial study undertaken to 1) gain an understanding of the new methodology, required inputs, and limitations, 2) conduct preliminary input sensitivity studies and review sensitivity studies performed by others, and 3) summarize implementation strategies undertaken or planned at other state highway agencies. Based on this investigation, general recommendations for SCDOT MEPDG implementation are proposed.</td></tr>
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Chapter 1

Introduction

Background Information

This report summarizes an investigation conducted to provide preliminary guidance to the South Carolina Department of Transportation (SCDOT) in their upcoming effort to adopt new highway pavement design procedures set forth in the *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, Final Report* (NCHRP, 2004), now referred to as the *Mechanistic-Empirical Pavement Design Guide* (MEPDG). Documentation of the new design methodology appears in *Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition* (AASHTO, 2008). The most recent version for the MEPDG software (Version 1.10, September 2009) is available through the Transportation Research Board (TRB) website. AASHTO has initiated an effort to develop updated and more user-friendly interactive software (AASHTO DARWin-ME^{TM}). This software is not expected to be available before mid-2011.

For the design of new pavement structures, the SCDOT currently uses the well known and still widely used empirical equations based on the regression analysis of 1958-1961 AASH(T)O Road Test data. These equations are documented in the AASH(T)O *Interim Guide for the Design of Pavement Structures* (1972) and the *AASHTO Guide for Design of Pavement Structures* (1986 and 1993). As used by the SCDOT, the design of flexible pavement structures requires inputs that include soil support value, design lane equivalent single axle loads (ESAL), regional factor (assumed to be 1.0), terminal serviceability value, and material layer coefficients.

For the design of rigid pavement structures, input requirements include a modulus of subgrade reaction, ESAL, PCC modulus of rupture and elasticity, load transfer coefficient, and serviceability values. Design computations are simple, the design inputs are few and relatively easy to obtain, and years of experience with the method has produced a generally high level of confidence in the resulting designs. However, problems arise with increasing traffic levels and when new pavement materials are introduced to the pavement designs. The procedure produces conservative results that are not optimally cost-effective. Comprehensive lists of the inherent shortcomings of AASHTO Guide designs are given in *Pavement Lessons Learned from AASHO Road Test and Performance of Interstate Highway System, Circular E-C118* (TRB, July 2007) and *Mechanistic-Empirical Pavement Design Guide. A Manual of Practice, Interim Edition* (AASHTO, 2008).

The new MEPDG methodology evolved from NCHRP Project 1-37A, *Development of the 2002 AASHTO Guide for Design of New and Rehabilitated Pavement Structures: Phase II*. The project was awarded in 1996 and the objective was to develop a design methodology that utilizes state-of-the-art mechanistic-based models and databases relevant to the current state-of-knowledge of highway pavement performance. MEPDG methodology is based on software-generated pavement responses (stresses, strains, and deflections) computed using detailed traffic loading, material properties, and environmental data. The responses are used to compute incremental damage over time. Using currently available software, design is an iterative process based on analysis software results for trial pavement structures proposed by the designer. A trial design is analyzed for adequacy against input performance criteria. The output of the analysis software is a prediction of distresses and smoothness against set reliability values. If the predictions do not meet the desired performance criteria at the given reliability, the trial design is revised and the evaluation is repeated. MEPDG methodology provides for three hierarchical levels of design inputs to allow the designer to match the quality and level of detail of the design inputs to the level of importance of the project (or to best utilize available input data). MEPDG procedures allow the designer to control incremental adjustments to the pavement structure and the specification of each performance criterion used in the design process. To predict performance over a design life, the MEPDG method uses over 100 inputs to model traffic loading, material properties, and environmental factors. In additional, detailed climatic data (including hourly temperature, precipitation, wind speed, relative humidity, and cloud cover) from 851 weather stations across the United States are embedded in the MEPDG software. An *Enhanced Integrated Climatic Model* (EICM) uses climatic data to simulate internal pavement temperature, moisture, and freeze-thaw conditions as a function of time.

Outputs from individual analyses are levels of pavement distresses and smoothness, not required pavement layer thicknesses. Therefore, MEPDG procedures represent a major change in the way pavement design is performed. Successful implementation of MEPDG procedures requires an understanding of the new methodology and additional resources to quantify the multitude of inputs (such as, additional and/or new field and laboratory testing and data collection protocols). In addition, regional/local calibrations and experience to optimize the designs and instill confidence in the methodology are needed.

Currently, MEPDG procedures are calibrated using the U.S. LTPP database supplemented by data obtained from the Minnesota Road Research Project (Mn/Road) and other state and federal agency research projects (AASHTO, 2008). A comprehensive review of the MEPDG is given in *Independent Review of the Mechanistic-Empirical Pavement Design Guide and Software, Research Digest Results 307* (NCHRP, 2006). This 2006 review cites concerns related to the variability and *questionable* reliability of designs, states that the *soundness of the underlying engineering principles varies considerably*, and notes the need for *further calibration/validation work.* The review also lauds the MEPDG as a substantial and innovative *piece of research that, with further work being undertaken, could be developed into a powerful design tool.*

Project Objectives

The overall objective of this investigation was to gather sufficient information about the new MEPDG to provide useful preliminary implementation recommendations to the SCDOT. To meet this overall objective, a review of MEPDG documentation was conducted to develop an understanding of the new methodology. Secondly, a literature review was conducted to gather information on published technical reviews of MEPDG methodology, MEPDG input sensitivity studies conducted by others, and other state highway agency (SHA) MEPDG implementation efforts (already undertaking or planned). In addition, preliminary sensitivity analyses were conducted using MEPDG inputs for representative South Carolina conditions. These sensitivity studies allowed for familiarization with the MEPDG software (as currently available), assessment of currently available and currently unavailable but desirable inputs, and provided data for some general observations regarding input sensitivity. Sensitivity results were compared to sensitivity studies reported by others and provided guidance for establishing priorities for new or alternative input data collection methodologies or research programs needed for implement the MEPDG in South Carolina.

Chapter 2

MEPDG Overview

MEPDG Design Approach

The general design approach applied by the MEPDG introduces a drastic change from previous pavement design methods. Instead of producing a required pavement structure, the MEPDG uses an initial assumption of a trial pavement structure to produce performance predictions. Inputs specifying the geometry of a trial pavement structure are needed along with traffic, climate, and materials inputs. MEPDG software computes pavement responses to load (stresses and strains) which are used to compute damage (distresses and loss in rideability) over time. The Enhanced Integrated Climate Model (EICM) uses climatic data to simulate changes in material properties caused by environmental factors. The design is performed in an iterative process. If the output of distress predictions exceeds a user specified desirable level, the assumed pavement structure is modified and the MEPDG performance predictions are repeated. The structural design is revised until the structure meets user specified performance criteria or the design engineer is satisfied. Ideally, the final step in producing a design using the MEPDG analysis software is considering all other reasonable alternative solutions. Ideally, to produce the optimum design, an engineering and life cycle cost analysis of alternative solution needs to be performed (NCHRP, 2004).

The multitude of inputs and computational capabilities in the MEPDG allows for a range of options in choosing a design strategy. Conventional flexible pavements, deep strength flexible pavements, full-depth hot mix asphalt (HMA) pavements, semi-rigid pavements, full depth reclamation, and HMA overlays comprise the HMA surface type options. Conventional flexible pavements, deep strength flexible pavements, and full-depth HMA pavements were the only pavements calibrated for the MEPDG. Jointed plain concrete pavement (JPCP), continuously reinforced concrete pavements (CRCP), JPCP overlays, CRCP overlays, and restoration of JPCP comprise the rigid pavement surface type options. All of the rigid pavement options were calibrated for the MEPDG.

Performance Indicators

When the design strategy is selected and all the information necessary for a trial structure is input into the program, MEPDG software analyzes the pavement's performance throughout its design life. The performance indicators in the software represent significant pavement distresses calculated by the software's structural response model and transfer functions. The structural response model calculates the critical pavement responses through mechanistic models embedded within the software. Empirical transfer functions convert these critical pavement responses into performance indicators that are evaluated throughout the design life.

For HMA pavements, the performance indicators are longitudinal (surface-down) cracking, alligator (bottom-up) cracking, transverse (thermal fracture) cracking, and rutting. Fatigue fracture is included for chemically stabilized layers. For rigid pavement structures, the performance indicators may include mean joint faulting and load related transverse slab cracking for JPCP and punchouts for CRCP. Functional performance for all pavements is defined by time (pavement age) dependent pavement roughness quantified as a predicted International Roughness Index (IRI). IRI is predicted using a regression equation with computed pavement distresses, initial (as constructed) IRI, and "site/climate" factors as the primary independent variables.

For flexible pavement, alligator cracking is bottom-up fatigue cracking. Repeated loading causes cracks that begin at the bottom of the HMA layer to spread up to the surface. Bending of the HMA layer results in tensile stresses and strains developing cracks at the bottom of the layer. An increase in predicted alligator cracking may be due to: higher wheel loads and tire pressures, inadequate HMA layers for the predicted magnitude and repetitions of the loading, or weaknesses in base layers resulting from high moisture contents, soft spots, or poor compaction issues (NCHRP, 2004). Alligator cracking is calculated as a percent cracking of total lane area in the MEPDG.

Longitudinal cracking is surface-down fatigue cracking. Stresses and strains develop at the surface of the pavement due to tension from wheel loadings. These stresses and strains create and spread longitudinal cracking. Aging of the HMA surface creates a stiffness which worsens this effect. A shearing effect is created from tire contact pressure that can combine with the tension from the loading to create cracking. Longitudinal cracking is calculated as feet of cracking per mile in the MEPDG.

Transverse cracking is thermal cracking (thermal fracture). Transverse cracking is a non-load related cracking mechanism. These cracks are created because of asphalt hardening, seasonal and daily temperature differences, or exposure to constant cold weather. Transverse cracks on the pavement surface usually appear perpendicular to the pavement centerline (NCHRP, 2004). Transverse cracking is calculated as feet of cracking per mile in the MEPDG.

Rutting is calculated in the MEPDG as a permanent deformation in the pavement structure along the wheel path. The deformation is caused by a vertical compression in any or all of the pavement layers. This compression may be a result of traffic loading, poor compaction of any of the layers during the construction stage, or shearing of the pavement caused by the wheel loading (AASHTO, 2008). Rutting is calculated in inches of deformation in the MEPDG.

For JPCP, mean transverse joint faulting is a measurement of the differential deflection across a joint. This value can vary greatly from joint to joint so predicted faulting is the average faulting for all transverse joints in the pavement. This distress can be caused by repeated heavy axle loads, poor joint load transfer efficiency, free moisture below the PCC slab, erosion of the base, subbase, subgrade, or shoulder material, and upward curling of the slab (AASHTO, 2008). Faulting is calculated in inches.

Bottom-up transverse cracking is caused by a tensile bending stress at the bottom of the PCC slab, midway between two transverse joints. Repeated heavy axle loading and high positive temperature gradient (top of slab is warmer than bottom of slab) cause fatigue damage to occur along the bottom of the slab. This fatigue damage results in a transverse crack in the pavement that can spread to the surface. Bottom-up transverse cracking is combined with top-down transverse cracking and calculated as percent of slabs cracked.

Top-down transverse cracking is caused by fatigue damage at the top of the PCC slab. A high negative temperature gradient (bottom of slab is warmer than top of slab) combined with simultaneous high axle loading at opposite ends of a slab produces a high tensile stress at the top of the slab. This tensile stress produces a transverse crack initiating at the surface of the pavement. Top-down transverse cracking is combined with bottom-up transverse cracking and calculated as percent of slabs cracked.

For CRCP, the predicted number of medium and high-severity punchouts per mile is computed based on the number of predicted cracks, predicted mean crack width, etc.

Predicted International Roughness Index (IRI) is used to quantify the overall serviceability of the pavement design. The MEPDG predicts IRI (in inches/mile – average along both wheel paths) using an empirical function. Different empirical functions are used for flexible pavement structures, JPCP and CRCP.

Computational Methodology

For flexible pavement structures, the Jacob Uzan Layered Elastic Analysis (JULEA) multilayer elastic computer program is used to calculate the pavement responses needed for distress predictions. For rigid pavements, the finite element analysis program ISLAB2000 is used to compute needed pavement responses.

Pavement responses are converted to distress predictions through transfer functions in the software. These distress predictions are calibrated using data from existing pavement databases, with most of the information coming from the long term pavement performance (LTPP) database. This database provides long term data analysis for a wide range of structures containing a variety of material, traffic, and environmental conditions across the country. None of the LTPP sites used for MEPDG calibration are located in South Carolina. The MEPDG provides the option to adjust the calibration factors for the distress prediction functions if there are agency specific regression constants or local data sets available. NCHRP Project 1-40b: *User Manual and Local Calibration Guide for the Mechanistic-Empirical Pavement Design Guide and Software* was completed in February 2009 and is being reviewed by an AASHTO Joint Technical Committee.

MEPDG Flexible Performance Prediction Equations

For flexible pavement designs, the MEPDG divides the structural layers of the design into sublayers. The JULEA program then calculates the critical responses in each sublayer. The EICM is used to adjust the pavement layer modulus values with time through hourly changes in temperature and moisture conditions. A dynamic modulus is calculated as a function of time and depth in the HMA layers by dividing the temperature of the sublayers into groups for each month of the design life. A normal distribution is assumed and the average temperature within each group division is used to compute the dynamic modulus of the sublayer. The dynamic modulus is used for fatigue damage and permanent deformation calculations by computing strains at critical depths within each layer caused by traffic loading. Transverse cracking is calculated using hourly EICM HMA temperature estimates and computing HMA properties, such as creep compliance, to determine the stress in the surface HMA layer. The smoothness prediction of IRI is calculated empirically in the MEPDG software based on the combination of the primary distresses. The following equations show the computational steps in the MEPDG to calculate distresses and are taken from the *Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition* (AASHTO, 2008). The equations were nationally calibrated from field testing using LTPP data and show what calibration coefficients are required to perform local calibration of the distress predictions.

The procedure to compute rutting, or plastic vertical deformation, in HMA layers is shown in Equation 2.1.1 below. The total rutting of the pavement structure is a simple summation of the permanent vertical deformation of each layer. The calculation uses an accumulation of plastic vertical deformation based on critical plastic vertical strain, specific pavement conditions, and truck loadings.

$$\Delta_{p(HMA)} = \varepsilon_{p(HMA)} h_{HMA} = \beta_{1r} k_z \varepsilon_{r(HMA)} 10^{k_{1r}} n^{k_{2r}\beta_{2r}} T^{k_{3r}\beta_{3r}} \quad (2.1.1)$$

where:

$\Delta_{p(HMA)}$ = Accumulated permanent or plastic vertical deformation in the HMA layer/sublayer, in.,

$\varepsilon_{p(HMA)}$ = Accumulated permanent or plastic axial strain in the HMA layer/sublayer, in/in.,

$\varepsilon_{r(HMA)}$ = Resilient or elastic strain calculated by the structural response model at the mid-depth of each HMA sublayer, in/in.,

h_{HMA} = Thickness of the HMA layer/sublayer, in.,

n = Number of axle-load repetitions,

T = Mix or pavement temperature, °F,

k_z = Depth confinement factor,

$k_{1r,2r,3r}$ = Global field calibration parameters (from the NCHRP 1-40D recalibration; k_{1r} = -3.35412, k_{2r} = 0.4791, k_{3r} = 1.5606), and

$\beta_{1r}, \beta_{2r}, \beta_{3r}$ = Local or mixture field calibration constants; for the global calibration, these constants were all set to 1.0.

$$k_z = (C_1 + C_2 D)0.328196^D \quad (2.1.2)$$

$$C_1 = -0.1039(H_{HMA})^2 + 2.4868H_{HMA} - 17.342 \quad (2.1.3)$$

$$C_2 = 0.0172(H_{HMA})^2 - 1.7331H_{HMA} + 27.428 \quad (2.1.4)$$

where:

D = Depth below the surface, in., and

H_{HMA} = Total HMA thickness, in.

Equation 2.2.1 represents the field-calibrated mathematical equation for rutting in the foundation and all unbound pavement layers.

$$\Delta_{p(soil)} = \beta_{s1} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_o}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{n}\right)^{\beta}} \tag{2.2.1}$$

where:

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

n = Number of axle-load applications,

ε_o = Intercept determined from laboratory repeated load permanent deformation tests, in/in.,

ε_r = Resilient strain imposed in laboratory test to obtain material properties $\varepsilon_o, \varepsilon_r$, and ρ, in/in.,

ε_v = Average vertical resilient or elastic strain in the layer/sublayer and calculated by the structural response model, in/in.,

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

k_{s1} = Global calibration coefficients; k_{s1} = 1.673 for granular materials and 1.35 for fine-grained materials,

ε_{s1} = Local calibration constant for the rutting in the unbound layers; the local calibration constant was set to 1.0 for the global calibration effort, and

β_{s1} = Local or mixture field calibration constants; for the global calibration, these constants were all set to 1.0.

$$Log\beta = -0.61119 - 0.017638(W_c) \tag{2.2.2}$$

$$\rho = 10^9 \left(\frac{C_o}{\left(1-(10^9)^{\beta}\right)}\right)^{\frac{1}{\beta}} \tag{2.2.3}$$

$$C_o = Ln\left(\frac{a_1 M_r^{b_1}}{a_9 M_r^{b_9}}\right) = 0.0075 \tag{2.2.4}$$

where:

W_c = Water content, %,

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

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

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

Alligator cracks are assumed to initiate at the bottom of HMA layers, while longitudinal cracks are assumed to initiate at the surface of the pavement. For both load related cracking models, the approach to calculate the allowable number of axle-load applications needed for the incremental damage index is shown using Equation 2.3.1.

$$N_{f-HMA} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{HMA})^{k_{f3}\beta_{f3}} \tag{2.3.1}$$

where:

N_{f-HMA} = Allowable number of axle-load applications for a flexible pavement and HMA overlays,

ε_t = Tensile strain at critical locations and calculated by the structural response model, in./in.,

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

k_{f1}, k_{f2}, k_{f3} = Global field calibration parameters (from the NCHRP 1-40D re-calibration; k_{f1}=0.007566, k_{f2}=-3.9492, and k_{f3}=-1.281), and

β_{f1}, β_{f2}, β_{f3} = Local or mixture specific field calibration constants; for the global calibration effort, these constants were set to 1.0.

$$C = 10^M \tag{2.3.2}$$

$$M = 4.84\left(\frac{V_{be}}{V_a+V_{be}} - 0.69\right) \tag{2.3.3}$$

where:

V_{be} = Effective asphalt content by volume, %,

V_a = Percent air voids in the HMA mixture, and

C_H = Thickness correction term, dependant on type of cracking.

For alligator (bottom-up) cracking:

$$C_H = \frac{1}{0.000398 + \frac{0.003602}{1 + e^{(11.02 - 3.49 H_{HMA})}}} \tag{2.3.4}$$

For longitudinal (top-down) cracking:

$$C_H = \frac{1}{0.01 + \frac{12.00}{1 + e^{(15.676 - 2.8186 H_{HMA})}}} \tag{2.3.5}$$

where:

H_{HMA} = Total HMA thickness, in.

Using the calculation for allowable number of axle-load applications shown above, the MEPDG calculates an incremental damage index (ΔDI) to predict the load related cracking. Equation 2.4 shows how the damage index is computed by dividing the actual number of loads by the allowable number of loads within the specified time increment. The cumulative damage index (DI) for the design life of the pavement is calculated at each critical location by summing the incremental damage indices.

$$DI = \sum (\Delta DI)_{j,m,l,p,T} = \sum \left(\frac{n}{N_{f-HMA}}\right)_{j,m,l,p,T} \tag{2.4}$$

where:

n = Actual number of axle load applications within a specific time period,

j = Axle-load interval,

m = Axle-load type (single, tandem, tridem, quad, or special axle configuration),

l = Truck type using the truck classification groups included in the MEPDG,

p = Month, and

T = Median temperature for the five temperature intervals or quintiles used to subdivide each month, °F.

Once the cumulate damage index is predicted, transfer functions are used to convert the data into either alligator cracking area using Equation 2.5.1 or longitudinal cracking length using Equation 2.6.

$$FC_{Bottom} = \left(\frac{1}{60}\right)\left(\frac{C_4}{1+e^{(C_1C_1^*+C_2C_2^*Log(DI_{Bottom}*100))}}\right) \tag{2.5.1}$$

where:

FC_{Bottom} = Area of alligator cracking that initiates at the bottom of the HMA layers, % of total lane area,

DI_{Bottom} = Cumulative damage index at the bottom of the HMA layers, and

$C_{1,2,4}$ = Transfer function regression constants; C_4=6,000; C_1=1.00; C_2=1.00.

$$C_1^* = -2C_2^* \tag{2.5.2}$$

$$C_2^* = -2.40874 - 39.748(1 + H_{HMA})^{-2.856} \tag{2.5.3}$$

where:

H_{HMA} = Total HMA thickness, in.

$$FC_{Top} = 10.56\left(\frac{C_4}{1+e^{(C_1-C_2Log(DI_{Top}))}}\right) \tag{2.6}$$

where:

FC_{Top} = Length of longitudinal cracks that initiate at the top of the HMA layer, ft/mi,

DI_{Top} = Cumulative damage index near the top of the HMA surface, and

$C_{1,2,4}$ = Transfer function regression constants; C_1=7.00; C_2=3.5; and C_4=1,000.

Thermal cracking predictions use calculations of changes in cracking due to cooling cycles, shown in Equation 2.7.1.

$$\Delta C = A(\Delta K)^n \tag{2.7.1}$$

where:

ΔC = Change in the crack depth due to a cooling cycle,

ΔK = Change in the stress intensity factor due to a cooling cycle, and

A,n = Fracture parameters for the HMA mixture.

$$A = 10^{k_t \beta_t (4.389 - 2.52 Log(E_{HMA} \sigma_m n))} \tag{2.7.2}$$

$$n = 0.8\left[1 + \frac{1}{m}\right] \tag{2.7.3}$$

where:

k_t = Coefficient determined through global calibration for each input level (Level 1 = 5.0; Level 2 = 1.5; and Level 3 = 3.0),

E_{HMA} = HMA indirect tensile modulus, psi,

σ_m = Mixture tensile strength, psi,

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

β_t = Local or mixture calibration factor.

$$K = \sigma_{tip}[0.45 + 1.99(C_o)^{0.56}] \tag{2.7.4}$$

where:

σ_{tip} = Far-field stress from pavement response model at depth of crack tip, psi, and

C_o = Current crack length, ft.

To get the degree of thermal cracking, the MEPDG uses a relationship shown in Equation 2.7.5.

$$TC = \beta_{t1} N\left[\frac{1}{\sigma_d} Log\left(\frac{C_d}{H_{HMA}}\right)\right] \tag{2.7.5}$$

where:

TC = Observed amount of thermal cracking, ft/mi,

βt_1 = Regression coefficient determined through global calibration (400),

$N[z]$ = Standard normal distribution evaluated at [z],

σ_d = Standard deviation of the log of the depth of cracks in the pavement (0.769) in.,

C_d = Crack depth, in., and

H_{HMA} = Thickness of HMA layers, in.

To calculate smoothness, the MEPDG uses a calculation of IRI which combines the effects of the other distress models. The assumption is surface distress will cause an increase in roughness. Equations 2.8.1 and 2.8.2 show how the MEPDG predicts IRI over time for flexible pavement designs.

$$IRI = IRI_o + 0.0150(SF) + 0.400(FC_{Total}) + 0.0080(TC) + 40.0(RD) \tag{2.8.1}$$

where:

IRI_o = Initial IRI after construction, in./mi,

SF = Site factor,

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

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

RD = Average rut depth, in.

$$SF = Age[0.02003(PI + 1) + 0.007947(Precip + 1) + 0.000636(FI + 1)] \quad (2.8.2)$$

where:

Age = Pavement age, yr,

PI = Percent plasticity index of the soil,

FI = Average annual freezing index, °F days, and

Precip = Average annual precipitation or rainfall, in.

The standard error computations, representing a function of the average predicted distresses, are shown for each performance prediction. Equations 2.9.1 through 2.9.3 show the standard error for rutting. These equations were based on estimations rather then actual rutting measurements of the LTPP test sections. During the global calibration process, trenches of rut depths were unavailable for all of the test sections. Equations 2.10 and 2.11 provide the standard error for alligator and longitudinal cracking predictions. None of the LTPP test sections were cored to determine whether the load related cracks started at the top or bottom of the HMA layers. Equations 2.12.1 through 2.12.3 give the standard error for thermal cracking, depending on the hierarchical input level used.

$$s_{e(HMA)} = 0.1587(\Delta_{HMA})^{0.4579} + 0.001 \quad (2.9.1)$$

$$s_{e(Gran)} = 0.1169(\Delta_{Gran})^{0.5303} + 0.001 \quad (2.9.2)$$

$$s_{e(Fine)} = 0.1724(\Delta_{Fine})^{0.5516} + 0.001 \quad (2.9.3)$$

where:

Δ_{HMA} = Plastic deformation in the HMA layers, in.,

Δ_{Gran} = Plastic deformation in the aggregate and coarse-grained layers, in., and

Δ_{Fine} = Plastic deformation in the fine-grained layers and soils, in.

$$s_{e(Alligator)} = 32.7 + \frac{995.1}{1+e^{2-2Log(FC_{Bottom}+0.0001)}} \quad (2.10)$$

$$s_{e(Long)} = 200 + \frac{2300}{1+e^{1.072-2.1654Log(FC_{Top}+0.0001)}} \quad (2.11)$$

$$s_e(Level\ 1) = -0.0899(TC + 636.97) \quad (2.12.1)$$

$$s_e(Level\ 2) = -0.0169(TC + 654.86) \quad (2.12.2)$$

$$s_e(Level\ 3) = -0.0869(TC + 453.98) \quad (2.12.3)$$

MEPDG Rigid Performance Prediction Equations

For JCP, the MEPDG calculates either bottom-up or top-down transverse slab cracking and eliminates the possibility of both types of cracking occurring on the same slab. Equation 2.13 shows the prediction of transverse cracking for both bottom-up and top-down modes.

$$CRK = \frac{1}{1+(DI_F)^{-1.98}} \quad (2.13)$$

where:

CRK = Predicted amount of bottom-up or top-down cracking (fraction), and

DI_F = Fatigue Damage calculated using procedure below.

Miner’s hypothesis is used for fatigue damage accumulations and is shown in Equation 2.14.1.

$$DI_F = \sum \frac{n_{i,j,k,l,m,n,o}}{N_{i,j,k,l,m,n,o}} \quad (2.14.1)$$

where:

DI_F = Total fatigue damage (top-down or bottom-up),

$n_{i,j,k...}$ = Applied number of load applications at condition i,j,k,l,m,n,o

$N_{i,j,k...}$ = Allowable number of load applications at condition i,j,k,l,m,n,o

i = Age (accounts for change in PCC modulus of rupture and elasticity, slab/base contact friction, deterioration of shoulder LTE),

j = Month (accounts for change in base elastic modulus and effective dynamic modulus of subgrade reaction),

k = Axle type (single, tandem, and tridem for bottom-up cracking; short, medium, and long wheelbase for top-down cracking),

l = Load level (incremental load for each axle type), and

m = Equivalent temperature difference between top and bottom PCC surfaces.

n = Traffic offset path, and

o = Hourly truck traffic fraction.

The applied number of load applications is based on traffic conditions, design life, and temperature differentials throughout the slab. The allowable number of load applications is based on the applied stresses, strength of the slab, and is determined using Equation 2.14.2.

$$\log\left(N_{i,j,k,l,m,n}\right) = C_1 \cdot \left(\frac{M_{Ri}}{\sigma_{i,j,k,l,m,n}}\right)^{C_2} \qquad (2.14.2)$$

where:

$N_{i,j,k...}$ = Allowable number of load applications at condition i,j,k,l,m,n,

M_{Ri} = PCC modulus of rupture at age i, psi,

$\sigma_{i,j,k,...}$ = Applied stress at condition i,j,k,l,m,n,

C_1 = Calibration constant, 2.0, and

C_2 = Calibration constant, 1.22.

The total cracking prediction is calculated by summing each incremental accumulation and combined using Equation 2.15.

$$TCRACK = \left(CRK_{Bottom-up} + CRK_{Top-down} - CRK_{Bottom-up} \cdot CRK_{Top-down} \cdot 100\%\right) \quad (2.15)$$

where:

$TCRACK$ = Total transverse cracking (percent, all severities),

$CRK_{Bottom-up}$ = Predicted amount of bottom-up transverse cracking (fraction), and

$CRK_{Top-Down}$ = Predicted amount of top-down transverse cracking (fraction).

To calculate the mean transverse joint faulting, an incremental approach is used. For the current month, the faulting from each of the previous months from the start of the pavement life is summed using Equations 2.16.1-2.16.4 below.

$$Fault_m = \sum_{i=1}^{m} \Delta Fault_i \quad (2.16.1)$$

$$\Delta Fault_i = C_{34} * (FAULTMAX_{i-1} - Fault_{i-1})^2 * DE_i \quad (2.16.2)$$

$$FAULTMAX_i = FAULTMAX_0 + C_7 * \sum_{j=1}^{m} DE_j * Log(1 + C_5 * 5.0^{EROD})^{C_6} \quad (2.16.3)$$

$$FAULTMAX_0 = C_{12} * \delta_{curling} * \left[Log(1 + C_5 * 5.0^{EROD}) * Log\left(\frac{P_{200} * WetDays}{p_s}\right)\right]^{C_6} \quad (2.16.4)$$

where:

$Fault_m$ = Mean joint faulting at the end of month m, in.,

$\Delta Fault_i$ = Incremental change (monthly) in mean transverse joint faulting during month i, in.,

$FAULTMAX_i$ = Maximum mean transverse joint faulting for month i, in.,

$FAULTMAX_0$ = Initial maximum mean transverse joint faulting, in.,

EROD = Base/subbase erodibility factor,

DE_i = Differential density of energy of subgrade deformation accumulated during month i,

$\delta_{curling}$ = Maximum mean monthly slab corner upward deflection PCC due to temperature curling and moisture warping,

P_s = Overburden on subgrade, lb,

P_{200} = Percent subgrade material passing #200 sieve,

WetDays = Average annual number of wet days (greater than 0.1 in. rainfall), and

$C_{1,2,3,4,5,6,7,12,34}$ = Global calibration constants (C_1 = 1.29; C_2 = 1.1; C_3 = 0.001725; C_4 = 0.0008; C_5 = 250; C_6 = 0.4; C_7 = 1.2).

$$C_{12} = C_1 + C_2 * FR^{0.25} \tag{2.16.5}$$

$$C_{34} = C_3 + C_4 * FR^{0.25} \tag{2.16.6}$$

where:

FR = Base freezing index defined as percentage of time the top base temperature is below freezing (32°F) temperature.

For each incremental month, the linear temperature difference is computed at 11 equally spaced points throughout the PCC slab for each hour using the ICM data. The temperature gradient is calculated using Equation 2.17 and is used with the base modulus and other information to determine the slab curling and warping for the month.

$$\Delta T_m = \Delta T_{t,m} - \Delta T_{b,m} + \Delta T_{sh,m} + \Delta T_{PCW} \tag{2.17}$$

where:

ΔT_m = Effective temperature differential for month m,

$\Delta T_{t,m}$ = Mean PCC top-surface nighttime temperature (from 8:00 p.m. to 8:00 a.m.) for month m,

$\Delta T_{b,m}$ = Mean PCC bottom-surface nighttime temperature (from 8:00 p.m. to 8:00 a.m.) for month m,

$\Delta T_{sh,m}$ = Equivalent temperature differential due to reversible shrinkage for month m for old concrete (i.e., shrinkage is fully developed), and

ΔT_{PCW} = Equivalent temperature differential due to permanent curl/warp.

The transverse joint load transfer efficiency (LTE) is calculated using Equation 2.18.

The MEPDG has values included in the software that assume the LTE of specific materials depending on how the joints are connected.

$$LTE_{joint} = 100(1 - \left(1 - \frac{LTE_{dowel}}{100}\right)\left(1 - \frac{LTE_{agg}}{100}\right)\left(1 - \frac{LTE_{base}}{100}\right)) \qquad (2.18)$$

where:

LTE_{joint} = Total transverse joint LTE, %,

LTE_{dowel} = Joint LTE if dowels are the only mechanism of load transfer, %,

LTE_{base} = Joint LTE if base is the only mechanism of load transfer, %, and

LTE_{agg} = Joint LTE if aggregate interlock is the only mechanism of load transfer, %.

To calculate the maximum faulting, the differential energy from truck loading, shear stress at slab corner, and maximum dowel bearing stress are used:

$$DE = \frac{k}{2}(\delta_L^2 - \delta_U^2) \qquad (2.19.1)$$

$$\tau = \frac{AGG*(\delta_L - \delta_U)}{h_{PCC}} \qquad (2.19.2)$$

$$\sigma_b = \frac{\zeta_d*(\delta_L - \delta_U)}{d*dsp} \qquad (2.19.3)$$

where:

DE = Differential energy, lb/in.,

δ_L = Loaded corner deflection, in.,

δ_U = Unloaded corner deflection, in.,

AGG = Aggregate interlock stiffness factor,

K = Coefficient of subgrade reaction, psi/in.,

h_{PCC} = PCC slab thickness, in.,

ζ_d = Dowell stiffness factor = J_d*k*l*dsp,

d = Dowell diameter, in.,

dsp = Dowel spacing, in.,

J_d = Non-dimensional dowel stiffness at the time of load application, and

l = Radius of relative stiffness, in.

Load transfer test data from the Portland Cement Association is used to determine the loss of shear capacity, Δs. Traffic loading creates the loss of shear and the calculation is shown in Equations 2.20.1 and 2.20.2.

$$\Delta s = \begin{cases} 0 & if; w < 0.001 h_{PCC} \\ \sum_j \frac{0.005}{1.0+(\frac{jw}{h_{PCC}})^{-5.7}} \left(\frac{n_j}{10^6}\right)\left(\frac{\tau_j}{\tau_{ref}}\right) & if; jw < 3.8 h_{PCC} \\ \sum_j \frac{0.068}{1.0+6.0*(\frac{jw}{h_{PCC^{-3}}})^{-1.98}} \left(\frac{n_j}{10^6}\right)\left(\frac{\tau_j}{\tau_{ref}}\right) & if; jw > 3.8 h_{PCC} \end{cases} \tag{2.20.1}$$

where:

n_j = Number of applied load applications for the current increment by load group j,

w = Joint opening, mils (0.001 in.), and

$$\tau_j = \frac{AGG*(\delta_L-\delta_U)}{h_{PCC}} \tag{2.20.2}$$

where:

τ_j = Shear stress on the transverse crack from the response model for the load group j, psi.

τ_{ref} = Reference shear stress derived from the PCA test results, psi,

τ_{ref} = 111.1*exp{-exp[0.9988*exp(-0.1089 log J_{AGG})]}, and (2.20.3)

J_{AGG} = Joint stiffness on the transverse crack computed for the time increment.

$$DAM_{dow} = C_8 \sum_j \left(\frac{J_d*(\delta_L-\delta_U)*dsp}{d\ f'_c}\right) \tag{2.20.4}$$

where:

DAM_{dow} = Damage at dowel-concrete interface,

C_8 = Coefficient equal to 400,

n_j = Number of load applications for the current increment by load group j,

J_d = Non-dimensional dowel stiffness at the time of load application,

δ_L = Deflection at the corner of the loaded slab induced by the axle, in.,

δ_U = Deflection at the corner of the unloaded slab induced by the axle, in.,

dsp = Space between adjacent dowels in the wheel path, in.,

f'_c = PCC compressive strength, psi, and

d = Dowel diameter, in.

As with the flexible pavement IRI procedure, the JPCP smoothness prediction of IRI combines the initial profile of the pavement and the loss of smoothness from the other distress predictions, calibrated from LTPP data. Spalling is calculated using Equation 2.22.1.

$$IRI = IRI_I + C1 * CRK + C2 * SPALL + C3 * TFAULT + C4 * SF \quad (2.21.1)$$

where:

IRI	= Predicted IRI, in./mi,
IRI_I	= Initial smoothness measured as IRI, in./mi,
CRK	= Percent slabs with transverse cracks (all severities),
$SPALL$	= Percentage of joints with spalling (medium and high severities),
$TFAULT$	= Total joint faulting cumulated per mi, in., and
$C1$	= 0.8203
$C2$	= 0.4417
$C3$	= 0.4929
$C4$	= 25.24
SF	= Site Factor

$$SF = AGE(1 + 0.5556 * FI)(1 + P_{200}) * 10^{-6} \quad (2.21.2)$$

where:

AGE	= Pavement age, yr,
FI	= Freezing index, °F-days, and
P_{200}	= Percent subgrade material passing No. 200 sieve.

$$SPALL = \left(\frac{AGE}{AGE+0.01}\right)\left(\frac{100}{1+1.005^{-12*AGE+SCF}}\right) \quad (2.22.1)$$

where:

$SPALL$	= Percentage joints spalled (medium-and high-severities),

AGE = Pavement age since construction, yr, and

SCF = Scaling factor based on site-design, and climate-related.

$$SCF = -1400 + 350 \cdot AC_{PCC} \cdot (0.5 + PREFORM) + 3.4f'c \cdot 0.4 - 0.2\left(FT_{cycles} \cdot AGE\right) + 43H_{PCC} - 536WC_{PCC} \quad (2.22.2)$$

where:

AC_{PCC} = PCC air content, %,

AGE = Time since construction, yr,

$PREFORM$ = 1 if preformed sealant is present; 0 if not,

$f'c$ = PCC compressive strength, psi,

FT_{cycles} = Average annual number of freeze-thaw cycles,

H_{PCC} = PCC slab thickness, in., and

WC_{PCC} = PCC w/c ratio.

The standard error computations for IRI, cracking, and faulting are shown below:

$$s_{e(IRI)} = \left(Var_{IRIi} + C1^2 \cdot Var_{CRK} + C2^2 \cdot Var_{Spall} + C3^2 \cdot Var_{Fault} + Se2\right)^{0.5} \quad (2.23)$$

where:

$s_{e(IRI)}$ = Standard deviation of IRI

Var_{IRIi} = Variance of initial IRI (obtained from LTPP) = 29.16 (in./mi)^2,

Var_{CRK} = Variance of cracking (percent slabs)^2,

Var_{Spall} = Variance of spalling (obtained from spalling model) = 46.24, (percent joints)^2,

Var_{Fault} = Variance of faulting, (in./mi)^2, and

S_e^2 = Variance of overall model error = 745.3 (in./mi)^2.

$$s_{e(CR)} = -0.00198 * CRACK^2 + 0.56857CRACK + 2.76825 \quad (2.24)$$

CRACK = Predicted transverse cracking based on mean inputs (corresponding to 50 percent reliability), percentage of slabs, and

$s_{e(CR)}$ = Standard error of the estimate of transverse cracking at the predicted level of mean cracking.

$$s_{e(F)} = (0.00761 * Fault(t) + 0.00008099)^{0.445} \tag{2.25}$$

Fault (t) = Predicted mean transverse joint faulting at any given time t, in.

For CRCP, the total number of medium to high-severity punchouts per mile (PO) is predicted using a globally calibrated model. The prediction is based on the accumulated fatigue damage due to top-down stresses in the transverse direction. Critical stresses are calculated using neural net models as a function of slab thickness, traffic offset from the edge of the pavement, PCC properties, base course properties, base course thickness, subgrade stiffness, temperature gradients, and other factors.

Design Criteria and Reliability Level

Each performance indicator is checked against user specified design criteria, or threshold limits. Interstate projects require more stringent design criteria values when compared to secondary road projects. Comparisons of specified threshold limits against MEPDG performance predictions are used to determine whether a trial design is acceptable or needs to be adjusted. Recommended MEPDG design criteria (or threshold limits) are shown in Table 2.1. Values can be adjusted as deemed appropriate or necessary for a particular design.

Table 2.1: Design Criteria or Threshold Values Recommended for Use in Judging the Acceptability of a Trial Design (AASHTO, 2008)

Performance Criteria	**Maximum Value at End of Design Life**
Alligator Cracking (HMA)	Interstate: 10% lane area Primary: 20% lane area Secondary: 35% lane area
Rutting (HMA)	Interstate: 0.40 in. Primary: 0.50 in. Others (<45 mph): 0.65 in.
Transverse Cracking (HMA)	Interstate: 500 ft./mil Primary: 700 ft./mi Secondary: 700 ft./mi
Mean Joint Faulting (JPCP)	Interstate: 0.15 in. Primary: 0.20 in. Secondary: 0.25 in.
Percent Transverse Slab Cracking (JPCP)	Interstate: 10% Primary: 15% Secondary: 20%
IRI (All Pavements)	Interstate: 160 in./mi Primary: 200 in./mi Secondary: 200 in./mi

The MEPDG uses a statistical design reliability calculation to account for the variability in the output performance indicators. The reliability for each distress model has been calibrated using field data primarily from the LTPP database to determine the difference between predicted and observed distresses. Design reliability is defined as the probability that the predicted distresses will be less than the critical level over the design period (AASHTO, 2008). The reliability level for each performance indicator can be adjusted individually or they all can be set to the same value. The calculation of the reliability for each performance indicator depends on the standard error of the distress prediction. The designer may choose to adjust the design if the desired reliability is not reached after the analysis of a trial design is complete. Table 2.2 shows recommended reliability levels for different roadway classifications.

Further experience with the MEPDG may enable agencies to develop and calibrate design criteria and design reliability values for various pavement designs. Designs with strict design criteria and high reliability will have a higher cost. Design reliability should be selected in balance with the design criteria (AASHTO, 2008).

Table 2.2: Levels of Reliability for Different Functional Classifications of the Roadway (AASHTO, July 2008)

Functional Classification	**Level of Reliability**	
	Urban	**Rural**
Interstate/Freeways	95	95
Principal Arterials	90	85
Collectors	80	75
Local	75	70

Hierarchical Design Strategy

The hierarchical input level feature in the MEPDG provides flexibility in determining the required input parameters of the software. Three input levels are available for most of the material and traffic information required for design. This feature enables agencies to adopt the MEPDG using minimal and supplemental default inputs, and allows for adjustments in input data collection efforts depending on the scope of the project. Agencies with limited resources and limited materials and traffic information can begin to immediately take advantage of the analysis the MEPDG provides. Designs are subject to possible and unknown errors associated with both simplified (less detailed) input and the use of nationally (rather than locally) calibrated empirical prediction equations within the MEPDG.

Generally, Level 1 inputs provide the most accuracy and least amount of uncertainty. It should be noted that not all MEPDG prediction models have been calibrated for higher levels of input (see AASHTO, 2008). Level 1 input data are site-specific and require the most extensive laboratory and/or field testing. Level 2 input data are less comprehensive. Inputs may be selected from a database, extrapolated from limited testing, or estimated through correlations. Level 3 inputs provide the lowest level of accuracy. Level 3 inputs typical include default values and with minimal materials testing and data collection. No matter what input level (or mixture of input levels) is used, the computational methodology to predict distresses remains the same in the MEPDG software (NCHRP, 2004).

For traffic data, Level 1 analysis requires site-specific collection of vehicle count by class and by direction and lane. Monitoring of weight data needs to be collected at or near the project site for the development of axle load spectra distributions. For Level 2, site-specific collection of vehicle count and class needs to be completed, but a state or regional axle load spectra distribution assumption can be made. Level 3 traffic data requires an estimation of vehicle volume and correlates the volume to a default load spectra distribution value for analysis. For material inputs, Level 1 information is gathered from laboratory or field testing. Level 2 values are gathered using correlations from testing. Default values compose Level 3 inputs. Climatic data is not hierarchical.

MEPDG Inputs

Traffic, materials, and climate are the three main categories of input variables. MEPDG methodology for collecting and inputting traffic data does <u>not</u> incorporate the ESAL approach used in current SCDOT pavement design procedure. Instead, the MEPDG provides an opportunity to use weigh-in-motion (WIM) data and other site-specific inputs to produce an axle-load spectrum. Table 2.3 summarizes the input parameters required for MEPDG traffic data. The MEPDG is able to link with traffic collection software packages. WIM, automatic vehicle classification (AVC), vehicle counts, and trip generation models are four main sources of data used by agencies that can be incorporated into the MEPDG to develop an axle load spectrum for each axle type. Where detailed truck traffic collection is not available or limited, the MEPDG provides default values calibrated using the WIM data from mostly interstate highway and principal arterial LTPP sites.

Table 2.3: MEPDG Traffic Inputs

Site Specific Traffic Inputs
• Initial Two Way Average Annual Daily Truck Traffic (AADTT) • Percent Trucks in Design Lane • Percent Trucks in Design Direction • Operational Speed • Truck Traffic Growth
WIM Traffic Data
• Axle Load Distribution • Normalized Truck Volume Distribution • Axle Load Configurations • Monthly Distribution Factors • Hourly Distribution Factors
Other Inputs
• Dual Tire Spacing • Tire Pressure • Lateral Wander of Axle Loads

The MEPDG uses the detailed climatic information of the EICM to predict internal pavement temperature changes, changes in moisture content of each layer, etc. Embedded within the software are a multitude of weather stations, which provide this information for analysis. Table 2.4 shows the location of the South Carolina weather stations and the amount of information available from each station. For climatic analysis, a project site location (latitude and longitude) and depth to water table are necessary to run MEPDG software. Once the location is specified, the software selects six stations closest to the latitude and longitude of the pavement. Multiple stations can be selected to provide the necessary climatic data because some weather stations may have missing information.

Table 2.5 summarizes Level 3 material inputs required for MEPDG analysis. The MEPDG provides nationally calibrated default values for Level 3 analysis. Dynamic modulus, creep compliance, and indirect tensile strength are the mechanistic properties estimated from the Level 3 HMA inputs. The MEPDG provides default values for a variety of material types shown in Table 2.6.

Table 2.4: South Carolina MEPDG Climatic Inputs

Location	Description	Months of Available Data	Months Missing
Anderson, SC	Anderson County Airport	88	0
Charleston, SC	Charleston AFB/INTL Airport	116	0
Clemson, SC	Oconee County Regional Airport	98	8
Columbia, SC	Columbia Metropolitan Airport	116	0
Columbia, SC	Columbia Owens Downtown Airport	89	1
Florence, SC	Florence Regional Airport	83	0
Greenville, SC	Greenville Downtown Airport	82	0
Greenwood, SC	Greenwood County Airport	69	0
Greer, SC	Greenville-Spartanburg INTL Airport	116	0
North Myrtle Beach, SC	Grand Strand Airport	80	0
Orangeburg, SC	Orangeburg Municipal Airport	105	0
Rock Hill, SC	Rock Hill/York County Airport	85	1

Table 2.5: MEPDG Level 3 Material Inputs

<table>
<tr><th>HMA</th><th>PCC</th></tr>
<tr><td>• Aggregate Gradation
• Air Voids
• Effective Asphalt Binder Content
• Total Unit Weight
• Poisson's Ratio
• Dynamic Modulus
• Surface Shortwave Apsorptivity
• Reference Temperature
• Thermal Conductivity of Asphalt
• Heat Capacity of Ashpalt</td><td rowspan="3">• Elastic Modulus and/or Flexural Strength
• Poisson's Ratio
• Unit Weight
• Coefficient of Thermal Expansion
• Surface Shortwave Absorptivity
• Thermal Conductivity
• Heat Capacity
• PCC Zero-Stress Temperature
• Cement Type
• Cementious Material Content
• Water to Cement Ratio
• Aggregate Type
• Curing Method
• Ultimate Shrinkage
• Reversible Shrinkage
• Time to Develop 50% of Ultimate Shrinkage</td></tr>
<tr><th>Unbound Materials</th></tr>
<tr><td>• Gradation
• Resilient Modulus
• Poisson's Ratio
• Moisture Content
• Dry Density
• Atterberg Limits</td></tr>
</table>

Table 2.6: MEPDG Material Types

Asphalt Materials	PCC Materials
Stone Matrix Asphalt (SMA)	Intact PCC Slabs
Hot Mix Asphalt (HMA)	• High Strength Mixes
• Dense Graded	• Lean Concrete Mixes
• Open Graded Asphalt	Fractured Slabs
• Asphalt Stabilized Base Mixes	• Crack/Seat
Cold Mix Asphalt	• Break/Seat
• Central plant Processed	• Rubblized
• Cold In-Place Recycling	**Chemically Stabilized Materials**
Subgrade Soils	Cement Stabilized Aggregate
Gravely Soils (A-1;A-2)	Soil Cement
Sandy Soils	Lime Cement Fly Ash
• Loose Sands (A-3)	Lime Fly Ash
• Dense Sands (A-3)	Lime Stabilized Soils
• Silty Sands (A-2-4; A-2-5)	Open-Graded Cement Stabilized Aggregate
• Clayey Sands (A-2-6; A-2-7)	**Non-Stabilized Granular Base/Subbase**
Silty Soils (A-4; A-5)	Granular Base/Subbase
Clayey Soils; Low-plasticity Clays (A-6)	Sandy Subbase
• Dry-Hard	Cold Recycled Asphalt Mix (as aggregate)
• Moist-Stiff	• RAP (includes millings)
• Wet/Sat-Soft	• Pulverized In-Place
Clayey Soils; High-Plasticity Clays (A-7)	Full Depth Reclamation
• Dry-Hard	**Bedrock**
• Moist Stiff	Solid, Massive, and Continuous
• Wet/Sat-Soft	Highly Fractured, and Weathered

Chapter 3

Literature Review

Overview

An effort was made to obtain information about MEPDG implementation plans for U.S. state highway agencies (SHA). To take full advantage of the MEPDG's hierarchical inputs and perform the necessary investigations to locally calibrate the MEPDG's performance prediction equations, a substantial commitment of resources is necessary. Some states have committed to immediate implementation activities, such as new testing programs for developing material properties and traffic data and establishment of permanent calibration test sections. Some states claim to have already partially calibrated current MEPDG software for local conditions. Others have apparently decided to not implement the MEPDG or to postpone implementation until release of AASHTO DARWin-ME™ software.

Two groups were created in the early stages of the MEPDG release to facilitate implementation efforts. The FHWA created a Design Guide Implementation Team (DGIT) to promote implementation efforts by informing, educating, and assisting all interested agencies about the new design guide. A Lead States Group was created in conjunction with AASHTO, NCHRP, and FHWA activities related to the MEPDG. The Lead States Group contains representatives from state highway agencies that had early interest in MEPDG implementation. The group was formed to promote the growth of the

MEPDG and develop both short and long term implementation plans. Neither the DGIT nor the Lead State Group websites have been updated for many years.

Summarized below are significant and fairly resent SHA MEPDG implementation plans and studies found during the course of this investigation. Available SHA sensitivity analysis results are also summarized.

Florida (*Fernando, et al. 2007*)

This 2007 report summarizes a research effort to develop a MEPDG implementation plan for the Florida Department of Transportation (FDOT). The main research objectives were to develop a database for calibrating the MEPDG performance prediction models and propose a new FDOT pavement design method based on the MEPDG. Steps taken to achieve these objectives included: studying the required MEPDG inputs, performing sensitivity analyses, establishing and testing in-service pavement sections, and characterizing the state's soil and climate conditions.

Sensitivity analyses were performed on material property inputs at different hierarchical levels to determine how much time and effort should be focused on establishing a specific input. Two pavements, one flexible and one rigid, were used in the analyses. Both pavements were representative of *typical* Florida pavement, environmental, and traffic conditions. The flexible pavement was a four layer structure comprised of an asphalt concrete layer, limerock base, stabilized subgrade, and sand subgrade. The rigid pavement was a six layer structure with a JPCP slab, two existing AC layers, limerock base, stabilized subgrade, and sand subgrade. An AADTT of 70,000, 20 year design period, and Orlando climatic input were used for both pavements in the analyses. The sensitivity analyses was executed by adjusting each input within a reasonable range, running the MEPDG software, and noting the changes in performance predictions.

Variables with high sensitivity based on predicted performance were found to be: AC dynamic modulus, layer thickness, base modulus, subgrade modulus, thermal coefficient of expansion, joint spacing, dowel bar diameter, and PCC compressive strength. It was found that the pavement with a higher AC modulus were predicted to perform better. The results provided information on how to proceed with establishing the input database.

For verification and calibration of MEPDG software, specific Florida pavement conditions were studied. Pavement sections were selected based on the availability of traffic data and history of pavement performance based on the DOT pavement condition survey (PCS). A total of 31 calibration sections were chosen, consisting of 15 flexible and 16 rigid pavement sections. For the climatic and soil analyses, researchers divided Florida into several regions. This was done by collecting data from weather stations and soil survey reports across the state. The soil surveys helped determine the predominant soil type for each county to be used with the MEPDG. Analysis of the climatic data resulted in simplifying the use of climatic data into four regions throughout the state.

To gather information on inputs and how to test materials most effectively, falling weight deflectometer (FWD) data, the FDOT coring database, and video logs of the testing sections were studied. The results of the data collection helped determine the coring and trenching requirements to complete the field and laboratory tests. From AC cores, binder content, gradation, effective bitumen content, air voids content, and resilient modulus were identified as significant properties. From Portland cement concrete cores, coefficient of thermal expansion and compressive strength were identified as significant properties.

A conceptual pavement design guide was developed based on MEPDG and software. Conversion from pavement condition survey (PCS) data to MEPDG performance predictions was established to calibrate each distress model. Additional revisions to calibrations applied to the MEPDG's pavement performance predictions are planned based on additional information gathered from the calibration sections. For implementation, manuals of design tables and charts derived from MEPDG runs will be developed so they match current FDOT design method formats.

Indiana *(Olson, 2009)*

As of January 1, 2009, the Indiana Department of Transportation (INDOT) has mandated the use of the MEPDG as the design methodology for all new state highway and interstate pavement designs. Also, as of April 1, 2009, INDOT required all roadways administered by a local public agency and federally funded use the MEPDG for pavement designs. Current MEPDG software is being used until DARWin-ME™ is available. The INDOT Office of Research and Development has locally calibrated the software and coordinated with the INDOT Office of Pavement Engineering to include the calibration information in the Indiana Design Manual. A two day workshop was offered in March 2009 to explain the software and support the initial implementation process.

Maryland *(Schwartz, 2007)*

In 2007, a plan was developed to transition Maryland's State Highway Administration (MDSHA) from current flexible pavement design procedures to the MEDPG. The suitability of the MEPDG for Maryland conditions was studied.

Sensitivity analyses were performed to evaluate the MEPDG parameters and compare the new software to the Maryland's old pavement design procedure.

Implementation suggestions were made to help the transition from old design procedures to the MEPDG. It was recommended that all inputs be Level 3 parameters initially. The asphalt concrete properties are considered most important to be transitioned to Level 1 characterization. Development of a database of traffic and material properties is suggested for routine designs and to reduce the need for site specific laboratory testing.

The sensitivity analyses performed as part of this investigation had two stated objectives: 1) comparison of pavement designs from MEPDG and the current MDSHA pavement design procedure, and 2) to study the sensitivity of the MEPDG parameters for a better understanding of the program and to gather information on calibration and other implementation needs. MDSHA currently uses 1993 AASHTO Design Guide procedures. By comparing AASHTO and MEPDG designs, it is envisioned that predicted performance can be further studied by grouping and comparing the designs based on traffic and environmental categories. Only rutting and fatigue cracking were used for comparison in study. Results indicted that the longitudinal cracking model was not reliable and the IRI predictions were found to be insensitive to structural distresses.

All of the pavements in design method comparison study were three layer structures consisting of one asphalt concrete layer on top of a granular aggregate base and subgrade. A climatic location in Alabama, Arizona, Maryland, South Dakota, and Washington State were used to get a sample of different temperatures and precipitation levels. Low, medium, and high traffic levels, set to represent different road classifications, were used. For MEPDG designs, default inputs and reasonable assumptions were used to fill in the information gaps where input information was unavailable.

Design comparisons assume MEPDG performance predictions to be the correct and 1993 AASHTO designs should show the similar distresses if the two methods are compatible. Designs using the current method showed a high variability in distress predictions even though all of the 1993 AASHTO pavements were designed with the same change in serviceability. An underestimation of distress with high traffic levels was found. Pavements in the warmest regions, Alabama and Arkansas, showed poorer performance then the pavements in cooler zones. Part of the explanation for these results was the fact that the 1993 AASHTO design method was based on data being collected from the original AASH(T)O Road Test site in Ottawa, Ill. Cooler climate and lower traffic level might explain some of the design method inconsistencies.

The sensitivity of the MEPDG pavement performance predictions to input parameters was studied. The reference pavement used was a typical low volume design for Maryland conditions. Level 3 inputs were used in the study and varied from their reference (base) value. The studied parameters included: base and asphalt layer thickness, traffic, environment, material properties, performance model parameters, and design criteria.

The pavement layers were comprised of a 6.7 inch asphalt concrete layer, an 18.6 inch base layer, and a subgrade. The results showed an increasing base thickness corresponding to a slight decrease in fatigue cracking and a negligible change in rutting. With increasing asphalt thickness, a decrease in both fatigue cracking and rutting was shown.

The sensitivity of the vehicle class distribution was studied through a run of three MEPDG default distributions. The distributions were based on typical minor collector, minor arterial, and principal arterial traffic conditions. The analysis showed rutting and fatigue cracking increase with an increase of class 9 percent trucks as the vehicle class distribution changed from minor collector to principal arterial.

The climatic input was studied through analysis of different Maryland environmental differences. The shore, central region, mountains were the three locations chosen. The results showed rutting and fatigue cracking decreasing with an increase of temperature and precipitation. The ground water table location was studied at depths of 3, 7, and 15 feet in the central Maryland location. The overall sensitivity was found to be negligible.

The asphalt concrete material properties studied in the sensitivity analysis included: air voids, effective binder content, gradation, and binder type for asphalt concrete; and gradation, material classification, and resilient modulus for unbound base and subgrade. The parameters studied were found to have a significant influence on dynamic modulus. The three binder grades used represented typical Maryland asphalt concrete gradation conditions ranging from coarse to fine. Fatigue cracking and rutting were found to increase with binder grade. Binder content and air voids content were adjusted by +/- 10% of their reference values. Pavements with high binder content showed an increase in rutting and decrease in fatigue cracking. An increase in fatigue cracking and rutting was found with an increase in air voids content. It was concluded that additional research was necessary for the asphalt concrete rutting model.

Different subgrade soil types and resilient modulus of the subgrade and base were the unbound material properties studied. The results show a reduction in fatigue cracking and slight reduction in rutting with increase in the base layer resilient modulus. It was also concluded that the base layer resilient modulus change has little influence on the asphalt concrete layer rutting. The increase in subgrade resilient modulus showed a decrease in both fatigue cracking and rutting. The three different subgrade soil types studied were: an A-7-6 clay soil, an A-5 silty soil, and an A-2-4 sandy soil. The rutting and fatigue cracking outputs did not show any significant conclusions, but it was noted that the MEPDG is able to evaluate some kind of effect on the pavement performance due to the change of subgrade soil type.

MEPDG calibration coefficients were changed to observe how sensitive the performance predictions are to coefficient changes. It was concluded that the asphalt concrete rutting model is very sensitive to the field calibration coefficients. Results for the base rutting model less conclusive. For calibration to be successful, it was recommended that trench data must be used in lieu of making assumptions about the rutting performance throughout the pavement. It was concluded that local calibration is necessary for successful implementation. Suggested local calibration steps include: establishment of a number of in-service pavement sections for which quality input and performance data can be determined; procurement of significant input parameters as needed; and gathering of significant distresses from the MDSHA pavement management system.

Minnesota (*Cochran, et al. 2009*)

This 2009 report summarizes an investigation performed for the Minnesota Department of Transportation (MnDOT). The goals of the investigation were to: evaluate MEPDG default inputs, identify MEPDG software deficiencies, evaluate MEPDG performance prediction models for Minnesota conditions, and re-calibrate the MEPDG performance models for Minnesota conditions.

Information on Minnesota flexible and rigid pavements was gathered and compared to the inputs required to run MEPDG software. Data available from MnDOT and LTTP sections were combined to determine what information was available for MEPDG input. Typical design conditions were established from studying MnDOT pavement surveys. Common pavement cross sections, layer thicknesses, and binder grades were a few of the parameters studied to establish a database of information for typical MnDOT deigns.

Sensitivity analyses for flexible pavements were evaluated at a 20 year design life and two different traffic levels, 10 and 1 million ESALs. A base pavement structure was established for each traffic level. The input parameters studied included: climate, asphalt concrete thickness, asphalt binder grading, asphalt mix gradation, base thickness, subbase thickness, and subgrade type. Sensitivity runs were made by changing specific inputs within a reasonable range of the base values. After all of the runs were completed, each the distress prediction was graphed and statistical analyses were performed to rank input sensitivity. Output sensitivity to each input parameter was judged using the analysis of the distress predictions over time. For the 10 million ESAL analyses, longitudinal cracking was shown to be highly sensitive to asphalt concrete layer thickness and soil type. Climate, asphalt concrete layer thickness, and asphalt concrete binder grade were the three highly sensitive transverse cracking inputs. Asphalt concrete layer thickness was found to be the only highly sensitive input for rutting and alligator cracking performance predictions. For the 1 million ESAL pavement analyses, longitudinal cracking was found to be highly sensitive to asphalt concrete layer thickness and soil type again. Climate, asphalt thickness binder grade, and soil type were the highly sensitive transverse cracking inputs. Rutting was found to be highly sensitive to asphalt concrete layer thickness and alligator cracking was highly sensitive to asphalt concrete layer thickness and soil type.

Different versions of the MEPDG software were studied because the sensitivity analysis spanned a number of years. Comparisons were made between the performance predictions of the updated versions of the program. MEPDG software versions 0.615, 0.900, 0.910, and 1.003 were studied for improvements or changes to the performance prediction models. The sensitivity analysis of the newest version was found to show similar trends when compared to the results above. Version 1.003 was found to be a significant improvement over the previous versions, but the longitudinal cracking model still had questionable distress predictions. Therefore, the recommendation was to not adapt the longitudinal cracking model. The most recent version of the MEPDG software (Version 1.10, September 2009) was not included in the investigation.

The rigid pavement sensitivity analysis inputs studied included: traffic volume, PCC coefficient of thermal expansion, PCC modulus of rupture, base thickness, base type, subgrade type, joint spacing, edge support, slab width, dowel diameter, and climate. Traffic volume, slab thickness, base thickness, and coefficient of thermal expansion were found to be the most sensitive parameters for the rigid pavement cracking and faulting predictions.

Recalibration of the MEPDG performance prediction models was performed. Rutting, alligator cracking, transverse cracking, and IRI were studied for flexible pavements and faulting, cracking, and IRI models were studied for rigid pavements. The performance model recalibration steps included: obtaining pavement sections with known performance, gathering MEPDG inputs that closely represent the sections to be studied, comparing the predicted and measured distresses, and recalibrating the MEPDG models to reduce the error in the performance models when compared to the measured distresses.

Montana (*Montana Department of Transportation, 2007*)

In 2007, the MEPDG distress models and transfer functions were verified and calibrated for the Montana Department of Transportation (MDT). LTPP and MDT project sites were used in the study to gather field and laboratory information for calibration. A total of 52 pavement sections were studied. Comparisons between MDT and LTPP pavement information were made to determine what data can be used for verification of MEPDG default inputs and local calibration needs.

A secondary study comparing MDT pavement data to adjacent state information was performed to determine any major differences in input information or output predictions. The output performance comparison found significant differences between Montana’s sections and those of adjacent states. However, there was limited difference in the comparison of the traffic and material input information.

MDT traffic, environmental, and material information was gathered off of the pavement sections and analyzed for the development of an MEPDG input database. Actual pavement performance information was gathered from the sections for use in calibration of the distress prediction models. The measured and predicted distresses were plotted and compared to develop local calibration coefficients.

The final implementation phase is to include annual data collection and analyses. This phase will enable MDT to build on the initial data collected, continue the quality control, and update the calibration. It was suggested that the MEPDG improve the load related longitudinal cracking and rutting performance prediction models.

New Jersey (*Pavement Resource Program, 2006*)

A 2005 report summarized New Jersey Department of Transportation (NJDOT) activities related to MEPDG implementation. Training, regional calibration, traffic input sensitivity, and pavement evaluation support were identified as MEPDG implementation tasks being performed for the NJDOT. Material and traffic input courses focused on proper data collection, data analysis, and conversion of the data into information that can be used in the MEPDG. Regional calibration was put on hold because of issues with the software. Testing of asphalt concrete mixes in the field was still proceeding for the development of a material property database.

Sensitivity analyses focusing on traffic inputs was performed using data from five New Jersey LTPP sites. Analyses were performed by establishing a base pavement structure with MEPDG default traffic inputs. Runs were made by adjusting one traffic input using measured LTPP data, while all other inputs were held as MEPDG default values. The substitution of different measured parameters continued for all of the traffic inputs and LTPP sites. Finally, all of the inputs were modified to represent the actual traffic from the site given by the LTPP data.

The goal of the sensitivity analyses was to find which traffic parameters are most important in the data collection process. The results found the software's rutting prediction to be highly sensitive to the monthly adjustment factor and number of axles per truck. Alligator cracking was found to be sensitive to the hourly distribution of truck traffic and number of axles per truck. Longitudinal cracking was found to be sensitive to hourly distribution of truck traffic and the monthly adjustment truck factor. IRI was found to be insensitive to measured traffic inputs compared to the MEPDG defaults.

It was concluded that the MEPDG default inputs were not sufficient and the NJDOT will develop a new set of traffic inputs based on data from the LTPP sites. For material characterization, dynamic modulus and creep-compliance testing was performed to aid in calibration and verification of the MEPDG.

North Carolina (*Kim and Muthadi, 2007*)

Kim and Muthadi (2007) summarizes an investigation undertaken to develop a MEPDG implementation plan for the North Carolina Department of Transportation (NCDOT). The report includes a summary of differences between current NCDOT pavement design procedures and MEPDG practices. MEPDG input sensitivity analyses were performed and summarized. An input data collection strategy and training program were proposed.

Required information for current NCDOT pavement design procedures, including material inputs, traffic inputs, climatic inputs, design reliability levels, performance criteria, and pavement structures, was compiled. To compile this information, NCDOT engineers were interviewed from the department's pavement management unit (PMU), Geotechnical Engineering Unit, Materials and Tests Unit (M&T Unit), Traffic Forecasting Unit, and Traffic Survey Unit (TSU). Each unit was analyzed for what information they provide to the current pavement design process, and what information can be used as MEPDG inputs.

Information for sensitivity analyses was taken from the LTPP database. Where input information was not available, such as thermal conductivity and heat capacity for asphalt, MEPDG default values and typical NCDOT values were used. The analyses utilized the 27 North Carolina LTPP sections. The pavement structures were broken down by pavement type and climatic region. North Carolina climatic regions were broken into the mountains, piedmont, and coastal plain.

The only adjustments made in sensitivity analyses were materials and traffic inputs. The variation in input values for each LTPP section defined the material sensitivity analysis range. For each section, the base case was established and the input parameter to be studied is adjusted based on the LTPP range. Traffic sensitivity analyses were performed by changing the traffic inputs based on site specific, regional, or national values.

For flexible pavements, the study found IRI to be insensitive to traffic and material inputs. Alligator cracking was found to be sensitive to air voids and dynamic modulus. Air voids, dynamic modulus, and surface shortwave absorptivity were sensitive to longitudinal cracking. Rutting was sensitive to the asphalt concrete dynamic modulus.

For rigid pavements, IRI was sensitive to JPCP coefficient of thermal expansion, thermal conductivity, joint spacing, and dowel bar diameter. JPCP transverse cracking was found to be sensitive to coefficient of thermal expansion, heat capacity, thermal conductivity load transfer efficiency, and joint spacing. JPCP mean joint faulting was found to be sensitive to coefficient of thermal expansion, load transfer efficiency, and dowel bar diameter.

A number of recommendations were made for NCDOT implementation of the MEPDG. Based on the sensitivity analysis results and MEPDG requirements, changes to the NCDOT input data collection were outlined. Verification and calibration should be performed using information from the LTPP database. It was recommended that a local training program and a new pavement design manual be developed, and acceptable pavement performance criteria be established.

Virginia *(Virginia Department of Transportation, 2007)*

In 2007, Virginia's Department of Transportation (VDOT) outlined an MEPDG implementation plan. A stated goal was to immediately use the MEPDG to compliment current pavement design procedures. By December 2012, the VDOT wants to implement pavement design procedures using MEPDG.

Tasks associated with MEPDG implementation are broken down into a committee-specific tasks. The organizational structure includes a steering committee and several technical committees, each comprised of various engineers and consultants. The steering committee is to provide guidance, progress, and feedback to the technical committees and facilitates implementation. Technical committees perform implementation tasks and are comprised of traffic, materials characterization, verification, calibration, validation, data management, and implementation and training. The traffic committee is responsible for load spectra data collection and analysis. The soils and aggregate committee is responsible for characterizing and establishing a representative state-wide data base for the resilient modulus of soils and aggregates. The concrete and stabilized materials committee is responsible for characterizing the properties of current VDOT paving concrete mixtures by performing laboratory tests for the elastic modulus, modulus of rupture, compressive strength, and coefficient of thermal expansion. The asphalt concrete committee is responsible for characterizing all layers of VDOT asphalt mixes for complex modulus and creep compliance properties. The verification, calibration, and validation committee is responsible for determining the validity of the analysis and national default values for Virginia conditions and materials, performing calibration and validation of the MEPDG to determine whether the program provides a reasonable performance prediction. The implementation and training committee is responsible for marketing the MEPDG to the industry and providing training.

For immediate use of the MEPDG to augment current VDOT pavement design procedures, each committee is to focus on updating or creating MEPDG data such as truck factors, subgrade resilient modulus, elastic modulus, modulus of rupture, and compressive strength of concrete mixes. To finalize the fully functional MEPDG version, the focus of each committee will be creating quality input data, sensitivity analysis, verification, validation, local calibration, default VDOT values, and training.

Washington *(Pierce, 2007)*

The Washington State Department of Transportation (WSDOT) studied the MEPDG and established a procedure to calibrate the MEPDG software to local conditions. General implementation needs were found to be the sensitivity analyses, axle load spectra characterization, materials characterization, seasonal climate effects, field evaluation, calibration to local conditions, laboratory protocols, and establishing a link to the pavement management system.

In 2005, WSDOT evaluated their Weigh in Motion (WIM) sensors to evaluate what work needed to be done to develop proper traffic load spectra for the state's MEPDG software input. It was found that 11 sites had valid traffic information. No significant seasonal variations were found; therefore it was concluded that monthly gross vehicle weight distributions could be grouped into a yearly distribution. There were no significant differences with geographical location or with urban versus rural and interstate versus non interstate roadways.

In 2007, WSDOT evaluated their Axle Load Spectra data by analyzing different load patterns, from light to heavy. It was concluded that WSDOT could use one load spectrum for the entire state. Changes in alligator cracking, longitudinal cracking, and rutting were found to be negligible with adjustments made to the axle load spectrum.

Rigid pavement analyses found calibration coefficients to significantly affect the pavement performance predictions. For flexible pavements, HMA dynamic modulus was invest aged using data from seven projects. The objective of the dynamic modulus study was to develop a WSDOT dynamic modulus database and to determine the impact of the dynamic modulus on MEPDG performance predictions. For calibration, WSDOT plans on using available LTPP data. When the MEPDG is calibrated to Washington conditions, a database of inputs will be developed and linked through the WSDOT pavement management system.

Wisconsin *(Williams, et al. 2007)*

During 2004 through 2007, work was done for the Wisconsin Department of Transportation (WisDOT) to assess MEPDG implementation and MEPDG inputs. Measurements of the required MEPDG asphalt concrete material properties were made. A focus was placed on asphalt concrete dynamic modulus and flow number. Extensive testing on these properties was performed on various hot mix asphalt mixtures throughout the state.

The results of the material testing enabled a Level 1 dynamic modulus evaluation in the MEPDG. An analysis was performed on nineteen pavement structures throughout the state. The WisDOT provided actual constructed pavement structure and traffic information to be used. MEPDG analyses were performed using a number of pavement thicknesses while adjusting asphalt concrete binder content and air voids. Each pavement structure was also analyzed using current WisDOT design procedures. MEPDG and WisDOT results were compared. The comparison showed variation in air voids and binder content did not impact the failure of pavements designed with the current method. Pavements that did not meet performance criteria in the MEPDG did not necessarily predicted to fail using current WisDOT procedures.

Summary

The investigations cited above indicate a variety of MEPDG implementation strategies. For a surprisingly large number of SHAs, no published or otherwise generally accessible information related to MEPDG implementation could be found. The investigations summarized above, although relatively recent, may now be dated. Recommendations made by the authors may not have been implemented by the SHA.

Several investigations cite local calibration of the MEPDG to be a priority. The national values used in the program's initial calibration were cited as inadequate. Some states are using local LTPP data to calibrate while others are using field and laboratory tests on specific pavement sections. The longitudinal cracking model was cited as being inadequate or unreliable. The traffic load spectra analysis in the MEPDG was found to be an improvement over a design approach using ESALs. Some investigators proposed adjustments to agency traffic data collection and analysis procedures.

The WisDOT study provides extensive information on asphalt concrete material testing and input analysis. To simplify data collection, the FDOT study broke down climatic and soil properties into regions as an alternative to obtaining specific properties information for each case. The MDT study outlines specific calibration and validation steps. Comparison of existing and MEPDG design methods was an initial MEPDG verification approach used by some SHAs.

A number of general conclusions related to input sensitivity can be drawn from the different sensitivity analyses cited. The input parameters required to predict asphalt concrete dynamic modulus, such as air voids and binder content, were cited as significant for flexible pavement analyses. PCC coefficient of thermal expansion, PCC modulus of rupture, and slab thickness were cited as significant for rigid pavements. Layer thickness, soil type, base modulus, and subgrade modulus were also found to be significant material inputs. Accurate traffic inputs are important. Table 3.1 below summarizes significant MEPDG inputs cited.

Table 3.1: Summary of Significant MEPDG Inputs

State	Input
Florida	HMA Dynamic Modulus HMA Layer Thickness Base Modulus Subgrade Modulus Coefficient of Thermal Expansion Joint Spacing Dowel Bar Diameter PCC Compressive Strength
Maryland	HMA Layer Thickness Vehicle Class Distribution Climatic Location HMA Binder Content HMA Air Voids Subgrade Modulus
Minnesota	HMA Layer Thickness Soil Type Climatic Location HMA Binder Grade Traffic Volume Slab Thickness Base Thickness Coefficient of Thermal Expansion
New Jersey	Monthly Adjustment Factor Number of Axles Per Truck Hourly Distribution of Traffic
North Carolina	HMA Air Voids HMA Dynamic Modulus Surface Shortwave Absorptivity Coefficient of Thermal Expansion Thermal Conductivity Joint Spacing Dowel Bar Diameter

Chapter 4

Sensitivity Analysis

Introduction

This chapter describes sensitivity analyses performed using MEPDG software on five pavement structures (4 flexible, 1 rigid) provided by the SCDOT. Early analyses for as-built pavement structures (Kershaw County, I-20, flexible and Aiken County, I-520, JPCP rigid) were done using MEPDG Version 1.003 (April 2007). Other *representative* flexible pavement structures provided by the SCDOT (typical Richland County design, typical Greenville County design, and Greenville County design with cement stabilized base) were analyzed using software Version 1.003, then updated using software Version 1.10 (September 2009). A range of Level 3 input parameters were used. For baseline traffic data an initial AADTT and compound growth factor were used. Traffic class distribution was established by typical SCDOT practice of assigning percent distributions of FHWA class 5, 6, 8, and 9 vehicles. Axle load spectra are not available. Baseline material properties were either estimated, MEPDG software generated from input soil classification, material gradation, etc., or MEPDG default values. Specific experimentally measured mechanical properties are not available.

The purpose of the sensitivity analyses was to 1) use and become familiar with the MEPDG software as it currently exists, to 2) assess the reasonableness of predicted performance using Level 3 baseline inputs, 3) hopefully produce data that can be used to assess the relative importance of individual inputs, and 4) if possible, compare results with the published results given by others. A summary of the sensitivity analyses is presented below.

Kershaw County (I-20, Flexible)

A goal of this first flexible pavement sensitivity investigation using Kershaw County data was to input a wide range of input values (some to extremes beyond what might normally be deemed a reasonable range from baseline values) to make initial observations of changes in performance predictions. The as-built pavement structure consisted of 1.4 inches asphalt concrete surface course, 5.0 inches asphalt concrete binder course, 8.0 inches asphalt stabilized earth base course (sand-asphalt), and 9.0 inches earth type subbase. The pavement was constructed in September 1971.

Every complete (zero missing months of data) South Carolina station database from Table 2.4 was run with the baseline case being the Columbia weather station at 10 feet water table depth. The weather databases include: Anderson, Charleston, Columbia, Florence, Greenville, Greenwood, Greer, North Myrtle Beach, and Orangeburg. Each station was used using three different water table depths: 1, 10, and 20 feet, resulting in 27 climatic runs. The results from 78 traffic and material analysis runs are summarized in Table 4.1. The range selected for each input parameter was generally based on the suggested limits embedded in the program. Certain inputs produced little or no significant change in distress prediction when adjusted. These inputs include: construction starting month, lane width, wheel location, asphalt reference temperature. The selected inputs ultimately used for this sensitivity study are shown in Table 4.2. The inputs were organized into material, traffic, and climate categories and graphed based on distress prediction. IRI, alligator cracking, longitudinal cracking, and total rutting were the distress models studied. The initial IRI was assumed to be 93 in/mile. Figures 4.1 through 4.16 show Kershaw County sensitivity data during a 20 year period.

Table 4.1: Traffic and Material Sensitivity Summary (Kershaw County, Flexible)

Input Parameter	Value
Initial AADTT	500, 1000, 2000, 2560 (base), 4000, 8000, 10000, 20000, 25000
Percent Trucks Design Direction	40, 45, 50 (base), 55, 60
Percent Trucks Design Lane	50, 60, 70, 80 (base), 90, 100
Traffic Growth Rate	0%, Linear: 5.3%, Compound: 3%, 4%, 5.3% (base), 7%, 10%
Traffic Wander (in)	7, 8, 9, 10 (base), 11, 12, 13
Operational Speed (mph)	15, 30, 45, 60 (base), 75
Granular Base Depth (in)	1, 3, 6, 9 (base), 12, 15, 18
Subgrade Modulus (psi)	16000, 20000, 25000, 29500 (base), 35000, 40000, 42000
Granular Base Modulus (psi)	14000, 18000, 22000, 26000, 32000 (base), 35000, 37500
AC Surface Conventional Viscosity Grade	2.5, 5, 10, 20 (base), 30, 40
AC Binder Conventional Viscosity Grade	2.5, 5, 10, 20 (base), 30, 40
AC Base Conventional Viscosity Grade	2.5, 5, 10, 20 (base), 30, 40

Table 4.2: Selected Inputs Summary (Kershaw County, Flexible)

Category	Inputs	Values
Material	AC Surface Viscosity Grade	10, 20 (base), 30
	Subgrade Modulus (psi)	16000, 29500 (base), 42000
	Base Modulus (psi)	14000, 32000 (base), 37500
General Traffic	Traffic Wander (inch)	7, 10 (base), 12
	Operational Speed (mph)	45, 60 (base), 75
Volumetric Traffic	Traffic Growth Rate	10% Compound, 5.3% Compound (base), No Growth
	Percent Trucks in Design Lane	100, 80 (base), 50
	AADTT	1000, 2560 (base), 10000
Climate	Location, Depth to Water Table (ft)	Columbia 20, Columbia 10 (base), Florence 20, Florence 10

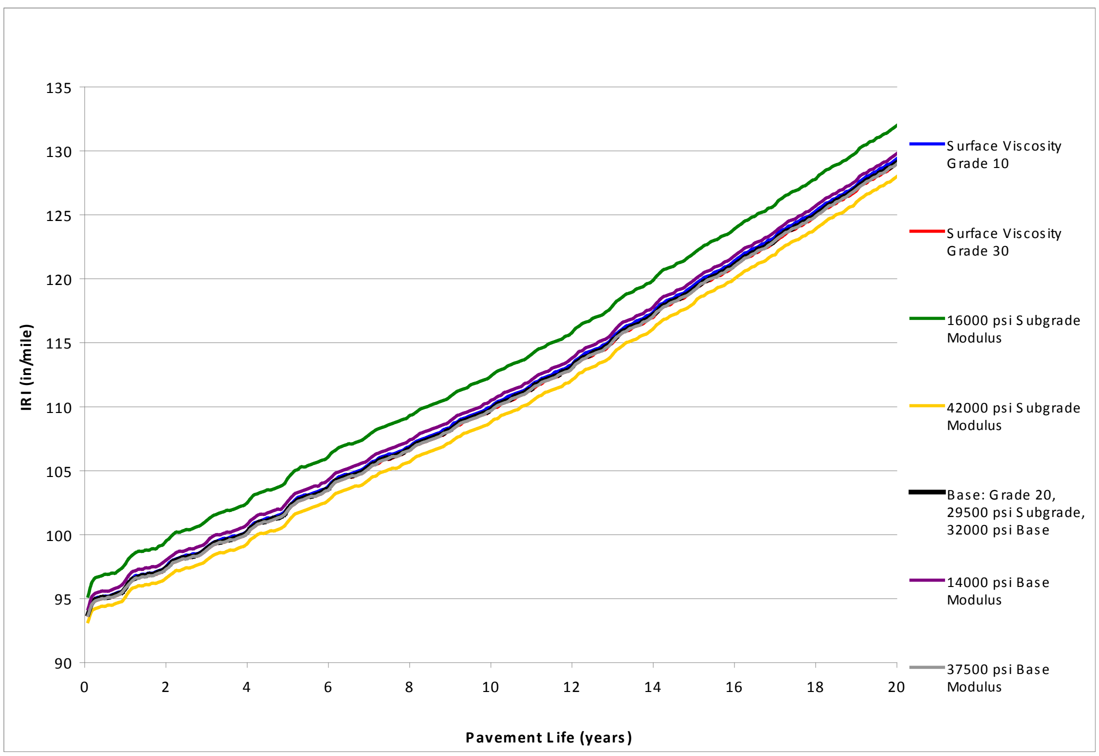

Figure 4.1: Kershaw County IRI Material Sensitivity

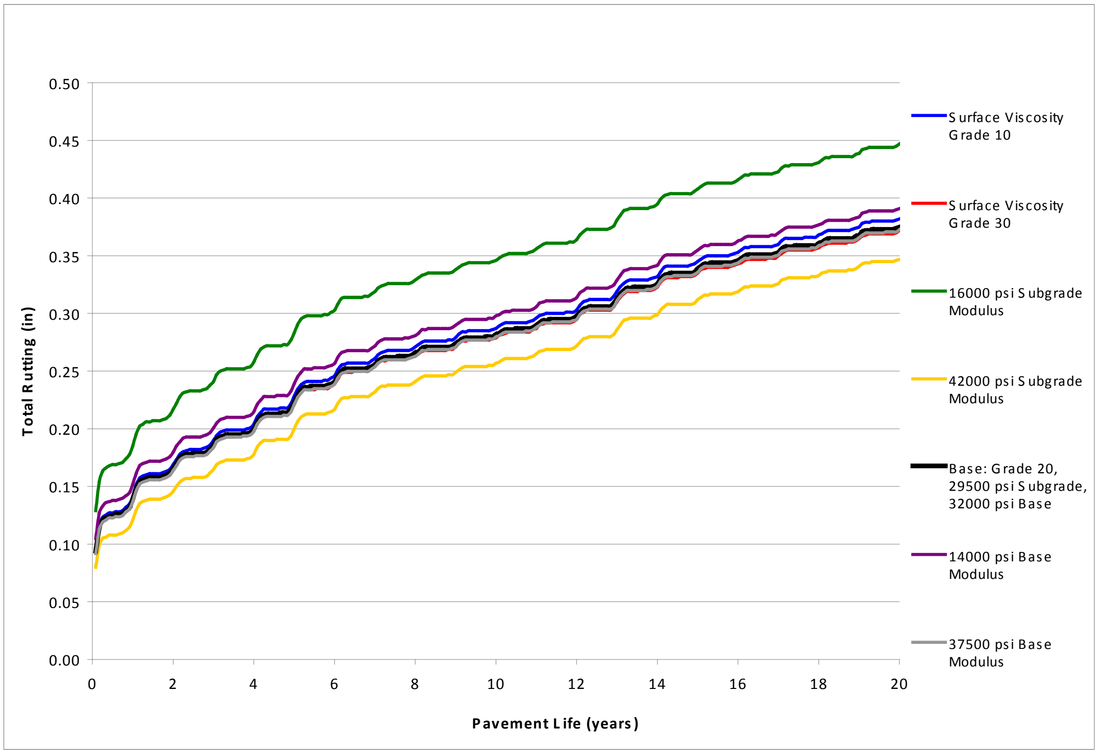

Figure 4.2: Kershaw County Total Rutting Material Sensitivity

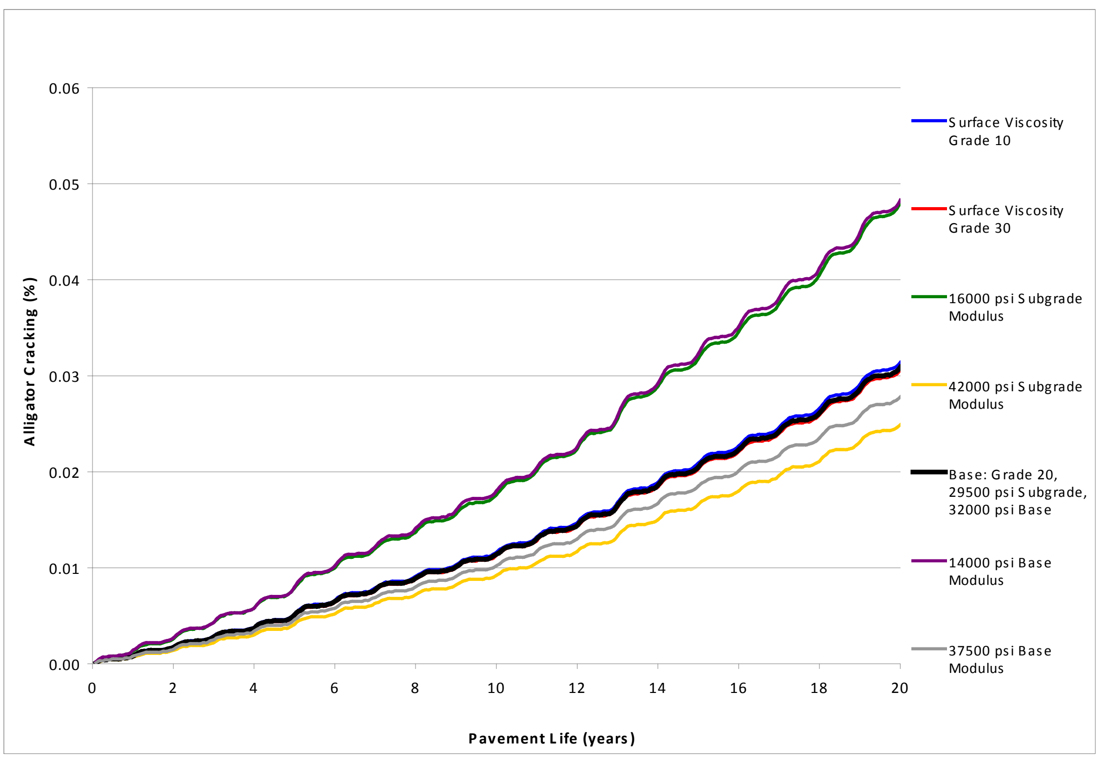

Figure 4.3: Kershaw County Alligator Cracking Material Sensitivity

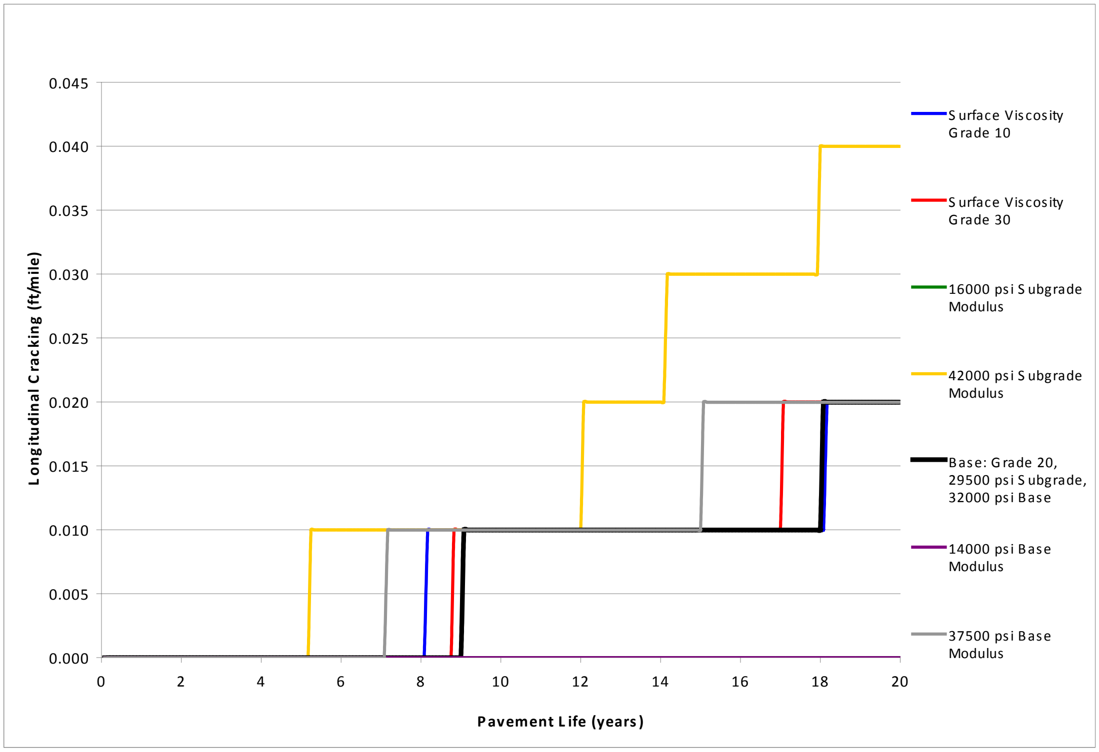

Figure 4.4: Kershaw County Longitudinal Cracking Material Sensitivity

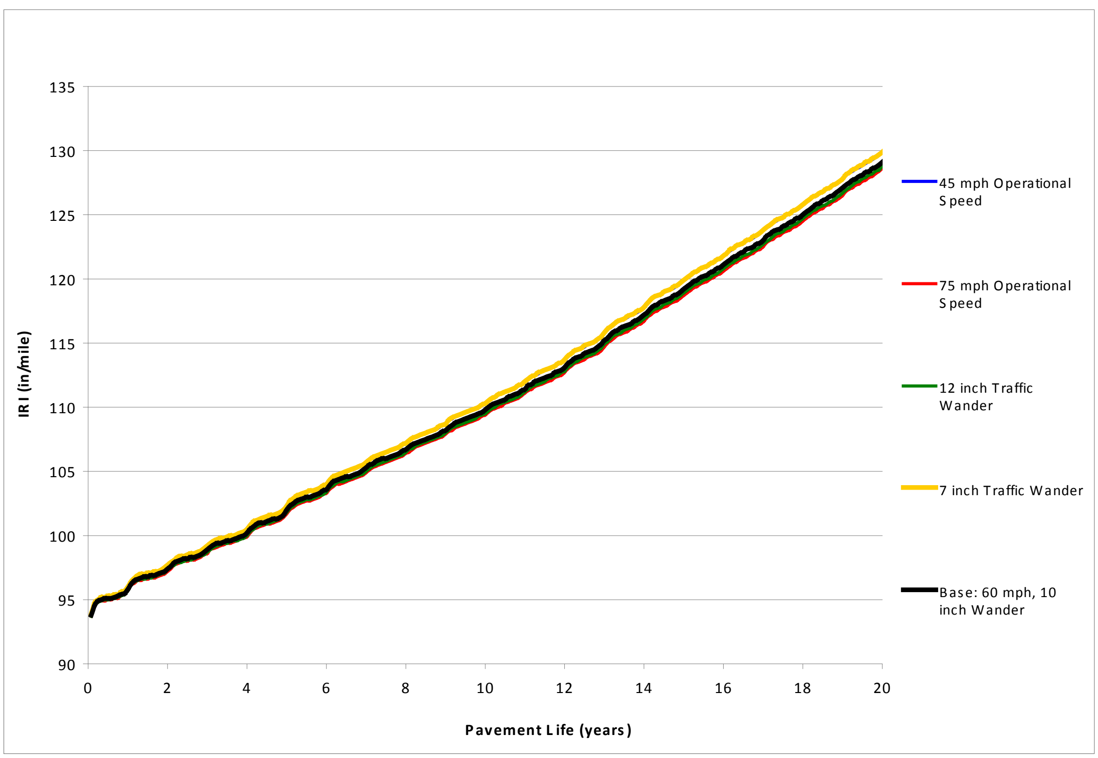

Figure 4.5: Kershaw County IRI General Traffic Sensitivity

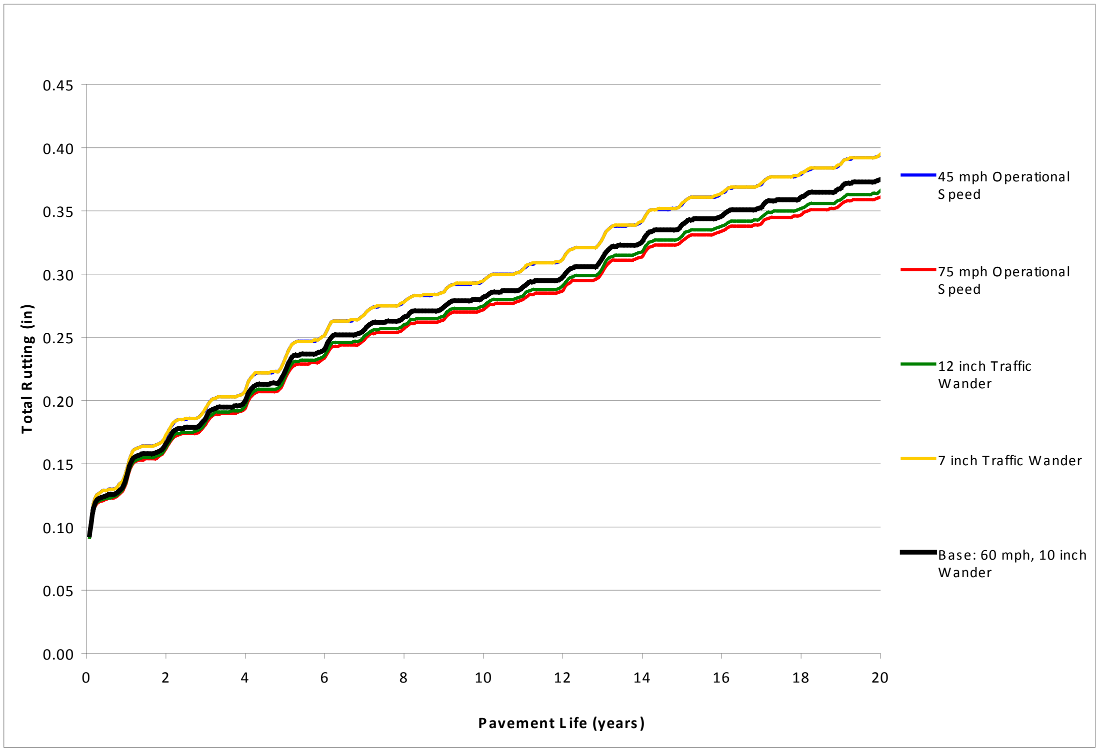

Figure 4.6: Kershaw County Total Rutting General Traffic Sensitivity

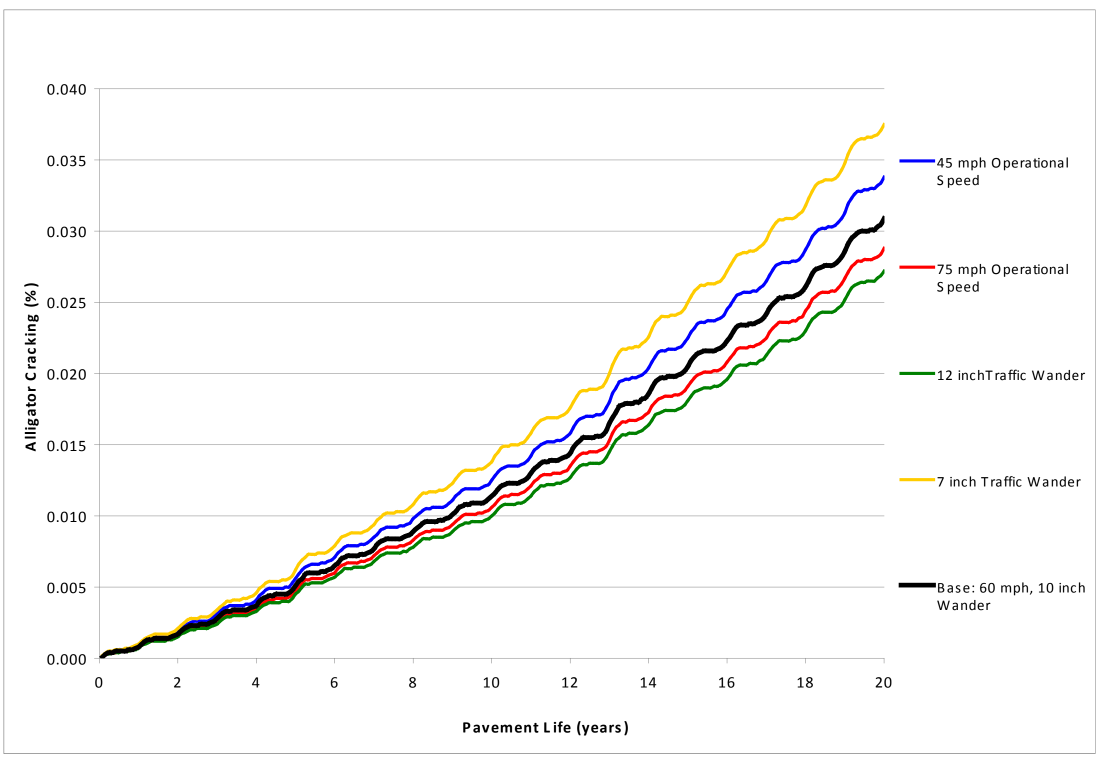

Figure 4.7: Kershaw County Alligator Cracking General Traffic Sensitivity

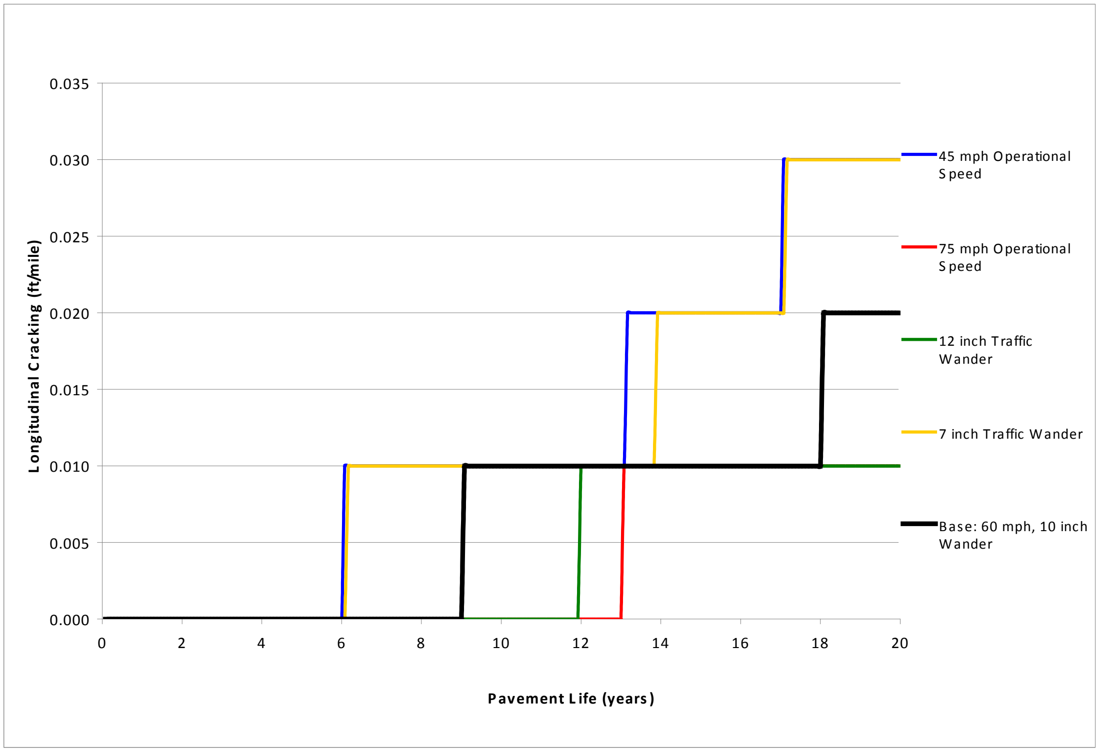

Figure 4.8: Kershaw County Longitudinal Cracking General Traffic Sensitivity

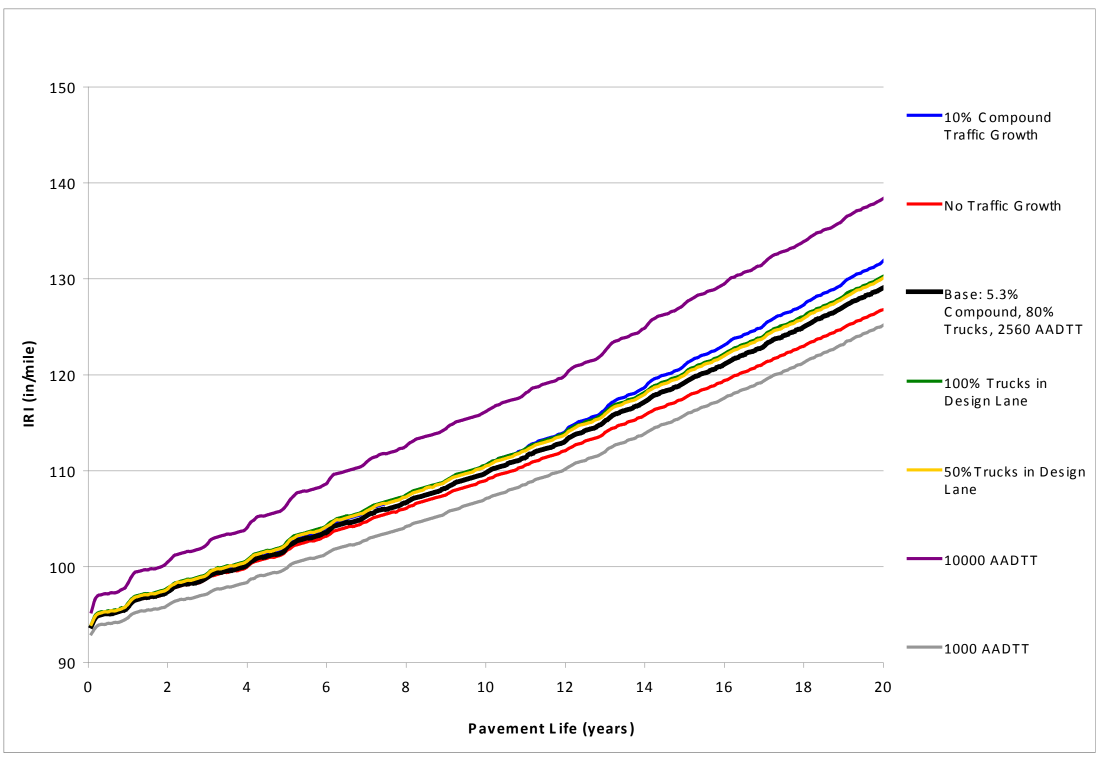

Figure 4.9: Kershaw County IRI Volumetric Traffic Sensitivity

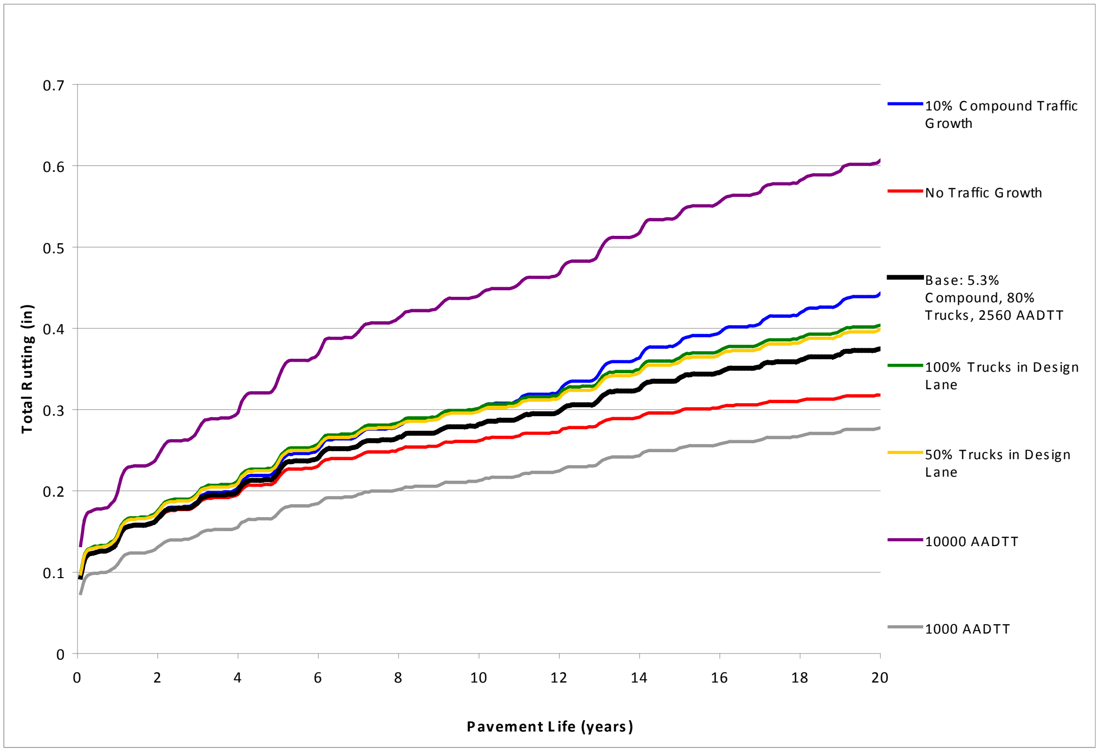

Figure 4.10: Kershaw County Total Rutting Volumetric Traffic Sensitivity

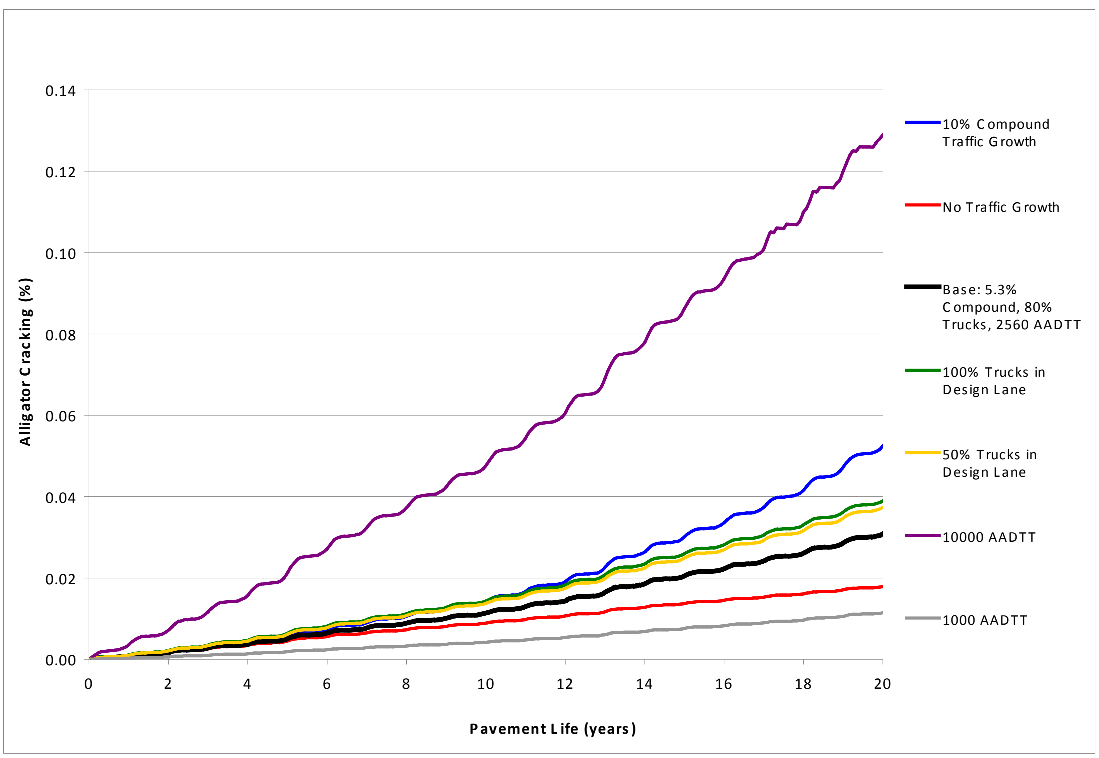

Figure 4.11: Kershaw County Alligator Cracking Volumetric Traffic Sensitivity

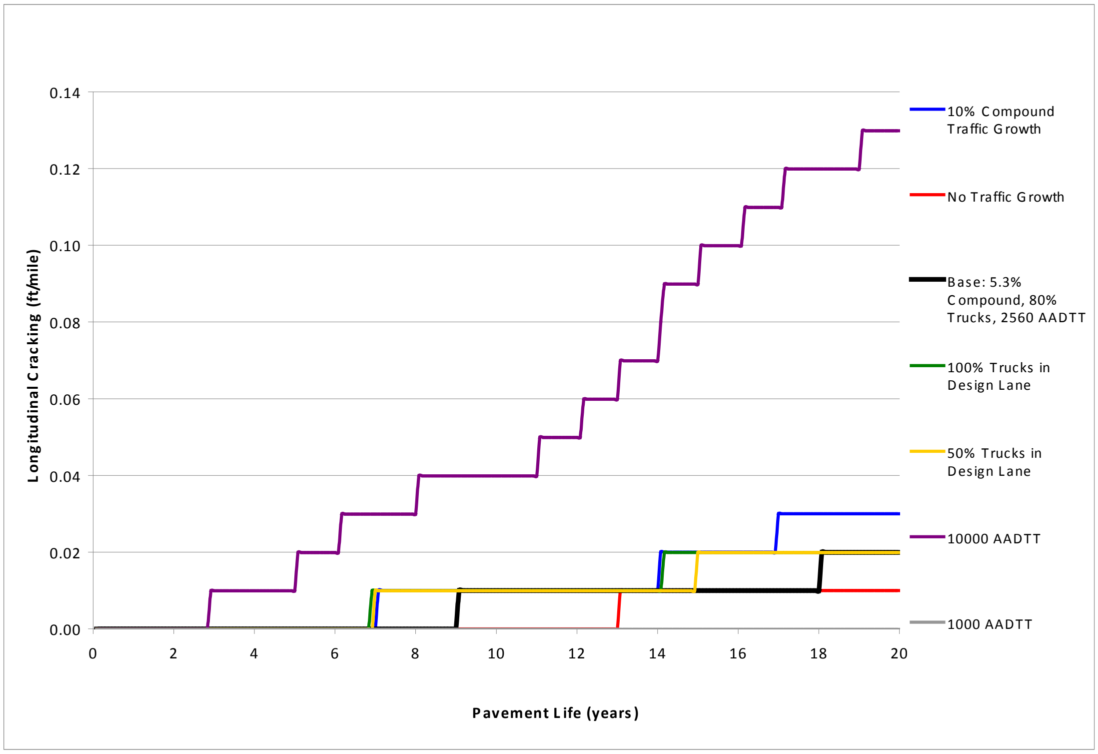

Figure 4.12: Kershaw County Longitudinal Cracking Volumetric Traffic Sensitivity

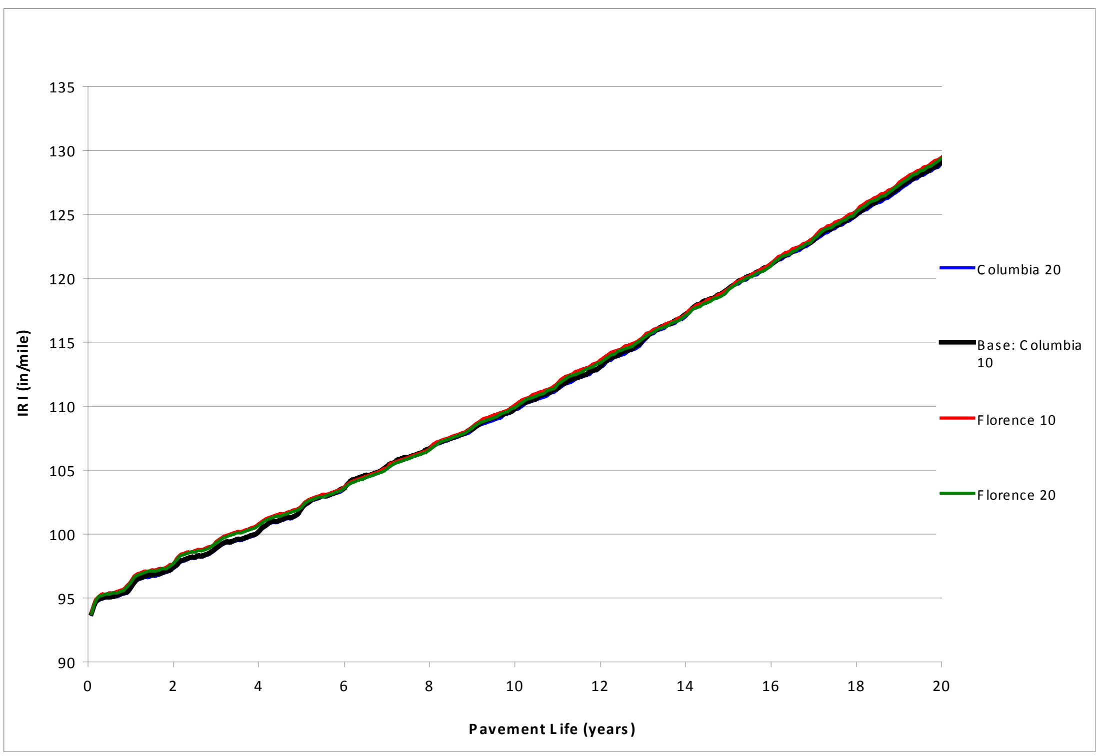

Figure 4.13: Kershaw County IRI Climatic Sensitivity

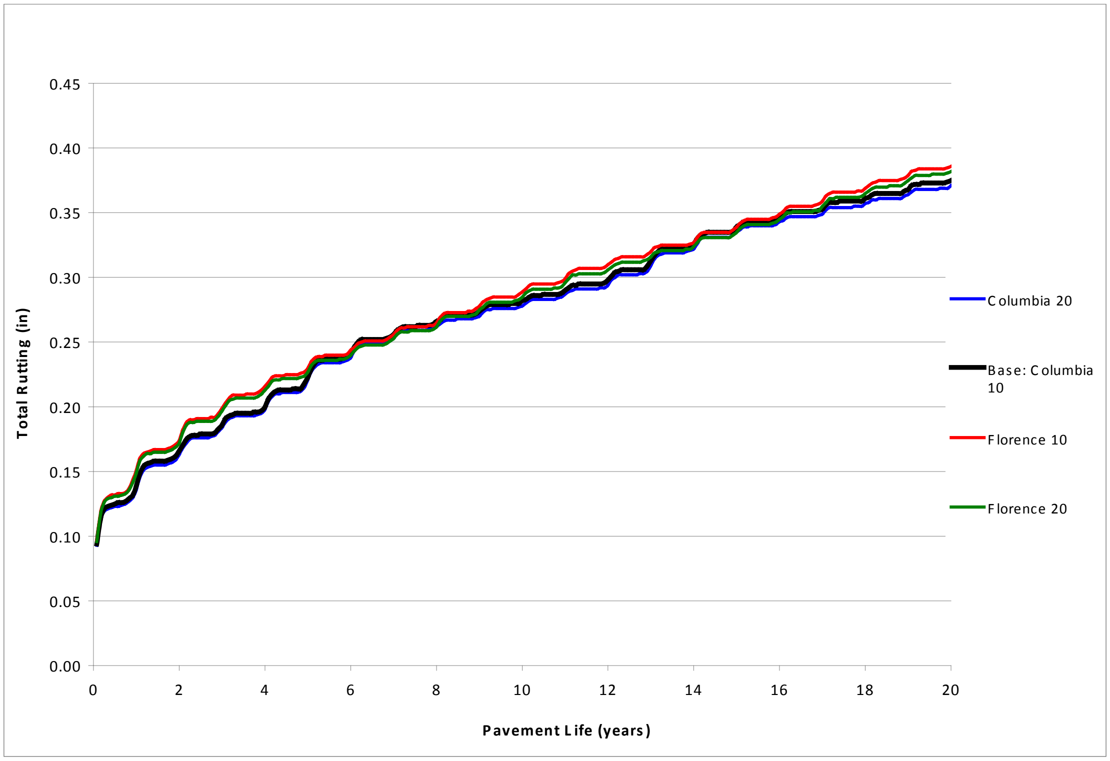

Figure 4.14: Kershaw County Total Rutting Climatic Sensitivity

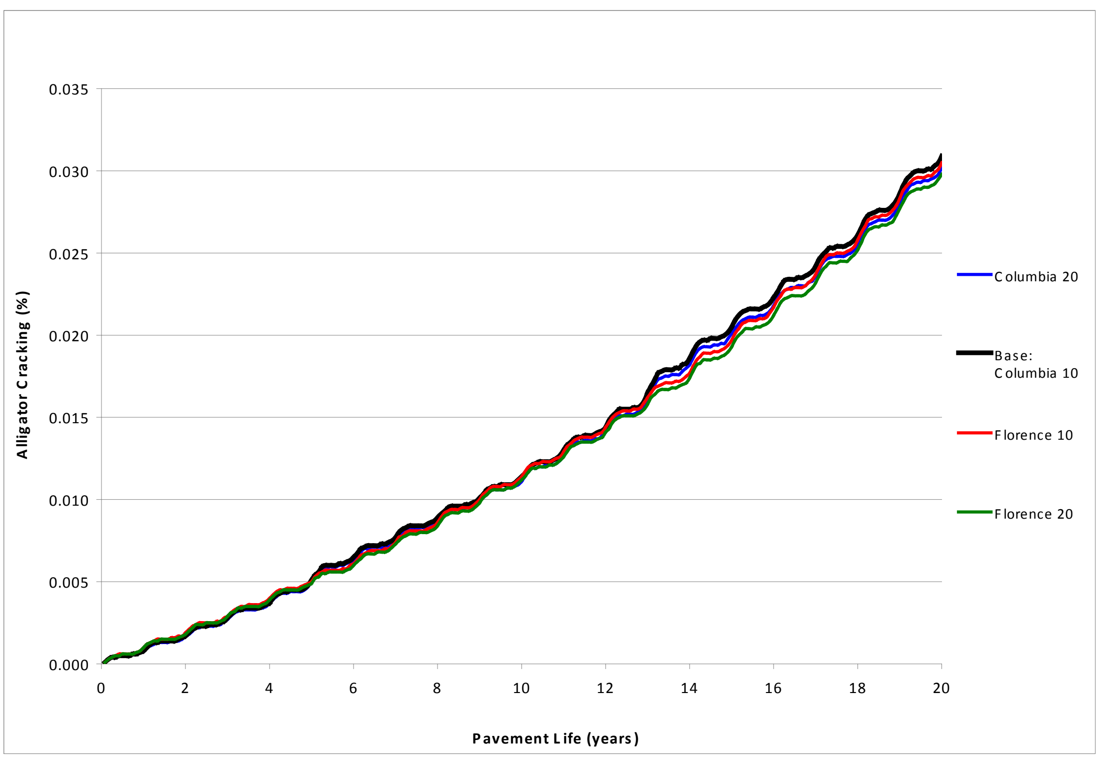

Figure 4.15: Kershaw County Alligator Cracking Climatic Sensitivity

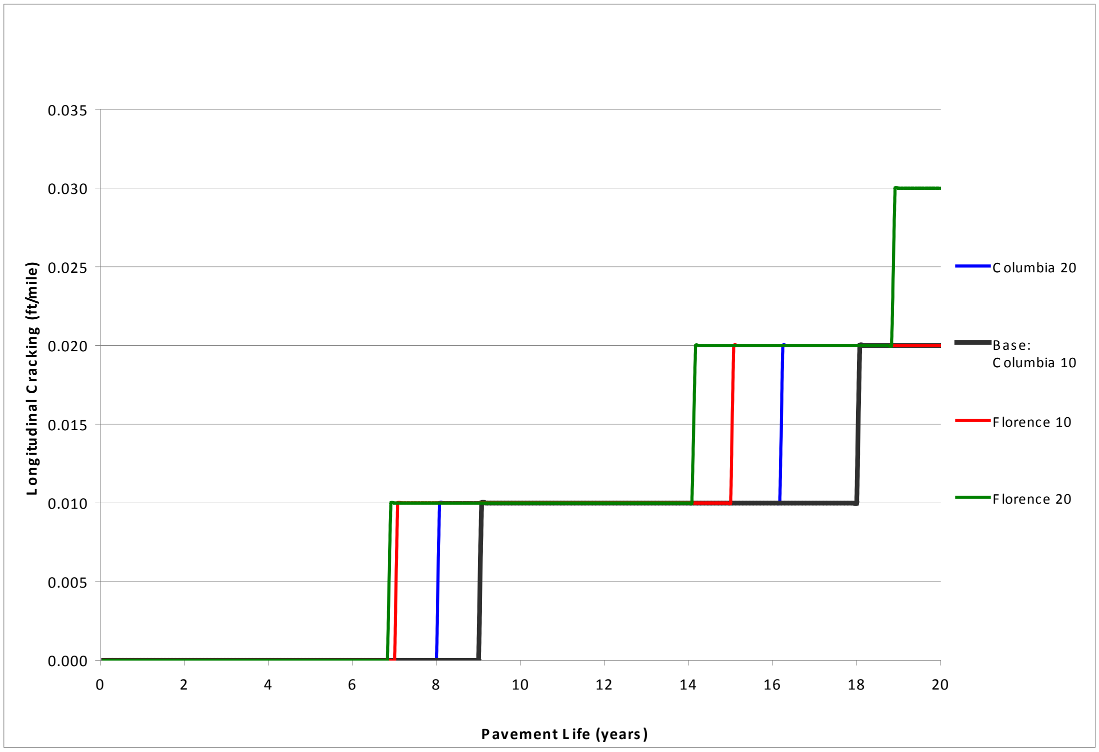

Figure 4.16: Kershaw County Longitudinal Cracking Climatic Sensitivity

Greenville County (Typical flexible)

The sensitivity analysis results for a Greenville County pavement was performed using Version 1.10 (September 2009) of the MEPDG software. The Greenville County structure provided by the SCDOT for analysis contained the following layers: 1.4 inches asphalt concrete surface course, 2.4 inches asphalt concrete binder course, 6.2 inches asphalt stabilized base course, 8.0 inches crushed stone, and an A-7-5 subgrade. The initial AADTT used was 2850. All other inputs used were either MEPDG default values or values provided by the SCDOT. The initial IRI value was assumed to 65 in/mile (SCDOT estimate for new construction). Alligator cracking, longitudinal cracking, total rutting, and IRI were the distress models evaluated. The input parameters utilized are shown in Table 4.3. Figures 4.17 through 4.32 show Greenville County pavement distress predictions during a 20 year period.

Table 4.3: Input Summary (Greenville County, Flexible)

Category	Inputs	Values
Material	AC Surface Viscosity Grade	10, 20 (base), 30
	Subgrade Modulus (psi)	8000, 13000 (base), 17500
	Base Modulus (psi)	20000, 25000 (base), 30000
General Traffic	Traffic Wander (inch)	7, 10 (base), 12
	Operational Speed (mph)	25, 45 (base), 75
Volumetric Traffic	Traffic Growth Rate	10% Compound, 3.5% Compound (base), No Growth
	Percent Trucks in Design Lane	50, 80 (base), 100
	AADTT	1000, 2850 (base), 10000
Climate	Location, Depth to Water Table (ft)	Greenville 1, Greenville 10 (base), Greenwood 1, Greenwood 10

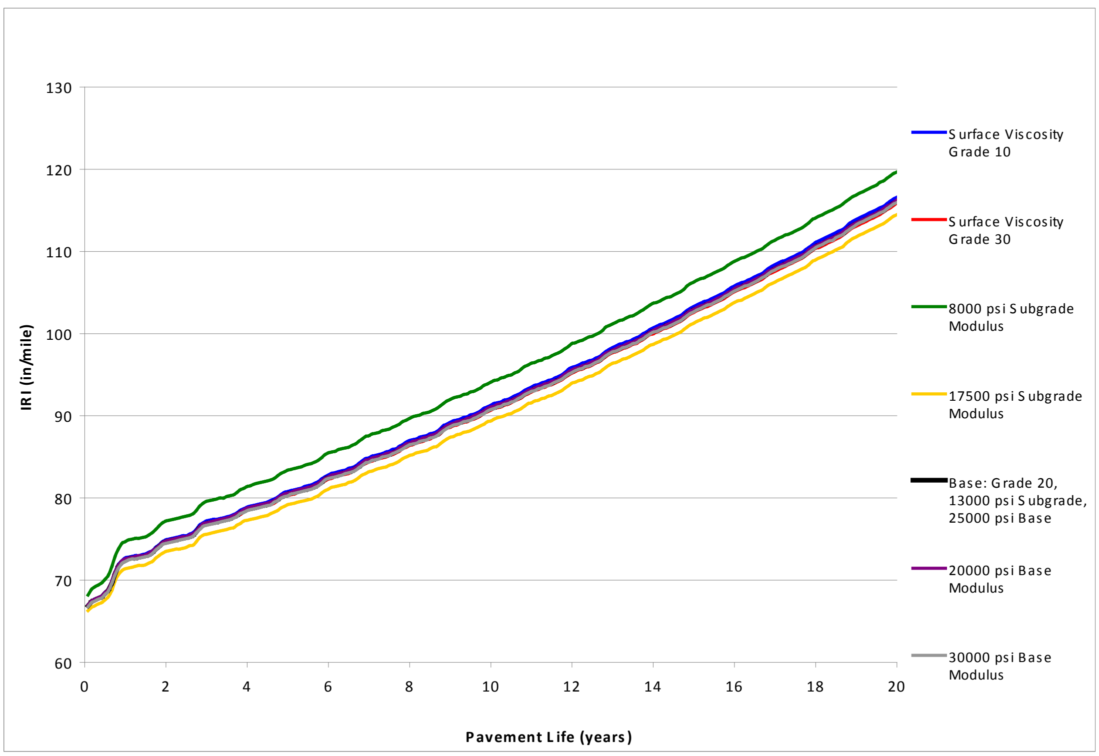

Figure 4.17: Greenville County IRI Material Sensitivity

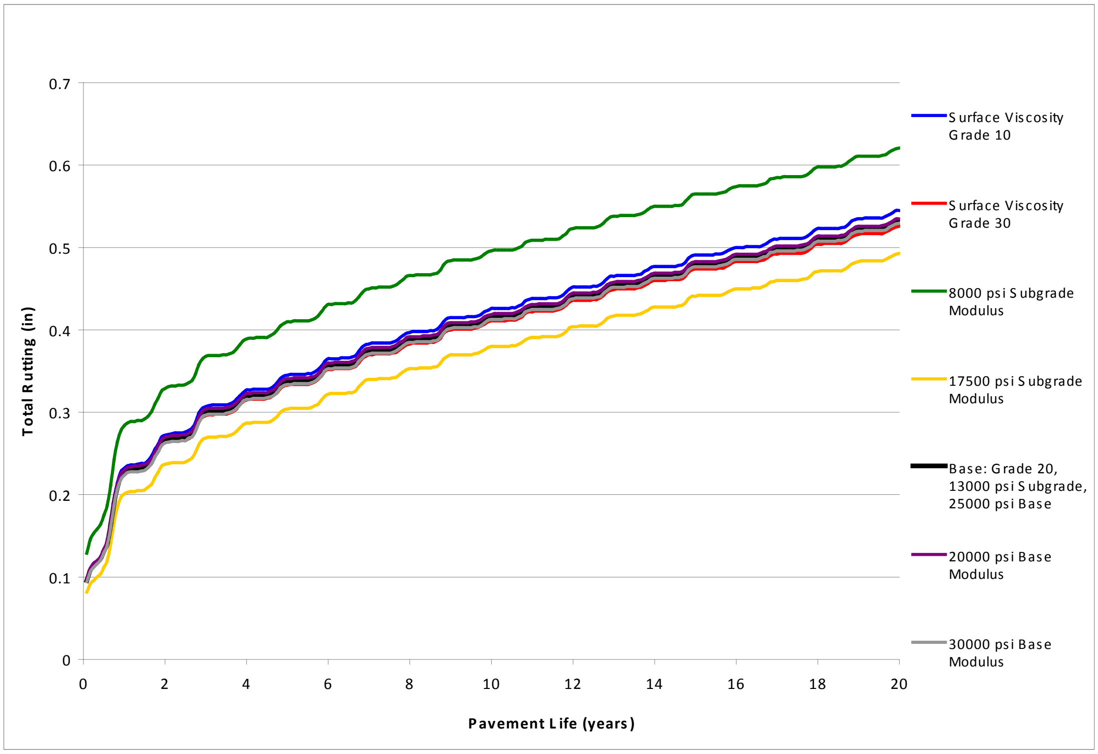

Figure 4.18: Greenville County Total Rutting Material Sensitivity

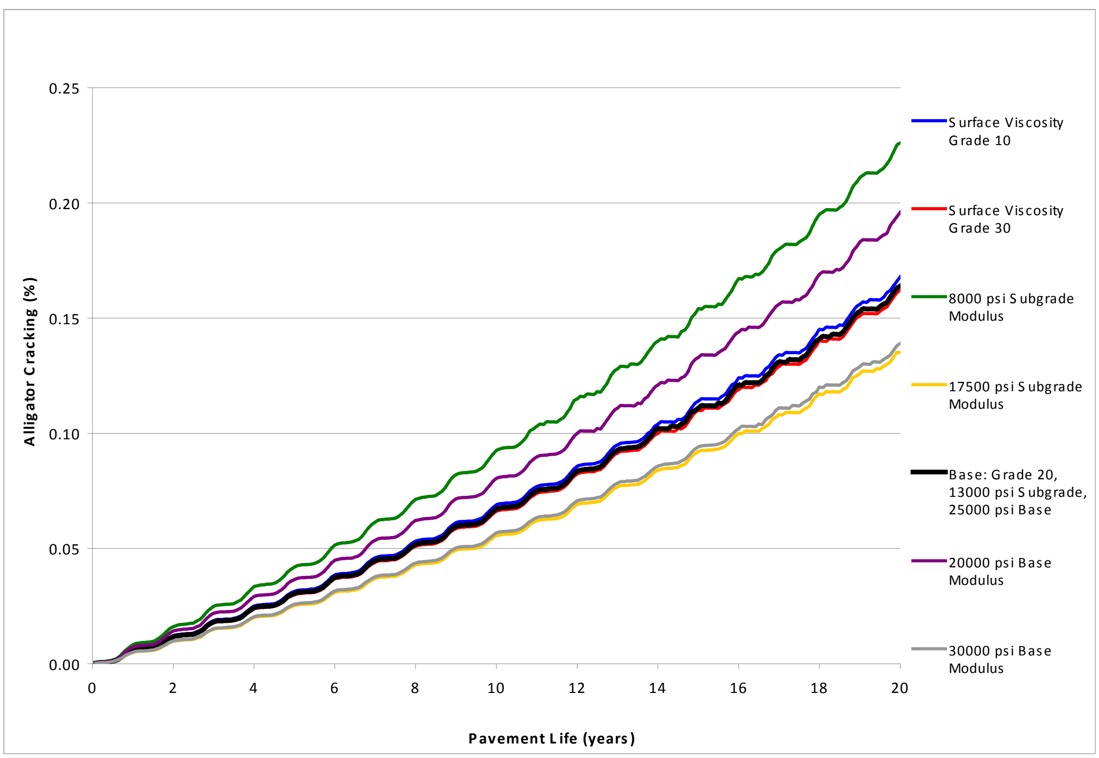

Figure 4.19: Greenville County Alligator Cracking Material Sensitivity

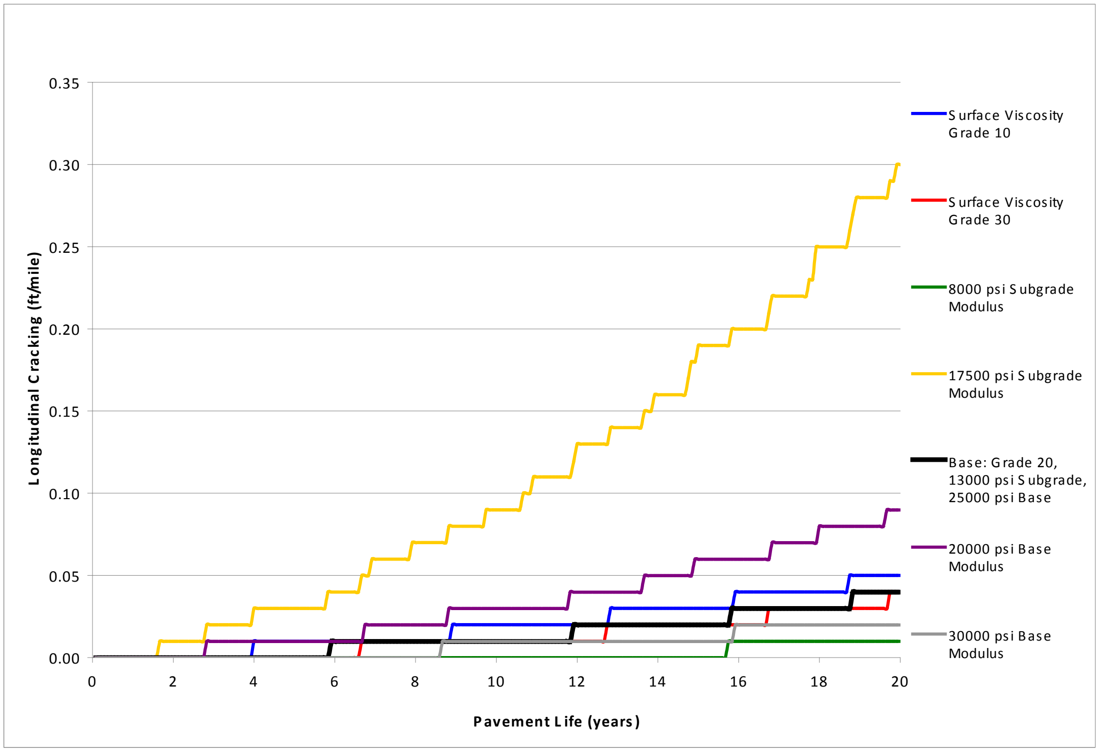

Figure 4.20: Greenville County Longitudinal Cracking Material Sensitivity

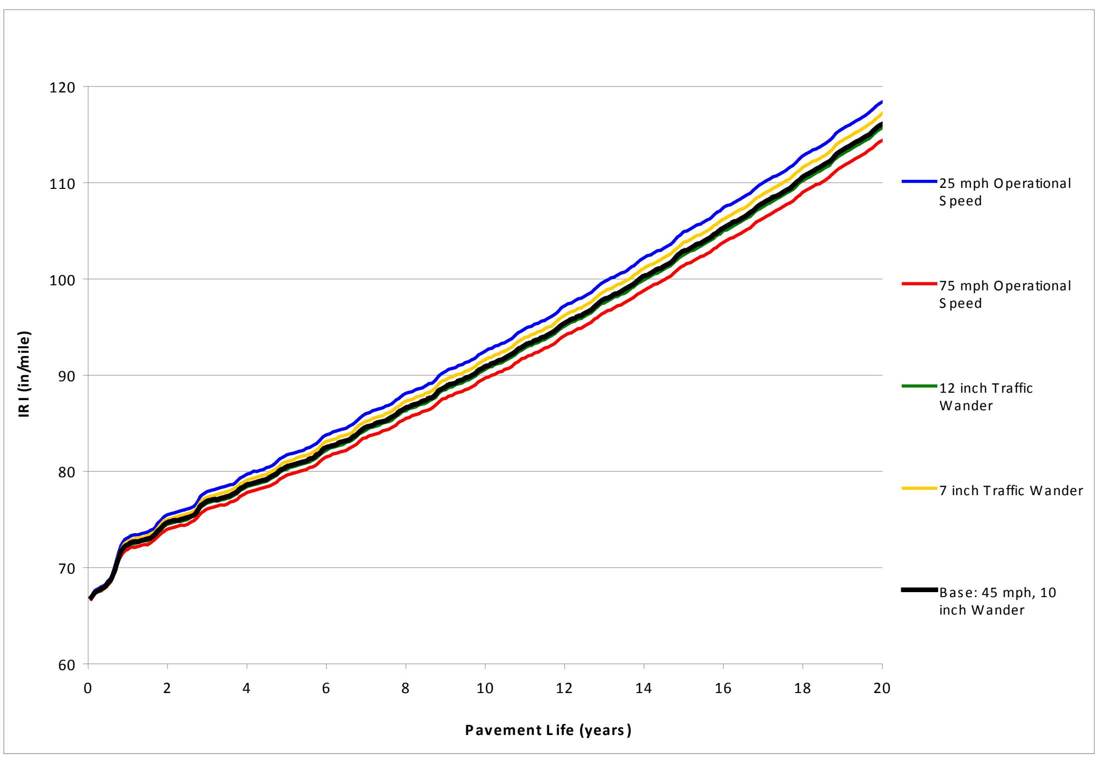

Figure 4.21: Greenville County IRI General Traffic Sensitivity

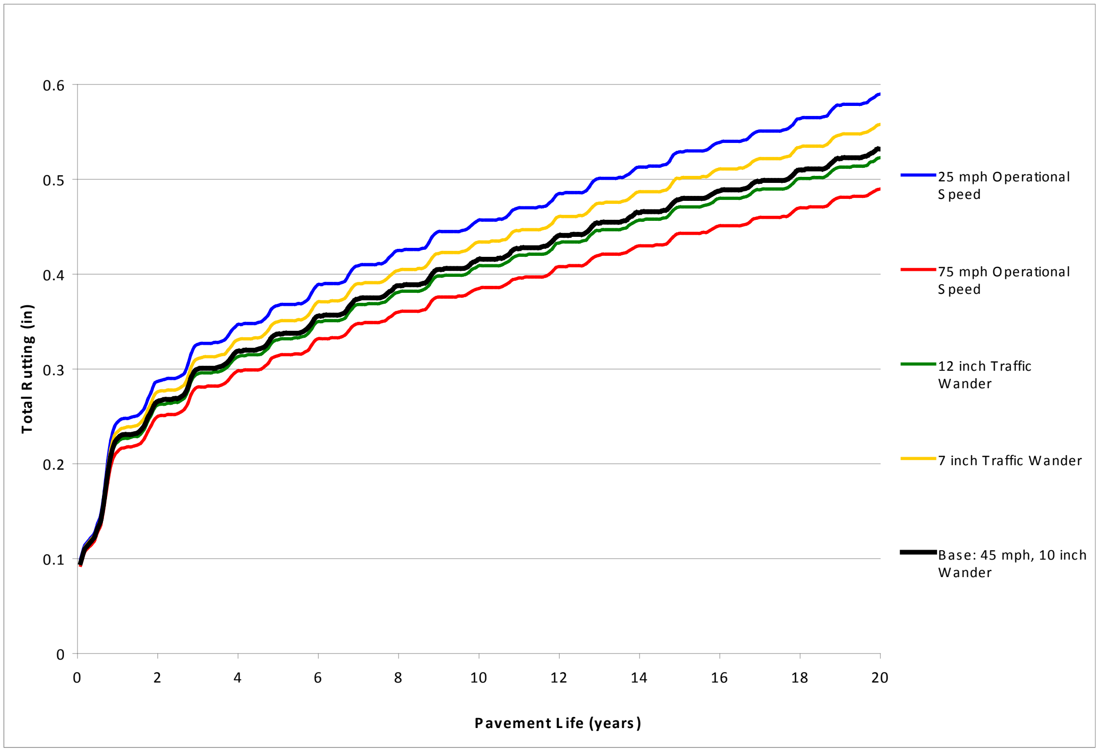

Figure 4.22: Greenville County Total Rutting General Traffic Sensitivity

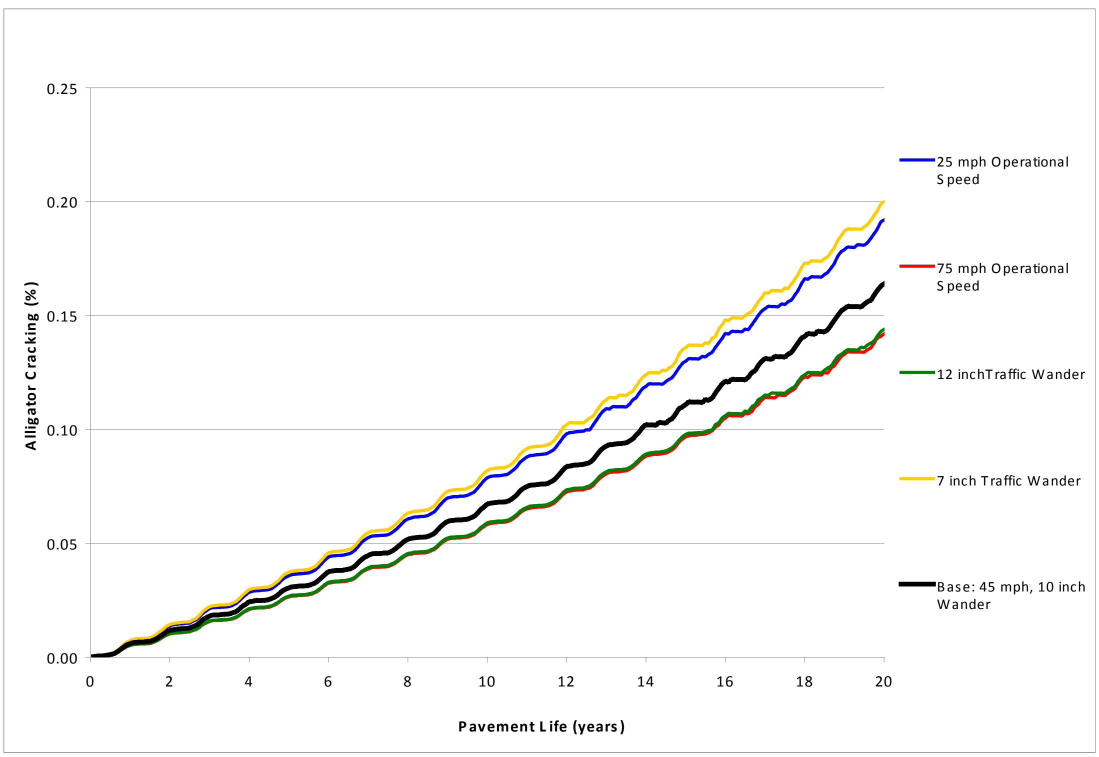

Figure 4.23: Greenville County Alligator Cracking General Traffic Sensitivity

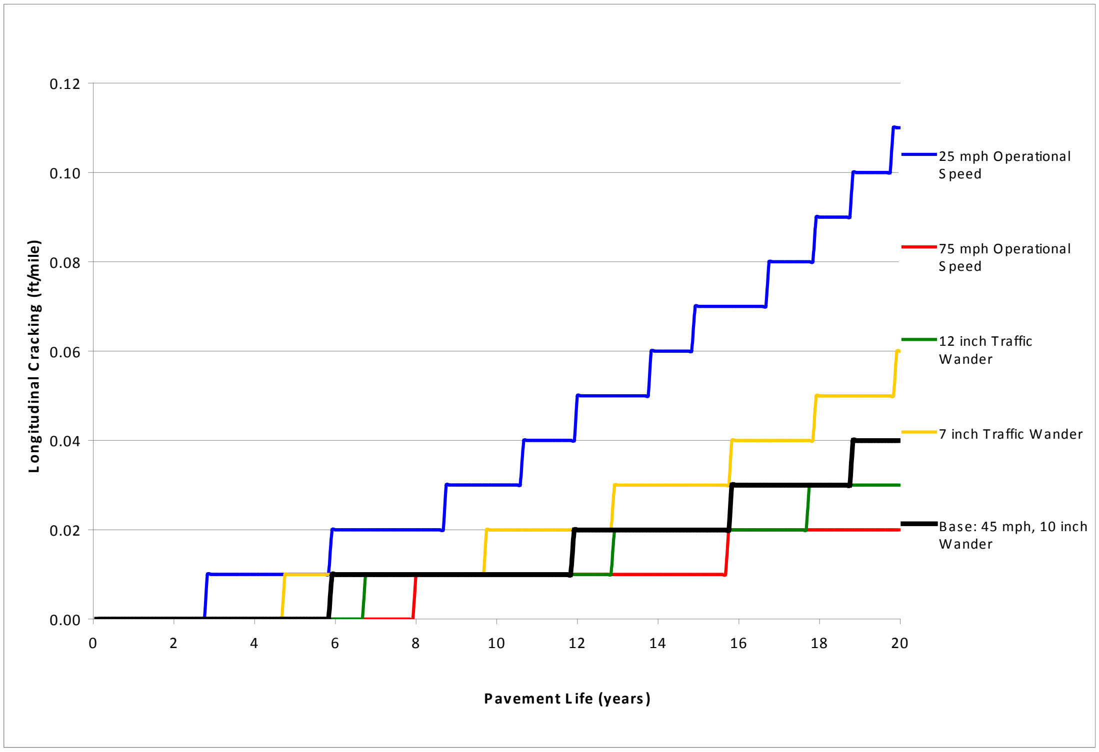

Figure 4.24: Greenville County Longitudinal Cracking General Traffic Sensitivity

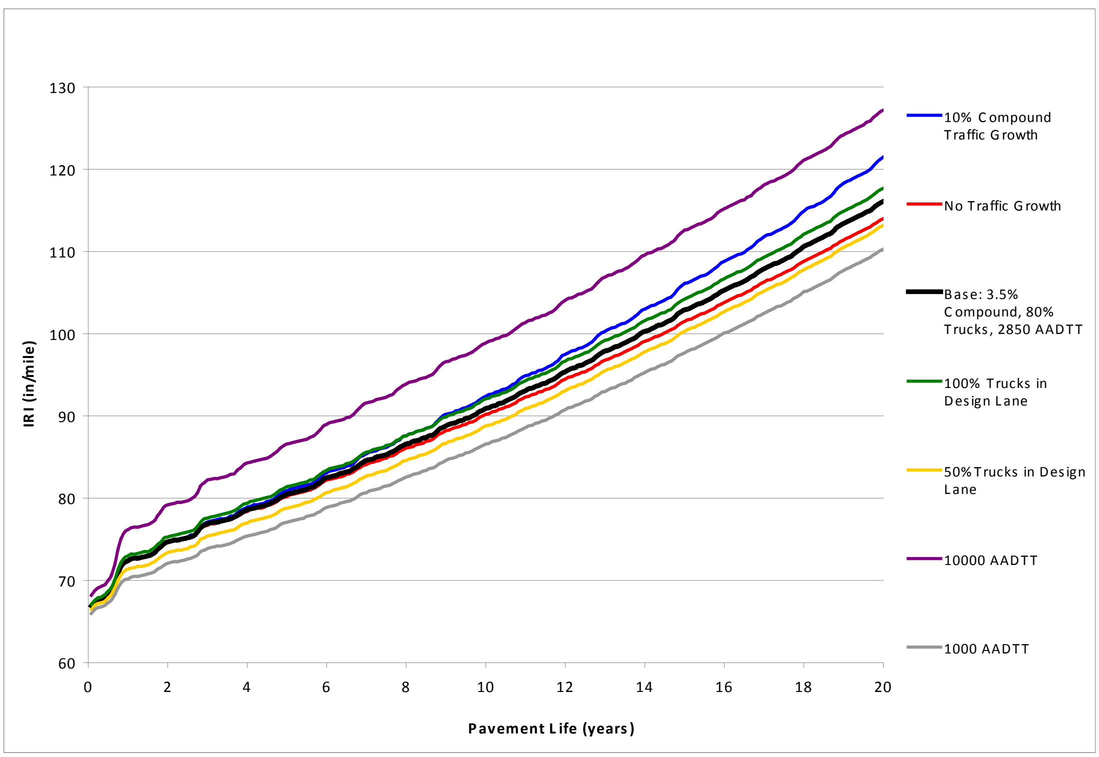

Figure 4.25: Greenville County IRI Volumetric Traffic Sensitivity

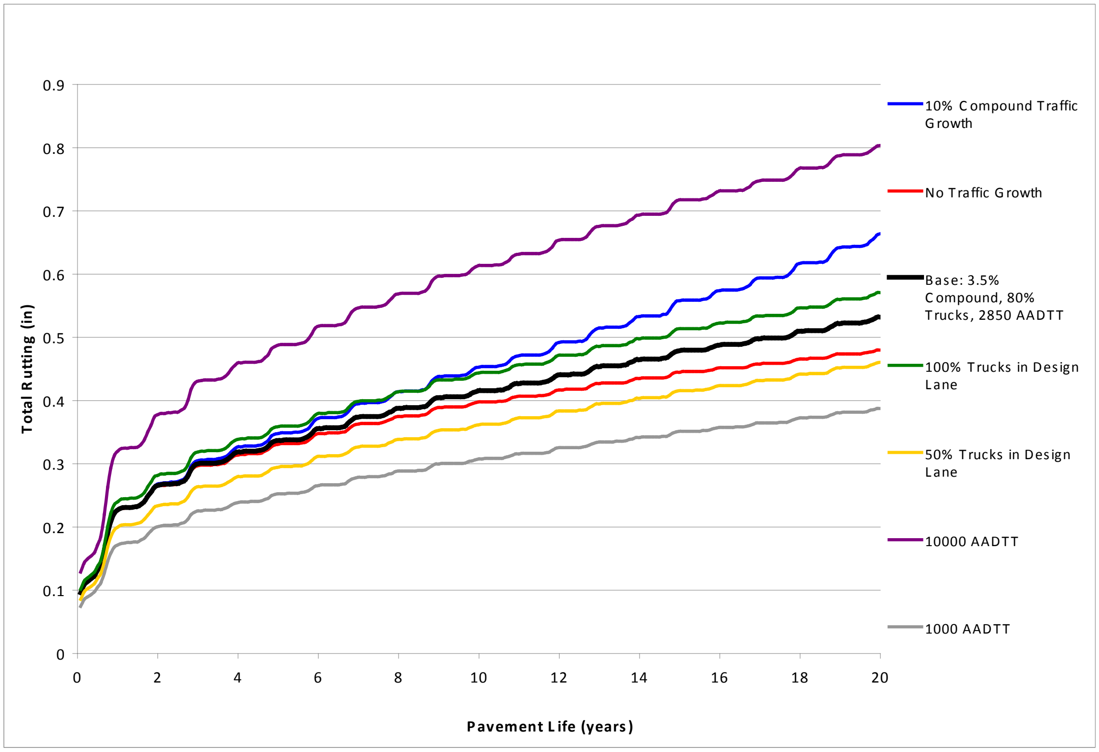

Figure 4.26: Greenville County Total Rutting Volumetric Traffic Sensitivity

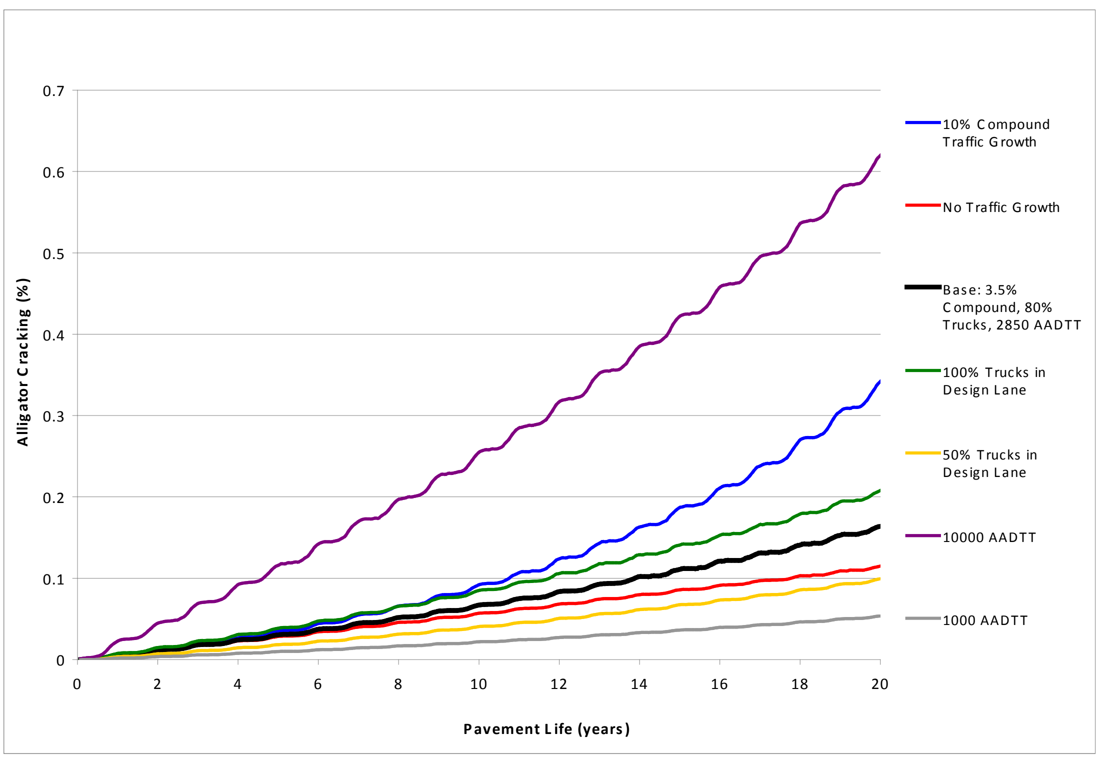

Figure 4.27: Greenville County Alligator Cracking Volumetric Traffic Sensitivity

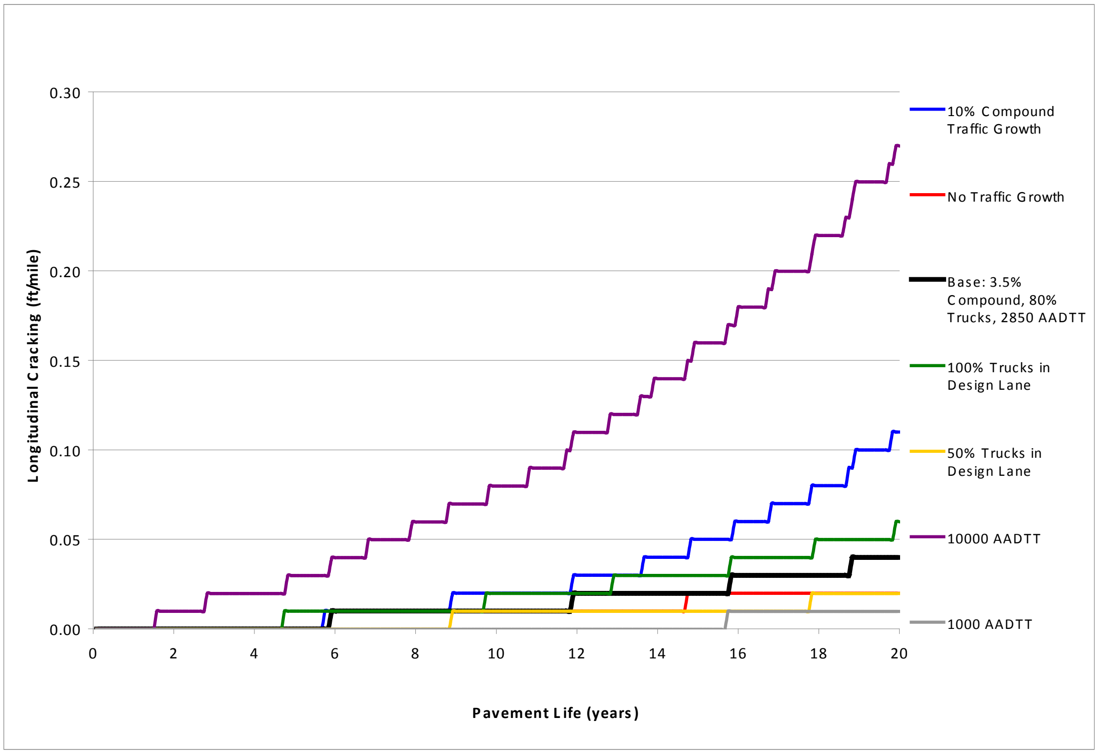

Figure 4.28: Greenville County Longitudinal Cracking Volumetric Traffic Sensitivity

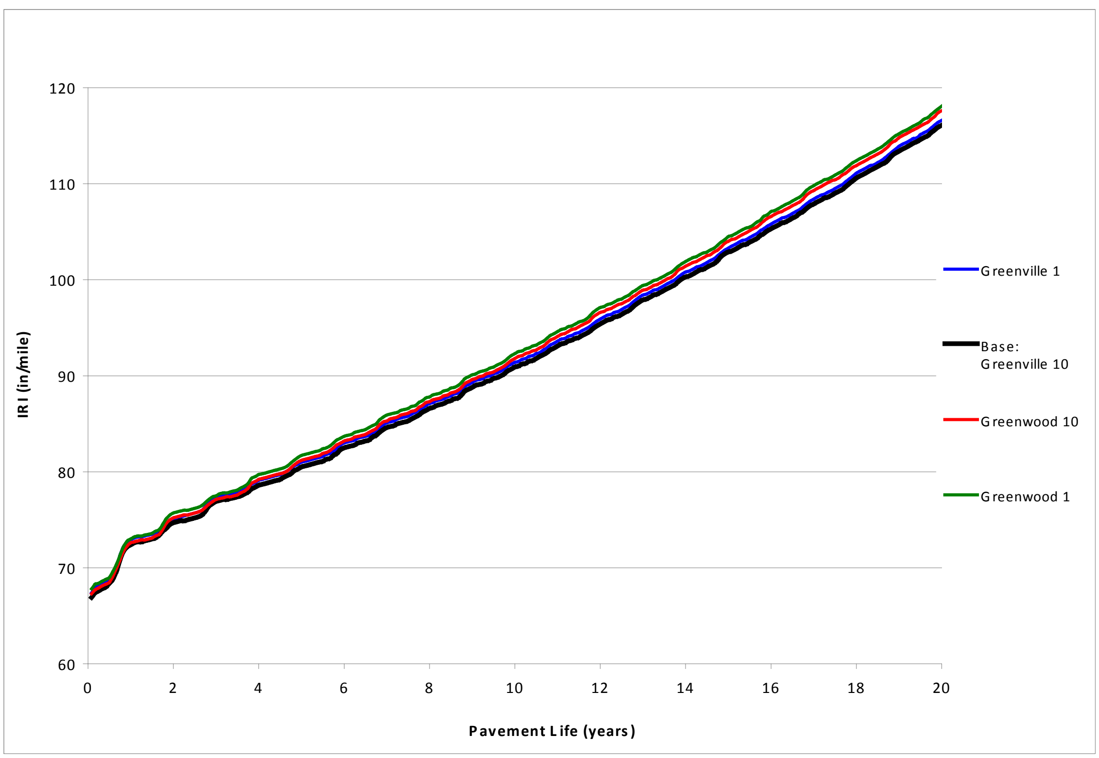

Figure 4.29: Greenville County IRI Climatic Sensitivity

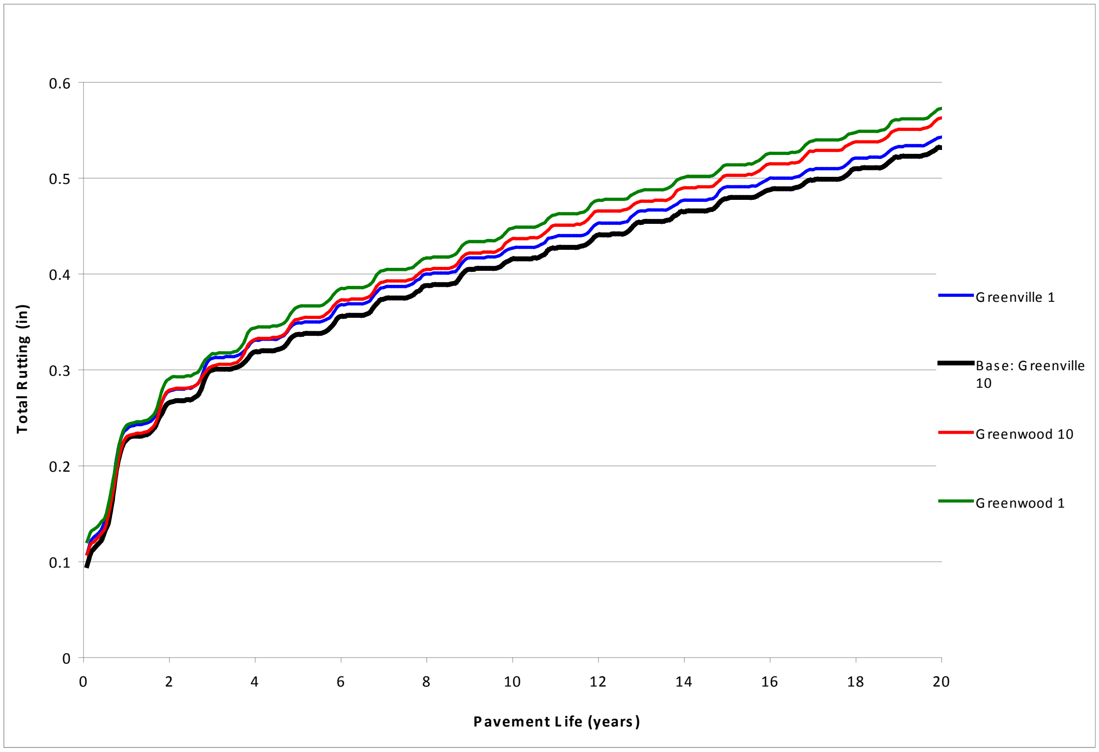

Figure 4.30: Greenville County Total Rutting Climatic Sensitivity

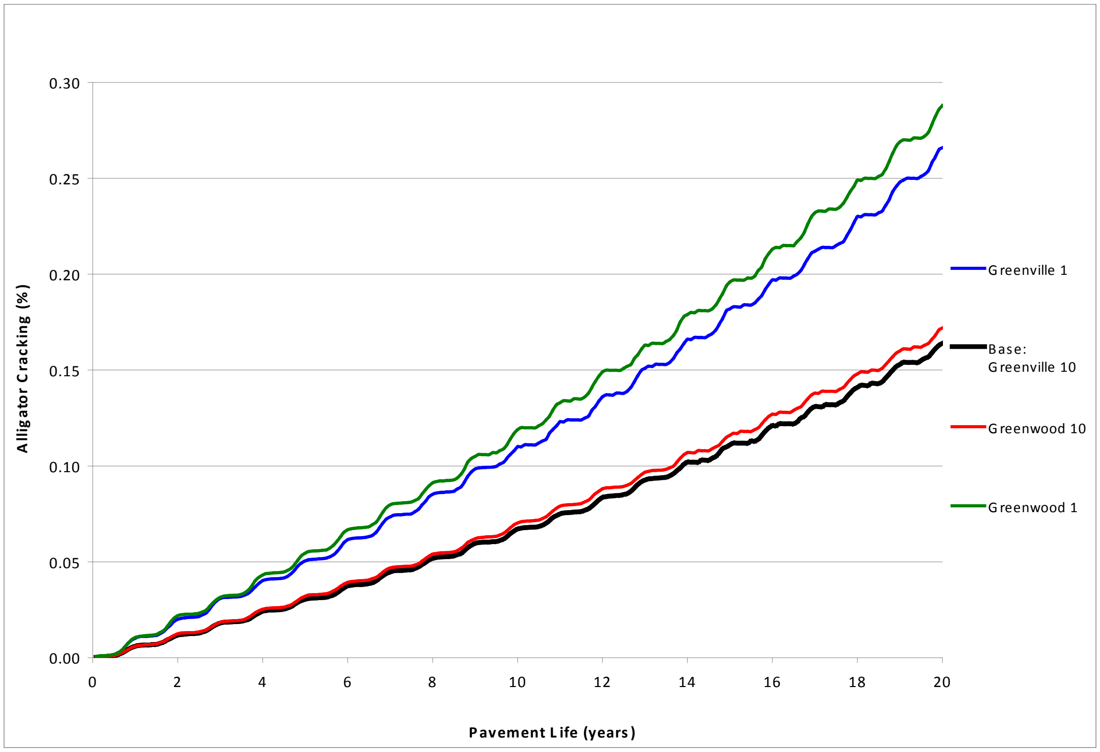

Figure 4.31: Greenville County Alligator Cracking Climatic Sensitivity

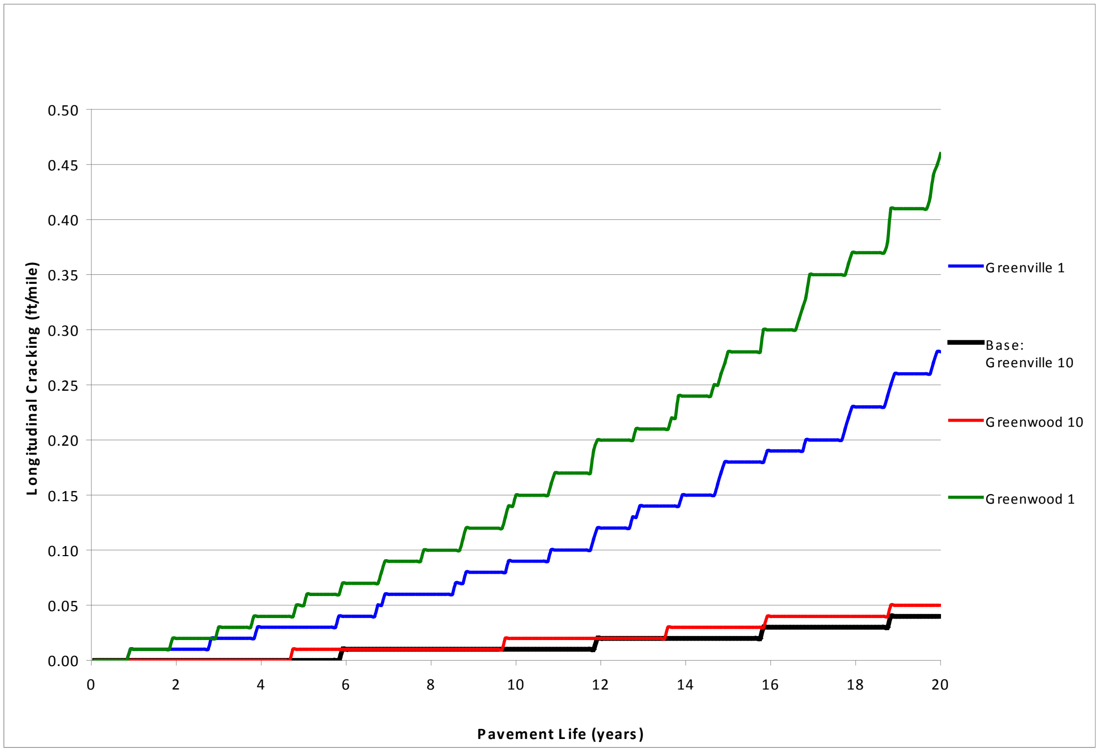

Figure 4.32: Greenville County Longitudinal Cracking Climatic Sensitivity

Richland County (Typical Flexible)

The sensitivity analysis results for a Richland County pavement was performed using Version 1.10 of the MEPDG software. This design represents a high volume roadway using SUPERPAVE asphalt binder gradation and other design features in accordance with current SCDOT procedures. The Richland County structure provided by the SCDOT for analysis contained the following layers: 3.8 inches asphalt concrete surface course, 7.1 inches asphalt concrete base course, 8.0 inches crushed stone, and an A-3 subgrade. The initial AADTT used was 6000. All other inputs were either MEPDG default values or values provided by the SCDOT. Alligator cracking, longitudinal cracking, total rutting, and IRI were the distress models evaluated. The initial IRI was assumed to be 65 in/mile. The input parameters utilized are shown in Table 4.4. Figures

4.33 through 4.48 show how the Richland County pavement distress prediction during a 20 year period.

Table 4.4: Input Summary (Richland County, Flexible)

Category	Inputs	Values
Material	AC Superpave Binder Grade	58/-22, 64/-22 (base), 72/-22
	Subgrade Modulus (psi)	14000, 24500 (base), 35500
	Base Modulus (psi)	20000, 25000, 30000 (base)
General Traffic	Traffic Wander (inch)	7, 10 (base), 12
	Operational Speed (mph)	45, 60 (base), 75
Volumetric Traffic	Traffic Growth Rate	10% Compound, 3.2% Compound (base), No Growth
	Percent Trucks in Design Lane	50, 80 (base), 100
	AADTT	1000, 6000 (base), 10000
Climate	Location, Depth to Water Table (ft)	Columbia 15, Columbia 25 (base), Orangeburg 15, Orangeburg 25

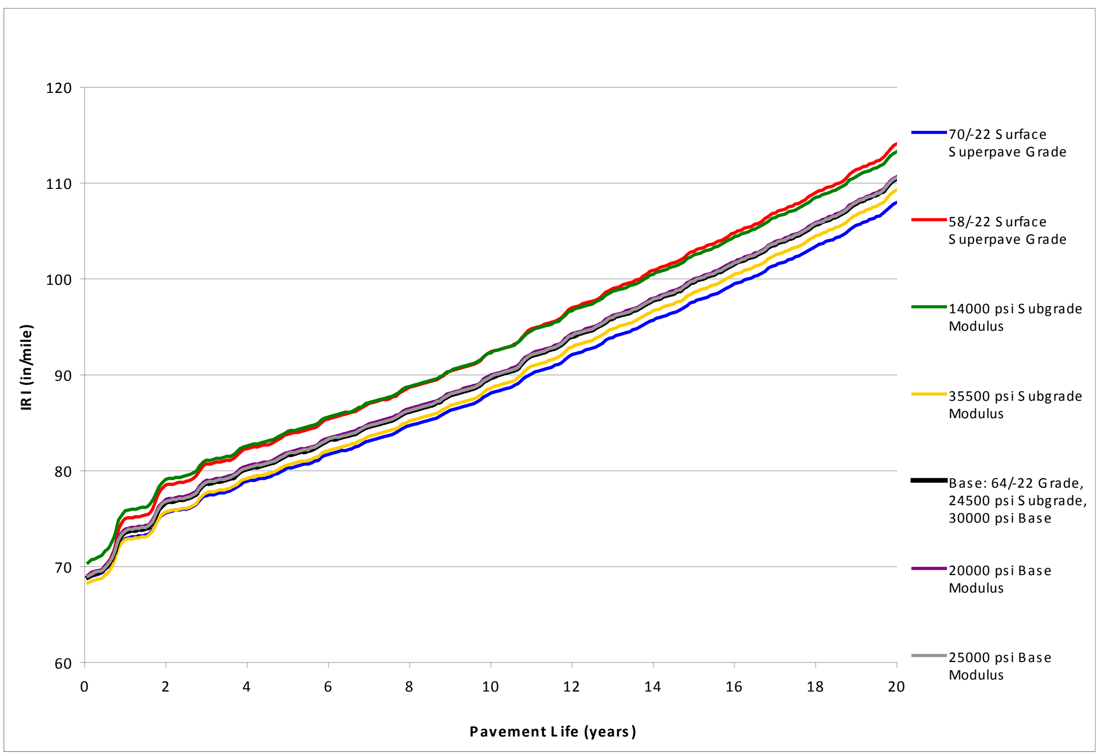

Figure 4.33: Richland County IRI Material Sensitivity

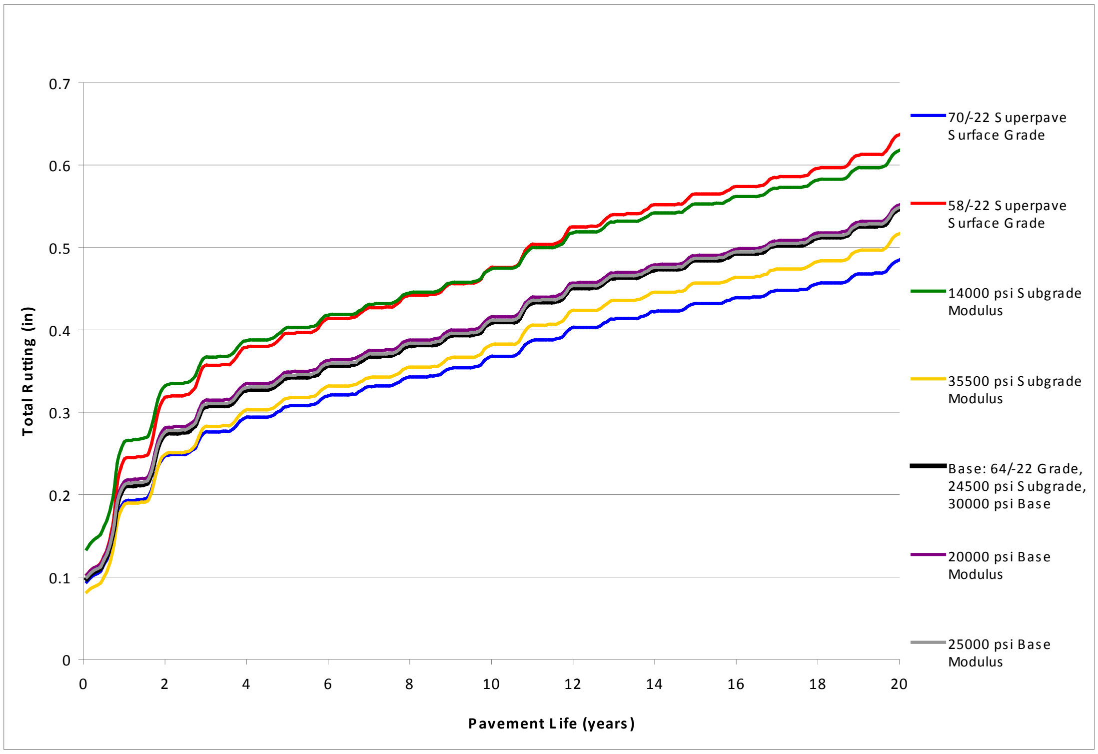

Figure 4.34: Richland County Total Rutting Material Sensitivity

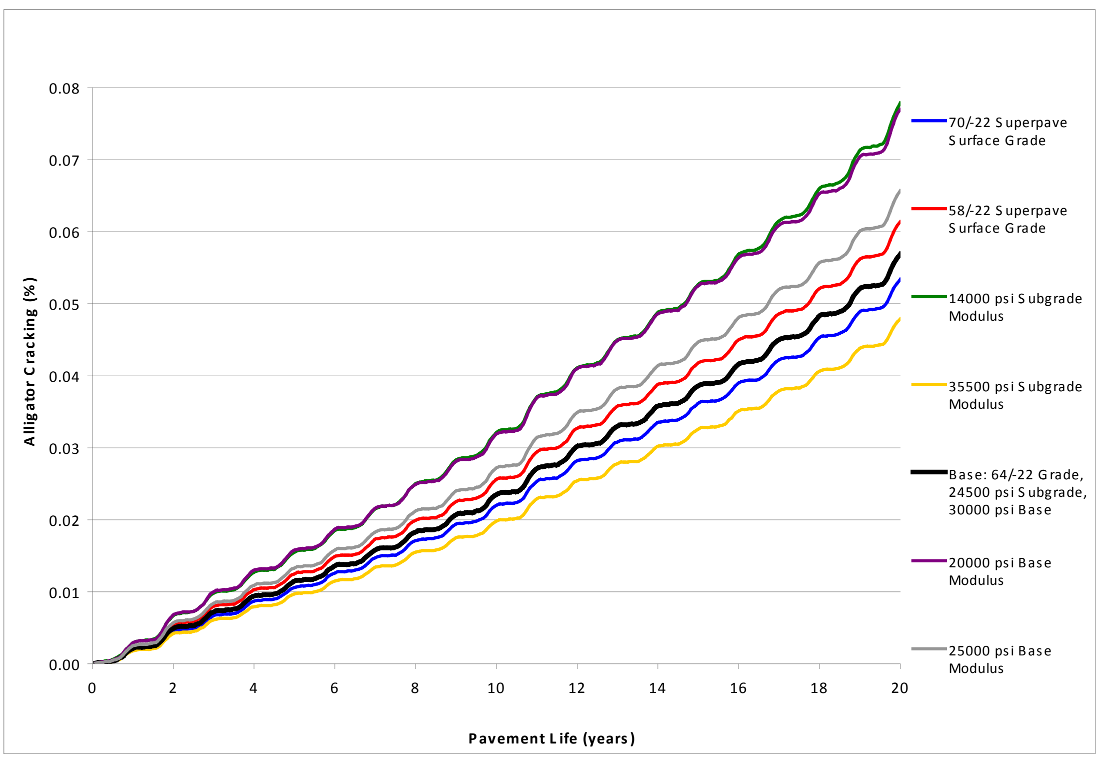

Figure 4.35: Richland County Alligator Cracking Material Sensitivity

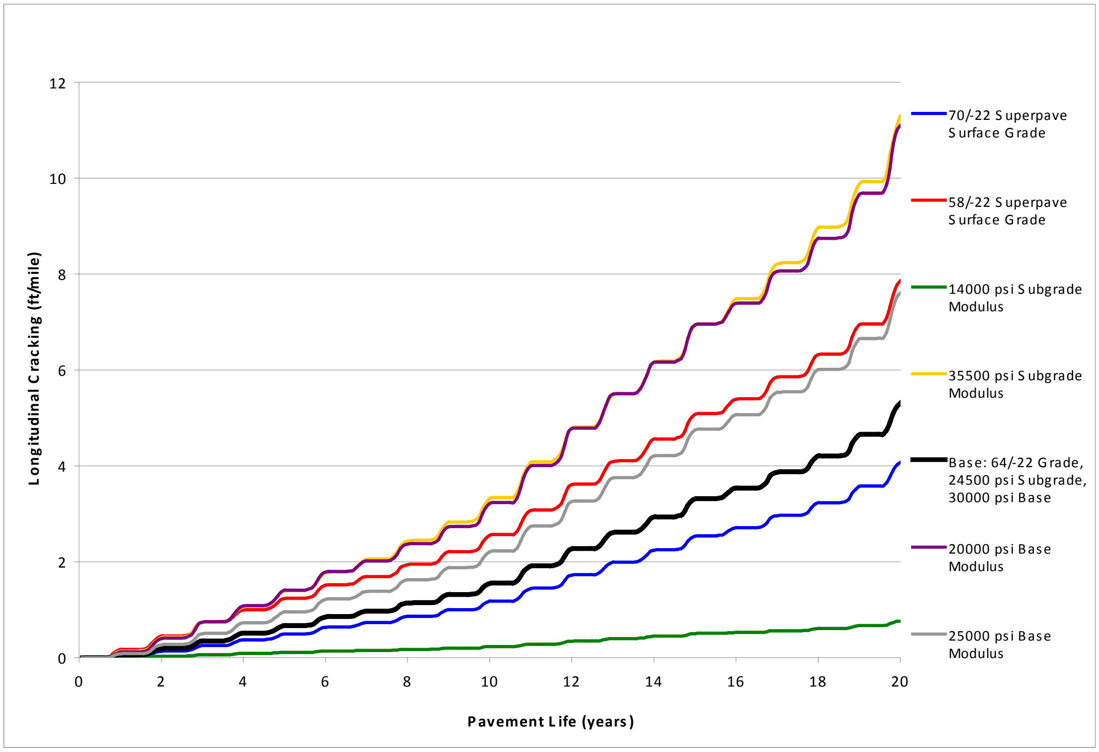

Figure 4.36: Richland County Longitudinal Cracking Material Sensitivity

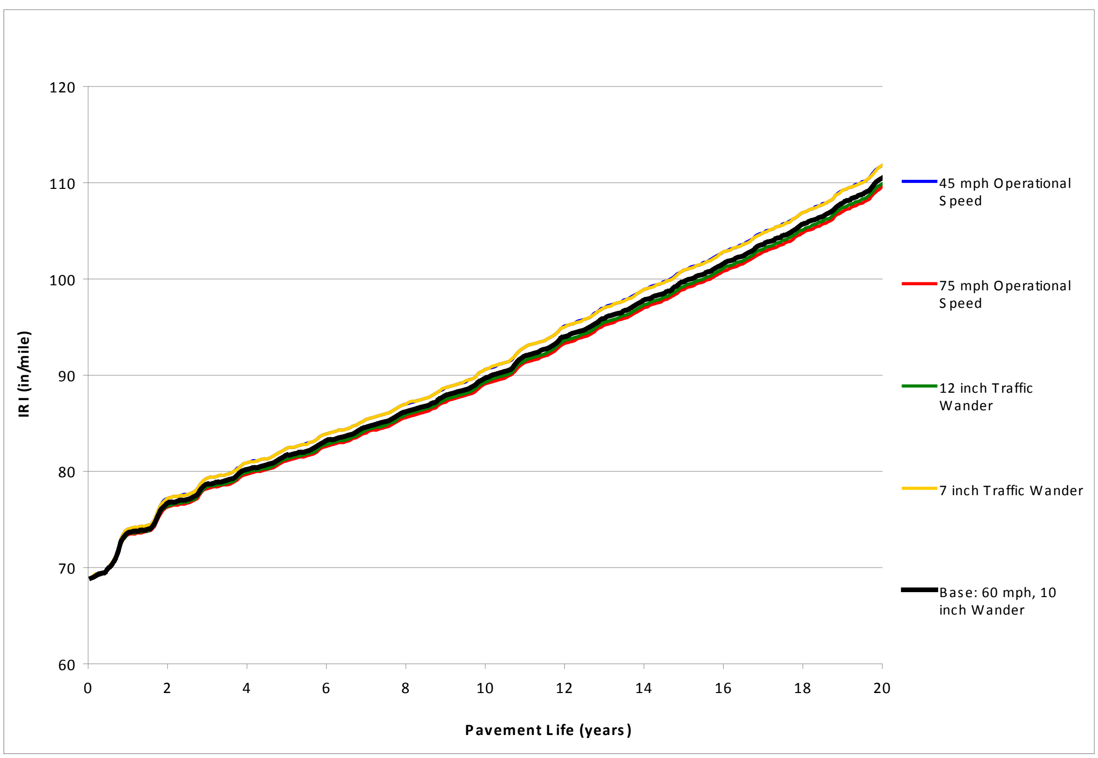

Figure 4.37: Richland County IRI General Traffic Sensitivity

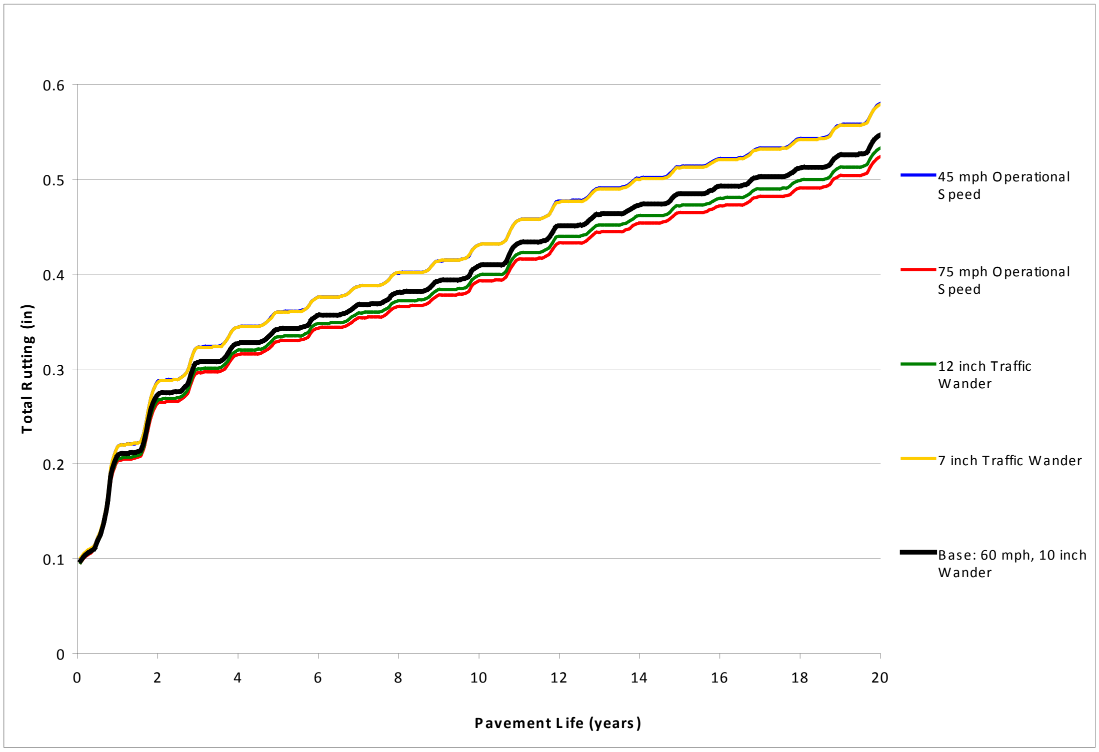

Figure 4.38: Richland County Total Rutting General Traffic Sensitivity

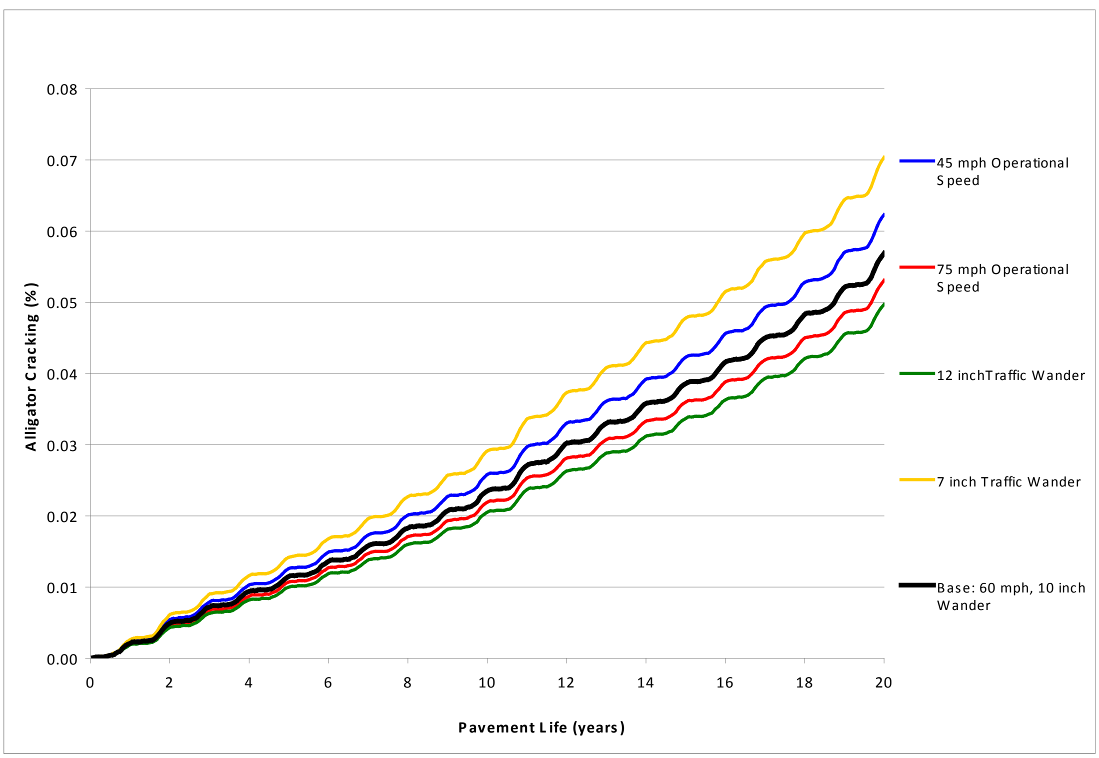

Figure 4.39: Richland County Alligator Cracking General Traffic Sensitivity

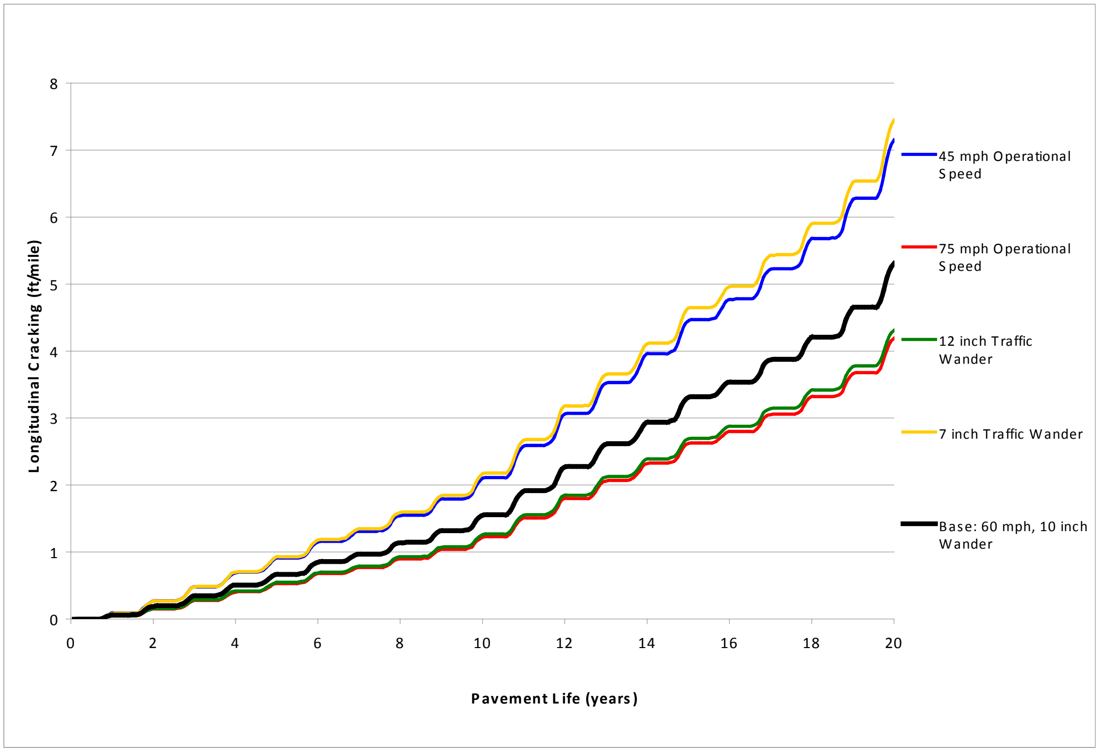

Figure 4.40: Richland County Longitudinal Cracking General Traffic Sensitivity

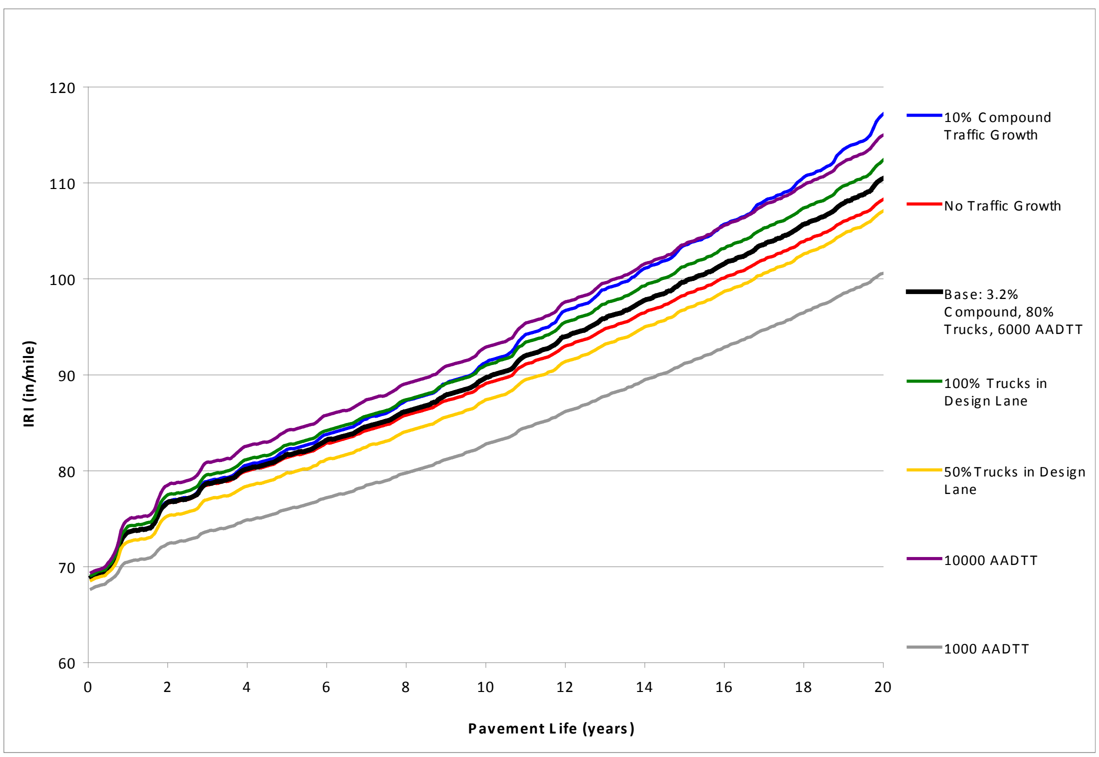

Figure 4.41: Richland County IRI Volumetric Traffic Sensitivity

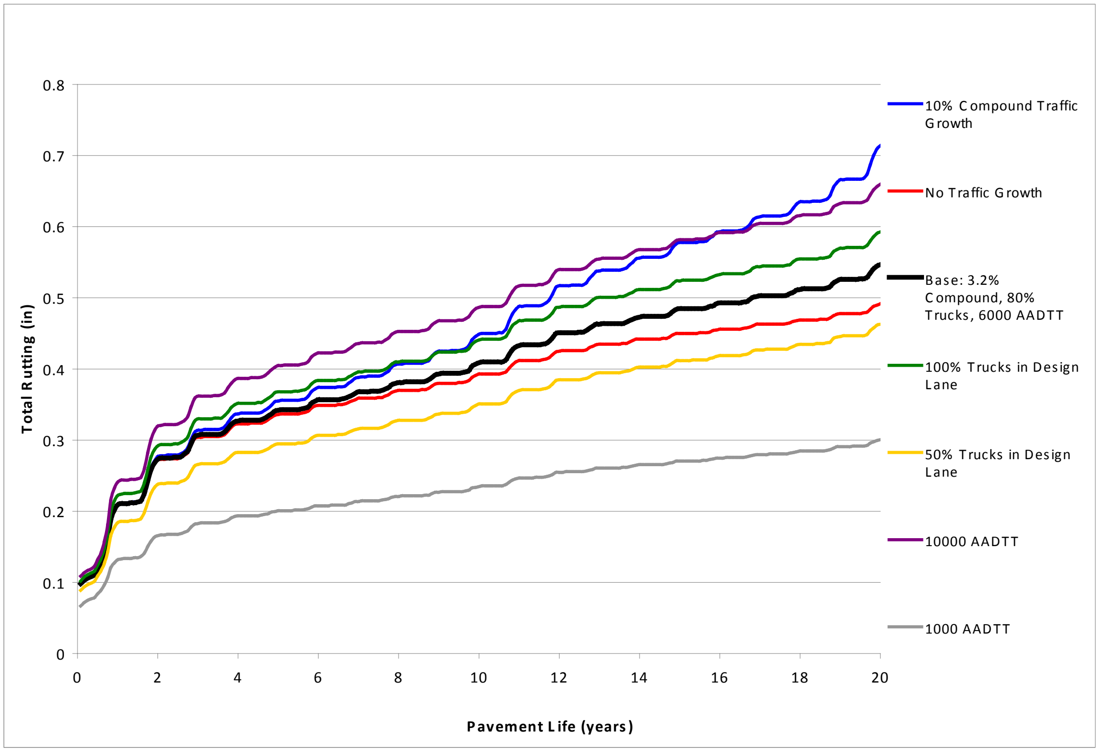

Figure 4.42: Richland County Total Rutting Volumetric Traffic Sensitivity

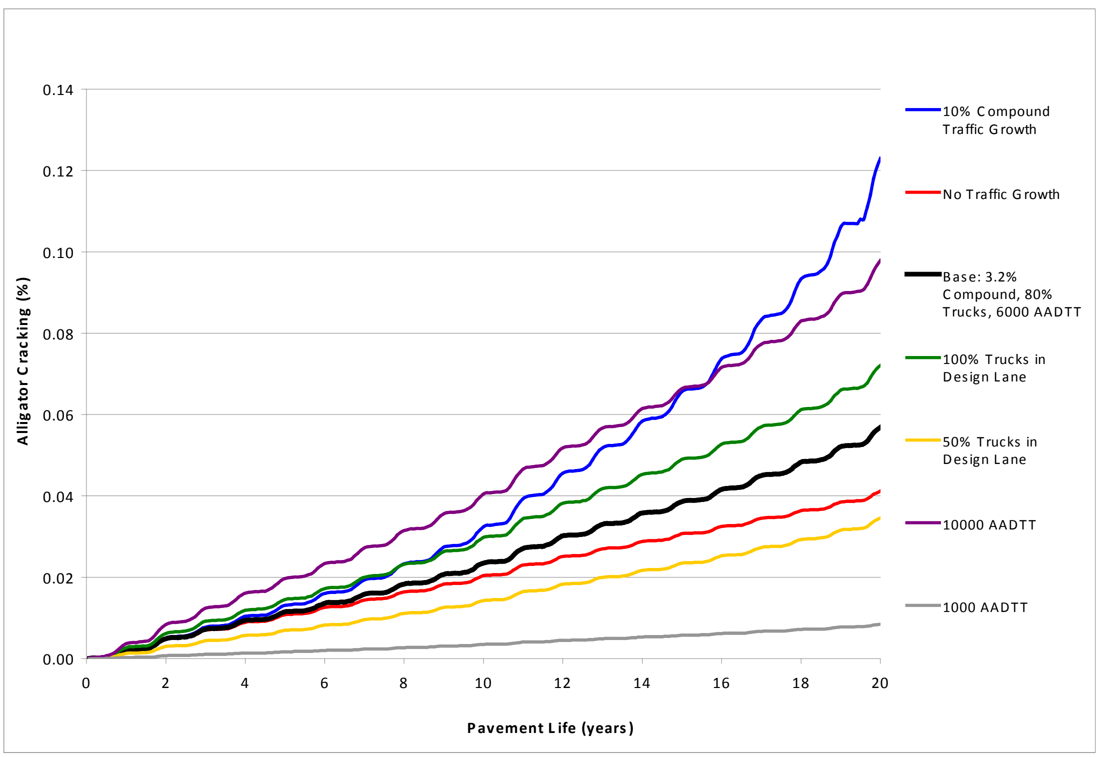

Figure 4.43: Richland County Alligator Cracking Volumetric Traffic Sensitivity

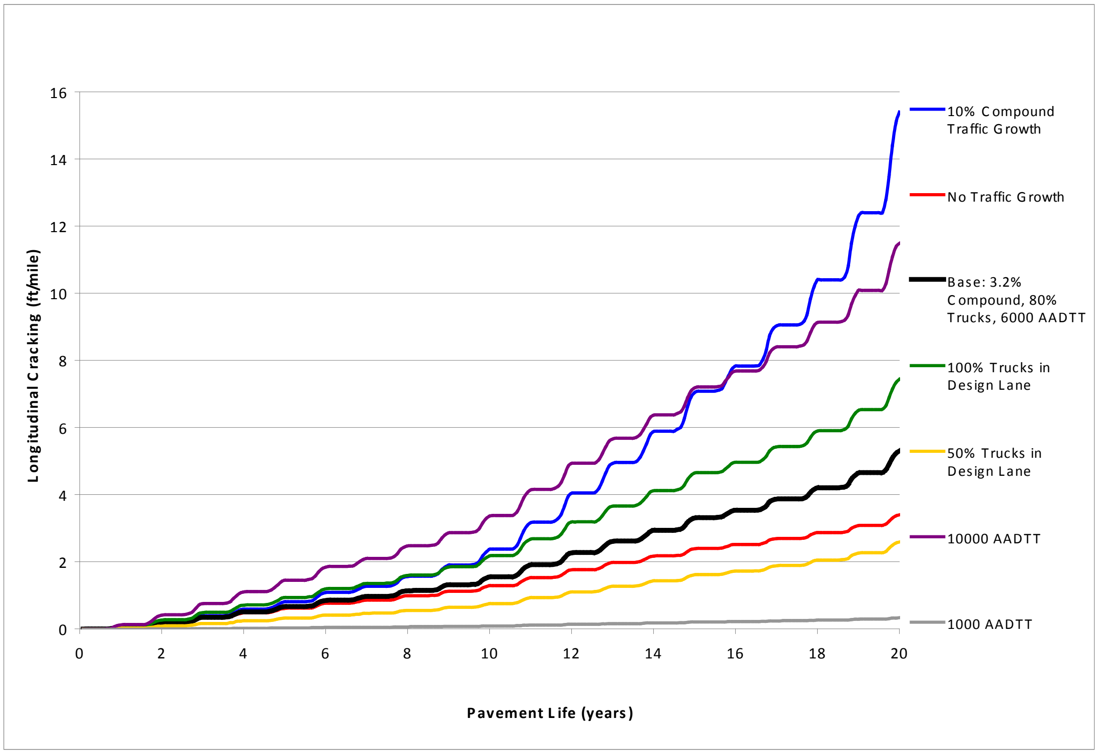

Figure 4.44: Richland County Longitudinal Cracking Volumetric Traffic Sensitivity

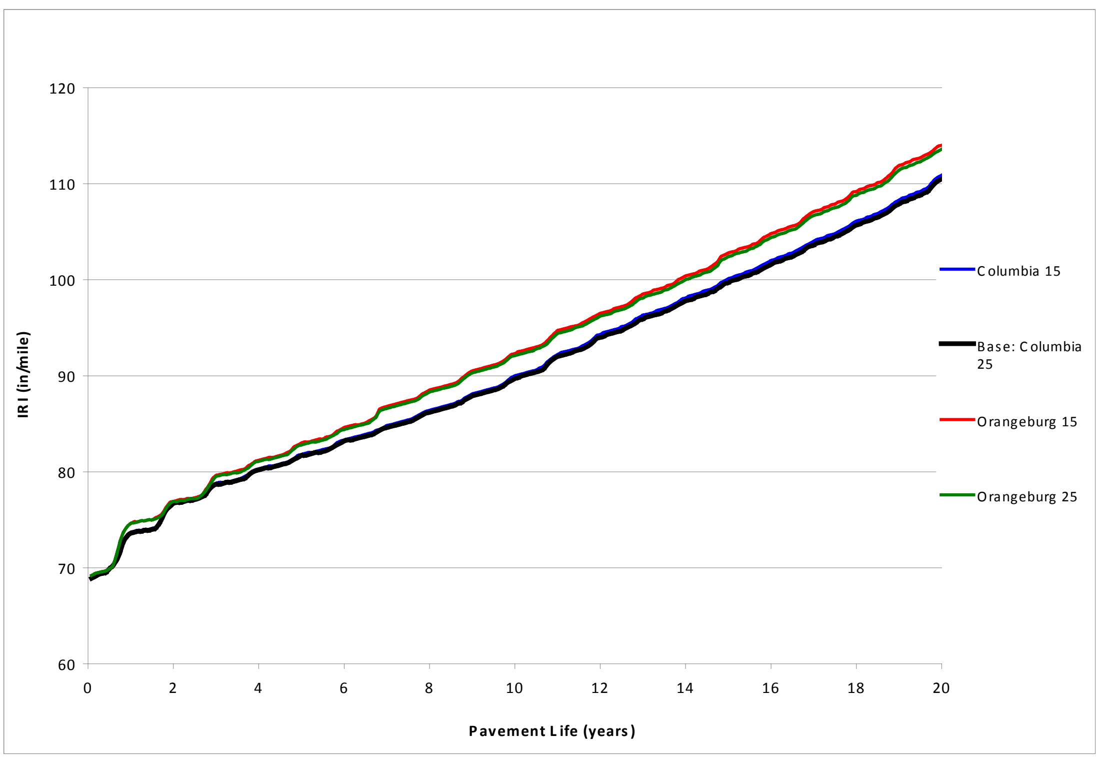

Figure 4.45: Richland County IRI Climatic Sensitivity

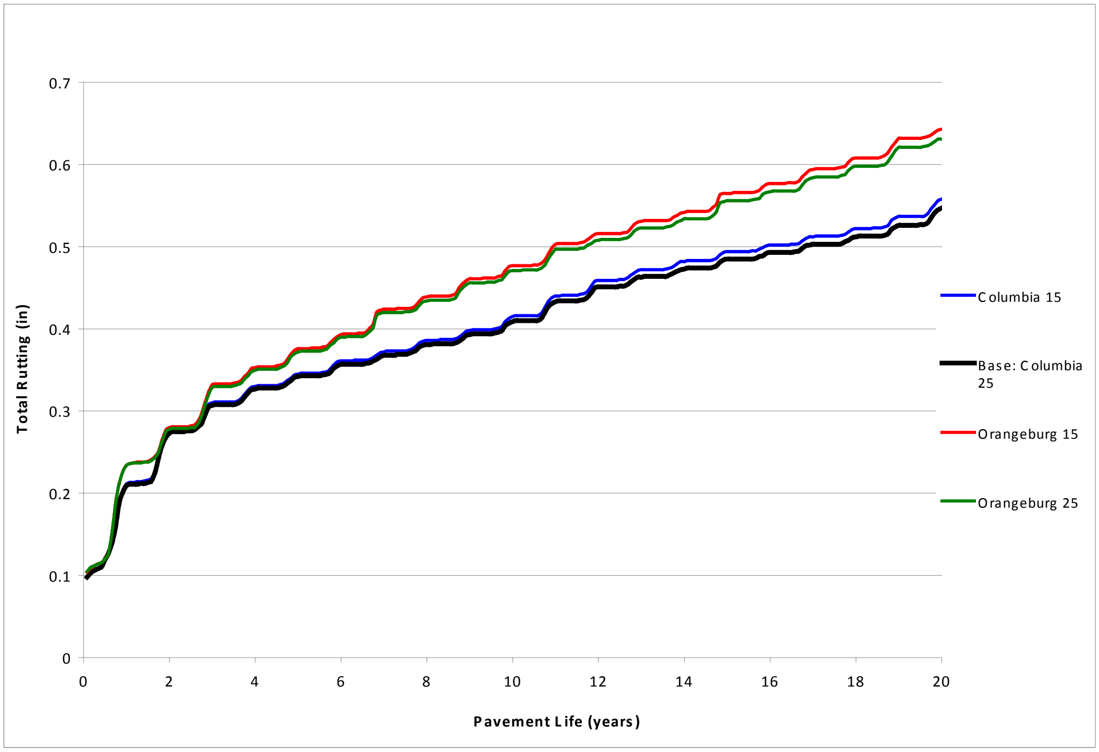

Figure 4.46: Richland County Total Rutting Climatic Sensitivity

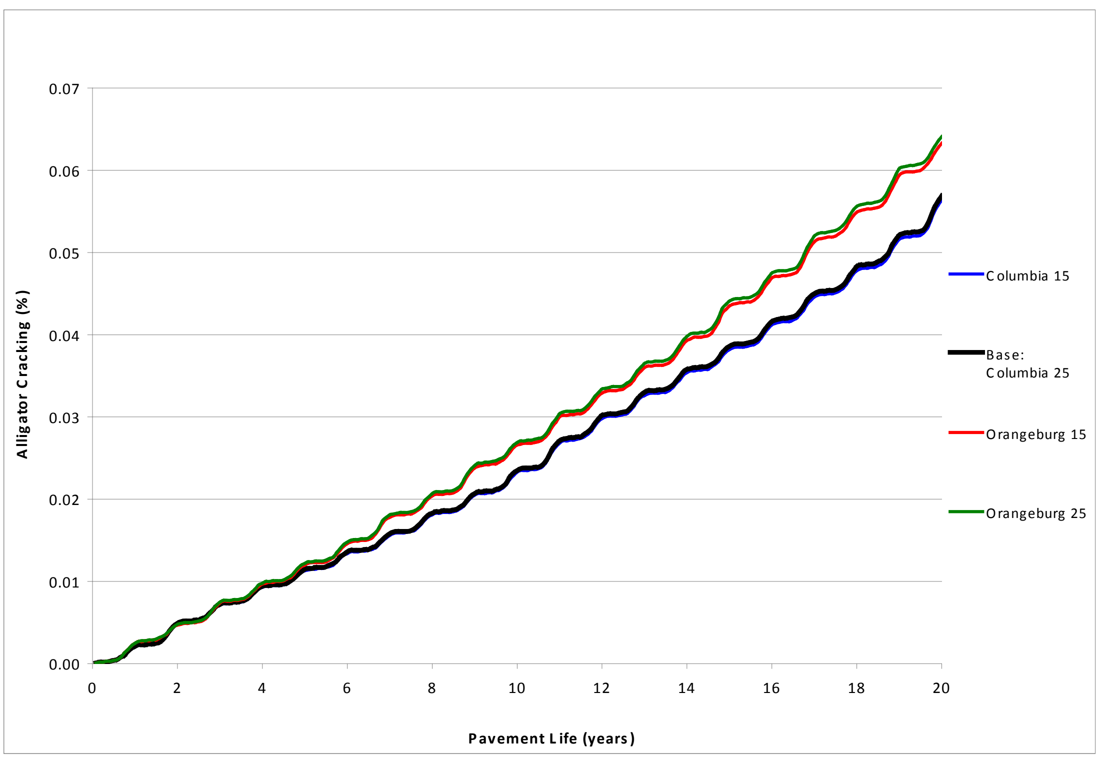

Figure 4.47: Richland County Alligator Cracking Climatic Sensitivity

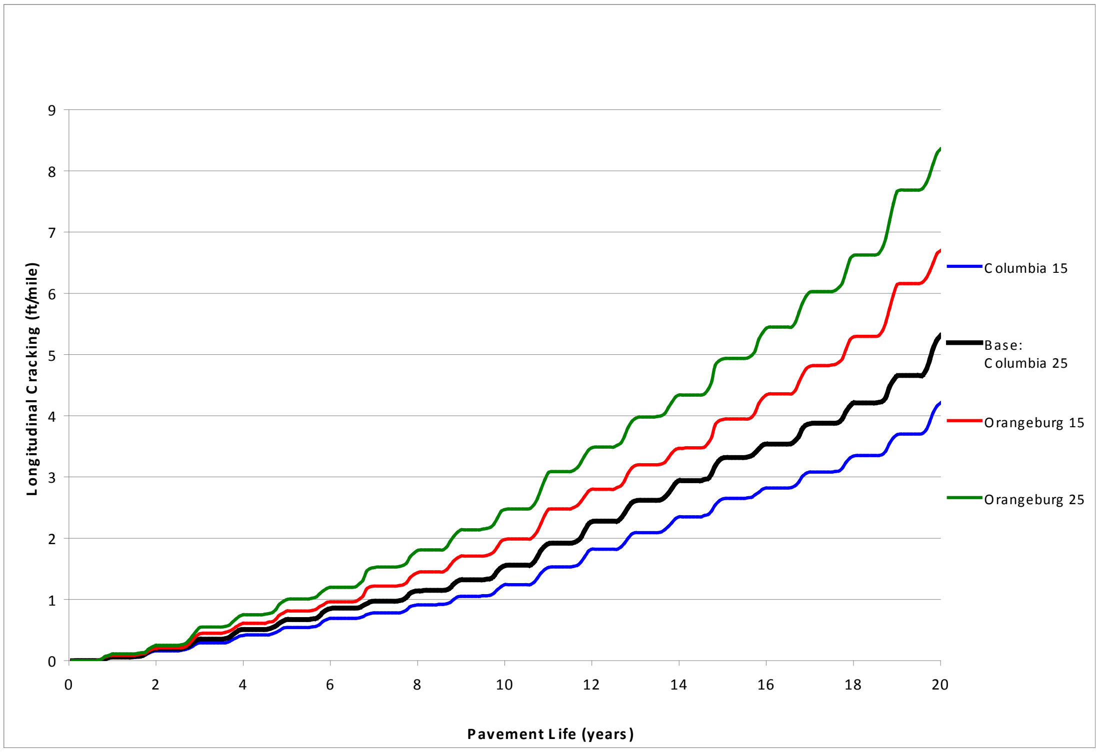

Figure 4.48: Richland County Longitudinal Cracking Climatic Sensitivity

Greenville County (Flexible with Cement Treated Base Layer)

The sensitivity analysis results for a Greenville County pavement utilizes a cement treated base (CTB) layer underneath the asphalt concrete layers. The analysis was performed using Version 1.10 of the MEPDG software. The software warns that CTB layers are not calibrated for flexible analyses. As a result, there are no output predictions of alligator cracking and longitudinal cracking. The Greenville County structure provided by the SCDOT for analysis contained the following layers: 1.4 inches asphalt concrete surface course, 4.3 inches asphalt concrete base course, 6.0 inches cement stabilized base, and an A-7-5 subgrade. The initial AADTT used was 2715. All other inputs were either MEPDG default values or values provided by the SCDOT. The initial IRI value was assumed to be 65 in/mile. Total rutting and IRI were the distress models evaluated. The input parameters studied are shown in Table 4.5. Figures 4.49 through 4.56 show Greenville County CTB pavement distress predictions during a 20 year period.

Table 4.5: Input Summary (Greenville County, Flexible, CTB)

Category	**Inputs**	**Values**
Material	AC Surface Viscosity Grade	10, 20 (base), 30
	Subgrade Modulus (psi)	8000, 13000 (base), 17500
	CTB Resilient Modulus (psi)	15000000, 2000000 (base), 2500000
General Traffic	Traffic Wander (inch)	7, 10 (base), 12
	Operational Speed (mph)	25, 45 (base), 75
Volumetric Traffic	Traffic Growth Rate	10% Compound, 10% Linear Growth, No Growth (base)
	Percent Trucks in Design Lane	50, 80 (base), 100
	AADTT	1000, 2715 (base), 10000
Climate	Location, Depth to Water Table (ft)	Greenville 15, Greenville 25 (base), Greer 25, Greer 15

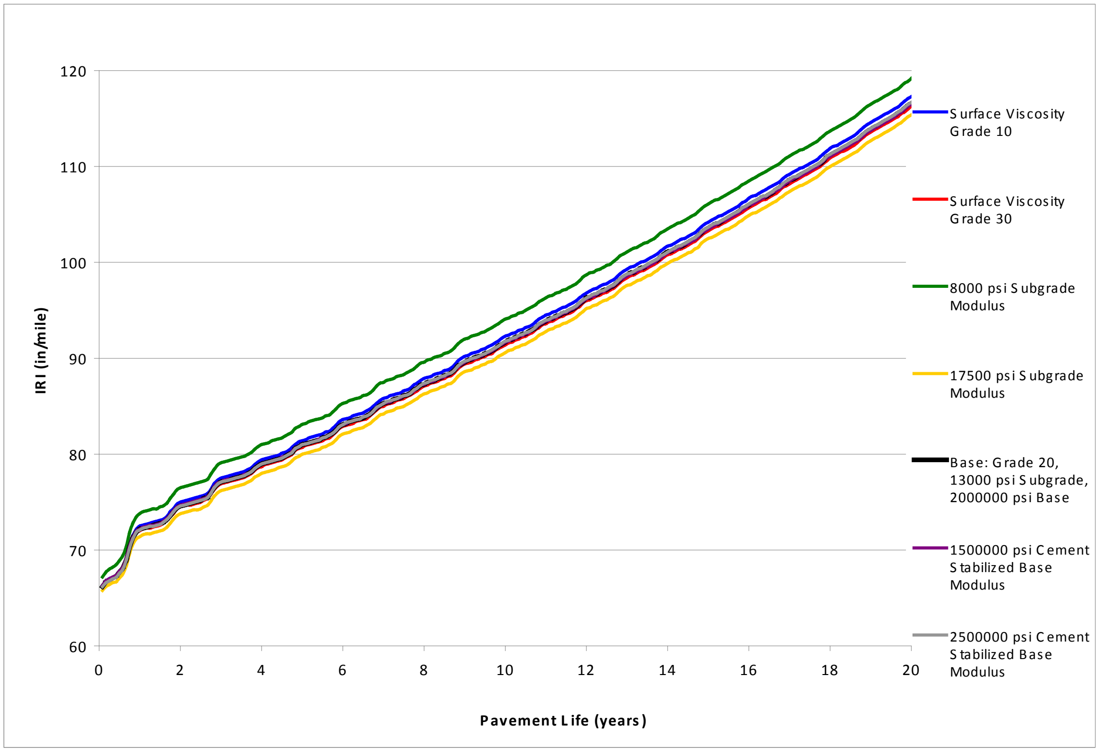

Figure 4.49: Greenville County CTB IRI Material Sensitivity

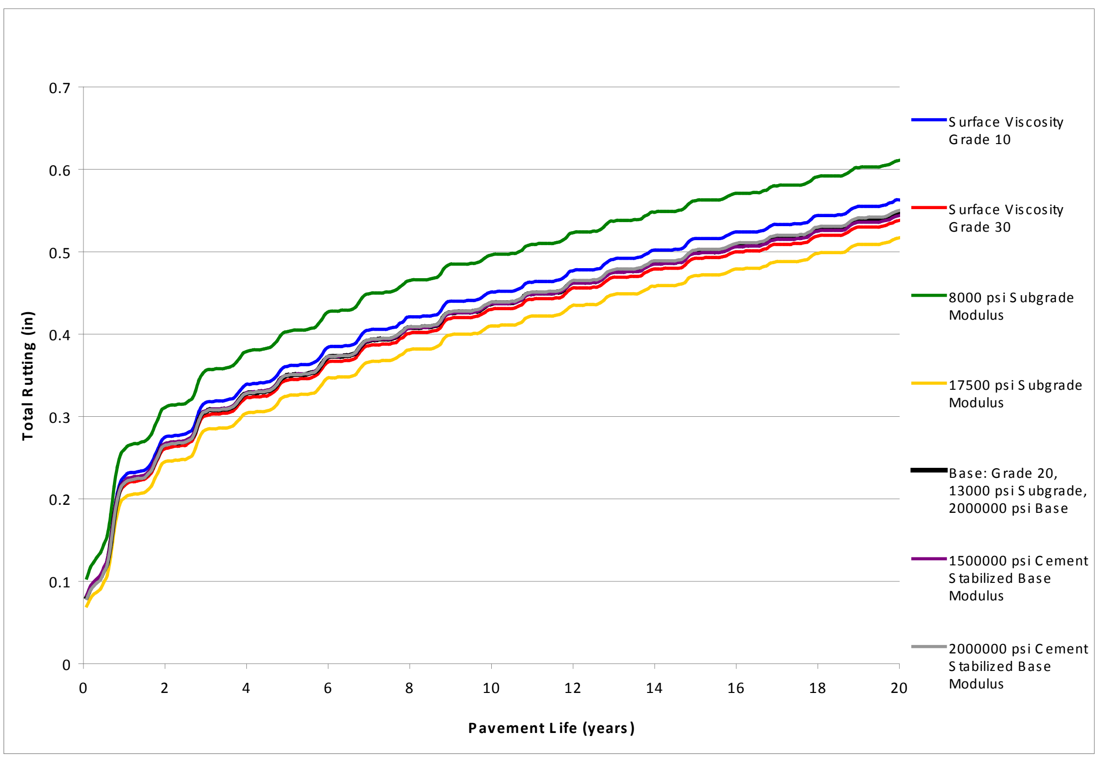

Figure 4.50: Greenville County CTB Total Rutting Material Sensitivity

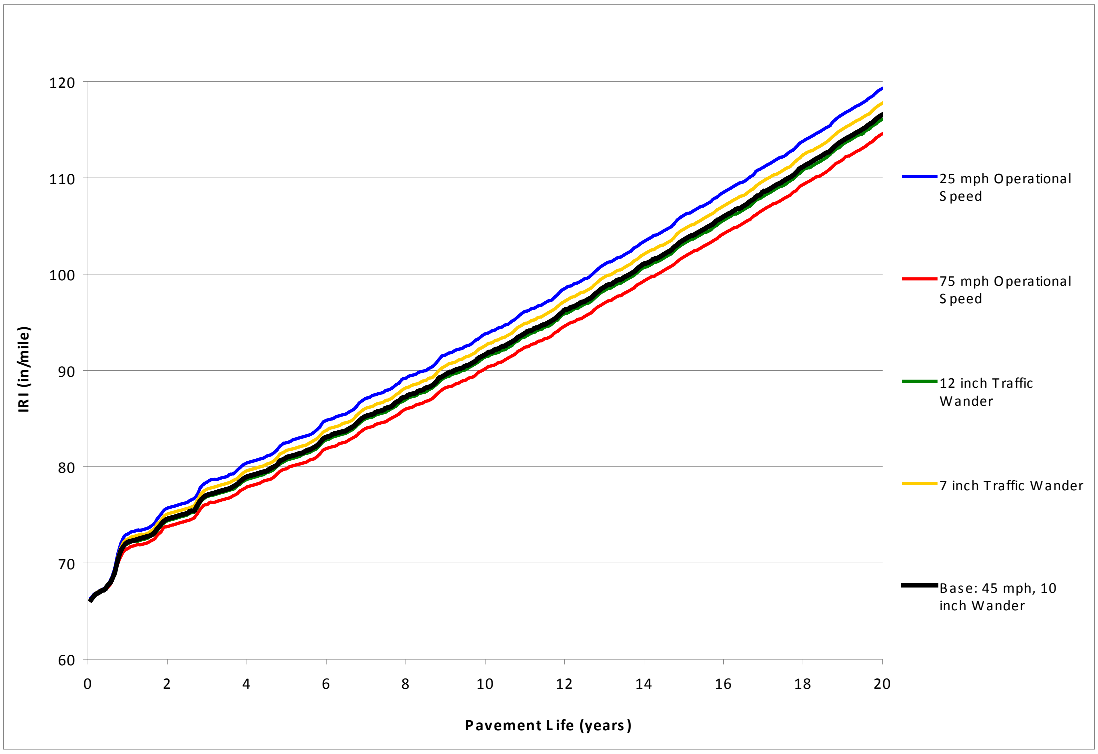

Figure 4.51: Greenville County CTB IRI General Traffic Sensitivity

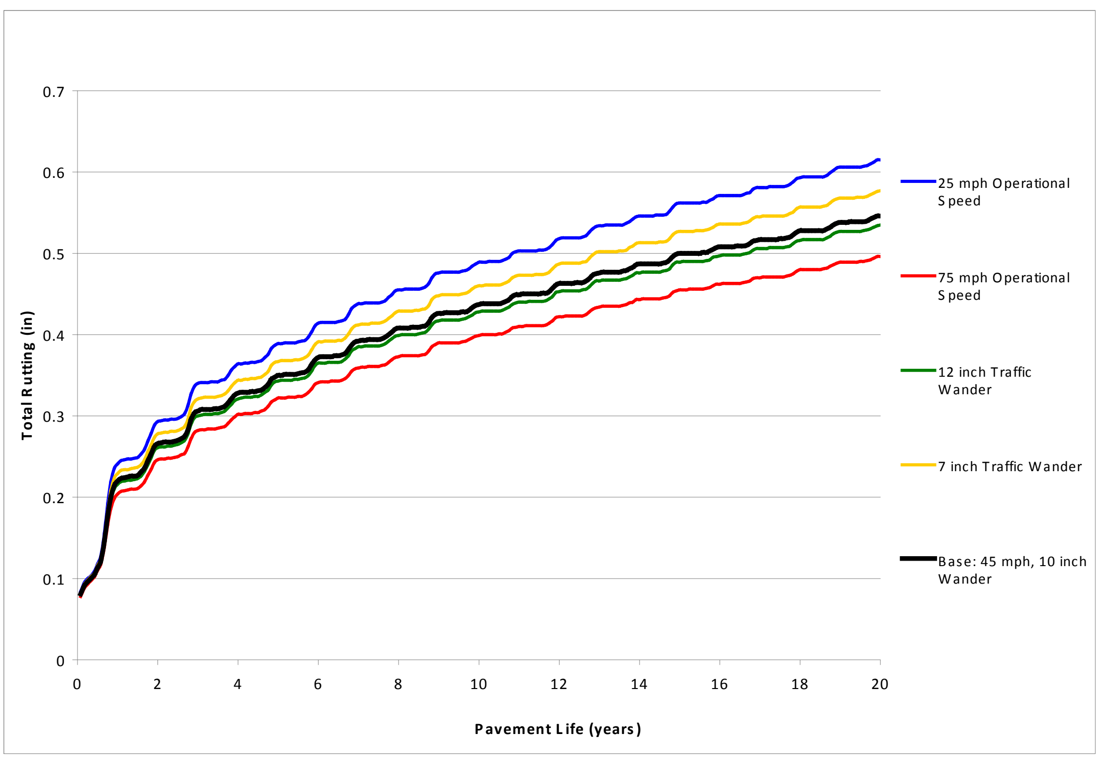

Figure 4.52: Greenville County CTB Total Rutting General Traffic Sensitivity

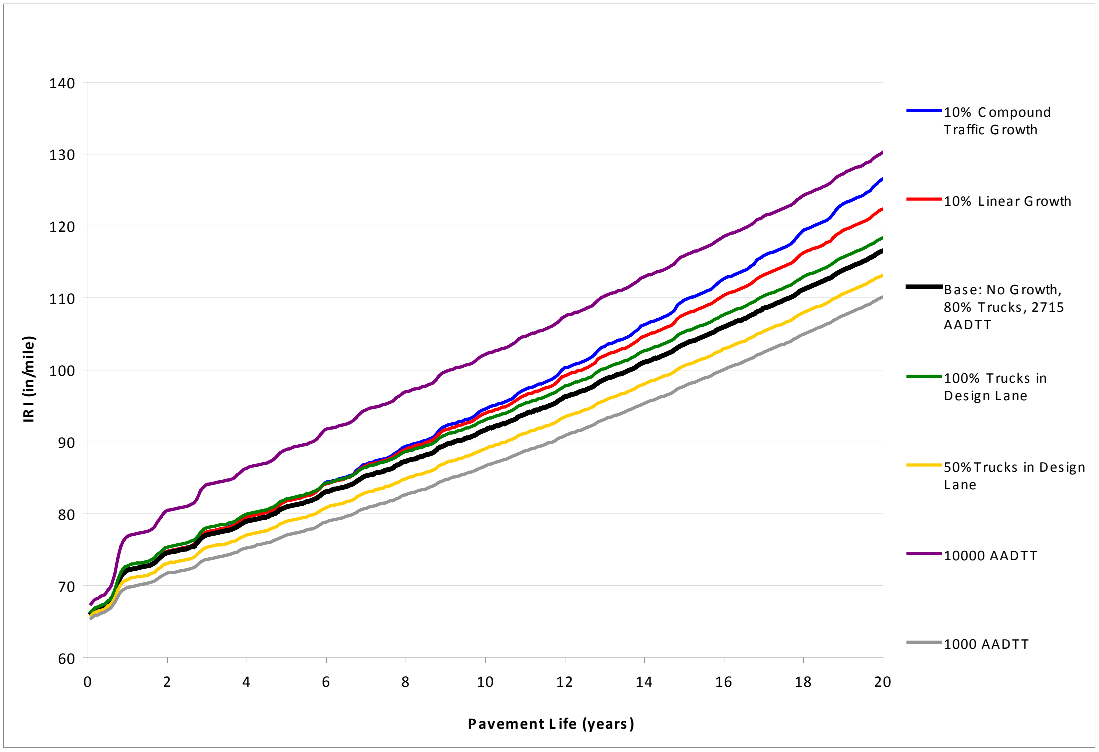

Figure 4.53: Greenville County CTB IRI Volumetric Traffic Sensitivity

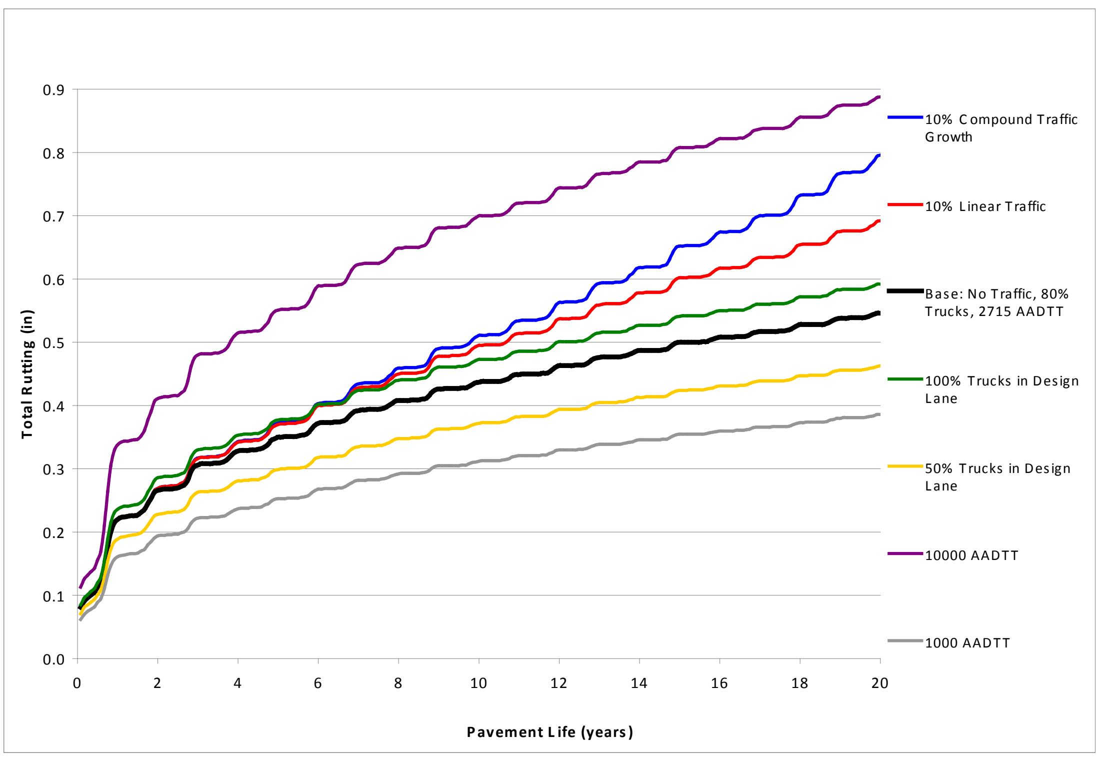

Figure 4.54: Greenville County CTB Total Rutting Volumetric Traffic Sensitivity

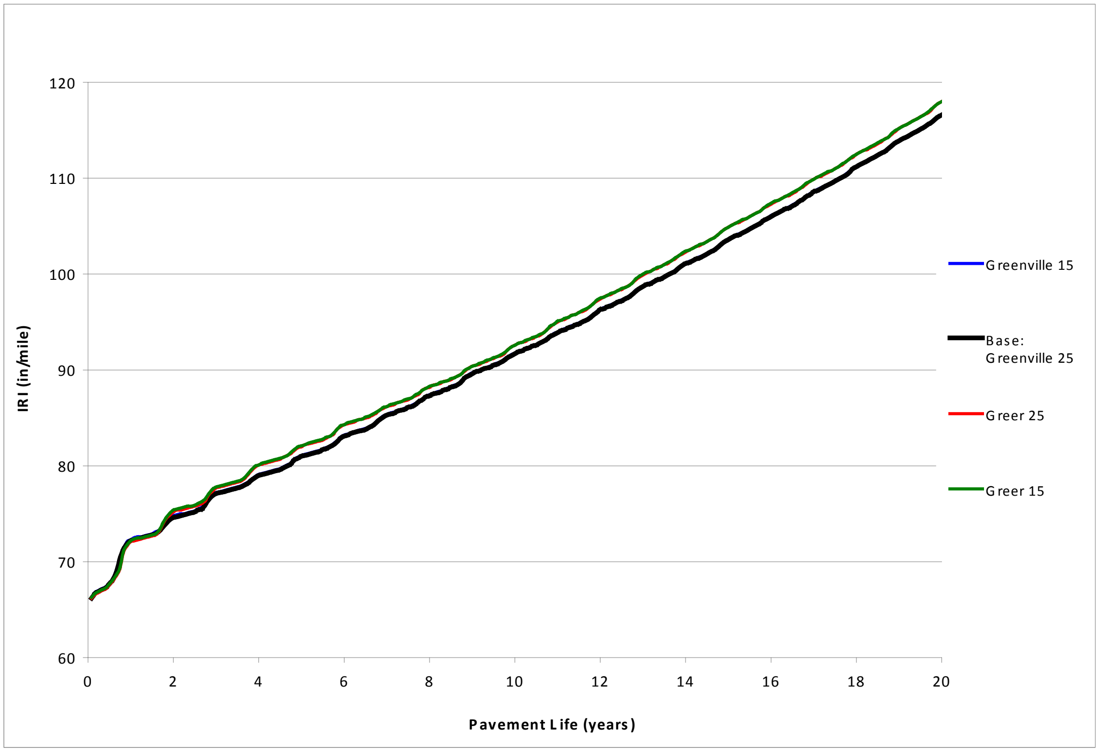

Figure 4.55: Greenville County CTB IRI Climatic Sensitivity

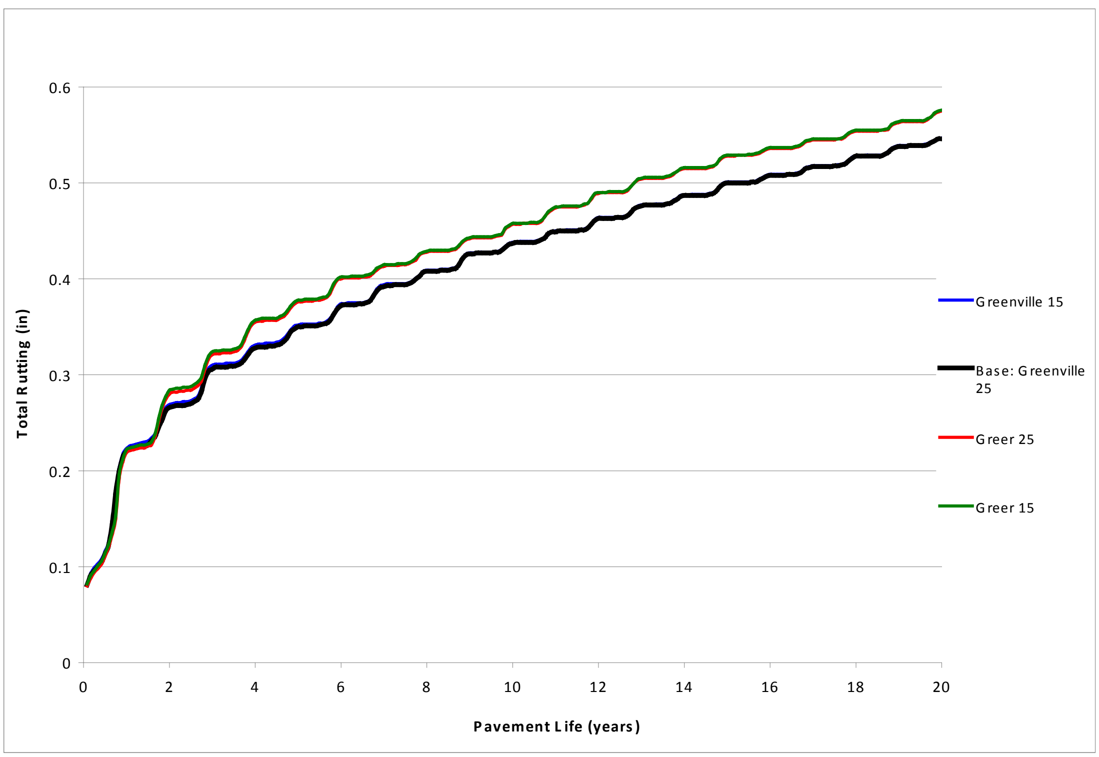

Figure 4.56: Greenville County CTB Total Rutting Climatic Sensitivity

Discussion of Results (Flexible Pavements)

The initial study analyzed a wide range of input values using MEPDG Version 1.003 with a SCDOT provided as-built pavement structure in Kershaw County. All of the South Carolina climatic stations with full sets of data were run at three water table depths. Little difference in performance predictions was observed (se Figures 4.13 – 4.15). Wide ranges traffic and material input parameters produced predictable results. Increases in traffic volume resulted in increases in distress predictions. Decreases in a layer moduli and layer thickness resulted in increases of distress predictions.

Table 4.6 summarizes the range of values for the Kershaw County predicted distresses (software Version 1.003). The alligator cracking and rutting graphs show a shape that represents the expected damage accumulation for each model. The substantial rutting increase initially is followed by a gradual rutting accumulation caused by continuous loading is expected (see for example Figure 4.2). The initial slow alligator cracking accumulation followed by an increase due to crack propagation also appears to represent an expected trend (see for example Figure 4.3). The predicted alligator cracking appears low (see Table 4.6). None of the predicted distresses exceed the limiting values recommended in the design guide. Large changes in input parameters produced relatively small response of rutting and alligator cracking performance predictions. The IRI model also appears to produce reasonable changes in distress prediction with changes in input parameters. Review of the numerical data used to produce the longitudinal cracking prediction graphs (for example Figure 4.4) shows an increase of accumulation of longitudinal cracking during warm weather months. The longitudinal cracking model is known to be unreliable and an update is planned. Table 4.6 shows very low predicted values of both longitudinal and alligator cracking.

Table 4.6: Range of Kershaw County Predicted Distresses

Distress	Min Value	Max Value	Limit
Longitudinal Cracking (ft/mi)	0	0.13	2000
Alligator Cracking (%)	0	0.13	25
Total Rutting (in)	0.278	0.607	0.75
IRI (in/mi)	125.2	138.4	172

Greenville County results (Figures 4.17 through 4.32) were made using software Version 1.10 (September 2009). Table 4.7 shows the range of values for the Greenville County predicted distresses. The structure layers and traffic levels are comparable to the Kershaw County design. The limited response in changes to weather station and water table depth data are similar to that observed for the Kershaw County pavement. Alligator cracking again appears to be under predicted, rutting predictions are higher, and longitudinal cracking predictions appear to be unreliable. IRI predictions appear reasonable.

Table 4.7: Range of Greenville County Predicted Distresses

Distress	Min Value	Max Value	Limit
Longitudinal Cracking (ft/mi)	0.01	0.46	2000
Alligator Cracking (%)	0.054	0.62	25
Total Rutting (in)	0.388	0.804	0.75
IRI (in/mi)	110.3	127.2	172

The Richland County results shown in Figures 4.33 through 4.48 were made using software Version 1.10. Table 4.6 shows the range of values for the Greenville County predicted distresses. The longitudinal cracking model was somewhat more responsive overall, especially with the climatic factor changes. The other distress predictions showed trends similar to the Greenville County pavement. None of the distresses exceed the limited values recommended in the design guide. All of the models still show limited distress prediction changes with respect to the input changes.

Table 4.8: Range of Richland County Predicted Distresses

Distress	Min Value	Max Value	Limit
Longitudinal Cracking (ft/mi)	0.35	15.4	2000
Alligator Cracking (%)	0.009	0.123	25
Total Rutting (in)	0.301	0.714	0.75
IRI (in/mi)	100.6	117.2	172

The Greenville County CTB results shown in Figures 4.49 through 4.56 were made using software Version 1.10. Table 4.9 shows the range of values for the Greenville County CTB predicted distresses. The software is not calibrated to analyze cement treated bases so there is no output of alligator or longitudinal cracking. The predicted rutting distress exceeds the limit value recommended in the design guide. It appears that rutting is over predicted. IRI and rutting output prediction trends are similar to the other Greenville and Richland County analyses.

Table 4.9: Range of Greenville County CTB Predicted Distresses

Distress	Min Value	Max Value	Limit
Total Rutting (in)	0.386	0.888	0.75
IRI (in/mi)	110.2	130	172

SCDOT pavement design personnel provided comments based on their preliminary analyses of flexible pavement structures using the MEPDG. The analyses included the typical Richland and Greenville County structures summarized in this chapter and other pavement sections. They observed a general trend of over prediction of rutting, under prediction of alligator (bottom-up) cracking and unreliable prediction of longitudinal (surface down) cracking.

Review of the data suggests that predicted distresses for the flexible pavements considered are most sensitive to traffic and material properties inputs and generally less sensitive to climatic (weather station) and ground water depth inputs. Increasing traffic increased predicted IRI, total rutting and alligator cracking. Longitudinal cracking predictions are presumed to be unreliable. Subgrade modulus is shown to have a significant impact on predicted IRI, total rutting, and alligator cracking (see for example Figures 4.17 – 4.19). Climatic data (selection of weather station location and depth to ground water table) are shown to have little significance (Figures 4.13 – 4.15) or demonstrable significance (see for example Figures 4.31 and 4.46). The sensitivity analysis results for flexible pavements presented here are not as comprehensive as those performed by others but do show trends that demonstrate the importance of traffic projections, material properties, and climatic inputs.

Aiken County (I-520, Rigid)

Early in the project, preliminary analyses were performed using an as-built rigid pavement structure in Aiken County (I-520). These preliminary analyses used software Version 1.003 (April 2007). Results from limited analyses provided some input sensitivity information.

The pavement structure consisted of 11-in. thick JPC slabs with slab lengths of 15 ft. Slab support is provided by a 2 inch thick HMA layer over 8 inches of crushed stone granular base. The subgrade soil is an A-2-4. The initial AADTT used was 2472 with 5% compound growth. Traffic distribution was based on assumed percentages for class 5, 6, 8 and 9 FHWA class vehicles. A 40 year time period was used. Pavement distresses predicted were IRI, percent slabs cracked, and mean joint faulting. To establish baseline input data, MEPDG Level 3 default values or values provided by the SCDOT were used. Base climate data was interpolated based on coordinates for Aiken, SC with a water table depth of 10 ft. In addition, data from other SC weather stations were used along with water table depths of 1 and 20 ft. For sensitivity analyses, selected input data values were varied, including slab thickness (8 to 15 inches), PCC Poisson's ratio (0.1 to 0.3), PCC modulus of rupture (500 to 700 psi), thermal conductivity (1 to 1.5 BTU/hr-ft-^{o}F), heat capacity (0.20 to 0.34 BTU/lb-^{o}F), and coefficient of thermal expansion, CTE (4.0 to 7.5 x 10^{-6} /^{o}F). Review of MEPDG software output suggests distress predictions are moderately sensitivity to heat capacity, modulus of rupture, and Poisson's ratio. MEPDG distress predictions appear to be most sensitive to variations in traffic volume (as would be expected), slab thickness (also, as would be expected) and coefficient of thermal expansion (CTE). Joint spacing and dowel diameter were not varied. These observations are generally consistent with significant inputs cited by others (Table 3.1, page 55).

Chapter 5

Summary and Implementation Guidelines

Summary

In this report, Chapters 1 and 2 provide general background information on MEPDG methodology. Chapter 2 presents current distress prediction equations. Review of these equations provides some insights into the complexity and empirical nature of converting computed pavement responses to predicted pavement distresses. Chapter 3 presents some of the different implementation strategies of other SHAs. Table 3.1 summarizes significant input parameters identified by sensitivity analyses. Chapter 4 presents the results of sensitivity analyses using MEPDG software [Versions 1.003 (May 2007) and 1.10 (September 2009)] applied to as-built and *representative* South Carolina pavement structures.

Implementation of the MEPDG procedures at the SCDOT will require significant time and resources. Effective use of new MEPDG procedures requires materials and traffic databases beyond Level 3 and MEPDG defaults information currently available at the SCDOT. A better understanding of MEPDG climatic models is needed. Immediate adoption of the MEPDG using only Level 3 and default inputs with current prediction models calibrated with limited national database information is not advised.

Use of MEPDG procedures will require local calibration of each pavement distress prediction. For new flexible pavement structures this includes: rutting (AC layers, unbound base, subbase, and subgrade layers, and total rut depth), fatigue cracking (surface-down longitudinal cracking and bottom-up alligator cracking), transverse (thermal) cracking, and IRI prediction. The current MEPDG prediction model for surface-down (longitudinal cracking) is known to be unreliable. For new rigid pavement structures, prediction models requiring local calibration include: faulting in JPCP, transverse cracking in JPCP (top-down and bottom-up cracking), edge punchouts for CRCP, and IRI prediction. The accuracy of IRI predictions for both flexible and rigid pavements depends on the accuracy of all other distress predictions. Guidelines on calibration and validation can be found in NCHRP Project 1-40b: *User Manual and Local Calibration Guide for the Mechanistic-Empirical Pavement Design Guide and Software* when it is released. The calibration process involves adjusting distress prediction equations in an attempt to minimize the differences between predicted and observed behavior. The model validation process involves collecting data sufficient to confirm the validity of the calibrated prediction models.

General MEPDG Implementation Recommendations

- Adopt agency initiative to move toward eventual adoption of the MEPDG. It is recommended that existing MEPDG software be utilized immediately (with Level 3, best guess, and default inputs) along with existing design procedures. This will provide several years of familiarity with MEPDG procedures prior to adoption. Design comparisons will also provide useful information on the general reasonableness of MEPDG output. It is envisioned that MEPDG procedures will not be adopted until sometime after updated interactive software (AASHTO DARWin-ME™) becomes available.

- Commence research and/or internal data collection efforts to eventually evolve from Level 3 to Level 2 or Level 1 inputs.
- Commence research and/or internal data collection activities necessary for calibration and validation of MEPDG output using data from a substantial database of planned or already in-service pavement sections with structures representative of flexible and rigid pavements in South Carolina. Sections should have detailed construction and traffic information and should be periodically monitored for the purpose of calibrating and validating MEPDG distress and smoothness predictions. A database consisting of no less then 20 in-service pavement sections is recommended.
- Establish a standing research steering committee to oversee and coordinate research projects and other internal data collection efforts associated with MEPDG implementation.

Recommended Investigations

- Traffic: Initiate a study to investigate the feasibility of developing full Level 1 axle load spectrum data using either existing WIM/AVC data or additional information. Sensitivity investigations show the anticipated direct relationship between traffic and predicted distresses. Load-related distress predictions can be no better than the quality of the traffic input data. Improving traffic inputs beyond currently available ESAL distribution should be considered a high priority.
- Material Properties: The SCDOT does not have a database of paving material properties required for higher level input. Currently, required MEPDG material

properties must be assumed or computed from classification, gradation, mix properties, etc. A database of actual measured mechanical properties for SCDOT paving materials is, by in large, nonexistent. A study is suggested to identify specific material properties to be measured, decide on the most effective testing protocols for measuring material properties, and execute material properties testing programs to generate the information necessary to create an SCDOT material properties database. Based on the results of this investigation, initial high priority efforts might be focused on testing to determine HMA complex modulus, PCC modulus and coefficient of thermal expansion, and base materials moduli.

- Subgrade Properties: Other than values backcalculated from FWD deflection testing, the SCDOT currently has no database of experimentally determined subgrade moduli. From previous SCDOT research projects, some general information is known about in-situ seasonal variations. A comprehensive investigation is recommended to determine in-situ modulus values for South Carolina subgrade soils and to provide comprehensive information on seasonal variations. Due to the uncertainties associated with known differences between moduli backcalculated using FWD deflections and laboratory values, is it recommended that the investigation include both field (FWD) and laboratory (triaxial) testing to reconcile observed differences for South Carolina subgrade soils. Seasonal FWD testing at in-service MEDPDG test sections can be used to monitor seasonal subgrade modulus changes.

- In-Service MEPDG Test Sections: To locally calibrate and validate MEPDG distress predictions, many SHAs have established in-service pavement test sections. It is recommended that the SCDOT establish a minimum of 20 pavement test sections for MEPDG calibration and validation. Where feasible, in-situ instrumentation is recommended. Instrumentation along with periodic testing (for example, FWD testing with multi-layer modulus backcalculation) and distress surveys can provide the information necessary to calibrate and validate MEPDG predictions.

References

American Association of State Highway and Transportation Officials (AASHTO). (2008). *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice.* AASHTO Designation: MEPDG-1. Washington, DC.

American Association of State Highway and Transportation Officials (AASHTO). (2008). *AASHTO DARWin-ME Project Proposal.*

Brown, S., Wilson, S., Darter, M., Larson, G., Witczak, M., and El-Basyouny, M. (2006). *Independent Review of the Mechanistic-Empirical Pavement Design Guide and Software.* NCHRP Research Results Digest Number - 307. Transportation Research Board, Washington, DC.

Cochran, G., Funk, N., Hoegh, K., Khazanovich, L., Marasteanu, M., and Velasquez, R. (2009). *Impementation of the MEPDG for New and Rehabilitated Pavement Structures for Design of Concrete and Asphalt Pavements in Minnesota*. Report MN/RC 2009-06. Minnesota Department of Transportation, St. Paul, MN.

Fernando, F. G., Oh, J., and Ryu, D. (2007). *Phase I of MEPDG Program Implementation in Florida.* Report D04491/PR15281-1. Texas Transportation Institute, College Station, TX.

Johnson, A. M. (2003). *SCDOT Pavement Design Guide.* South Carolina Department of Transportation, Columbia, SC.

Kim, R. and Muthadi, N. (2007). *Implementation Plan for the New Mechanistic-Empirical Pavement Design Guide*. NCDOT Project 2005-28. North Carolina State University, Raleigh, NC.

Montana Department of Transportation Performance Prediction Models. (2007). Online. <http://mdt.mt.gov/research/docs/research_proj/pave_model/final_presentation.pdf>

National Cooperative Highway Research Program (NCHRP). (2004). *Guide for Mechanistic-Empirical Design of New and Rehabilitated Structures.* NCHRP Report 01-37A. Transportation Research Board, Washington, DC.

Olson, R. (2009). *Mechanistic-Empirical Pavement Design Guide in Indiana*. ACEC Indiana Short List Volume 5, Issue 5. ACEC Indiana, Indianapolis, IN.

Pavement Resource Program Quarterly Progress Report. (2006). Center for Advanced Infrastructure & Transportation, Rutgers, the State University of New Jersey. Online. <http://www.state.nj.us/transportation/refdata/research/pdf/caitqrep/cait_06_q1_pavement_resourse.pdf>

Pierce, L. (2007). *New Pavement Design Guide*. Washington State Department of Transportation, Olympia, WA. Online. <http://nwpma-online.org/resources/07Fall_NewPavemtDesignGuide.pdf>

Schwartz, C. (2007). *Implementation of the NCHRP 1-37A Design Guide*. Report No. SP0077B41. Maryland State Highway Administration, Lutherville, MD.

Transportation Research Circular Number E-C118. (2007). *Pavement Lessons Learned From the AASHO Road Test and Performance of the Interstate Highway System.* Transportation Research Board, Washington, DC.

VDOT Preparation Plan for the Implementation of The Mechanistic-Empirical Guide for Design of New and Rehabilitated Pavement Structures. (2007). Online. <http://www.transportation.org/sites/desing/docs/PEPDG%20VDOT%2020Plan%20Jan%2007%20-17.pdf>

Williams, C.R., Robinette, C.J., Bausano, J., and Breakah, T. (2007). *Testing of Wisconsin Asphalt Mixtures for the Forthcoming AASHTO Mechanistic-Empirical Pavement Design Procedure.* Report No. WHRP 07-06. Wisconsin Department of Transportation, Madison, WI.

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