Preparation of the Implementation Plan of AASHTO Mechanistic-Empirical Pavement Design Guide (M-E PDG) In Connecticut

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Disclaimer

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Standard Conversions

SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
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16. Abstract

2002 Mechanistic-Empirical Pavement Design Guide is based on mechanistic-empirical (M-E) principles that provide a uniform platform for the design of flexible, rigid, and composite pavements. It considers design parameters for traffic, structure conditions, environment, and allows the user to specify a reliability Level of the predictions. The distress prediction models were originally calibrated to national averages using data from the Long-Term Pavement Performance (LTPP) effort. The distress models need to be recalibrated with data obtained locally in order to be applicable for the particular materials, construction practices, and environmental conditions encountered in Connecticut. Longitudinal (top-down fatigue), alligator (bottom-up fatigue), thermal cracking, asphalt rutting and total rutting prediction models were analyzed for all pavement designs. Statistical sensitivity analyses were conducted for all of the input ranges identified as pertinent for Connecticut including mix properties, environmental factors, underlying structures etc... All of the the inputs were then ranked according to their significance in order to establish target levels of detail as necessary for each input. Because this study only provides analyses based on a limited dataset, it is recommended that all of the M-EPDG models should be calibrated for use throughout the state. An implementation plan is presented along with course/training materials and recommendations for further analysis.

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Executive Summary

Introduction

Currently, the Connecticut Department of Transportation (ConnDOT) utilizes the AASHTO 1986 (1993) pavement design procedure, which is aimed primarily at determining the thickness of asphalt layer for a given truck traffic volume and subgrade and base layer strengths. This approach does not necessarily lead to the design of durable and economical pavements. The newly developed 2008 AASHTO Mechanistic-Empirical Pavement Design Guide (M-EPDG) offers pavement designers a modern computerized tool that allows them to achieve an optimal design by varying a wide range of material properties and other pavement features. The M-EPDG prediction models compute the amount of cracking, rutting, and roughness that will accumulate over the design life of the pavement, which is then compared with the performance threshold (maximum distress values specified by the agency) to develop the optimal pavement structure and materials based on trial designs.

Since the first evaluation version of the M-EPDG was released in 2002, the awareness of the M-EPDG by State Highway Agencies (SHA) nationwide has come a long way from skepticism and reluctance to a nationwide effort on the research and implementation of the guide. After the release of a new AASHTO 2008 Interim Pavement Design Guide, the final 1.1 version of the M-EPDG software was only available until 2011 when it was commercialized into the DARWin-ME™ package.

The new M-EPDG requires an extensive number of inputs associated with traffic, materials, and environmental variables. Those input values can be obtained from the data collected in the field, as well as from laboratory testing with varying levels of precision. Specifically, the M-EPDG provides three optional levels of hierarchy for the inputs. Level 1 data offer the highest reliability, but require site-specific data such as laboratory testing on collected soils or construction materials. Level 2 data provide intermediate accuracy, but require less site-specific testing. At Level 2, inputs may be selected based on previous tests that have been conducted on similar types of materials or other forms of agency experience. At Level 3, agencies select default values that represent typical averages for the geographic region where the design project is located. For a given paving project, all inputs do not have to be at the same input level. That is, an agency may choose input levels depending on the availability of different types of data and the resources available to support the data-collection efforts. To facilitate the decision on the level of input accuracy, a sensitivity analysis is usually conducted to rank the influence of a particular input on the variation in the output of a performance prediction model.

Adaptation of the M-EPDG to the local and state conditions may require calibration and validation of the prediction equations by using a set of multiple input parameters typical for a given location. It also warrants a preparation of the implementation plan, which is to be used for successful transition from currently used design procedures to a totally new and somewhat sophisticated M-EPDG approach. Therefore, ConnDOT has contracted the University of Connecticut (UConn) to prepare an M-EPDG implementation plan under State Planning and Research Project No. SPR-2274. This executive summary briefly summarizes the final project report and outlines the main findings and recommendations.

Summary of Sensitivity Analysis

Typical Design Inputs for Connecticut

Three typical pavement designs were considered in the sensitivity analysis: (1) newly constructed asphalt pavement, (2) asphalt-overlaid asphalt pavement, and (3) asphalt-overlaid Portland cement concrete (PCC) pavement. A total of 185 simulations of the M-EPDG software were run to determine the impact of change in site factors (climate, truck traffic volume, and subgrade type), pavement structure (layer thicknesses) and material properties on the variability in cracking, rutting, and roughness in each of the three typical pavement designs. Three climatic zones recognized in the analysis were coastal, inland, and high-hill regions of Connecticut. The analysis explored three traffic levels: Level 2, Level 3 medium, and Level 3 high with 1.9, 4.8, and 12.1 million ESALs, respectively, accumulated over 20 years of service. Subgrade moduli ranged between 10,000 and 20,000 psi.

The traffic levels corresponded to the three highway functional classes recognized by ConnDOT, namely, Interstate highways, non-Interstate highways, and local arterials. Accordingly, three typical pavement structures were modeled with 8, 10, and 12 inches of asphalt supported by 10 to 18 inches of granular base. Structure moduli ranged between 20,000 and 30,000 psi and corresponded to Gradings A, B, and C specified by ConnDOT. The aggregate gradations and volumetric properties associated with asphalt, base, and subgrade materials were kept fixed for each structure, base, and subgrade type included in the analysis, while their values were obtained from ConnDOT specifications. The details on the typical Connecticut inputs for the M-EPDG sensitivity analysis are provided in Chapter 4 of the report, whereas the testing matrix for the sensitivity analysis is described in Chapter 5.

Sensitivity Analysis Approach

The performance indicators for the analyses were chosen based on the distress types predicted by the M-EPDG models. Thus, longitudinal (top-down fatigue), alligator (bottom-up fatigue), thermal cracking, asphalt rutting and total rutting prediction models were analyzed for all pavement designs. In addition, a reflection cracking model was evaluated for the asphalt-overlaid asphalt pavement design. The sensitivity analysis employed a "one-at-a-time" approach where, first, the baseline values for all variables were established and, next, the sensitivity of each nonfixed input variable was estimated by changing the value of the variable, calculating the resulting pavement performance using the M-EPDG software, and then comparing the predicted pavement performance to the established baseline performance for the given design. The input values were changed from "Baseline" to "Low" and "High" as shown in Table 5.3.

The analysis of the sensitivity results explored two types of evaluation. First, the qualitative assessment of the "stock" charts was performed where the relative effect of each input was estimated by the length of a vertical line connecting the outputs corresponding to "Low", "Baseline", and "High" input values (Figure 5.1). In the second phase of the investigation, a multiple analysis of variance (ANOVA) was conducted where the significance of an individual M-EPDG input was evaluated by the magnitude of the calculated F-ratio associated with the input. The F-ratio measured variation in the output caused by the variation in the individual input being investigated. Effectively, the higher the calculated F-ratio, the greater the effect that input had on the model output. The F-ratio was found statistically significant if its p-value did not exceed the established level of confidence (α=0.05). The logF parameter was utilized to normalize the effect of inputs and rank their importance for predicting a particular performance indicator (cracking, rutting, or IRI). More details on the approach and the individual analysis for each prediction model can be found in Chapter 5, whereas this executive summary provides the main findings based on the overall sensitivity of each of the three pavement designs typical for Connecticut as outlined below.

Sensitivity of New Asphalt Pavement Design

- Elements of the AC layer structure, such as thickness and volumetric properties of the HMA mix, appear to govern the pavement performance the most in a specified location.
- For a specified functional road class in Connecticut, the truck traffic volume appears to have more effect on rutting than it does on cracking. Note that only longitudinal and thermal cracking, both being non-load related, were predicted at a noticeable level for all new AC designs.
- The binder performance grade and subgrade support showed a high influence on rutting, and thus on roughness in terms of IRI.
- Granular base-related inputs did not yield any significant effect on pavement performance, most likely, due to the relatively high modulus prescribed by ConnDOT specifications and the substantial thickness considered in the sensitivity analysis.

Sensitivity of Asphalt-Overlaid Asphalt Pavement Design

- Location (which identifies climate zone) and traffic volume appear to be important for an optimal overlay design. The traffic volume has a lesser impact for the low-volume roads in the colder high-hill locations.
- The pre-overlay condition of the existing surface should be considered first to reduce cracking susceptibility, whereas the milled thickness is expected to affect overall performance of the overlay.
- When rutting is of a greater concern, the total amount of asphalt rutting in the existing surface is the most influencing input on the M-EPDG prediction.
- The AC overlay thickness shows to be an important factor in the cracking and rutting outputs, while a moderate contributor for IRI predictions.
- The overlay mix and binder properties show a high influence on rutting and a low influence on cracking, which makes them moderately important when IRI predictions are concerned.
- For the analyzed range of unbound layer properties, neither the cracking model , rutting model, nor IRI model appear to be sensitive to subgrade and base moduli nor to base thickness.

Sensitivity of Asphalt-Overlaid Portland Cement Concrete Pavement Design

- Overall, the M-EPDG cracking predictions show the highest sensitivity to anticipated traffic load, project location, fractured PCC slab support, and thickness of the overlay. Volumetric properties of the asphalt mix, subgrade stiffness, and thickness of the fractured PCC layer show moderate influence on the predicted cracking values.

- The M-EPDG rutting prediction models are highly sensitive to all site factors (truck traffic volume, climate, and subgrade), AC layer thickness, and AC binder properties. The volumetrics of the asphalt mix and the stiffness of fractured PCC affect rutting predictions to a moderate degree.
- The IRI output appears to be mostly controlled by the location (climate zone) of the project, whereas the AC layer inputs, subgrade, and traffic volume show lesser influence on IRI.
- In general, base modulus does not show any significant influence on any of the distresses considered in this analysis.

It is understood that the stated conclusions are only valid for the specific range of parameters evaluated in this study. It is anticipated that some of the sensitivity trends shown here may change after re-calibration of the M-EPDG distress prediction models. In addition, it should be noted that a "moderate" ranking of some inputs does not diminish their significance for the design. The ranking is used for further recommendations on data collection to meet required level of hierarchy as explained in the next section (see also Chapter 6).

Recommended Input Levels

The sensitivity analysis of the M-EPDG prediction models allowed for different degrees of impact of the input variables on the predicted output value of a particular distress. Based on a ranking of an input for a targeted design (New AC, AC-overlaid AC, or AC-overlaid PCC pavement), the recommended level of hierarchy, and a corresponding scope of testing required to meet that level should be established as part of the M-EPDG implementation process. The description of the hierarchical levels is provided in Chapter 3. Following is the summary of the tentative recommendations based on the results of this study, while more details on the assigned hierarchy levels are included in Chapter 6:

- Truck traffic volume expressed in Average Annual Daily Truck Traffic (AADTT) appears to be the most important input after climate and therefore, it should be treated as the highest level of the hierarchy (Level1). The data may be site specific or alternatively may be generated from vehicle count, traffic forecast, or trip generation.
- The asphalt related inputs are recommended to be determined on Level 2, which would require measuring G* and phase angle for RTFO-aged binder at a minimum of 3 temperatures as well as providing gradation parameters of the asphalt mix.
- Subgrade modulus can be determined at Level 2 by either correlation with CBR or R-values, or measuring resilient modulus directly in triaxial test.
- Base modulus can be obtained using Level 3 default AASHTO classification. Note that in this study, medium strength bases were considered (20,000 to 30,000 psi)
- For AC-overlaid AC pavement design, milled thickness appears to be a critical input and should be surveyed as well as total rutting in the existing surface.

Implementation Plan

The concluding task of this Project was to compile a roadmap for the implementation of the M-EPDG by ConnDOT. This roadmap includes a step-by-step outline of the activities and processes that should be undertaken to facilitate a change in design philosophy by adapting a mechanistic-empirical approach to pavement design. The plan consists of 10 general steps, some of which have been or will be completed concurrently. It should be noted that this chapter only describes tentative activities proposed by the UConn Research Team that should be finalized and approved by ConnDOT's M-EPDG Implementation Team.

1. Conduct sensitivity analysis of M-EPDG inputs.
2. Recommend MEPDG input levels and required resources to obtain those inputs.
3. Assemble a ConnDOT M-EPDG Implementation Team and develop and implement a communication plan.
4. Conduct staff training.
5. Develop formal ConnDOT-specific M-EPDG-related documentation.
6. Develop and populate a central database with required M-EPDG input values.
7. Align distress data Collection in Connecticut with the M-EPDG defined performance indicators.
8. Calibrate and validate M-EPDG performance prediction models to local conditions.
9. Define the long-term plan for adopting the M-EPDG design procedure as the official ConnDOT pavement design method.
10. Develop a design catalog.

The list of the above activities necessary for successful implementation was developed based on previous work (Saeed 2003, Yut et al. 2007,Hoerner et al. 2007) and customized to address ConnDOT specific needs. The explanation of each implementation step is provided in Chapter 8.

Recommendations on the M-EPDG Calibration

To evaluate the calibration needs for Connecticut, the Research Team identified the now terminated LTPP SPS-9A project located on Connecticut State Route 2 as a viable source of information. Well-documented construction history, pavement performance, and laboratory testing data exists to provide real values for climatic, traffic, and material-related inputs that were used in the M-EPDG trial runs. Once the predicted deterioration curves were obtained, they were superimposed with the field trends to evaluate the errors. Based on the magnitude of prediction errors, the recommendations on the calibration were made for each of the prediction models included in the sensitivity analysis.

The following is a summary of the preliminary validation results for the chosen set of sections:

- **Longitudinal Cracking:** The M-EPDG predicted zero top-down fatigue for all sections at a reliability of 50 percent, whereas the condition survey revealed very few low severity cracks in the wheel path of 2 out of 6 sections. Because of prevalent zero values, it is impossible to correlate predicted output with the field measurements. Therefore, calibration of a model with such a high built-in error appears impractical in general. Ultimately, more research should be done with use of better suited candidate sections where the extent of longitudinal cracking in the wheelpath is noticeable.
- **Alligator Cracking:** Due to relatively low truck traffic volume (580-600 AADTT in one direction) as well as due to the "deep-strength" nature of the pavement structure on Route

2, no alligator cracking developed during the 12 years of service. The zero-values by the M-EPDG should not be, however, attributed to good quality of predictions. Instead, it is recommended to choose a different set of sections for the calibration of the fatigue cracking models because this type of distress is not typical for CT State routes.

- **Thermal (Transverse) Cracking:** The M-EPDG thermal cracking model underestimated the extent of thermal failure on average (77 percent of measured) at moderate goodness of fit (R-squared=0.48). It is well established that thermal cracking is one of the main distresses on the asphalt surfaces in Connecticut. Thus it is strongly recommended to consider this model for calibration.
- **Total Rutting:** In general, the M-EPDG underestimated total rutting for the given dataset as only 25 percent of the measured rutting. However, the relatively high goodness of fit for the linear trends (R-squared=0.69) suggest that a scaling factor can be applied to rutting predictions to adapt the model to the Connecticut environment. Ultimately, a larger dataset involving a wide range of traffic volumes and layer thicknesses should be utilized during the calibration.
- **IRI:** An error analysis revealed no association between predicted and measured values for the given set of sections, which might be due to the combined low predictability demonstrated by fatigue cracking models. On the other hand, it might be a result of discrepancy in IRI measurements. At any rate, the calibration of the IRI model is possible if the field measurements are consistent with growth in roughness with pavement age.

It should be noted that this study only provides examples based on a limited dataset, whereas the statewide calibration and validation study needs to be implemented as a part of the M-EPDG implementation process outlined in Chapter 8. In summary, it appears that all of the M-EPDG models should be calibrated. Special consideration should be given to the fatigue (longitudinal and alligator) predictions where very low values were predicted for thick pavements. It is recommended that ConnDOT allocate the resources for calibration and validation of all the M-EPDG models to facilitate creation of the design catalog, which in turn will save time and finances in the future pavement design activities.

CHAPTER 1 Introduction

The road infrastructure in the U.S. has been aging at an accelerated rate while at the same time, the monetary and material resources for preserving roads from further deterioration have become limited. Given these conditions, employing proper pavement design using state-of-the-art techniques and proper construction are crucial to ensure durability and satisfactory pavement performance.

For the moment, the Connecticut Department of Transportation (CTDOT) utilizes the AASHTO 1986 (1993) Pavement Design Guide, which is aimed primarily at determining the thickness of asphalt layers or concrete slabs for a given truck traffic volume, as well as subgrade and base layer strengths. This procedure is based on the empirical equation developed under the AASHO Road Test program, established with a very limited variation of materials in a particular climate of Ottawa, Illinois, in the late 1950s (Smith et al. 2004). Over the past 40 years, the dramatic changes in truck axle loads and configuration, as well as a large variation in pavement material properties throughout the states, have warranted changes in design philosophy. As a result, the AASHTO 2008 Mechanistic-Empirical Pavement Design Guide (M-EPDG) has evolved as an advanced procedure supported by sophisticated software (AASHTO 2008). The main advantage of the M-EPDG compared to previous AASHTO procedures is that a designer has the ability to vary the material mechanical properties and numerous other design inputs, in addition to the layer thickness, in order to predict the development of various distresses based on realistic climatic data. These predictions can then be used to optimize the pavement structure and material properties (NCHRP 2004).

Following the release of the AASHTO Mechanistic-Empirical Pavement Design Guide (M-EPDG) 2008, the CTDOT contracted the University of Connecticut (UConn) to prepare comprehensive plans for the implementation and adaptation of the MEPDG to Connecticut conditions. This report summarizes the efforts of the UConn Research Team under State Planning and Research (SPR) Project No. SPR-2274 awarded in July 2011.

Problem Statement

The distress prediction models that are an integral module of the M-EPDG procedure were originally calibrated to national averages using data from the Long-Term Pavement Performance (LTPP) study. As recommended by the M-EPDG project team, the distress models need to be re-calibrated with data obtained locally in order to be applicable for local materials, construction practices, and environmental conditions. The following general steps are suggested (NCHRP 2004):

- Achieve full support of the departmental personnel.
- Select procedures to obtain all inputs and establish local defaults for the inputs.
- Complete training of the staff involved in the pavement design.
- Acquire necessary equipment and computer software.
- Calibrate/Validate the M-EPDG software to local conditions.

Since only one Connecticut pavement section was included in the LTPP dataset for the calibration of the M-EPDG distress prediction models, the need for calibration of the M-EPDG software to different Connecticut local conditions seems to be indisputable.

Objectives

The main objective of this project was to prepare comprehensive implementation plans of the M-EPDG in Connecticut for asphalt pavements. In addition, the research team identified the short and long term needs for complete and efficient adaptation of this pavement design procedure. Lastly, in order to help local engineers to familiarize themselves with the M-EPDG, practical training materials and guidelines were developed.

Organization of the Report

This report opens with an introduction, problem statement and project objectives. Chapter 2 explores the ongoing and completed M-EPDG implementation activities on a national (Federal) and state (State) level with emphasis on the Northeast Region of the U.S. Chapter 3 provides an overview of the M-EPDG design inputs and distress prediction models. Chapter 4 summarizes typical traffic volumes, pavement features, and site conditions for Connecticut, followed by the summary of the analysis of M-EPDG input sensitivity in Chapter 5. The detailed report on the analysis of sensitivity and ranking of inputs in terms of their influence on the distress prediction is provided in Appendix A. Chapter 6 discusses needs for additional data collection protocols to meet M-EPDG requirements, while Chapter 7 explores needs in local calibration of the M-EPDG distress models, and provides guidelines for future calibration efforts. Chapter 8, in conjunction with Appendix B, summarizes efforts on development of training materials for pavement design personnel. The report is concluded with Chapter 9 providing the proposed step-by-step M-EPDG implementation plan and recommendations for future research.

CHAPTER 2 Literature Review of M-EPDG Implementation Activities on the National and State Level

To be successful in the implementation of M-EPDG in Connecticut, an understanding of the underlying concepts of the guide as well as more information about the guide's state-of-the-practice for adaptation and calibration on a local level are required. The research team has conducted an inclusive literature review to explore the history of the AASHTO Pavement Design Guide and the concepts of the M-EPDG, and has summarized the implementation activities on both the federal and local levels with an emphasis on the northeast region of the U.S.

History of AASHTO Pavement Design Guide

In the late 1950s, the AASHO road test was constructed in Ottawa, Illinois for the primary purpose of developing a fair tax scheme for different vehicle types based on fuel consumption (Galal and Chehab 2005; Smith, Zimmerman, and Finn 2004). Based on the design data from those test sections and the measured traffic and performance histories, the first AASHO interim pavement design guide was published in 1972. The 1972 design guide introduced many innovative design concepts that still serve the pavement design community, such as the present serviceability index (PSI), traffic damage factors and equivalent single axle loads (ESALs), and the structural number (SN). The 1972 Guide was revised in 1986 and again in 1993, the latter revision only focusing on pavement overlay design procedures.

Nationally, the majority of State DOTs have adopted a version of the AASHTO design guide as their method for developing new and rehabilitated designs for their pavement structures. In a 2004 survey of state agency pavement design practices, 24 of the 49 responding agencies (51 percent) indicated that they use the 1993 AASHTO guide, 3 agencies (6 percent) stated that they still use the 1972 AASHO guide, 14 agencies (29 percent) use a combination of AASHTO and State practices, while the remaining eight agencies (16 percent) use another design procedure (FHWA 2004). The 2004 survey of New England states and their neighbors showed that Connecticut, New Jersey, New York and Pennsylvania used the 1993 AASHTO procedure, while Maine, Massachusetts and New Hampshire used the AASHTO guide in combination with their own design procedures.

Need for Development of the M-EPDG

While the original versions of the AASHTO Guide for Design of Pavement Structures (i.e., 1972, 1986, and 1993) have served the pavement design community well, they were based on the empirical results of one road test in the late 1950s with the following shortcomings (NCHRP 2004):

- Traffic load limitations
  - Just over 1 million axle load replications
  - Outdated truck characteristics, such as suspensions, axle configurations, and tire design and configuration
- Environmental effect limitations
  - One location; Ottawa, Illinois
  - Short duration of the project (two years)

- Materials deficiencies
  - One hot mix asphalt (HMA) mixture
  - One Portland cement concrete (PCC) mixture
  - Two unbound, dense granular base/subbase
- Performance deficiencies
  - Previous versions of the AASHTO guide are thickness oriented, while pavements often require rehabilitation for reasons related to material properties (e.g., rutting, thermal cracking, joint faulting)

The obvious limitations of the empirical equations based on AASHO Road Test results created a need for the development of a new pavement design guide based on mechanistic engineering principles and relationships. In the mid-1990s, the research was initiated as NCHRP Project 1-37A under the oversight of an NCHRP technical panel that included state DOTs representing the Joint Task Force Panel, the HMA and PCC paving industries, academia, and FHWA (AASHTO 2004). The latest version 1.1 of the M-EPDG and its accompanying software were released in 2009 and were only available for evaluation and academic research through 2011 (TRB 2013).

Principles of M-EPDG and Implementation Needs

The main concept of the M-EPDG approach is to simulate the performance of the designed pavement in order to determine the expected accumulated damage on a monthly basis over the selected design period. Incremental damage calculations are based on monthly changes in traffic, climate, and material properties that are computed within the design software. Finally, the incremental damage accumulated on a monthly basis is converted into physical pavement distresses and expected smoothness using calibrated models that relate the damage to observable distresses (NCHRP 2004). For flexible pavements, performance is expressed in terms of longitudinal cracking, transverse cracking, fatigue (alligator) cracking, rutting, and smoothness (International Roughness Index [IRI]). For rigid pavements, performance is expressed in terms of faulting, cracking, IRI, and punchouts (for continuously-reinforced concrete pavements [CRCP] only). Figure 2.1 illustrates the inputs and analysis methods employed by the M-EPDG.

The fundamental differences between the new approach to pavement design and the approach used in the older versions of the AASHTO design guide include the following (NCHRP 2004):

- A trial design is proposed with input of the traffic, climate, subgrade, existing pavement condition for rehabilitation, and construction conditions for a new pavement or rehabilitation
- The trial design is checked for adequacy through the prediction of key distresses and smoothness. If the design does not meet desired performance criteria, it is revised and the evaluation process is repeated, as necessary
- The designer can optimize the design using different combinations of design features and materials for the prevailing site conditions

The distress prediction models that are an integral module of the mechanistic-empirical (M-E) procedure were originally calibrated to national averages using data from the Long-Term Pavement Performance (LTPP) effort. As recommended by the NCHRP 1-37A project team, the distress models need to be re-calibrated with data obtained locally in order to be applicable for the particular materials, construction practices, and environmental conditions encountered in a given state. The following general steps are suggested (NCHRP 2004):

- Achieve full support of the departmental personnel
- Select procedures to obtain all input and establish local data and defaults for inputs
- Complete training of staff involved in pavement design
- Acquire needed equipment and computer software
- Calibrate/Validate the M-EPDG software to local conditions

In knowing that only one pavement section from Connecticut was used for the national calibration of the M-EPDG distress prediction models, the need for calibration of the M-EPDG software to Connecticut local conditions was deemed to be required in the preliminary stage of this project.

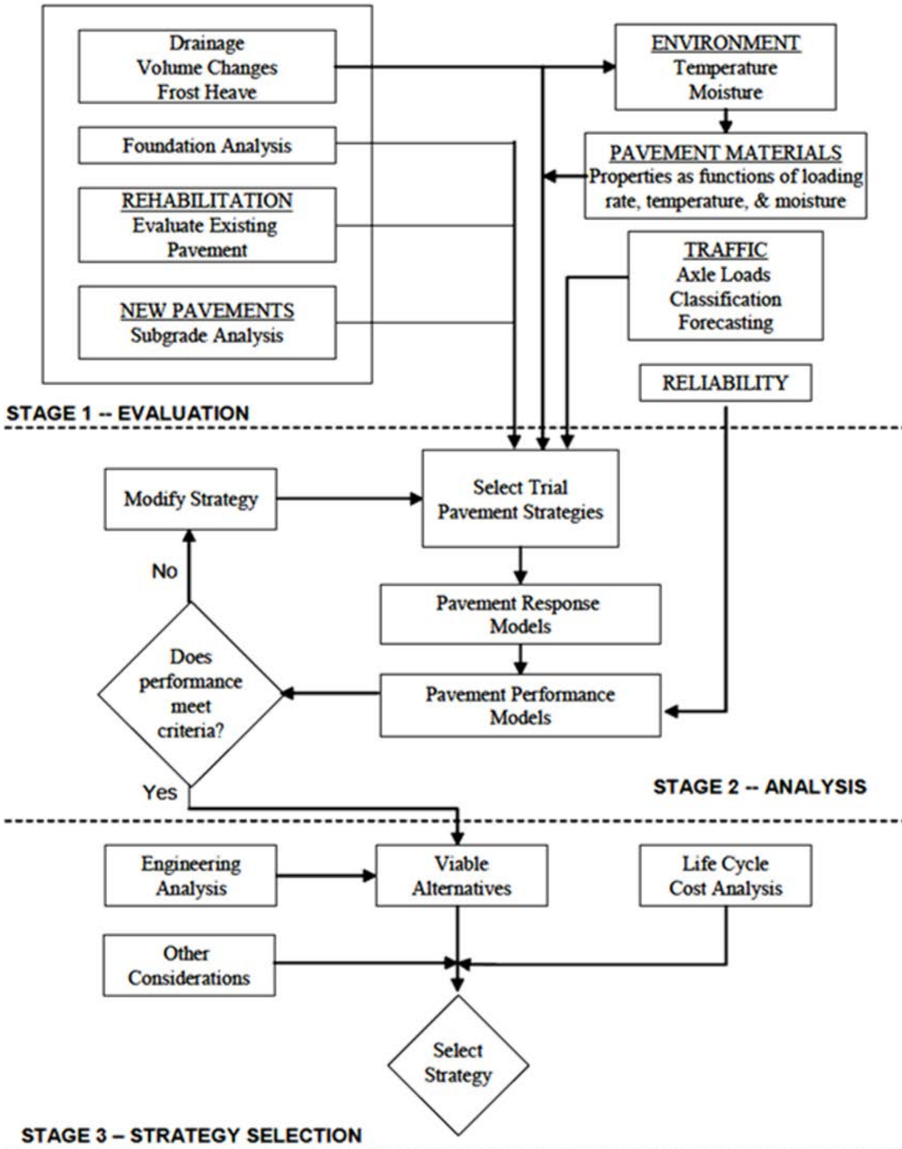

Figure 2.1. Flow chart of the M-EPDG (NCHRP 2004)

National Status of the M-EPDG Implementation Efforts

Since the first evaluation version of the M-EPDG was released in 2002, the awareness of the M-EPDG by the State Highway Agencies (SHA) nationwide has evolved from skepticism and reluctance to a national effort on research and implementation of the guide. Thus, the SHA's survey in 2004 showed that nationally, 80 percent of SHAs use AASHTO procedures alone or in combination with the local guides for their pavement design (FHWA). With regard to New England, only Maine had an M-EPDG implementation plan in place at that time. The skepticism about the advantages of the new M-EPDG procedure as compared with the long time accepted 1986 (1993) AASHTO Design Guide was justified by the absence of a reliable working version of the M-EPDG at the time of the survey (NCHRP 2006a).

After the release of a new AASHTO Interim Pavement Design Guide in 2008 called "Mechanistic-Empirical Pavement Design Guide: A Manual of Practice, Interim Edition," (the final 1.1 version of the M-EPDG software was only available until 2011 when it was commercialized into the DARWin-ME^{TM} package) a recent survey of 42 agencies shows that about 19 percent of them use M-EPDG on a periodic basis or have at least completed validation and local calibration. (Crawford 2011) The survey also indicated that about a third of SHAs are preparing input libraries (Crawford 2011). Twenty-nine percent of SHAs are still conducting research, while the rest of agencies (~36%) do not plan to implement M-EPDG in the immediate future (Crawford 2011).

Federal Efforts on Development and Improvement of the M-EPDG

Following the first release of the 2002 Design Guide under the NCHRP 01-37A Project in 2004, quite a few NCHRP activities were initiated to improve the accompanied M-EPDG software. The NCHRP 01-40 Project had the following primary objectives:

- Obtain independent review of the guide and accompanied software from a panel of pavement research specialists (NCHRP 2006a)
- Identify and eliminate software bugs and deficiencies(NCHRP 2006b)
- Develop a user manual of practice and local calibration guide (AASHTO 2010)
- Provide technical assistance for SHAs participating in implementation of the Guide

The main deliverable of this project is a commercialized AASHTO DARWin-ME^{TM} software. This software combines the distress predicting capabilities of the M-EPDG with user-friendly interfaces and the ability of report-generating of the DARWin software, which was based on the 1993 AASHTO predictive equations.

Along with the elimination of program-related bugs, the M-EPDG researchers were working on improving the distress-prediction models. Thus, an enhanced reflective cracking model was created under NCHRP 01-41, a rutting model was recalibrated nationally under the NCHRP 09-30A, and an attempt at improving top-down fatigue cracking predictions were made under NCHRP01-42A (Lytton et al. 2010, Von Quintus et al. 2012, Roque et al. 2010). Those alternative models are offered as options in the latest available version 1.1 of the M-EPDG software.

Concurrently with local investigations of the M-EPDG sensitivity, which will be discussed later in this report, an attempt on global sensitivity analysis was made under the NCHRP 01-47 (Schwartz et al. 2011). In contrast with multiple local sensitivity studies, the project targeted all possible pavement designs (new and rehabilitated HMA and plain/reinforced PCC) in five major US climates (Hot-Wet, Hot-Dry, Cold-Wet, Cold-Dry, and Temperate). Over 41,000 M-EPDG software runs were performed to determine Normalized Sensitivity Indices (NSI) of 25 to 35 inputs evaluated in this study. The NSI values were computed as a ratio of the percentage change in predicted distress over the percentage change in a design input normalized to the design limit of the distress (Schwartz et al. 2011). It should be noted here that the UConn research team did not adopt this method but rather used a limited statistical approach, which proved robust yet easily interpretable in local sensitivity studies (Yut et al. 2007, Hoerner et al. 2007, Velasquez et al. 2009, Daniel et al. 2012). It is also worth noting that the NCHRP 01-47 researchers used one-at-a-time approach in changing inputs, which might not account for interaction between critical inputs; therefore, the adaptation of the results of NCHRP 01-47 for local SHAs should be limited to the specific ranges of inputs used in the study.

Local Implementation and Calibration of the M-EPDG

This section summarizes the well-documented M-EPDG implementation and calibration activities on the SHA level. The emphasis is made on the championing states and the northeast region of the U.S.

Mississippi

Mississippi is one of a very few states that pioneered implementation of the M-EPDG as early as 2002 (Saeed and Hall 2003). The two-phase implementation project initiated by the Mississippi DOT consisted of developing an implementation plan in Phase I and actually implementing the design guide in Phase II. In Phase I, the implementation plan included familiarizing DOT staff with the M-EPDG, establishing the scope of pavement types and rehabilitation activities of interest to the DOT, developing a factorial experiment design, recommending test sections for use in calibrating and validating performance models, preparing a detailed plan for the Phase II implementation, and estimating a budget for implementing the M-EPDG. The specific Phase II work plan in Mississippi included the following research tasks (Saeed and Hall 2003):

- Review all design inputs.
- Conduct an initial sensitivity analysis and compare with current DOT procedures.
- Provide guidance to carry out the required field and laboratory testing.
- Outline work related to obtaining all design inputs, including detailed traffic inputs, selection of performance criteria, and material testing.
- Establish default inputs where applicable.
- Calibrate and validate the distress prediction models with Mississippi pavement performance data.
- Conduct additional sensitivity analysis and comparison of the design guide procedure with current Mississippi DOT design procedure results.
- Prepare detailed design and training manuals for training and future reference.
- Customize the design guide software to include Mississippi-calibrated performance models and default inputs.
- Provide training to Mississippi DOT staff.

Iowa

The M-EPDG research and development along with implementation efforts in Iowa date back at least a decade. Coree et al. (2005) was one of the first research groups that applied importance rankings to the M-EPDG design inputs based on a one-at-a-time sensitivity analysis. They found that PCC mix thermal properties and strengths along with slab thicknesses were the most influencing inputs for rigid pavements, whereas the Performance Grade (PG) of binder and volumetric properties of HMA were the most influencing material properties for flexible pavements (Coree et al. 2005). Most of the work performed by the Iowa State University team involved in the M-EPDG implementation revolved around calibration of the M-EPDG rutting and longitudinal cracking models based on characteristics for Iowa inputs for HMA pavements (Kim et al. 2010, Kim et al. 2013). They also observed differences in cracking and faulting model calibrations between M-EPDG version 1.1 software and the commercial DARWin-ME, which, once again, required re-calibration (Kim et al. 2013).

Minnesota

Minnesota has been working on the development of the M-EPDG since as early as 1998. The University of Minnesota is assisting the Minnesota Department of Transportation (MnDOT) in its implementation efforts for the M-EPDG. A comprehensive sensitivity analysis has been conducted for both rigid (200,000 runs) and flexible pavements (2,000 runs) to determine the most significant factors on pavement performance (Velasquez et al. 2009). This study also identified the M-EPDG software deficiencies, evaluated at least five interim versions of the M-EPDG, and re-calibrated rutting and fatigue cracking models to reduce bias and error in performance prediction for Minnesota conditions (Velasquez et al. 2009). Another project with emphasis on the low volume PCC pavements in Minnesota evaluated prediction capabilities of the M-EPDG, re-calibrated transverse cracking models incorporated in the M-EPDG software, and developed catalogs of recommended design features (Yut et al 2007).

South Dakota

A very detailed analysis of the M-EPDG inputs for both PCC and HMA and a comprehensive investigation on the resources needed for successful implementation was completed by the South Dakota DOT (Hoerner et al 2007). The researchers conducted sensitivity analyses on about 80 design inputs required by the M-EPDG to predict behavior of flexible and composite pavements. The team also prioritized those inputs in terms of their significance for the prediction of the distresses and their importance for further data collection, material testing, and prediction model calibration efforts. As a result of the study, Level 1 and Level 2 inputs were recommended for highly significant inputs, whereas Level 3 inputs were found to be satisfactory for the inputs of mild and low significance. The preparation of the Pavement Management System data for use in calibration of the M-EPDG prediction models was identified as an essential part of the implementation process (Hoerner et al 2007).

Other States

Baus and Stires (2010) developed recommendations for implementation of the M-EPDG in South Carolina. One interesting outcome of this study is that it was recommended to establish a minimum of 20 in-service pavement sections either instrumented or periodically tested for validation and calibration of the M-EPDG distress prediction models.

Bayomy et al. (2012) reported emphasis on binder and mix characterization to establish Level 2 inputs for HMA pavements and subgrade characterization for Level 3 unbound material inputs in Idaho. They also recommended using at least 3 years of Weigh-in-Motion sites’ data to establish reliable traffic inputs.

U.S. North East Region Efforts on M-EPDG Research and Development

Connecticut

The Connecticut Department of Transportation was the lead state in a pooled fund project with the University of Connecticut to coordinate pavement activities in the northeastern United States. As a part of this effort, interviews of transportation agency staff in the northeast were conducted to determine needs in pavement and paving technology, and particularly, the issues with design of pavement systems, sub-systems and specification requirements (Dougan 2004). The two top ranked needs identified by the researcher in a pavement design category were (1) need in training of the personnel in M-EPDG and (2) evaluation of applicability of M-EPDG in New England (Dougan 2004)

Maine

The Maine Department of Transportation has constructed a weigh-in-motion pavement instrumentation (WIMPI) site along Rt. 16 in Guilford. The site will measure the actual distresses in the pavement layers due to traffic loads and climatic changes (Maine DOT 2005). The analysis of data from the instrumented pavement test section will be used to calibrate the Mechanistic-Empirical Pavement Design Guide (Maine DOT 2006).

New Jersey

Rutgers University conducted a sensitivity analysis to determine the impact of the Portland cement concrete Poisson's Ratio on the pavement performance as predicted by the M-EPDG in New Jersey. In addition, laboratory testing was completed to (1) assess the level of variability of Poisson's ratio of typical pavement materials (bound and unbound) and subgrade soils for various temperature, moisture, and stress conditions under laboratory conditions and (2) to develop a method of selecting the appropriate Poisson's ratio values for use in Mechanistic Pavement design (CAIT Date Unavailable).

Recently, the evaluation of input accuracy and performance data from seven LTPP sections in the state of New Jersey was sponsored by the New Jersey Department of Transportation (NJDOT) (Mehta et al. 2008). The objective of the study was to provide the state agency with the tools and the knowledge needed to successfully implement the design guide. A case-by-case comparison was conducted between predicted and measured performance data for every section and each distress, such as rutting, fatigue cracking, longitudinal cracking, transverse cracking and roughness. The analysis determined conditions where the Level 3 inputs may not be appropriate.

New York

The New York State DOT (NYSDOT) has undertaken projects with objectives to review the guide and its associated software, to comment on the Guide/software, to coordinate the Department's AASHTO review processes, to develop an implementation plan, and ultimately to adopt the new AASHTO Pavement Design Guide in New York State (NYSDOT 2002).

Pennsylvania

The Pennsylvania Department of Transportation (PennDOT) collects HMA performance data from six instrumented pavement test sections (Anderson et al. 2003). This data will be used for calibration of the AASHTO pavement design procedures. PennDOT also is working on instrumenting a PCC pavement project which will help toward local calibration of the M-EPDG.

New England Transportation Consortium (NETC)

In 2006, the New England Transportation Consortium (NETC) initiated two projects aimed toward implementation of the MEPDG. In the first study, NETC contracted the University of Connecticut in Storrs to test commonly used HMA mixtures throughout New England to determine their respective dynamic moduli master curves (Jackson et al. 2011). The results of physical modulus testing were compared to predicted modulus values from three different theoretical modulus models. Comparisons of predicted |E*| values from the Mechanistic-Empirical Pavement Design Guide (MEPDG) and physical testing indicates the predicted |E*| values may be off by as much as 100% for New England Mixes. The analysis of scaling factors showed that there is potentially a constant scaling factor that could be applied to all New England mixes, regardless of aggregate source, and binder type (Jackson et al. 2011)

In another study sponsored by NETC, the University of New Hampshire in Durham has developed guidelines for the implementation of the M-EPDG in New England and New York with focus on flexible pavements and AC overlays (Daniel et al. 2012). Only the LTPP sections from the six New England States and New York State were included in the sensitivity analysis (Daniel et al. 2012). The research team recommended using specific values for some design inputs and Level 3 default values for the others. It's worth noting that the final NETCR 87-06 report provides very detailed recommendations for each group of inputs (Daniel et al. 2012). It is also worth noting that for Connecticut, three climatic zones were recognized (Groton-New London, Bridgeport, and Hartford), which yielded significant difference in thermal cracking. However, the range of binder PG values used in this study for Connecticut (52-22, 58-22, and 64-22) is questionable in knowing that 64-22 and 64-28 have mainly been used for years in these areas. One important conclusion from this study is that the researchers recognized that differences between New England states are great enough to necessitate a closer look on the range of input values for each individual state in the region.

Closing Remarks

The vast majority of SHAs use empirical AASHTO procedures for pavement design. Recently, the new Mechanistic-Empirical Pavement Design Guide has been developed to assist the designers in optimizing the design through the use of different combinations of design features and materials for the prevailing local conditions. Before adopting the M-EPDG, a better understanding of its concepts, as well as a comprehensive implementation plan, are needed to make it work on the state/project level.

The literature search allowed tracking of the most recent M-EPDG implementation activities in New England, as well as in the SHAs that are in the most advanced phase of implementation. The review of the information available from the Transportation Research Board and DOT publications indicated the following general steps to be made for preparation of a comprehensive implementation plan:

- Identify design inputs relevant for the local typical pavement designs
- Conduct sensitivity analysis of selected inputs to establish their significance for the M-EPDG distress prediction models
- Estimate resources needed for the local agency to collect data that is needed to establish the design input values on a desired level of prediction accuracy
- Prepare implementation plan including M-EPDG-related activities, such as staff training and M-EPDG-related local guidance and specifications

The Research Team used the information obtained from the literature search to develop the research methodology for this project.

CHAPTER 3 Review of M-EPDG Design Inputs and Distress Prediction Models

The M-EPDG software incorporates distress prediction models that are based on the correlation between accumulated damage in pavement layers due to traffic loads and temperature gradients. The damage computations utilize numerous inputs related to the thermal, mechanical and volumetric properties of bound and unbound materials. Depending on the desired reliability of the distress prediction, an agency may be required to obtain different levels of detail on a particular material property. To facilitate understanding of the M-EPDG hierarchical input system broad categories of the climatic, traffic, and material inputs are covered in the following section, while the detailed list of input levels for each hierarchical level is provided in Appendix A.

Hierarchical Approach to Inputs in the M-EPDG

The M-EPDG was developed using a hierarchical approach to provide pavement designers with flexibility in making decisions on desirable levels of detail for design inputs. A level of detail would depend on a criticality of a project (e.g., high-volume interstate versus low-volume local collector) and availability of resources for obtaining required data. (NCHRP 2004). The Guide defines the following three levels of inputs:

1. Level 1 provides the highest accuracy with the lowest degree of uncertainty. Therefore, it requires project-specific inputs obtained from either laboratory or field testing (e.g., binder/mix master curves), site-specific axle load distribution data, or nondestructive deflection testing. Due the extensive amount of time and resources required to perform a Level 1 design, it is recommended only for where a low likelihood of failure is warranted, and for research projects and forensic studies.

2. Level 2 yields an intermediate level of accuracy similar to the typical procedures associated with the AASHTO 1986 design guide. The inputs for Level 2 can be selected from an agency database, obtained from a limited testing program, or estimated through empirical relationships. For instance, an HMA dynamic modulus can be estimated from binder viscosity, aggregate strength, and volumetric properties of the HMA mixture.

3. The use of Level 3 inputs results in the lowest level of accuracy, therefore, it is recommended for typical projects with low variation in material properties and low traffic volumes. The M-EPDG software incorporates default values, but the software allows the average values for a particular region to be used instead of the national default values.

Inputs of mixed hierarchical levels can be used in the same project (i.e., Level 2 binder data along with Level 3 subgrade data). The computation process, however, remains the same regardless of the quality of the input data.

Climatic Inputs

The previous versions of the AASHTO pavement design guide (e.g., 1993, 1986) addressed differences in climate by applying seasonal adjustments to material moduli using drainage coefficients. The M-EPDG introduces a one-dimensional finite element model, which is called Enhanced Integrated Climatic Model (EICM), to compute temperature and moisture gradients through each pavement layer and the subgrade (NCHRP 2004). The EICM model requires quite a few input parameters that can be divided into the following categories:

- General Information: construction dates for each pavement layer, open-to-traffic dates, and type of design (new or rehabilitated, HMA, or PCC)

- Weather-Related Inputs (hourly values): air temperature, precipitation, wind speed, percent sunshine, and relative humidity over the pavement design life (all obtained from weather stations throughout the US).

- Depth of Groundwater Table (obtained from boreholes for Levels 1 and 2 or from the National Resources Conservation Service reports for Level 3 inputs)

- Drainage and Surface Properties: surface short wave absorptivity, water infiltration potential of the pavement (none, minor, moderate, and extreme levels for 0, 10, 50, and 100 percent of precipitation entering the pavement, respectively), drainage path length, and the pavement cross slope.

- Pavement Material Properties: layer thickness; thermal conductivity (K) and heat capacity (Q) for HMA and PCC layers; specific gravity, saturated hydraulic conductivity, maximum dry unit weight, dry thermal conductivity, heat capacity, plasticity index, gradation, optimum gravimetric water, and equilibrium gravimetric water content for unbound materials of base and subgrade.

Traffic Inputs

The M-EPDG procedure utilizes axle load spectra data to compute the total design 18 Kip Equivalent Axle Loads (ESALs). This requires the following inputs:

- Base year truck-traffic volume (the year used as the basis for design computations)
- Vehicle (Class 4 to 13 truck) operational speed
- Truck-traffic directional and lane distribution factors
- Truck class distribution
- Axle load distribution factors
- Axle and wheel base configurations
- Tire characteristics and inflation pressure
- Truck lateral distribution factor
- Truck traffic growth factors

Material Related Inputs

All materials considered in the M-EPDG can be divided into two large groups: bound materials (HMA, PCC, Stabilized Bases) and unbound materials (granular bases/subbases and subgrade).

The inputs related to those materials are further classified into (1) critical response inputs, (2) transfer function inputs, and (3) climatic modeling inputs. Table 3.1 provides a summary of the material-related inputs.

Table 3.1. Summary of Required Material-Related Inputs for the M-EPDG

Material Category	Input category		
	Critical Response Inputs	Distress/Transfer Inputs	Climatic Modeling Inputs
HMA materials (surface, binder, and base courses)	• Time-temperature dependent dynamic modulus of elasticity • Poisson's ratio	• Tensile strength • Creep compliance • Coefficient of thermal contraction	• Surface shortwave absorptivity (for HMA surface only) • HMA thermal conductivity • HMA heat capacity • Binder viscoelastic properties
PCC materials (surface only)	• Time-dependent elastic modulus • Poisson's ratio • Unit Weight • Coefficient of thermal expansion	• Modulus of rupture • Compressive strength • Split tensile strength • Cement type • Cement content • Water-cement ratio • Ultimate shrinkage • Reversible shrinkage	• Surface shortwave • Absorptivity • Thermal conductivity • Heat capacity
Chemically and cementitiously stabilized materials (lean PCC, cement/ lime/ fly ash stabilized bases and soils)	• Elastic modulus • Poisson's ratio • Unit weight	• Minimum resilient modulus • Modulus of rupture • Base erodibility	• PCC thermal conductivity • PCC heat capacity
Unbound base/subbase and subgrade materials	• Seasonally adjusted resilient modulus • Poisson's ratio • Unit weight • Coefficient of lateral pressure	• Gradation (% passing) • Base erodibility	• Plasticity index • Gradation (% passing) • Specific gravity • Hydraulic conductivity • Optimum moisture content • Soil-water curve parameters
Recycled PCC materials	• Resilient modulus • Poisson's ratio	• Base erodibility	• Thermal conductivity • Heat capacity
Recycled asphalt pavement (RAP) (plant-processed)	Same as for HMA surface		
Cold RAP (aggregate)	Same as for unbound materials with no moisture sensitivity		
Bedrock	• Elastic modulus • Poisson's ratio • Unit weight	None	None

Rehabilitation-Specific Inputs

The M-EPDG considers three main categories of pavement rehabilitation design: (1) Restoration of Jointed Plain Concrete Pavement (JPCP), (2) HMA overlay of existing HMA or PCC pavement, and (3) PCC overlay of existing PCC or HMA pavement. Each category includes several sub-categories based on the type of existing pavement treatment (e.g., do nothing, HMA milling, PCC rubblization, etc.). Since the main rehabilitation technique in Connecticut is an HMA overlay of either milled HMA or PCC pavement, the rehabilitation-specific inputs for those two sub-categories are discussed in this section (See Table 3.2). Note that the material-related inputs for both existing and new layers are listed in Table 3.1. For this research the main focus was HMA overlay of existing HMA and HMA overlay of rubblized PCC. In future research, the scenario of HMA overlay of PCC should be examined.

Table 3.2. Summary of Required Rehabilitation-Specific Inputs for the M-EPDG

Rehabilitation Design Type	Rehabilitation-Specific Inputs
HMA overlay of existing HMA pavement	Milled thickness [in] Geotextile presence [True/False] Pavement rating of the existing HMA surface [Good to very Poor] Total rutting of the existing HMA surface [in]
HMA overlay of existing rubblized PCC pavement	<u>HMA overlay-related inputs</u> Milled thickness [in] Geotextile presence [True/False] Pavement rating of the existing HMA surface [Good to very Poor] Total rutting of the existing HMA surface [in] <u>Rubblized PCC-related inputs</u> Elastic resilient modulus of the fractured slab [psi] Type of fracture [Rubblization]

Overview of the M-EPDG Distress Prediction Models

To evaluate pavement performance during its design life, the M-EPDG procedure utilizes three stages on which (1) The monthly cumulative damage is computed from the hourly critical pavement responses to the traffic and environmental loads, (2) The amount of distress for each month is predicted by statistical distress-damage models calibrated on the LTPP performance data, and (3) The distress trend over the service life at a specified level of reliability is produced in tabulated and graphic format. Figure 3.1 shows a simplified schematic of the distress prediction process. The distress values are compared with the performance thresholds, or maximum values specified by an agency, to make a decision on the acceptance of a particular design or needs in an alternative one.

Figure 3.1. Flow chart of predicting distresses from cumulative damage through transfer functions.

While the detailed documentation of the distress prediction models can be found elsewhere (NCHRP 2004, AASHTO 2008), this report summarizes information pertinent to pavement design in Connecticut. Effectively, the following distress models for new and rehabilitated HMA pavements are discussed:

- Longitudinal cracking (top-down fatigue)
- Alligator cracking (bottom-up fatigue)
- Reflective cracking (for rehabilitated HMA pavement only)
- Thermal (transverse) cracking
- Rutting in asphalt layer
- Total rutting
- Roughness in terms of International Roughness Index (IRI)

Longitudinal (Top-Down Fatigue) Cracking

Although most of the traffic load-related fatigue cracking propagates from the bottom of the HMA layer up to its surface in the direction of traffic, it has been commonly accepted that the top-down fatigue (See figure 3.2) can develop in the longitudinal direction due to the following factors (Roque2010):

- Bending-induced surface tension away from the tire in thin to medium HMA layers
- Shear-induced near-surface tension at the tire edge in thicker HMA layers

- HMA aging, which accelerates development of both bending and shear-induced damage

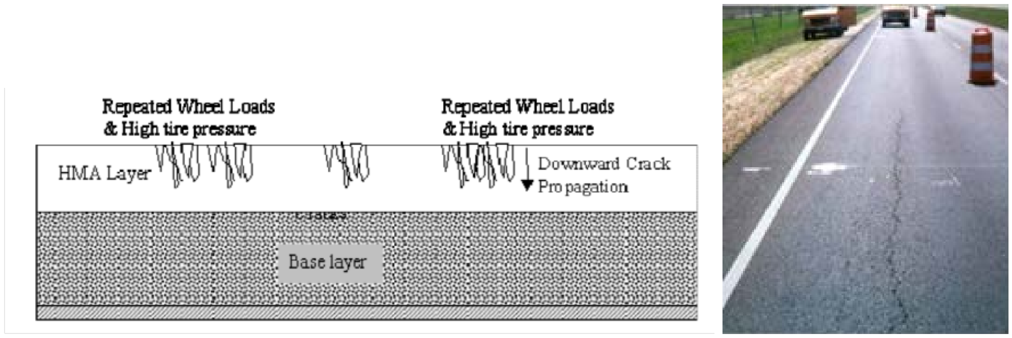

Figure 3.2. Top-down fatigue cracking schematic (left [MEPDG 1.100]) and image (right).

The M-EPDG algorithm utilizes an equation (See Figure 3.3) to directly predict longitudinal cracking (FC_{top}) from the damage due to cumulative traffic load and the elastic and volumetric properties of the HMA mixture. The damage computed using Miner's principle (See Figure 3.4), is used to predict the number of ESALs at failure (N_f as shown in Figure 3.5).

The regression coefficients C1 through C4 can be adjusted by an agency for local calibration. Note that the standard deviation equation shown in Figure 3.3 was developed from the LTPP data and it shows very low reliability for prediction of cracking. For example, 10 ft/mi of longitudinal cracking at 50-percent reliability will correspond to 1923 ft/mi at one standard deviation (84-percent reliability), which nears the default performance threshold of 2000 ft/mi recommended by the M-EPDG. Such a low correlation between predicted FC top values and field measurements may be explained by an inconsistency in cracking definitions (wheelpath versus non-wheelpath) and section boundaries (e.g., longitudinal joints on one or both sides of the lane taken into consideration).

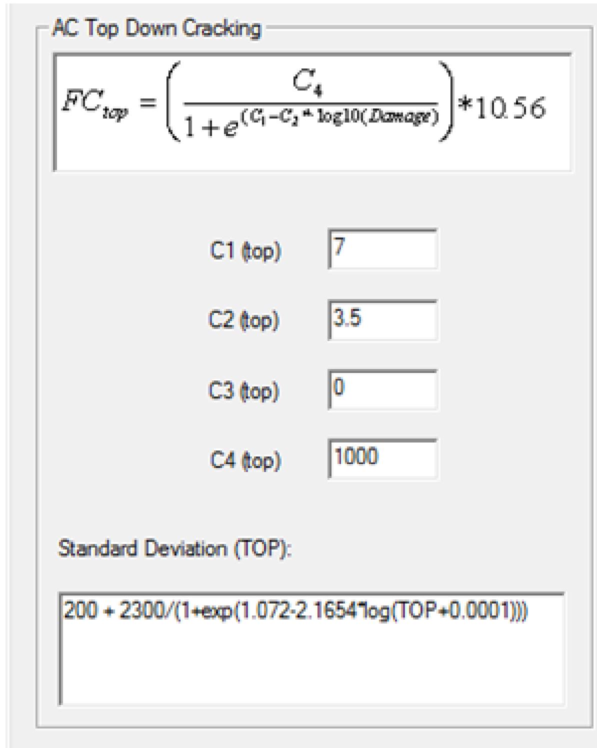

Figure 3.3. HMA Top-Down Cracking Model (MEPDG 1.100)

$$Fatigue\ Damage = \sum_i \sum_j \sum_k \sum_l \sum_m \sum_n \frac{n_{ijklmn}}{N_{ijklmn}}$$

where:

n_{ijklmn} = Applied number of load applications at condition i,j,k,.
N_{ijklmn} = Allowable number of load applications at condition i,j,

i = Age
j = Season
k = Axle combination
l = Load level
m = Temperature gradient
n = Traffic path

Figure 3.4. Fatigue Damage Calculation.

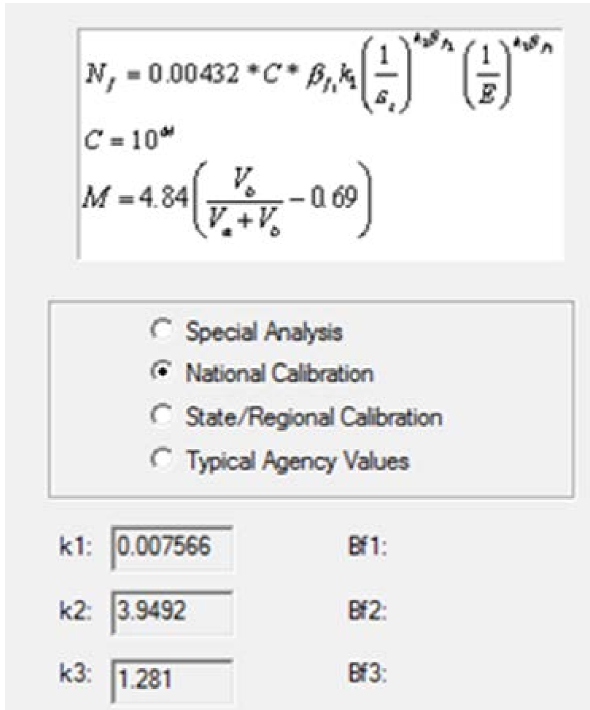

Figure 3.5. HMA Fatigue Model (MEPDG 1.100)

The asphalt fatigue model (Asphalt Institute) equation shown in Figure 3.5 allows for the local calibration by switching from the National Calibration option to the state/regional calibration option and changing the coefficients β_{f1} through β_{f3}.

Alligator (Bottom-Up Fatigue) Cracking

This type of fatigue cracking develops due to repeated bending of the HMA layer under traffic. This bending results in tensile stresses, which cause cracks that initiate at the bottom of the layer and will increase with continued loadings until the cracks propagate to the surface of the layer (See Figure 3.6). The most common reasons for alligator cracking are (NCHRP 2004):

- Inadequate HMA thickness or strength for the traffic magnitude and repetitive loading
- Higher wheel loads and higher tire pressures
- Soft spots or areas in unbound aggregate base materials or in the subgrade soil
- Weak aggregate base/subbase layers caused by inadequate compaction or increases in moisture contents, and/or an extremely high ground water table

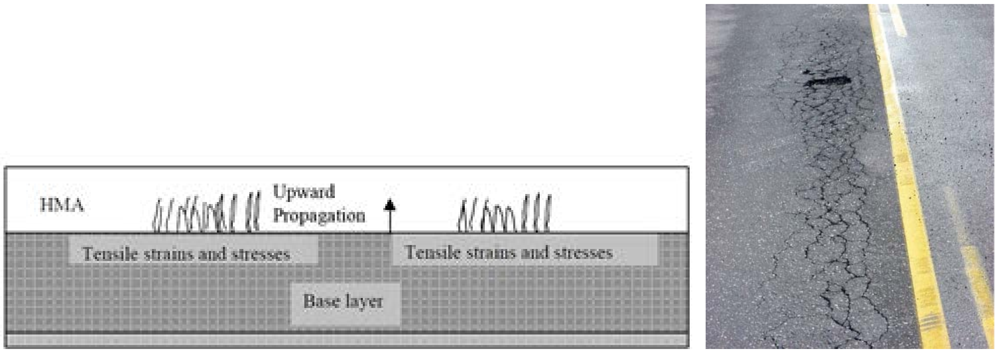

Figure 3.6. Alligator (bottom-up fatigue) cracking schematic (left [MEPDG 1.100]) and image (right).

Similar to the longitudinal cracking, the M-EPDG directly predicts bottom-up (alligator) fatigue cracking from the damage (D) calculated as shown in Figures 3.4 and 3.5. However, the model incorporates another major factor, HMA layer thickness (h_{AC}), into the equation (See Figure 3.7). The reliability of the prediction based on the LTPP database is indicated by the standard deviation equation, i.e. 10 percent alligator cracking predicted at 50 percent reliability will yield 24 percent of area failed, which is very close to 25-percent default threshold for this distress. This also suggests the need for calibration of the alligator cracking model on local data.

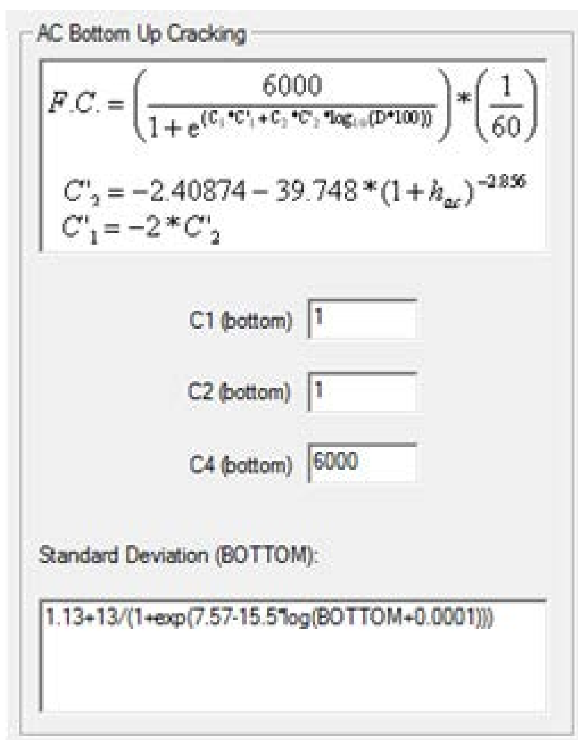

Figure 3.7. Bottom-Up Fatigue Model (MEPDG 1.100)

Reflection cracking
It is worth noting that, although the reflection cracking prediction model has been incorporated into M-EPDG software since as early as version 0.9, very little or no documentation is provided in both the original (NCHRP 2004) and the most updated manual of M-EPDG Practice (AASHTO 2008). One of the reasons for that is that the reflection cracking model was never nationally calibrated due to the lack of data from the LTPP sections. Therefore, this section refers to the NCHRP Report 669 (Lytton et al. 2010) for definitions and mechanisms of reflection cracking, while the quantitative features of the model are inferred from the screen shots of the M-EPDG program user interface (M-EPDG 1.100)

Reflective (NCHRP 2004) or reflection (Lytton et al. 2010) cracking can be defined as the cracking in a pavement overlay that is caused by fatigue propagating through the overlay due to movements of some form in the vicinity of existing cracks or joints in the underlying pavement (Lytton et al. 2010). There are three possible mechanisms of the reflection crack development (Figure 3.8):

1. Traffic load-induced fatigue occurs due to excessive deflection of an overlay above an underlying crack or joint resulted in the vertical stress concentration.
2. Thermally-induced fatigue develops in an overlay due to horizontal expansion or contraction of an existing crack or joint.
3. Surface-initiated cracking due to non-linear temperature gradient from the top down to the pavement structure.

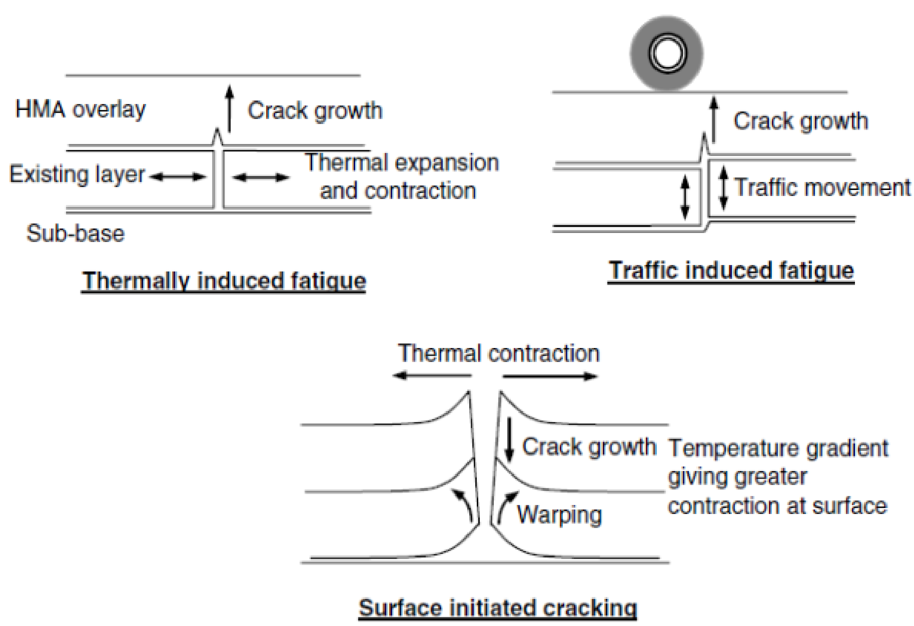

Figure 3.8. Mechanisms of Reflection Cracking (Lytton et al. 2010 after Nunn 2008)

The prediction equation for reflective cracking (RC) in Figure 3.9 includes age of pavement (t), overlay thickness (h_{ac}), and its effective thickness (H_{eff}) as independent variables. The Heff parameter is not calculated but rather a recommended value based on the existing pavement type (flexible or rigid) and quality of load transfer (good or poor). Further, the M-EPDG provides recommendations for calibrating coefficients (See Figure 3.9). Once again, the calibration of the reflection cracking model may be critical since reflective cracking is one of the most common distresses in Connecticut, especially in cases when no milling is performed before the overlay, and thin overlays have been placed.

$$RC = \frac{100}{1 + e^{c.a + d.b.t}}$$

RC = Percent of cracks reflected, %
t = Time, years
h_{ac} = Overlay thickness(in)
a = 3.5 + 0.75(Heff)
b = -0.688584 – 3.37302(Heff)$^{-0.915469}$
c = 1
d = Calibration parameter (user input)

	AC over AC	AC over Rigid, Good Load Transfer	AC over Rigid, Poor Load Transfer
Heff	h_{ac}	$h_{ac} - 1$	$h_{ac} - 3$

Heff	Recommended Calibration Parameter - d	
	Delay Cracking by 2 years	Accelerate Cracking by 2 years
< 4"	0.6	3
4 – 6"	0.7	1.7
> 6"	0.8	1.4

Reflective cracking c: 1.000

Reflective Cracking d: 1.000

Figure 3.9. Reflective Cracking Model (M-EPDG 1.100)

Thermal Cracking

Cracking in flexible pavements due to cold temperatures or temperature cycling is commonly referred to as thermal cracking (Figure 3.8). Thermal cracks typically appear as transverse cracks on the pavement surface roughly perpendicular to the pavement centerline. These cracks can be caused by shrinkage of the HMA surface due to low temperatures, hardening of the asphalt, and/or daily temperature cycles. There are two types of non-load related thermal cracks: transverse cracking and block cracking. Transverse cracks usually occur first and are followed by the occurrence of block cracking as the asphalt ages and becomes more brittle with time.

The M-EPDG thermal cracking model only predicts the amount of transverse cracking by relating the ratio of the crack depth (C) over the asphalt layer thickness (h_{ac}) to cracking frequency (C_f) through calibration coefficients (Figure 3.10). The incremental increase in crack depth (ΔC) is computed from the change in the stress intensity factory (ΔK) and HMA mix stiffness parameters A and n, which are, in turn, a function of undamaged mix tensile strength (σ_m), and mix stiffness (E) (NCHRP 2004, Appendix HH).

It appears that the current version of the thermal cracking model in M-EPDG does not yield meaningful results because of frequent crashes due to missing or inconsistent temperature data in the climatic files created by the EICM (Marasteanu et al., 2007, Hoerner et al., 2007, Velasquez et al., 2009). Apparently, an additional research effort is needed to improve the thermal cracking predictions by the M-EPDG. Nevertheless, the sensitivity analysis of the thermal cracking model for Connecticut was performed under this project, as discussed in the next chapter, to verify previous findings.

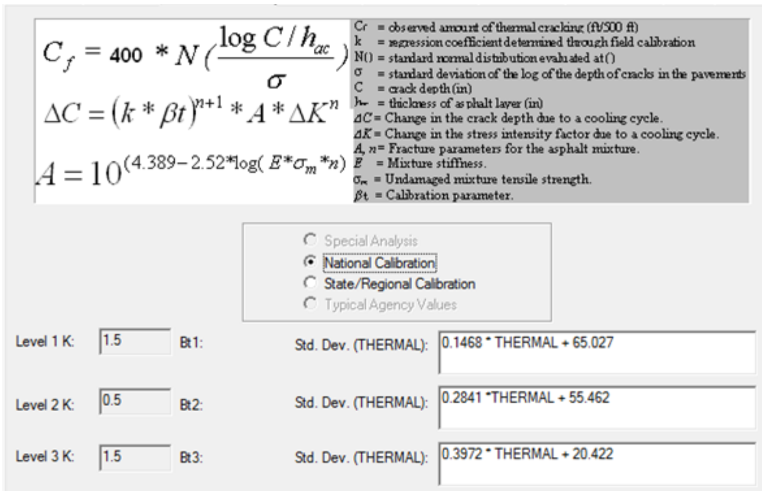

Figure 3.10. Thermal Cracking Model (MEPDG 1.100)

Asphalt Concrete (AC) Rutting

In pavement design, rutting refers to surface depression(s) in the pavement layer (normally, in the wheelpaths) due to irrecoverable plastic deformation. This permanent deformation may occur by two mechanisms:

1. One-dimensional densification or vertical compression (Figure 3.11a) is generally a result of compaction of the mat or underlying layers to compact under the traffic due to excessive air voids or inadequate compaction.
2. Lateral flow or plastic movement (Figure 3.11b) in HMA mixes with inadequate shear strength is characterized by shear upheavals on either side of the depression.

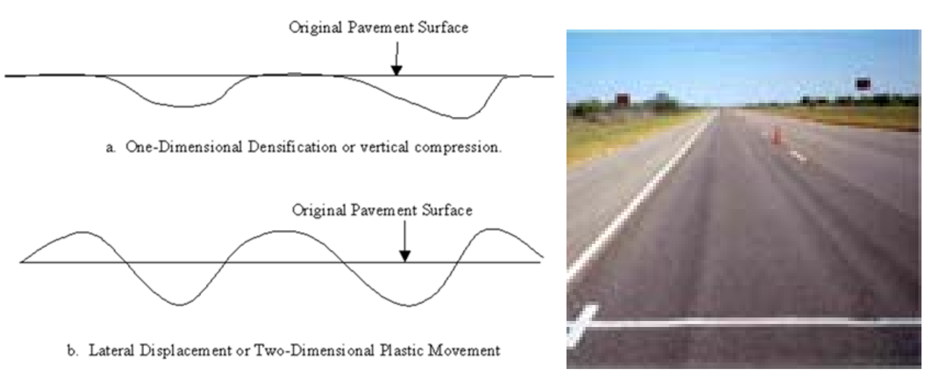

Figure 3.11. Rutting Schematics (left) and image (right)

The M-EPDG incremental damage approach requires an estimation of rutting for each sub-season at the mid-depth of each sub-layer within the pavement system. Each sub-season represents 20% of the frequency distribution of the pavement temperature over a given analysis period. The permanent deformation of each individual sub-layer is computed by separate algorithms for unbound and bound materials from the plastic strain accumulated at the end of each sub-season. The overall permanent deformation (*PD*) for a given season is the sum of permanent deformations for each individual layer (a product of plastic strain (ε_p) and sublayer thickness (*h*)) as in Equation [3.1].

$$PD = \sum_{i=1}^{nsublayers} \varepsilon_p^i h^i \qquad [3.1]$$

The plastic strain model for AC layers depicted in Figure 3.12 allows for computation of plastic strain from the resilient strain of HMA mix, layer temperature, number of traffic load repetitions, and total AC layer thickness through a regression equation (NCHRP 2004, Appendix GG). The model allows users to use local calibration coefficients to adapt the model to local conditions. Note that it is assumed that asphalt layers have no moisture content (NCHRP 2004, Appendix GG).

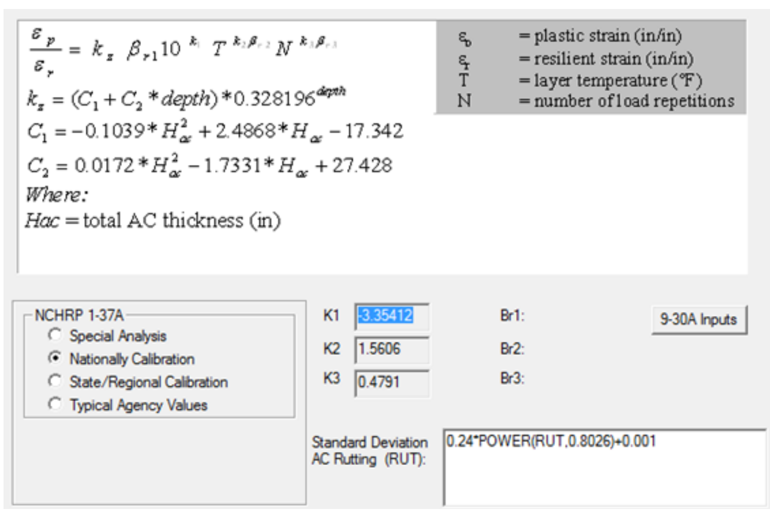

Figure 3.12. AC Rutting Model (MEPDG 1.100)

Unbound Material Rutting

Similarly to the AC rutting model, the M-EPDG predicts the permanent deformation in unbound layers (base/subbase, and subgrade) from resilient properties of the materials and layer thicknesses. The difference in the approach to unbound materials is that the temperature term is taken out of the equation (NCHRP 2004, appendix GG). It should be noted that the moisture effect on rutting in unbound materials is indirectly incorporated with a correlation of the water content with the plasticity index and the percentage of aggregates passing #200 (NCHRP 2004, Appendix GG). Figure 3.13 provides a screenshot of the base/subgrade rutting model from the M-EPDG user interface.

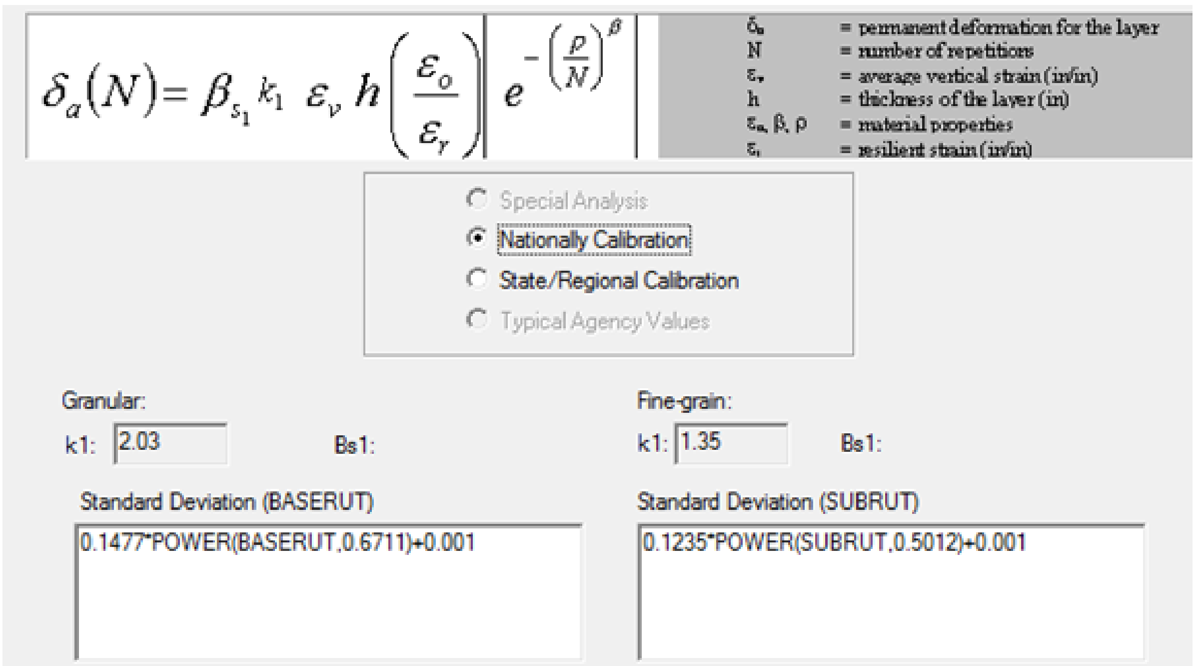

Figure 3.13. Base/Subgrade Rutting Model (MEPDG 1.100)

Total Rutting

The M-EPDG software reports separate values for each pavement layer for total rutting of the pavement structure. Total rutting is computed as the sum of permanent deformations of individual layers including AC, granular base/subbase, and subgrade. Since it seems impractical to measure rutting in unbound layers, and where it is difficult to separate asphalt and base/subbase layers' contribution to rutting, in most cases, the sensitivity analysis in this project only utilized AC and total rutting as performance indicators.

International Roughness Index (IRI)

Pavement surface smoothness has long been used nationwide by the road authorities as a measure of functional adequacy of pavements. Smoothness can be defined as the variation in surface elevation that induces vibrations in traversing vehicles (NCHRP 2004). The International Roughness Index (IRI) is one common way of measuring variations in road surface profile.

The M-EPDG utilizes linear regression models to predict the IRI over the design period. The model treats the initial IRI, rutting, bottom-up/top-down fatigue cracking, thermal cracking, and site factors as independent variables. The site factors include subgrade and climatic factors to account for the roughness caused by shrinking or swelling soils, and frost heave conditions. IRI is estimated incrementally over the entire design period. It should be noted that IRI is not a distress, and therefore it serves as an indicator of functional serviceability rather than a measure of structural integrity, albeit a strong correlation of IRI with some distresses has been found in the field. For instance, the regression coefficients in Figure 3.14 indicate rutting as a primary contributor to changes in longitudinal profiles over time (C1=40).

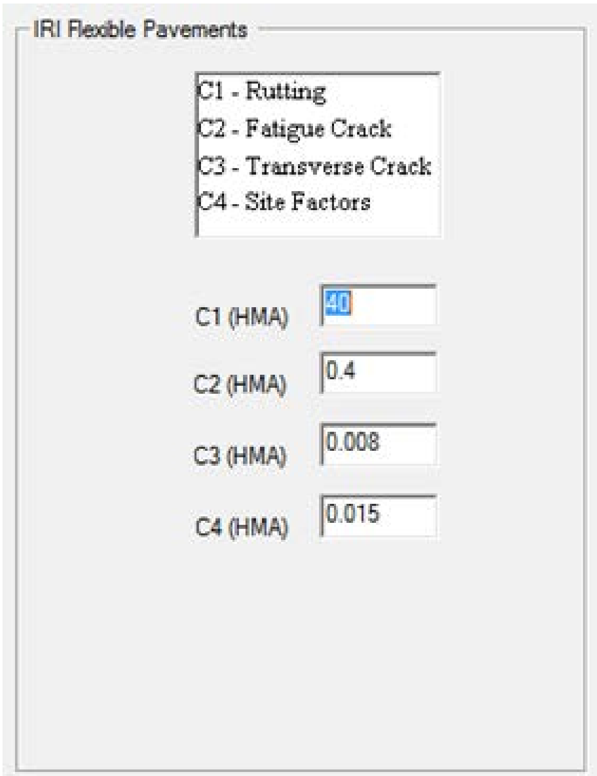

Figure 3.14. IRI Model for Flexible Pavements (MEPDG 1.100)

The independent panel of reviewers has recommended excluding IRI from the M-EPDG, arguing that IRI is a tool for pavement management and its inclusion contradicts mechanistic approaches to pavement design in the M-EPDG (NCHRP 2006). Nevertheless, the most current version of the M-EPDG software, as well as DARWin-ME^{TM} package, does include the nationally calibrated IRI model. Furthermore, ConnDOT has been intensively using automated profile measurements since as early as 1987. This historic data can be used to perform a local calibration of the IRI model. Therefore, the research team of this project included this parameter in the analysis of sensitivity.

CHAPTER 4 Identification of Typical Traffic Volumes, Pavement Features, and Site Conditions for Connecticut

One of the most important tasks in preparing the M-EPDG implementation plan for the Connecticut DOT was to identify the scope and range of design inputs to be evaluated in the sensitivity analysis of the M-EPDG software. In view of the vast number of inputs used by the distress prediction models, a decision had to be made on (1) what variables could be fixed to constant values, (2) what inputs actually varied around the state, and (3) what ranges of variable inputs should be used in the sensitivity analysis.

To facilitate the decision, the research team explored two venues. First, a panel meeting was arranged to interview pavement management and design personnel at the ConnDOT headquarters and obtain as much information as possible on typical pavement design configurations and pavement structures. Second, the ConnDOT pavement-related construction specifications and special provisions were thoroughly studied to complete an assessment of the traffic and material-related inputs' scope and range.

This chapter provides a summary of typical climatic conditions, pavement features, and material properties, whereas Appendix A includes detailed descriptions and ranges of inputs considered for the sensitivity analysis.

Typical Pavement Design in Connecticut

During the interview with ConnDOT pavement design and management personnel, three typical pavement designs were identified: (1) Newly constructed AC pavements, (2) AC-overlaid AC pavements, and (3) AC-overlaid rubbilized PCC pavements. It was recognized that the most common pavement maintenance/rehabilitation activity in Connecticut is a 2 inch overlay placed over existing pavement with or without preliminary milling of the existing surface. ConnDOT also acknowledged that AC-overlaid PCC or repaired PCC is also an important pavement design consideration.

Climatic Zones in Connecticut

According to the U.S. Department of Agriculture (USDA), Connecticut territory falls under four plant hardiness zones that differ in extreme minimum annual temperatures (See Figure 4.1). The southern coast is the warmest (zone 7A [-18 to -15 °C]), whereas the northwest and northeast hills are the coldest (zone 5B [-26 to -23 °C]). The rest of the state is rated either as 6A or 6B (-23 to -18 °C). After consultations with ConnDOT, the research team adopted three climatic zones and explored the climatic data available for use in M-EPDG.

Three climatic input files (Climate I, II, and II for shore, inland, and mountain zones respectively) were created by the interpolation of the temperature, precipitation, and wind data as shown in Table 4.1. Note that not enough data was available for weather stations (Poughkeepsie, NY, and Pittsfield, MA) in the vicinity of Litchfield County (Northwest CT). Therefore, the data from Worcester, MA, station were used for simulating Climate III (mountain). The elevation and groundwater table data were interpolated for each climate as well.

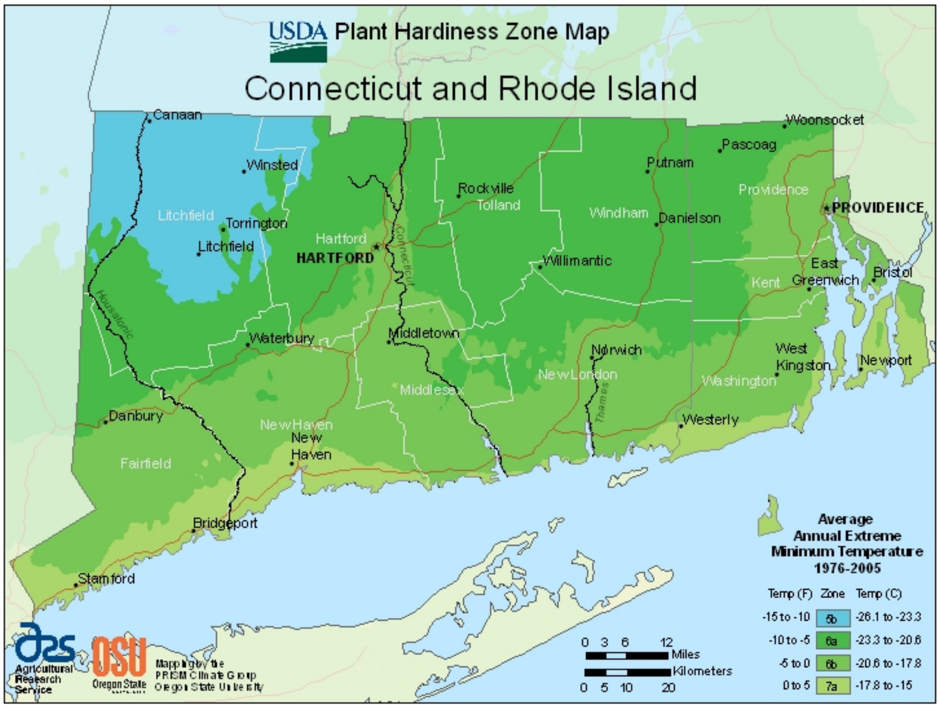

Figure 4.1. Connecticut Climatic Zones (http://planthardiness.ars.usda.gov/PHZMWeb/#)

Table 4.1. Summary of the M-EPDG climatic data

Climate ID	Climate Name	Weather Station Locations	Elevation [ft]	Depth of Groundwater Table [ft]
Climate I	SHORE	Bridgeport, CT New Haven, CT Groton, CT	11	20
Climate II	INLAND	Hartford, CT Willimantic, CT	18 247	20
Climate III	MOUNT	Worcester, MA	1,009	20

Main Traffic Variables

The traffic-related variables in the M-EPDG were chosen based on the typical functional classification and corresponding traffic levels prescribed for Superpave design in Connecticut (Table M.04.03-4). Accordingly, three levels of traffic (Level 3 High, Level 3 Medium, and Level 2) were utilized to establish the number of ESALs over the design life and calculating initial Average Annual Daily Truck Traffic (AADTT) values. In addition, a separate speed value was established for each traffic level. The annual truck traffic growth was fixed at 2 percent based on consultation with ConnDOT pavement design personnel. Table 4.2. summarizes the general traffic-related inputs, while Appendix A provides a full description of vehicle class distribution, hourly truck traffic distribution, axle configuration, and other required inputs.

Table 4.2. General traffic inputs

Highway Functional Class	**Traffic Level (**Table M.04.03-4	**Design Life ESALs [million]**	**Initial AADTT [trucks]**	**Speed [mph]**
Interstate HWY	Level 3 High	12.1	2500	70
Non-Interstate HWY	Level 3 Medium	4.8	1000	55
Local Arterial	Level 2	1.9	400	40

Subgrade Properties in Connecticut

The Research Team has explored the available literature to determine typical subgrade properties in Connecticut. Historically, three types have been identified as representative soils based on the percentage of aggregate passing the #10 and the #4 sieves, as shown in Table 4.3 (Long 1992). It should be noted that, although subgrade types per AASHTO classification do not vary significantly across the state (Malla 2006), considerable seasonal variations in resilient modulus values should be anticipated in Connecticut. Finally, the subgrade modulus values in the sensitivity analysis ranged between 10,000 and 20,000 psi (See Table 4.3)

Table 4.3. Subgrade properties

Subgrade ID	**Percent Passing #10 (Long1992)**	**Percent Passing #4 (Long1992)**	**AASHTO Class**	**Mix Dry Density**	**Resilient Modulus Range [psi] (NCHRP 2004)**	**Assigned Resilient Modulus [psi]**
Soil A	75	8.7	A-1-b	123.3	6,000 – 16,000	10,000
Soil B	62	8.8	A-1-b	126.5	8,000 – 20,000	15,000
Soil C	50	11.2	A-1-b	142.5	10,000 – 30,000	20,000

Typical Pavement Structures in Connecticut

During the interview with ConnDOT professionals and based on the information available elsewhere, typical values for such parameters as layer thickness, binder PG, and aggregate gradations, were identified for further use in the analysis of M-EPDG sensitivity design inputs. Table 4.4 shows major design parameters for 8-, 10-, and 12-in thick newly constructed asphalt pavements. Note that aggregate gradation and air voids for Superpave HMA mixes were obtained from Table M.04.03-3, whereas binder content was obtained for each mix type and traffic level from Table M.04.02-5 (ConnDOT Specifications, FORM 816, Division III, Section

M.04). Table 4.5 provides basic material inputs for the three types of granular base material considered in this project. More specific inputs related to pavement layer material properties can be found in Appendix A.

Table 4.4. Baseline pavement structures and mix properties

Design Parameter	Structure I (3+5+0)	Structure II (4+6+0)	Structure III (3+3+6)
HMA Layer Thicknesses [in]			
Surface HMA	3	4	3
Binder HMA	5	6	3
Base HMA	0	0	6
Asphalt Binder Inputs			
Surface AC Binder PG	64-22	64-22	64-22
Binder AC Binder PG	64-22	64-22	64-22
Base AC Binder PG	64-22	64-22	64-22
HMA Mix Properties[1]			
Surface AC Mix Type/ NMAS	S0.375	S0.375	S0.375
Binder AC Mix Type	S0.5	S0.5	S0.5
Base AC Mix Type	Granular Base A[2] + 2% PG 64-22		
Air Voids [percent]	4 (for all AC layers)		
Asphalt Binder Content[3] [percent]			
Surface AC	5.4-5.5	5.4-5.5	5.4-5.5[3]
Binder AC	4.8-4.9	4.8-4.9	4.8-4.9
Base AC	2	2	2

[1]See Table M.04.03-3 for gradation and volumetrics
[2]See Table 4.5 for granular base properties
[3]Depends on traffic level (See Table M.0.4.02-5)

Table 4.5. Basic granular base material properties (after CTDOT Section M.04)

Input	Grading A	Grading B	Grading C
Aggregate Gradation (Percent Passing Sieve)			
125 mm (5 in)	100	100	
90 mm (3.6 in)		90-100	
37.5 mm(1 ½ in)	55-100	55-95	100
19 mm(3/4 in)			15-80
6.3 mm(1/4 in)	25-60	25-40	25-60
4.15 mm (#4)	20-52	20-52	20-52
2 mm (#10)	15-45	15-45	15-45
0.425 mm (#40)	5-25	5-25	5-25
0.15 mm (#100)	0-10	0-10	0-10
0.075 mm (#200)	0-5	0-5	0-5
Plasticity Index	1	1	1
Assigned Modulus [psi]	30,000	25,000	20,000

For the AC-overlaid pavements (both AC over AC, and AC over PCC), a 3.5-in thick overlay was considered as a base case for sensitivity analysis. The material properties of the overlay varied as described in Appendix A. The underlying 10-in (4+6+0) thick existing AC pavement (before 2-in milling occurred) was considered for the AC over AC analysis. The AC over PCC pavement analysis required input of the resilient modulus for rubblized PCC slab (500,000 psi) and for the underlying slab thickness (9-in).

CHAPTER 5 Summary of the Sensitivity Analysis of M-EPDG Inputs

The M-EPDG software utilizes numerous inputs (more than 80 for flexible pavements) to characterize traffic, climate, site conditions, pavement features, and material properties. Those inputs do, however, have varying impacts on the predicted distress values. Furthermore, the same input can affect the trends of different distresses differently. One simple example is an AADTT, which has a tremendous effect on the fatigue distress values, but has no effect on thermal cracking in the asphalt layer. Therefore, one of the central tasks of this project was to conduct a sensitivity analysis of the M-EPDG inputs for the typical pavement designs in Connecticut.

Development of Testing Matrix

Based on the consultations with ConnDOT professionals and the previous experience of the research team, a set of eight basic design scenarios was developed to address variations in traffic volume, climatic conditions, and subgrade soil type for each pavement design, as shown in Table 5.1. Twenty four baseline M-EPDG runs were performed for each of the three typical pavement designs (New AC, overlaid AC, and overlaid PCC pavements) with a total of 72 baseline projects. The last available M-EPDG version 1.100 was used to predict pavement performance, which was expressed in terminal values of predicted distresses. Table 5.2 lists the performance indicators used for each pavement design type.

Table 5.1. Summary of basic design scenarios for sensitivity analysis

Scenario	Pavement Design Type	Climate Type	Subgrade	Traffic Level
1	New AC Overlaid AC Overlaid PCC	Climate I (Shore) Climate II (Inland) Climate III (Mountain)	Soil A	Level 2
2				Level 3 Medium
3				Level 3 High
4			Soil B	Level 2
5				Level 3 Medium
6				Level 3 High
7			Soil C	Level 2
8				Level 3 Medium

Table 5.2. Performance indicator models for sensitivity analysis

Design Type/ Pavement Type	Performance Indicator Model
– New AC – Overlaid PCC (AC over rubblized PCC)	– Longitudinal cracking (top-down fatigue) – Alligator cracking (bottom-up fatigue) – Thermal cracking – AC layer rutting – Total rutting – IRI
– Overlaid AC (AC over milled AC)	– Longitudinal cracking (top-down fatigue) – Alligator cracking (bottom-up fatigue) – Reflection Cracking – Thermal cracking – AC layer rutting – Total rutting – IRI

While creating M-EPDG design projects, the research team had to decide which inputs would be kept fixed and which would be varied, as listed in Appendix A. Although it was desirable to analyze the effect of all inputs on the predicted performance, due to budget and time constraints, it was not possible. For the same reason, it was not possible at the time to analyze the full factorial of the inputs and their interactions. Therefore, the team decided to vary the inputs or sets of inputs that are only relevant to ConnDOT design procedures. For example, only the AADTT variable was used in the analysis of the effect of traffic, while the effect of other variables, such as vehicle class and hourly truck traffic distribution, were kept fixed.

In the second step of creating the sensitivity test matrix, a "one-at-a-time" approach to sensitivity analysis was implemented for this project. Effectively, once the baseline values for variables were established, the sensitivity of each nonfixed input variable was estimated by changing the value of the variable, calculating the resulting pavement performance using the M-EPDG software, and then comparing the predicted pavement performance to the established baseline performance for the given design. The input values were changed from "Baseline" to "Low" and "High" as shown in Table 5.3. The full description of the 185 M-EPDG sensitivity runs can be found in Appendix C.

Sensitivity Analysis Approach

In general, a sensitivity analysis of a prediction model explores the magnitude of change in a model response, or outputs, relative to the magnitude of change in individual predictors, or inputs. Two venues of the analysis - qualitative and quantitative – were explored in this project. The qualitative approach involved plotting summary charts in a "stock" format where the outputs for low and high input values are connected by a vertical line to the "base" performance value, centered against the input name on the x-axis (See Figure 5.1). The labels correspond to the input values. Effectively, the longer the line, the greater the effect of a particular input on the output value. For example, the chart in Figure 5.1 indicates that the use of binder PG 64-22 instead of

PG 70-22 results in higher rutting, whereas binder PG76-22 yields lower rutting, if the rest of the variables are fixed. When comparing the length of vertical lines, one can reasonably conclude that the greatest influence on total rutting in new AC pavement is from AADTT , followed by pavement structure (STRUCT), and modulus of subgrade (ES). At the same time, base layer thickness (HBASE) and strength (EB) show very little effect as compared with other variables.

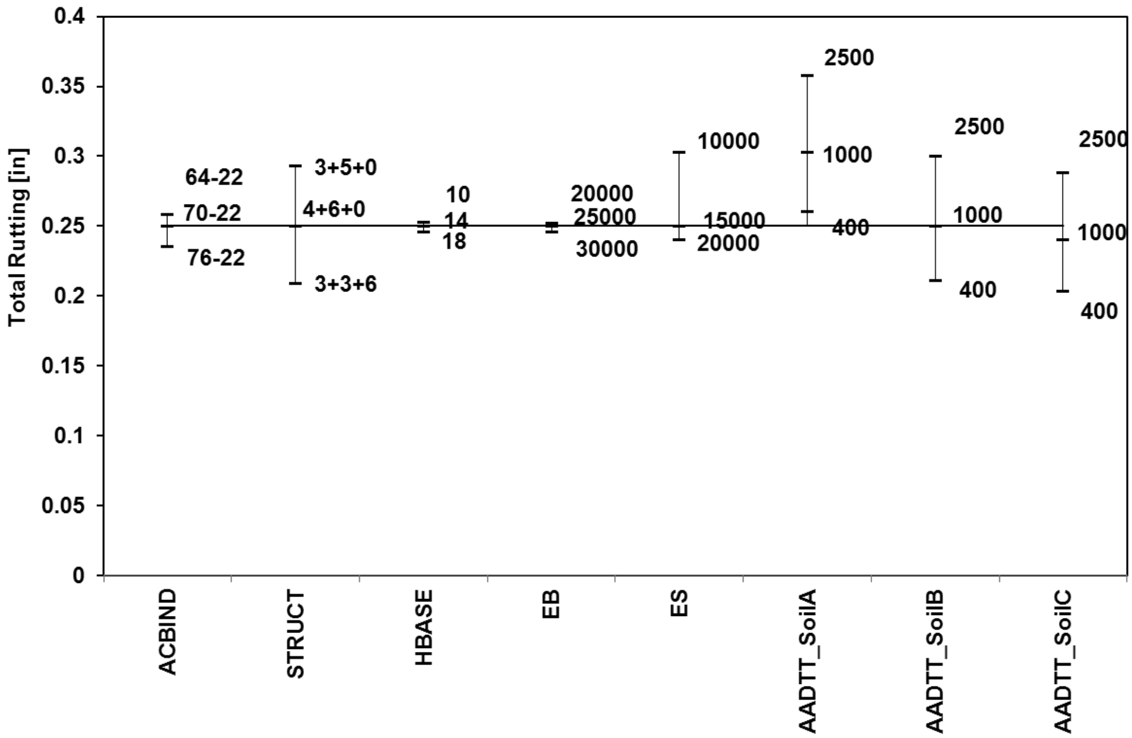

Figure 5.1. Example summary chart of relative sensitivity of total rutting model for new AC design in coastal climate.

One historic approach used by M-EPDG researchers to quantify the sensitivity employs numerical analysis of the ratio of the percentage of change in output over the percentage change in input (either direct or normalized values [Coree et al. 2005, Schwartz et al. 2007]). Another approach was initiated by Yut et al. (2006) and further adopted by Velasquez et al. (2009) in Minnesota and Hoerner et al. (2007) in South Dakota. This approach utilizes multiple analyses of variance (ANOVA) and it is believed to be more robust in terms of proving statistical significance of the effect of an individual factor on an output of the prediction model. Therefore, the ANOVA approach to sensitivity analysis was also used in this project.

In an ANOVA, the significance of an individual M-EPDG input is indicated by the magnitude of the calculated F-ratio associated with the input. Specifically, the F-ratio which is associated with a given M-EPDG input is computed using the following equation:

$$F = \frac{MSE_{input}}{MSE_{model}} \quad [5.1]$$

where:
F = F-ratio
MSE_{input} = Mean square error of the mean predicted distress output associated with the individual input under question
MSE_{model} = Mean square error of the mean predicted distress output when all the inputs are in the model
In other words, the F-ratio measures variation in the output caused by the variation in the individual input being investigated and thus, the higher the F-ratio, the greater the effect of that input on the model output. A statistical significance of such an effect is evaluated by a p-value, or level of confidence. Due to a relatively small sample size in this study, a level of confidence $\alpha=0.05$ was selected. Effectively, an input is ranked as being of low importance if the p-value for its F-ratio is statistically insignificant (smaller than 0.05).

Table 5.3 presents an example of an ANOVA analysis for total rutting sensitivity to inputs depicted in Figure 5.1. Note that statistically significant inputs in Table 5.1 can be distinguished by the order of magnitude of their F-ratios. Further, one can reasonably assume that, although AADTT is higher in rank than SUBGRADE, which is followed by STRUCT, all three inputs may be equally important because they represent independent input categories (traffic, site condition, and pavement thickness). Therefore, it was decided to use logF to assess the importance of the inputs. Where LogF is less than 0.5 it is low importance; LogF = 0.5 to 1 is moderate importance; LogF= from greater than 1 to 3 is high importance; and, LogF greater than 3 is critical importance. Figure 5.2 illustrates the concept where log(F=3.16)=0.5 and log(F=10)=1 separate low-, moderate-, and high-importance categories of inputs exist. The inputs with logF greater than 3 are considered critical in the analysis.

Table 5.3. Example of input significance for the total rutting model for new AC pavements

Order No.	Predictor Index	Predictor Name	F	p-value	logF	Statistical Significance	Assigned Importance
1	**AADTT**	Initial average annual daily truck traffic	549.3	0	2.74	Yes	High
2	**SUBGRADE**	Subgrade Modulus	292.96	0	2.47	Yes	High
3	**STRUCT**	Pavement Structure	159.27	0	2.20	Yes	High
4	**CLIMATE**	Climate (Location)	49.82	0	1.70	Yes	High
5	**ACBIND**	AC binder PG	14.49	0	1.16	Yes	High
6	**HBASE**	Base thickness	1.39	0.2645	0.14	No	Low
7	**BASE**	Base Modulus	1.38	0.2674	0.14	No	Low

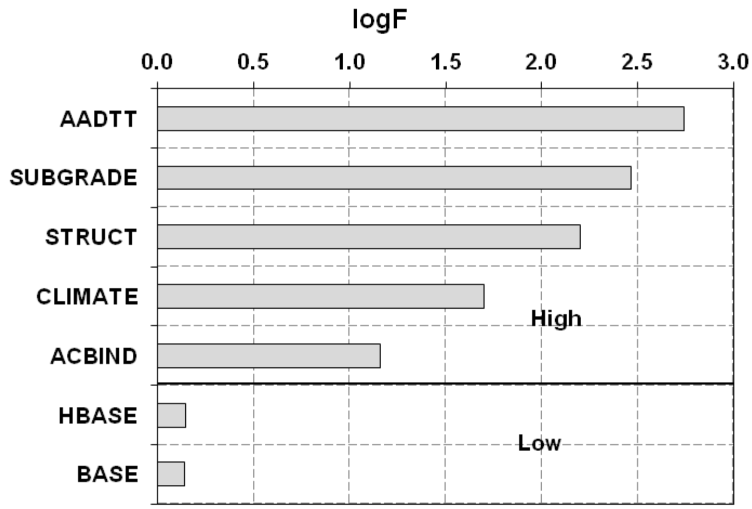

Figure 5.2. Illustration of importance ranking for total rutting model for New AC.

Table 5.4. One-at-a-time testing matrix for sensitivity analysis (Climate II, Traffic Level 3 Medium, Soil B)

Input Index	Subbase/ Base Thick, [in]	Subbase/ Base Type*	Pavement Structure *	Surface AC/ Overlay AC Binder PG	Overlay AC Thick. [in]	Overlay AC mix type [NMAS, in]	Mill Thick. [in]	Existing Pavement Rating	Existing Total Rutting	Rubblized PCC Resilient Modulus [psi]	Existing PCC Thick. [in]
	New AC Inputs				N/A	N/A	N/A	N/A	N/A	N/A	N/A
Low	10	Grading A	3+5+0	64-22							
Base	14	Grading B	4+6+0	70-22							
High	18	Grading C	3+3+6	76-22							
	Overlaid AC Inputs									N/A	N/A
Low	10	Grading A	4+6 (Fixed)	64-22	2	S0.375	1	Poor	0		
Base	14	Grading B		70-22	3.5	S0.5	2	Fair	0.5		
High	18	Grading C		76-22	5	S1	4	Good	1		
	Overlaid PCC Inputs										
Low	10	Grading A	N/A	64-22	2	S0.375	N/A	N/A	N/A	200,000	8
Base	14	Grading B		70-22	3.5	S0.5				500,000	9
High	18	Grading C		76-22	5	S1				100,0000	10

Sensitivity Results for New AC Pavement Design

The sensitivity analysis for new AC design targeted the effect of the design inputs on the variability in outputs for the following prediction models:

- Longitudinal cracking (top-down fatigue)
- Alligator cracking (bottom-up fatigue)
- Thermal cracking
- AC layer rutting
- Total rutting
- IRI

A total of 47 M-EPDG runs for a 20-year design life were conducted to analyze the effect of climate, truck traffic volume, subgrade, and other design parameters (as indicated in Table 5.4). Table A.8 summarizes fixed inputs for the new AC design, whereas the relative effect and ANOVA results are discussed separately for each performance indicator.

Analysis of the Longitudinal Cracking Model for New AC Design

The relative effect of the investigated inputs on the predicted longitudinal cracking after 20 years of service is shown in Figures 5.3 and 5.4. Note that for the base design, M-EPDG predicted an identical value of 0.45 ft/mi for the shore and inland climates (Climate I and II, respectively) and a half as high value of 0.2 ft/mi for the northwest and northeast hills for Connecticut (Climate III). While both values are very close to zero, one should recall that at 84 percent reliability, the predicted total longitudinal crack length would reach 257 ft/mi based on the standard deviation model shown in Figure 3.3.

The relative effect charts in Figures 5.3 and 5.4 indicate the following predictive trends:

- **Effect of climate:** Slightly lower top-down fatigue is expected in colder Climate III.

- **Effect of traffic:** While in general, low to medium truck traffic volume would not significantly affect the longitudinal cracking growth, it can clearly be seen that high AADTT volumes of 2500 trucks per day do have more of an effect than the other parameters. Furthermore, such an effect increases growth exponentially with an increase in subgrade strength from Soil A (10,000 psi) to Soil C (20,000 psi).

- **Effect of subgrade soil:** It appears that the increase in soil modulus may result in a visible increase in top-down fatigue. This can be explained by larger tensile strains on the asphalt surface caused by stiffer support conditions (NCHRP 2004); however, this seems to be relatively counterintuitive and contradictory to common practice.

- **Effect of asphalt layer inputs:** The relative charts clearly indicate the structure of asphalt layers as the major factor in development of top-down fatigue cracking with a substantial increase in cracking with a 1-in reduction in thickness of both surface and binder courses (3+5 versus 4+6 structure). Also, it can be observed that binder PG yields no effect of longitudinal cracking. Previous studies assumed that 4-in and thinner asphalt layers suffer primarily from bottom-up fatigue cracking due to traffic loads. This type of fatigue is predicted by a separate model for alligator cracking (Hoerner et al. 2007).

- **Effect of granular base inputs:** As expected, neither base thickness nor base modulus show any effect on the predicted longitudinal cracking values.

The ANOVA results in Table 5.5 mainly agree with the observations from the relative effect charts as they show the combination of layer thicknesses in pavement structure (STRUCT) as a sole important factor. Recall that for the new AC design, the asphalt material properties, such as aggregate gradation, air voids, and binder content were fixed at typical Connecticut values (see Table 4.4).

Table 5.5. ANOVA of inputs for the longitudinal cracking model new AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	STRUCT	224.18	0	2.35	Yes	High
2	CLIMATE	1.55	0.2276	0.19	No	Low
3	SUBGRADE	0.79	0.4612	-0.10	No	Low
4	AADTT	0.23	0.7927	-0.64	No	Low
5	ACBIND	0.01	0.9965	-2.00	No	Low
6	BASE	0.01	0.9972	-2.00	No	Low
7	HBASE	0.01	0.998	-2.00	No	Low

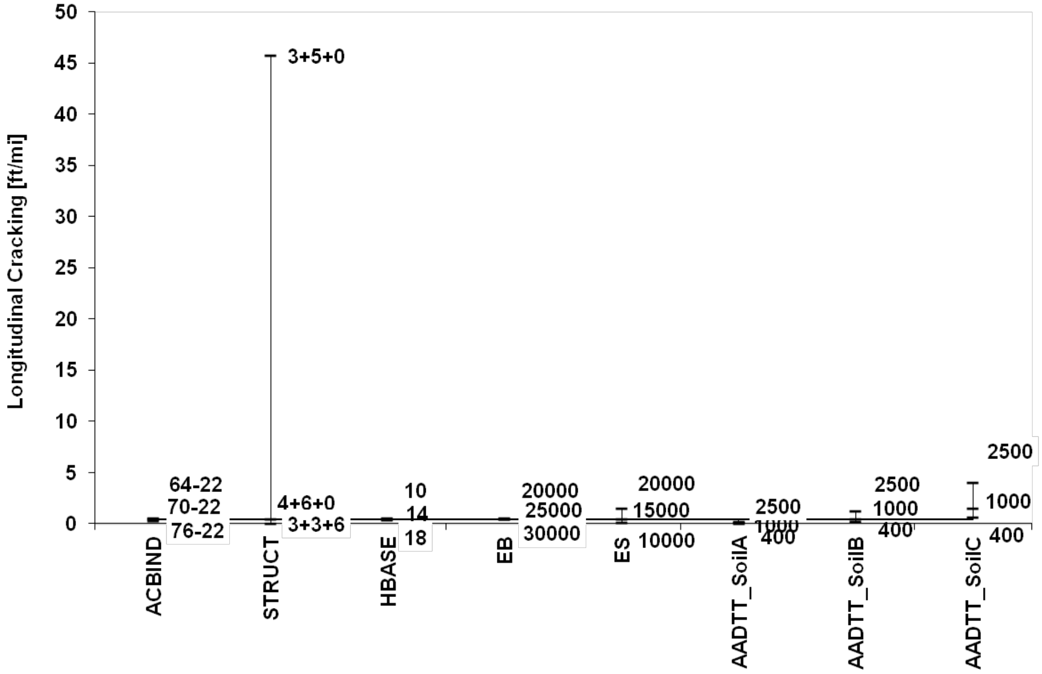

Figure 5.3. Relative effect of variables on longitudinal cracking in New AC design located in SHORE and INLAND climate

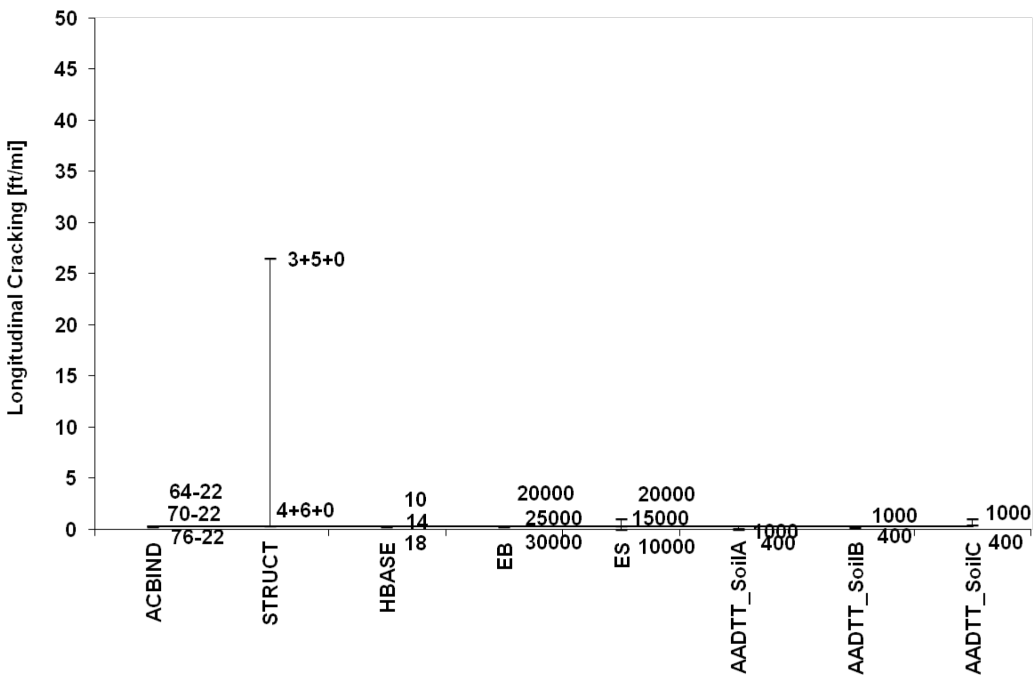

Figure 5.4. Relative effect of variables on longitudinal cracking in New AC design located in MOUNT climate

Analysis of Fatigue Cracking Model for New AC Design

Figures 5.5 and 5.6 show relative effect charts for the fatigue cracking model. The important observations from those charts are following.

Effect of climate: The M-EPDG model predicted very similar levels of bottom-up fatigue cracking for all climates (0.12 and 0.11 percent area covered for SHORE/INLAND and MOUNT climates correspondingly) when the baseline design is considered. It is also notable that virtually no fatigue cracking is predicted for the thinnest AC structure (0.5 percent area for 3”+5”+0”) at medium AADTT of 1000 trucks as well as for the highest AADTT of 2500 trucks using the baseline pavement structure (4”+6”+0”). Lastly, it appears that the effect of base thickness in the MOUNT climate is reduced as compared with that of SHORE/INLAND climates, which may be attributed to the difference in moisture distribution through the base thickness.

Effect of traffic: Obviously, traffic volume has a significant impact on bottom-up fatigue cracking, which grows exponentially with an increase in AADTT. Once again, similarly to the top-down fatigue, an interaction can be observed between AADTT and subgrade type.

Effect of subgrade soil: As expected, weaker subgrade support (lower modulus [ES]) results in visibly higher levels of fatigue. Nevertheless, this effect is significantly lower than that of traffic (AADTT) and AC structure (STRUCT).

Effect of AC layer inputs: Undoubtedly, the reduced total thickness of asphalt pavement is the major contributor to the increase in fatigue cracking while the other variables discussed in here are kept at fixed values. A higher cracking value for the 3”+3”+6” structure as compared with that for 4”+6”+0” can be neglected here, since all the values are very close to zero. A negligible effect of the binder PG (ACBIND) is explained by the fact that fatigue is more controlled by the low-temperature PG (-22 for all binders), while the high-temperature PG primarily controls permanent deformation.

Effect of granular base inputs: The effect of base thickness is visible, although very small, and apparently due to a relatively thick AC structure. Also, it appears that in the MOUNT climate, the effect of base strength is visibly higher than that in the SHORE/INLAND climate.

The suggested ranking of significance in Table 5.6 is based on the level of the F-statistic calculated from the ANOVA analysis. It assigns high importance to STRUCT, AADTT, ES, while recognizing the moderate importance of CLIMATE.

Table 5.6. ANOVA of inputs for the fatigue cracking model in new AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	STRUCT	494.43	0	2.69	Yes	High
2	AADTT	440.66	0	2.64	Yes	High
3	SUBGRADE	21.6	0	1.33	Yes	High
4	CLIMATE	3.05	0.0612	0.48	Yes	Low/Moderate
5	BASE	2.13	0.1354	0.33	No	Low
6	HBASE	0.31	0.7331	-0.51	No	Low
7	ACBIND	0.17	0.8411	-0.77	No	Low

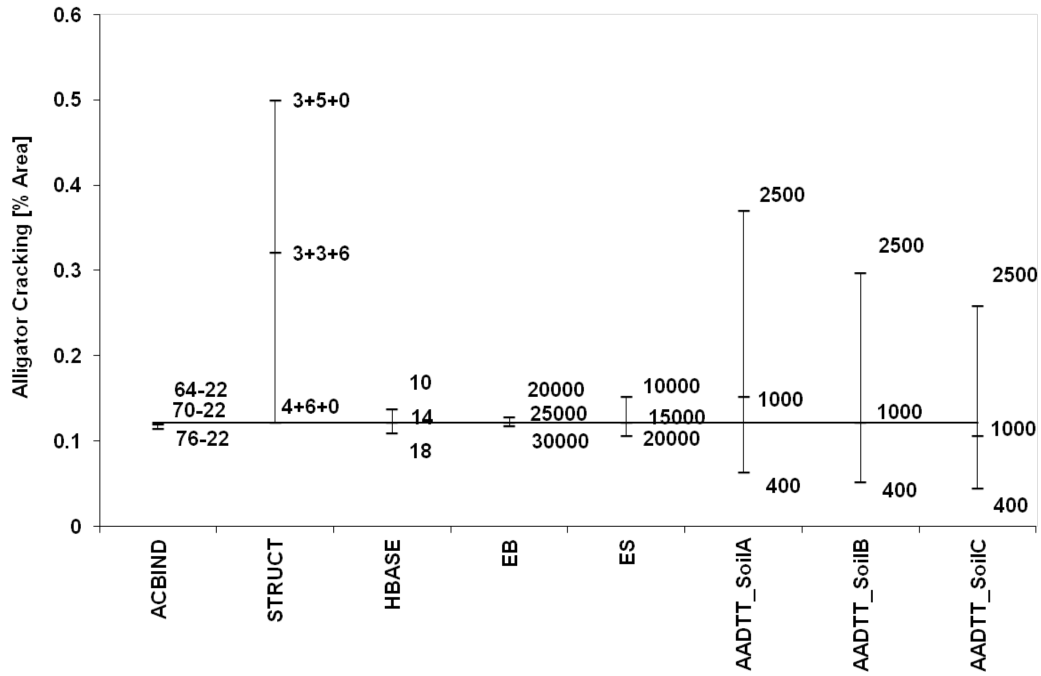

Figure 5.5. Relative effect of variables on fatigue cracking in New AC design located in SHORE and INLAND climate

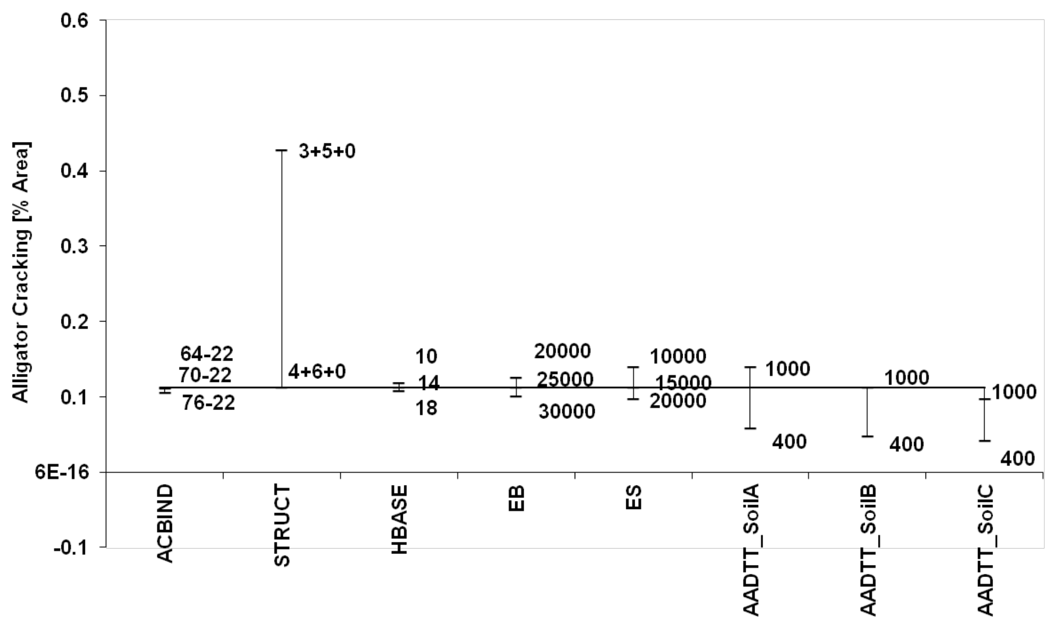

Figure 5.6. Relative effect of variables on fatigue cracking in New AC design located in MOUNT climate

Analysis of Thermal (Transverse) Cracking Model for New AC Design

Previous studies indicated that the thermal cracking model in the preliminary versions of the M-EPDG software (0.914 and earlier) was often crashing or yielding unreasonable results (Hoerner et al. 2007, Velasquez et al. 2009). However, the M-EPDG version 1.100 evaluated in this project appeared bug-free, at least for the range of inputs used for the sensitivity analysis. Therefore, the research team decided to include this model in the discussion.

Effect of climate: It is notable that the M-EPDG simulations for the coastal and low inland areas (SHORE and INLAND climates) predicted virtually no thermal distress for any combination of the inputs (Figure 5.7). On the other hand, presumably colder MOUNT climates representing northwest and northeast hills in Connecticut yielded noticeable yet not critical amounts of transverse cracking (313ft/mi) for the baseline design (Figure 5.8).

Effect of subgrade and base: Since thermal cracking is modeled as a response to a temperature gradient that initiates from the top of the pavement structure, it is not expected to propagate to the well-protected unbound layers (subgrade and base). Indeed, even for a colder MOUNT climate, the M-EPDG shows a very small yet visible effect of base modulus (EB) with no effect of base thickness (HBASE).

Effect of AC layer inputs: As discussed in Chapter 3, the main predictors in the thermal cracking model are asphalt thickness and tensile properties of binder and HMA. It is no surprise then that STRUCT and ACBIND show great relative effect on the outcome even at the zero baseline value for milder climates. As expected, the thicker structure along with higher tensile properties from the selected PG binders better withstand the thermal gradient. Nevertheless, it is strongly recommended that thermal models be carefully calibrated before use in design.

Effect of traffic: No effect of traffic volume is shown since only environmental loading is considered in the thermal cracking prediction.

The ANOVA results shown in Table 5.7 support the observations of the relative effect with the exception of the STRUCT variable. Apparently, the variation in total asphalt thickness (8 to 12 in) is not sufficient to affect the variation in thermal cracking as compared with CLIMATE and ACBIND variable.

Table 5.7. ANOVA of inputs for the thermal cracking model in new AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	CLIMATE	198.04	0	2.30	Yes	High
2	ACOLBIND	6.25	0.0051	0.80	Yes	Moderate
3	STRUCT	1.13	0.3351	0.05	Yes	Low
4	AADTT	0.01	0.9964	-2.00	Yes	Low
5	BASE	0.01	0.9987	-2.00	No	Low
6	HBASE	0.01	1	-2.00	No	Low
7	SUBGRADE	0.01	1	-2.00	No	Low

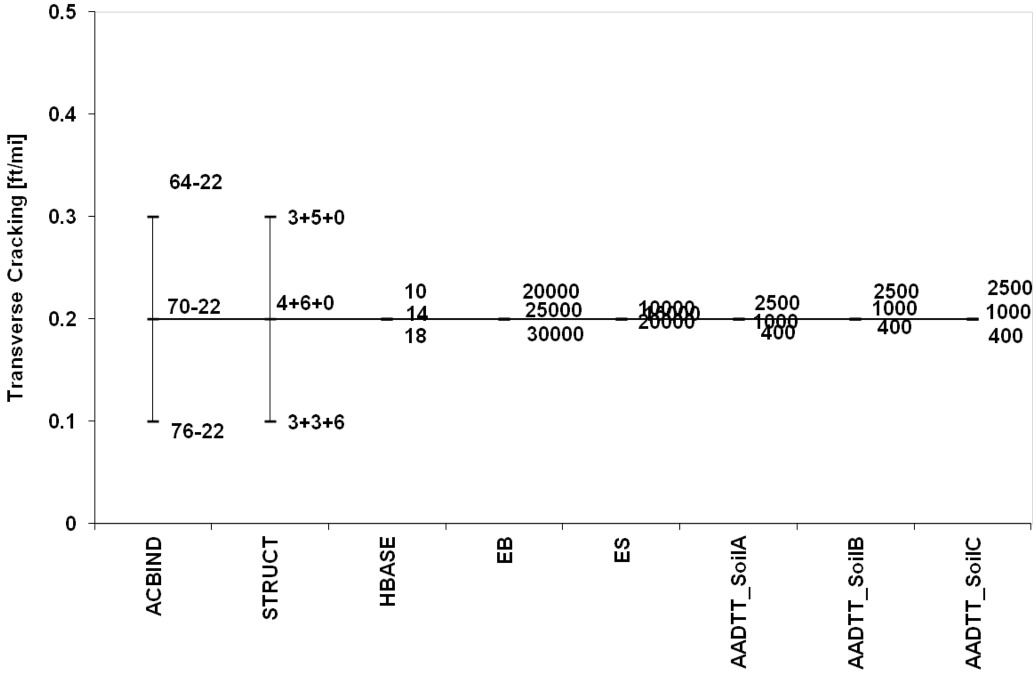

Figure 5.7. Relative effect of variables on thermal cracking in New AC design located in SHORE and INLAND climate

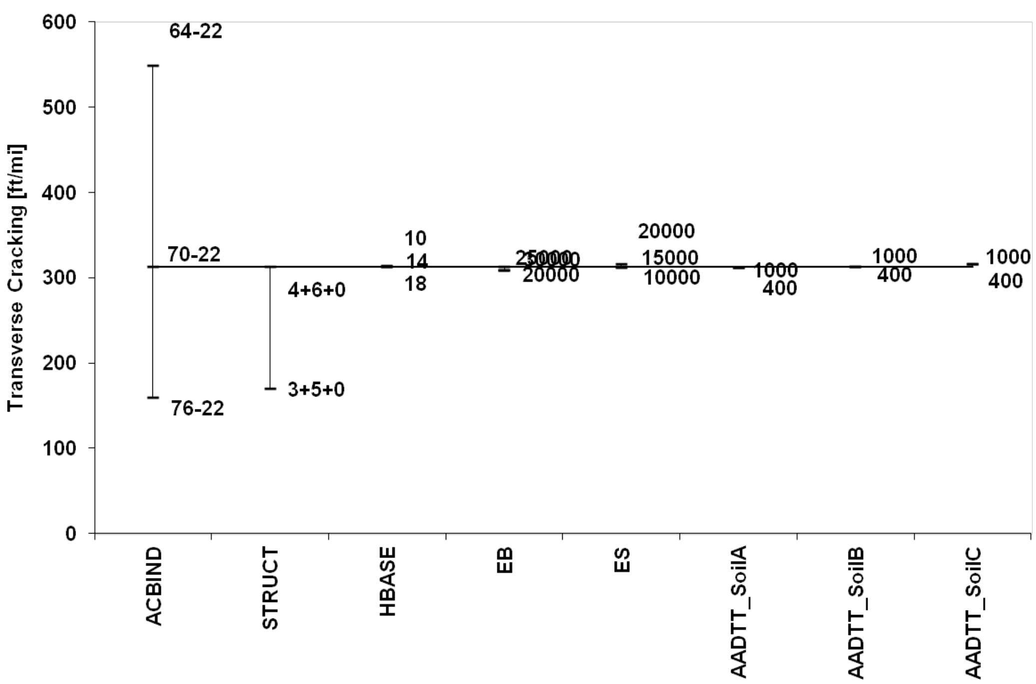

Figure 5.8. Relative effect of variables on thermal cracking in New AC design located in MOUNT climate

Analysis of AC Rutting Model for New AC design

Figures 5.9 and 5.10 show the relative effects of the traffic and pavement structure-related inputs on the output of the M-EPDG prediction model for AC rutting. Those effects are discussed next, followed by the summary of the statistical significance of the inputs.

Effect of climate: In general, a barely visible AC rutting level was predicted by the baseline M-EPDG runs for all three climates, yet with lower rutting for MOUNT climate (0.06 inches as compared with 0.07 inches for SHORE and INLAND), which would be anticipated at lower average temperatures.

Effect of traffic: For all climates, the truck traffic volume (AADTT) appears to be the most influencing factor, but to a slightly lesser degree for a colder MOUNT climate.

Effect of subgrade soil: Although the subgrade stiffness (ES) yields very small effects on the M-EPDG rutting prediction for the AC layer, as expected, a stiffer subgrade may result in higher shear strain for the asphalt layer, which is reflected in slightly higher permanent deformation. Once again, the observations in the field may contradict the outcome predicted by the M-EPDG.

Effect of AC layer inputs: As expected, the binder PG and asphalt layer structure inputs (STRUCT) are the primary material-related contributors to the terminal AC rutting level in newly constructed HMA pavements.

Effect of granular base input: Although counterintuitive, the stiffer granular base may result in slightly higher rutting outputs in the AC layer, provided the rest of the inputs are kept fixed. In addition, the increase in base thickness barely reduced AC rutting.

It should be noted that despite the very small absolute values of AC rutting, the relative change in the output appears statistically significant for all inputs in Table 5.8, with the exception of base strength (BASE) and thickness (HBASE). Although rutting in Connecticut has mostly been observed on interstate highways, the calibration of the rutting model may result in a slightly different order of importance depending on local range of inputs.

Table 5.8. ANOVA of inputs for the AC rutting model in new AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	AADTT	1865.52	0	3.27	Yes	High
2	CLIMATE	132.69	0	2.12	Yes	High
3	ACBIND	65.65	0	1.82	Yes	High
4	STRUCT	62.97	0	1.80	Yes	High
5	SUBGRADE	6.72	0.0037	0.83	Yes	Moderate
6	BASE	2.19	0.1284	0.34	Yes	Low
7	HBASE	0.31	0.7335	-0.51	No	Low

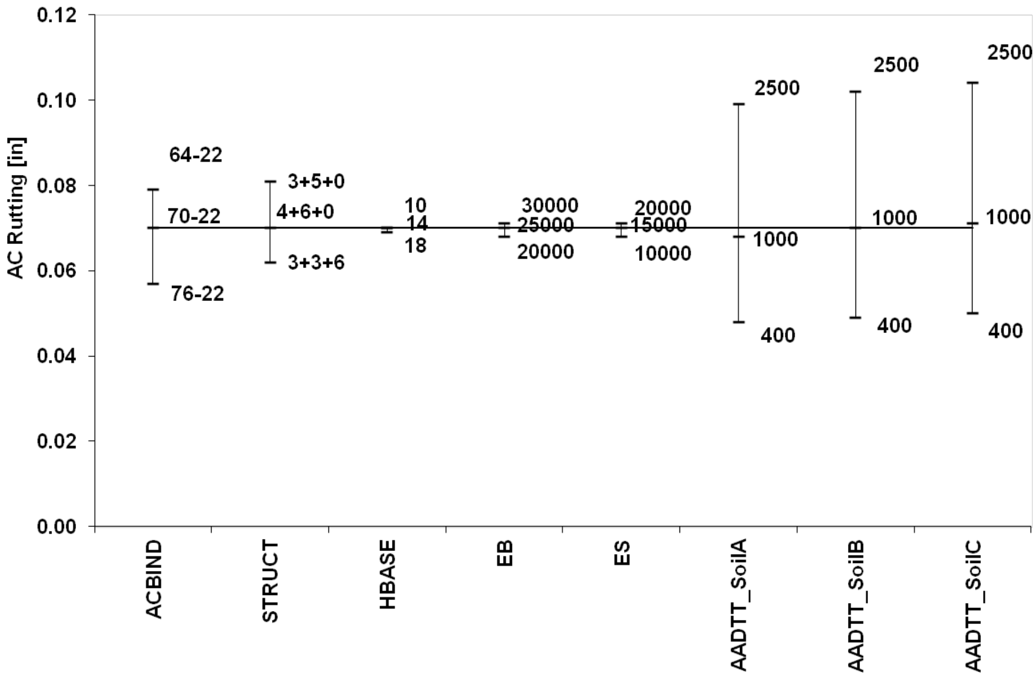

Figure 5.9. Relative effect of variables on AC rutting in New AC design located in SHORE and INLAND climate

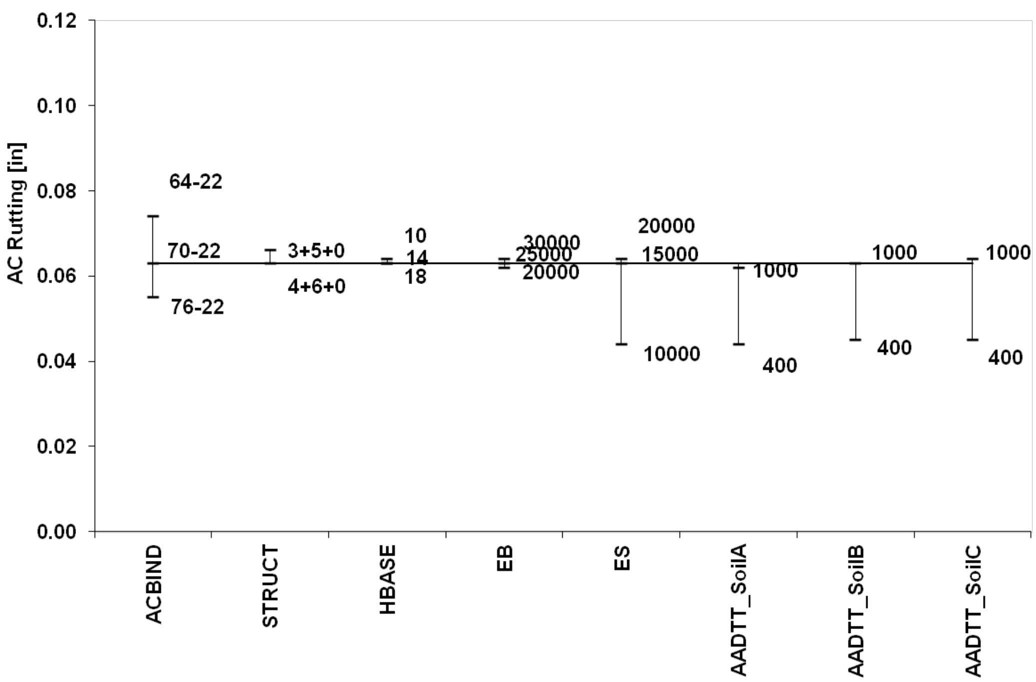

Figure 5.10. Relative effect of variables on AC rutting in New AC design located in MOUNT climate

Analysis of the Total Rutting Model for New AC design

As discussed in Chapter 3, the M-EPDG algorithm predicts total rutting as a sum of permanent deformations contributed by the AC, unbound granular layers, and the subgrade. The relative effect charts in Figures 5.10 and 5.11 indicate generally low total rutting values (maximum 0.36 in) as compared with the default rutting failure threshold of 0.75 in. Nevertheless, no effect of any individual variable can be neglected, as discussed next.

Effect of climate: The M-EPDG baseline predictions for the total rutting yielded very small yet consistent differences between coastal/inland climates (0.250 in) and the mountain climate (0.272 in). Indeed, all sensitivity runs for the MOUNT climate yielded terminal total rutting values averaging 0.022-in.higher than the corresponding runs for the other climates.

Effect of traffic: Obviously, for the given range of inputs, the truck traffic volume has shown the greatest influence on the total rutting.

Effect of subgrade: According to the relative effect charts, weaker subgrade results in larger rutting, especially for soils with a modulus lower than 15,000 psi. Note that the decrease in modulus from 15,000 psi to 10,000 psi results in as much damage as an increase in traffic from 1,000 to 2,500 AADTT.

Effect of AC layer inputs: Both binder PG and AC layer thickness contributed to the AC-related portion of the total rutting, yet to a lesser degree than traffic and subgrade. It is shown that an increase in high-temperature PG from 70 to 76 results in a decrease in rutting.

Effect of granular base inputs: Neither base thickness nor its strength show any visible effect on total rutting for the given range of inputs (10-18 in thick base with modulus of 20,000 to 30,000 psi).

The ANOVA results shown in Table 5.9 mostly agree with the observations from the relative effect charts in Figures 5.10 and 5.11. Note that the CLIMATE variable yields a statistically significant F-ratio higher than 1 and, therefore, should be considered highly important.

Table 5.9. ANOVA of inputs for the total rutting model in new AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	AADTT	549.3	0	2.74	Yes	High
2	SUBGRADE	292.96	0	2.47	Yes	High
3	STRUCT	159.27	0	2.20	Yes	High
4	CLIMATE	49.82	0	1.70	Yes	High
5	ACBIND	14.49	0	1.16	Yes	High
6	HBASE	1.39	0.2645	0.14	No	Low
7	BASE	1.38	0.2674	0.14	No	Low

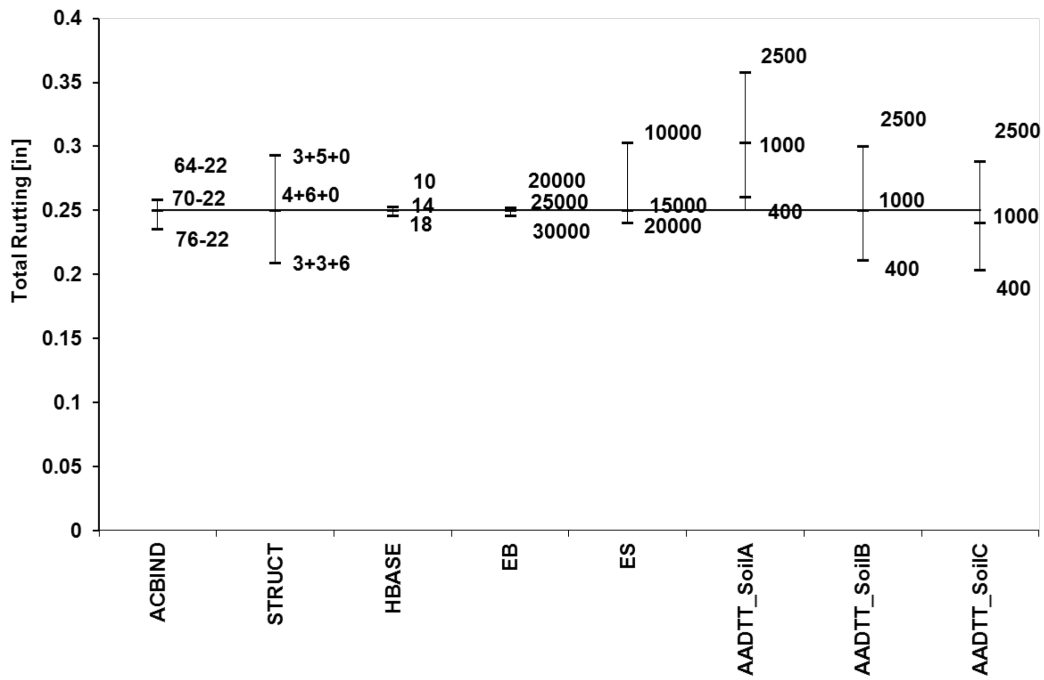

Figure 5.11. Relative effect of variables on total rutting in New AC design located in SHORE and INLAND climate

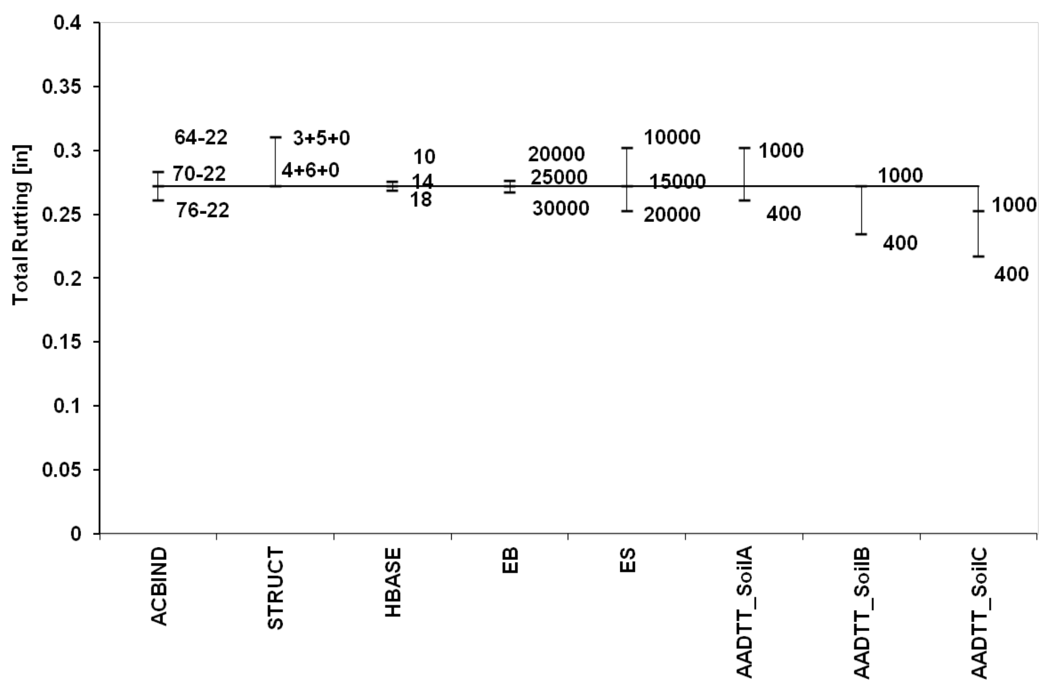

Figure 5.12. Relative effect of variables on total rutting in New AC design located in MOUNT climate

Analysis of IRI Model for New AC design

The IRI prediction model in the M-EPDG utilizes total rutting and fatigue as major predictors along with thermal cracking and site factors as complementary predictors. Therefore, one should expect that the inputs influencing the incorporated distresses will also affect the IRI output. Therefore, the sensitivity of IRI predictions was evaluated for the same set of inputs as for the other performance indicators. In knowing that IRI is routinely used by ConnDOT as a performance measure, the calibration of the IRI model may be of a particular interest in the process of the implementation of the M-EPDG in Connecticut.

From the relative effect charts in Figures 5.13 and 5.14, the following conclusions can be drawn:

- In the colder MOUNT climate, a new bottom-up constructed AC pavement is expected to have slightly higher roughness (103.3 in/mi) at the end of its service life as compared with SHORE and INLAND climatic zones (99.1 in/mi).

- The climate (CLIMATE) and traffic volume (AADTT) appear to influence the IRI output the most. The subgrade support (SUBGRADE), and asphalt layer thickness (STRUCT) are following in that order in terms of their effect on IRI (Table 5.10).

- Binder PG input (ACOLBIND) has a statistically significant effect on IRI, apparently due to its significant contribution to AC rutting and thermal cracking.

- For the given range of granular base thicknesses (HBASE) and moduli (EB), there is no evidence of the effect of base-related inputs on IRI.

Table 5.10. ANOVA of inputs for the IRI model in new AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	CLIMATE	364.37	0	2.56	Yes	High
2	AADTT	133.96	0	2.13	Yes	High
3	SUBGRADE	78.56	0	1.90	Yes	High
4	STRUCT	31.34	0	1.50	Yes	High
5	ACOLBIND	15.1	0	1.18	Yes	High
6	BASE	0.46	0.6368	-0.34	No	Low
7	HBASE	0.38	0.6839	-0.42	No	Low

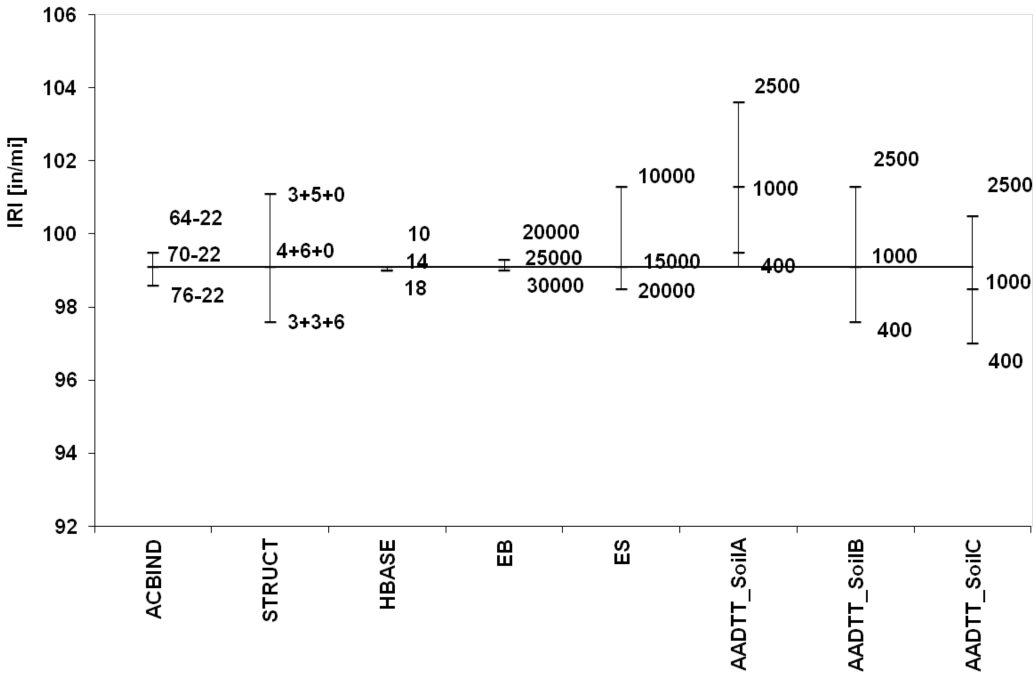

Figure 5.13. Relative effect of variables on IRI in New AC design located in SHORE and INLAND climate

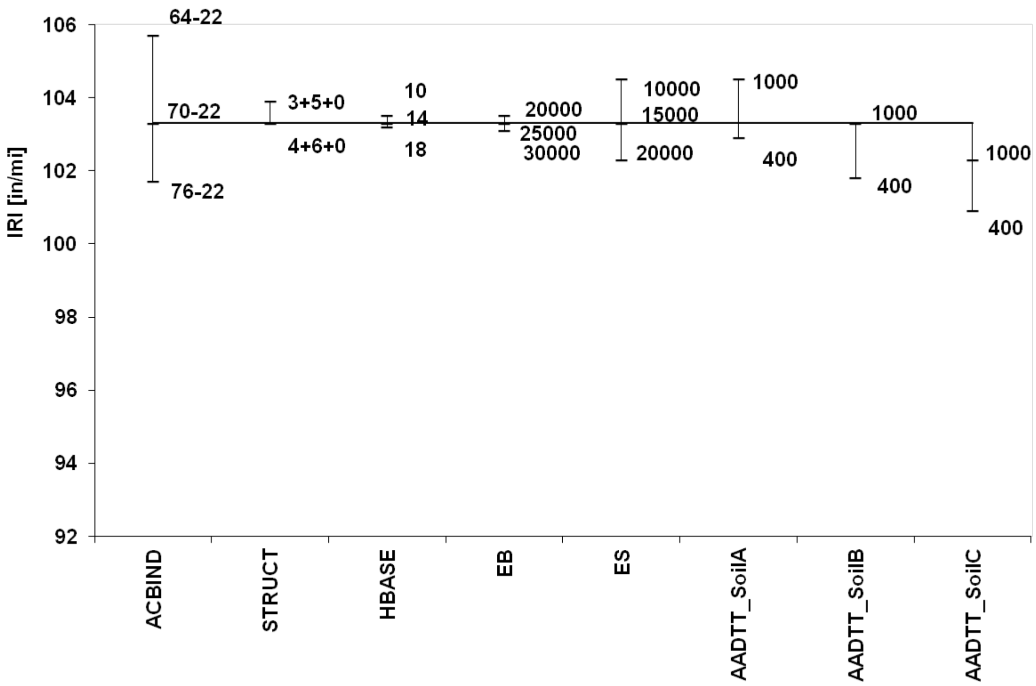

Figure 5.14. Relative effect of variables on IRI in New AC design located in MOUNT climate

Summary of the New AC Design Sensitivity to Inputs

As shown above, the distress prediction models for the new AC design showed high sensitivity to some inputs and low sensitivity to others. Although it is important to understand the degree of influence of an input on the predicted value of a particular distress, the optimal pavement design calls for addressing more than one performance indicator. Therefore, the research team evaluated a combined effect of each input on a group of distresses. For that purpose, a ranking parameter logF was averaged over all types of cracking (i.e. longitudinal, alligator, and thermal) to evaluate overall cracking ranking of inputs, while logF of AC rutting and total rutting were also averaged to evaluate the overall effect ranking of inputs on rutting. Table 5.11 summarizes mean logF values and the individual importance rankings for cracking, rutting, and IRI. Those rankings are illustrated in the sensitivity summary charts shown in Figures 5.15 through 5.17.

Table 5.11. Summary of the combined sensitivity ranking of new AC design inputs

Cracking Ranking			**Rutting Ranking**			**IRI ranking**		
Predictor	Mean logF	Importance	Predictor	Mean logF	Importance	Predictor	Mean logF	Importance
STRUCT	1.70	High	AADTT	3.01	High	CLIMATE	2.56	High
CLIMATE	0.99	Moderate	STRUCT	2.00	High	AADTT	2.13	High
AADTT	0.00	Low	CLIMATE	1.91	High	SUBGR.	1.90	High
SUBGR.	-0.26	Low	SUBGR.	1.65	High	STRUCT	1.50	High
ACBIND	-0.66	Low	ACBIND	1.49	High	ACBIND	1.18	High
BASE	-1.22	Low	BASE	0.24	Low	BASE	-0.34	Low
HBASE	-1.50	Low	HBASE	-0.18	Low	HBASE	-0.42	Low

In summary, the following should be noted as far as an optimal design of a new AC pavement in M-EPDG environment is considered:

- The location (climate) of a newly constructed AC pavement has very high influence on all performance indicators.
- The parameters of the AC layer structure, such as thickness and volumetric properties of the HMA mix appear to govern the pavement performance the most in a specified location.
- For a specified functional road class in Connecticut, the truck traffic volume appears to have more effect on rutting than it does on cracking. Note that only longitudinal and thermal cracking, both being non-load related, were predicted at noticeable levels for all new AC designs.
- The binder performance grade and subgrade support show high influence on rutting and roughness in terms of IRI.
- Granular base-related inputs did not yield any significant effect on pavement performance, most likely due to the relatively high modulus required by ConnDOT specifications, and large pavement thicknesses considered in the sensitivity analysis.

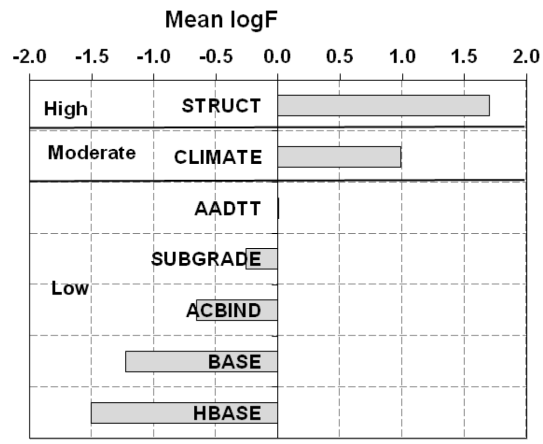

Figure 5.15. Cracking sensitivity to the new AC design inputs

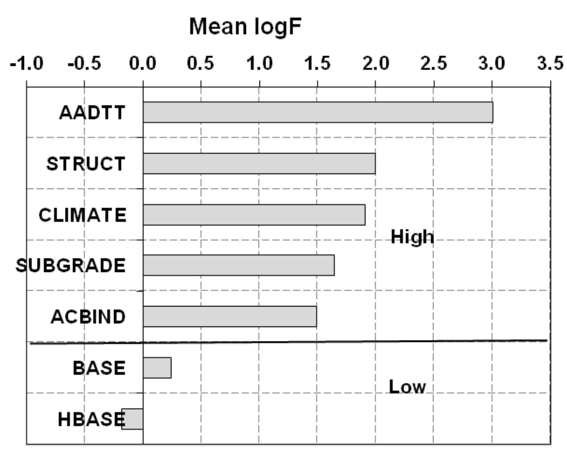

Figure 5.16. Rutting sensitivity to the new AC design inputs

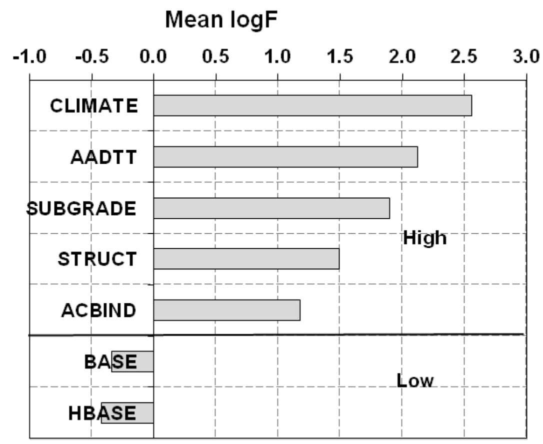

Figure 5.17. IRI sensitivity to the new AC design inputs

Sensitivity Results for AC-Overlaid AC Pavement Design

Asphalt overlays are the mainstay of pavement maintenance/rehabilitation activity in Connecticut. Therefore, the importance of the analysis of the AC-overlaid AC pavements cannot be over stated. As expected, additional inputs related to the existing/rehabilitated pavement condition and milling parameters are required by the M-EPDG for this type of design (see Table A.6). Also, for this design, not only overlay thickness but also the volumetric properties of the overlay mix were varied, while the existing pavement structure (3”+5”+0”) was kept constant. This was done because the output for all distress models appeared to have no sensitivity to thickness of the structures (3”+3”+6” and 4”+6”+0”). Effectively, a total of 72 AC-overlaid AC pavement designs were simulated to evaluate the sensitivity of the following prediction models to the variation of the inputs described in Table 5.4:

- Longitudinal cracking (top-down fatigue)
- Alligator cracking (bottom-up fatigue)
- Reflection cracking
- Thermal cracking
- AC layer rutting
- Total rutting
- IRI

The design life of 20 years was considered for all the M-EPDG runs with the same performance threshold parameters as for the newly constructed AC pavement (see Table A.3). The separate discussions of the sensitivity results for each prediction model are following with support of relative effect charts and ANOVA results. The discussed inputs are grouped into pre-existing conditions, overlay parameters, unbound material properties, and traffic-related inputs

Analysis of the Longitudinal Cracking Model for AC-Overlaid AC design
The relative effect charts for longitudinal cracking predictions are shown in Figures 5.18 through 5.20, whereas ANOVA results of the importance rankings are summarized in Table 5.12.

Effect of climate: It is notable that distinct longitudinal cracking values were predicted for all three climatic zones considered in this study. Specifically, consistently high cracking values were predicted for the INLAND climate followed by SHORE and MOUNT climates with baseline values of 19.4, 12.7, and 9.4 ft/mi, respectively. Recall that due to a very low expected reliability of the predictions, those values can reach hundreds of feet per mile, which will increase based upon the different climates.

Effect of pre-existing conditions: The relative effect charts indicate the pavement rating (PR) of the existing surface as the second most influencing factor of longitudinal cracking. Note that a dramatic increase in top-down fatigue damage is predicted for PR varying from FAIR to POOR. Interestingly, the longitudinal cracking is reported to be the major type of cracking in Long-Term Pavement Performance (LTPP) SPS-9 sections that were rated fair to poor before being overlaid. On the other hand, the milling depth (HMILL) appears to have a minor impact, which is expected since the propagation of the longitudinal cracking is modeled from the surface down. For the same reason, the variation in total rutting of the existing surface (TOTRUTEXIST) is shown to have no effect on the top-down fatigue. One important observation is that the relative effect of PR and HMILL changes with climate, while following the trend in baseline values. This indicates a significant level of interaction between those factors and the CLIMATE.

Effect of AC overlay inputs: The overlay thickness (HAC1) is shown to be the input with the largest effect. It was observed that longitudinal cracking is predicted to increase drastically if the overlay thickness drops from 3.5 to 2 inches. Once again, this high level of cracking was also observed on the LTPP SPS-9A sections overlaid with 2.5-in of asphalt. It is notable that for a 3.5-in thick AC overlay, both gradation (ACOLGRAD) and overlay binder PG (ACOLBIND) have a very limited effect on the predicted longitudinal cracking values.

Effect of unbound layer inputs: The granular base thickness (HBASE) shows no relative effect on the longitudinal cracking in the overlay, while a very small variation in the predicted output can be attributed to the change in base modulus (EB). Contrary to what we would expect to see, an increase in subgrade stiffness (ES) shows notable increase in top-down fatigue.

Effect of traffic volume: The effect of truck traffic volume (AADTT) appears to be the highest when pavement rating and overlay thickness are kept constant. Furthermore, the effect of AADTT is dependent on the location (CLIMATE) and subgrade support (ES), thus, indicating reasonable interaction between all site factors in their effect on the longitudinal cracking as predicted by the M-EPDG models.

All the conclusions above are supported by the statistical analysis of significance and the importance rankings assigned in Table 5.12.

Table 5.12. ANOVA of inputs for the longitudinal cracking model in AC-overlaid AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	HAC1	173.7	0.000	2.24	Yes	High
2	PR	65.69	0.000	1.82	Yes	High
3	AADTT	15.18	0.000	1.18	Yes	High
4	SUBGRADE	13	0.000	1.11	Yes	High
5	CLIMATE	6.46	0.003	0.81	Yes	Moderate
6	HMILL	1.79	0.178	0.25	No	Low
7	BASE	0.29	0.750	-0.54	No	Low
8	ACOLBIND	0.26	0.772	-0.59	No	Low
9	ACOLGRAD	0.13	0.876	-0.89	No	Low
10	HBASE	0.02	0.984	-1.70	No	Low
11	TOTRUTEXIST	0.01	0.995	-2.00	No	Low

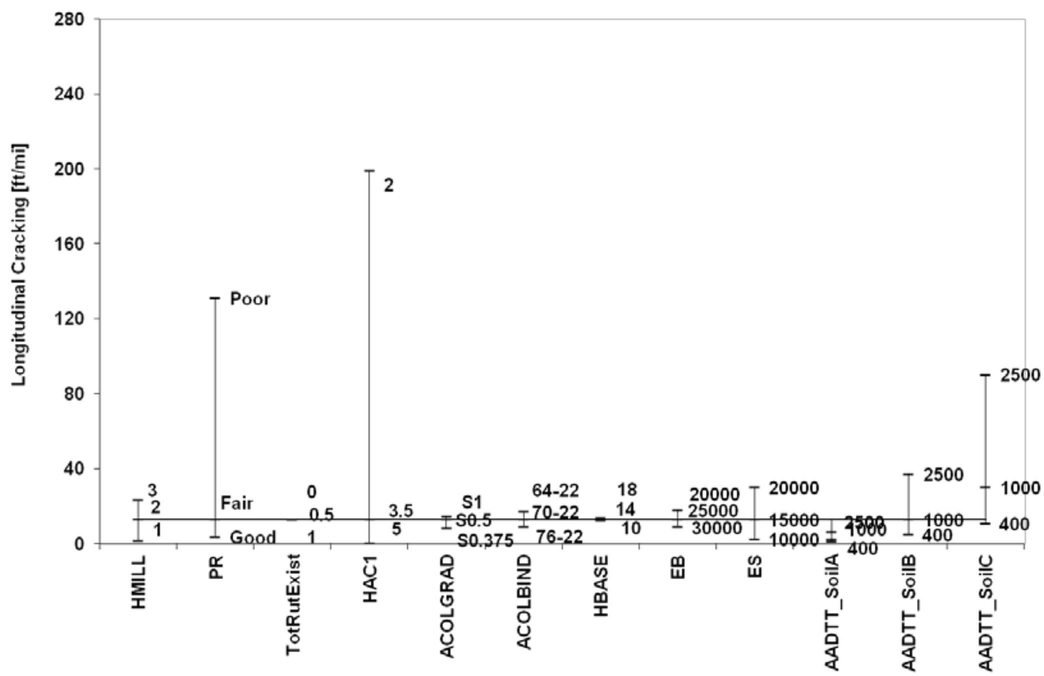

Figure 5.18. Relative effect of variables on longitudinal cracking in AC-overlaid AC design located in SHORE climate

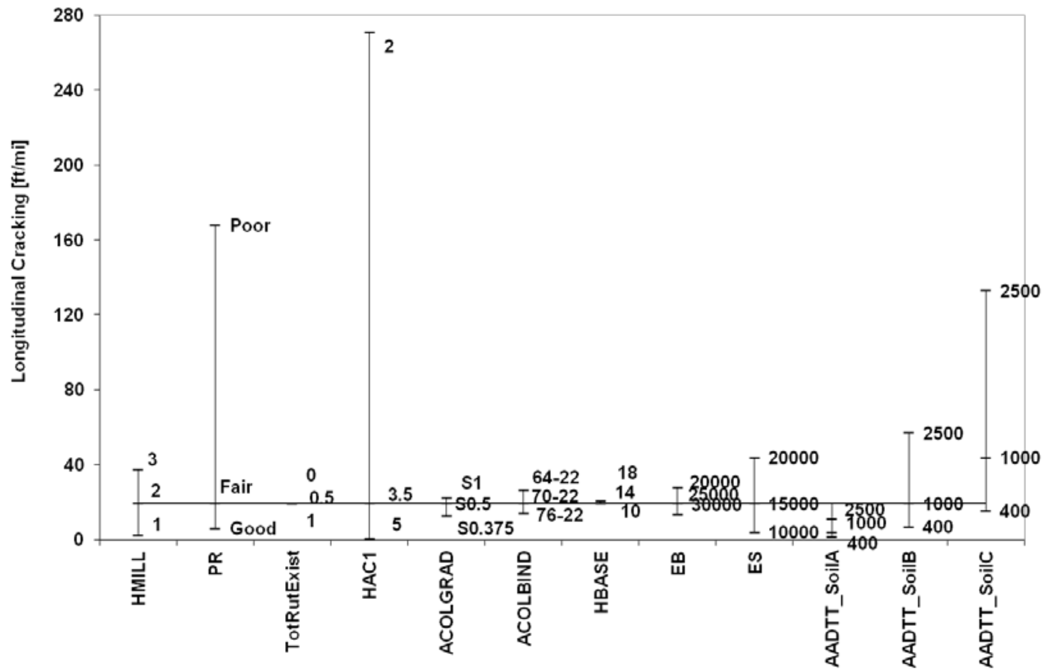

Figure 5.19. Relative effect of variables on longitudinal cracking in AC-overlaid AC design located in INLAND climate

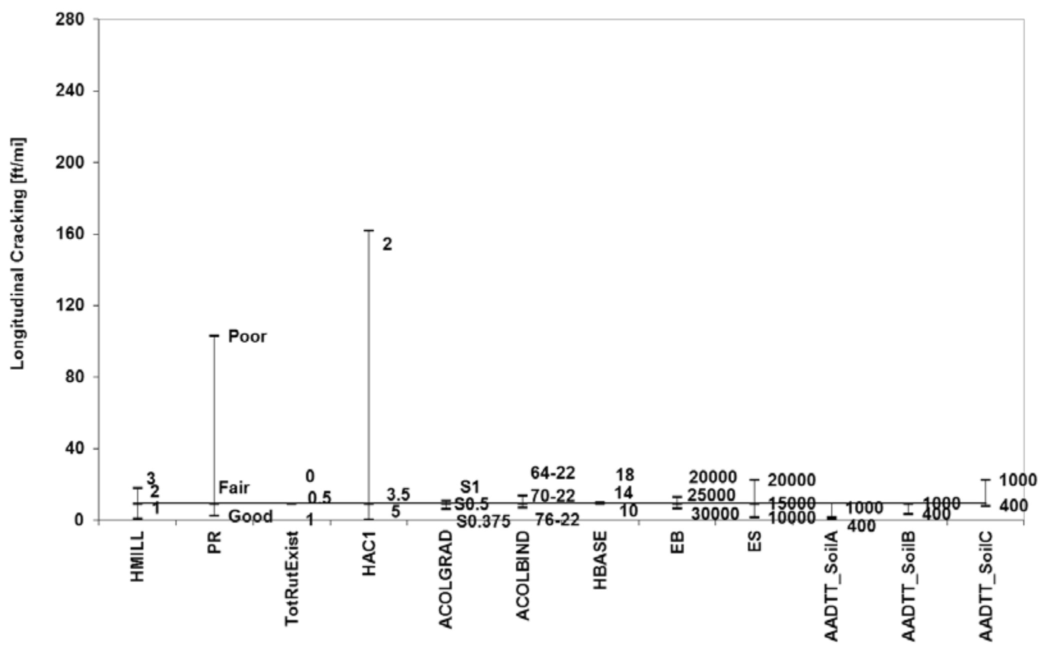

Figure 5.20. Relative effect of variables on longitudinal cracking in AC-overlaid AC design located in MOUNT climate

Analysis of Alligator Cracking Model for AC-Overlaid AC design

On average, the 72 M-EPDG project runs only yielded negligible values of alligator cracking with a mean of 0.02 percent area and maximum 0.3 percent area for an overlay over a pavement with poor surface condition rating. Therefore, no sensitivity results are presented in this chapter. It should be noted, however, that such an outcome is expected for a relatively thick, so called “deep strength”, AC pavement.

Analysis of Reflection Cracking Model for AC-Overlaid AC design

As discussed in Chapter 3, reflection cracking is expected to develop due to the propagation of cracks from the existing surface through the thickness of an overlay. Therefore, it is expected that the overlay thickness and stiffness as well as the pre-overlay pavement condition should be the primary contributors to the reflection cracking growth.

Figure 5.21 illustrates the relative effect of the pre-existing conditions, AC overlay parameters, unbound material inputs, and traffic on the terminal reflection cracking predicted in percent area covered after 20 years in service. Note that the M-EPDG predicted no difference in reflection cracking for the three climatic zones in CT when the other variables were kept fixed. Also it can be seen that only milled thickness (HMILL), pavement rating (PR), and overlay thickness (HAC1) influence the output, in that order. The rest of the inputs appear to be irrelevant to the M-EPDG predictions for reflection cracking, which is expected (see Figure 3.9).

The observations from Figure 5.21 are not necessarily supported by ANOVA results in Table 5.13. The HMILL, PR, and HAC1 factors appear to be over-exaggerated by the enormous F-ratios, which may be due to very small yet consistent standard deviation of the output values. Note that only 4 out of 24 runs yield results different from the baseline value of 34.3 percent. The significant p-values for such factors as AADTT, CLIMATE, ES, and EB (marginal) suggest that their effect cannot be neglected when the other parameters are kept fixed.

Table 5.13. ANOVA of inputs for the reflection cracking model in AC-overlaid AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	HMILL	17034709.46	0.000	7.23	Yes	Critical
2	PR	10358476.9	0.000	7.02	Yes	Critical
3	HAC1	538674.3	0.000	5.73	Yes	Critical
4	AADTT	494.24	0.000	2.69	Yes	High
5	ES	20.53	0.000	1.31	Yes	High
6	CLIMATE	14.27	0.000	1.15	Yes	High
7	EB	3.08	0.055	0.49	Yes	Moderate
8	ACOLBIND	1.01	0.373	0.00	No	Low
9	ACOLGRAD	0.45	0.638	-0.35	No	Low
10	HBASE	0.12	0.885	-0.92	No	Low
11	TOTRUTEXIST	0.01	0.987	-2.00	No	Low

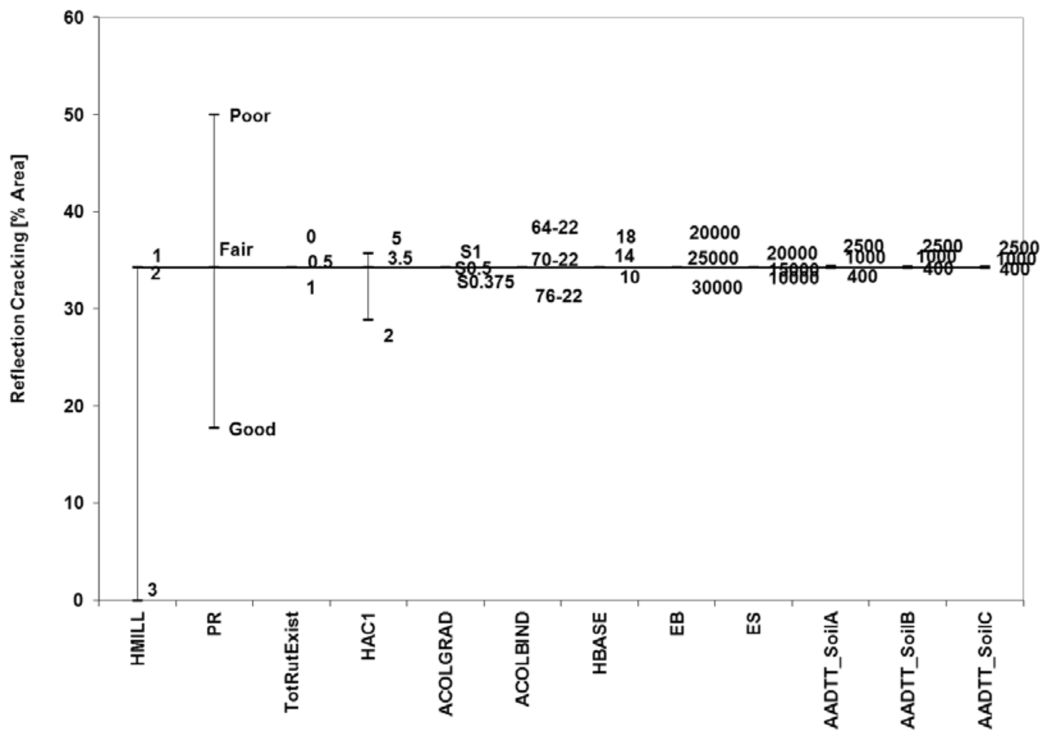

Figure 5.21. Relative effect of variables on reflection cracking in AC-overlaid AC design located in Connecticut

Analysis of Transverse Cracking Model for AC-Overlaid AC design

Transverse cracking in AC pavements is generally a result of thermal damage, which varies primarily due to temperature gradients in the pavement and temperature susceptibility of the asphalt mix. The sensitivity analysis of the thermal cracking model in this study consistently showed differences between the three climatic zones in Connecticut. The effect of climate and the other groups of inputs are discussed below with the support of relative effect charts in Figures 5.22 through 5.24 and the ANOVA results in Table 5.14.

Effect of climate: The baseline transverse cracking values for the coastal (SHORE), inland (INLAND) and northwest and northeast hills (MOUNT) locations were 3.4, 19.2, and 1190 ft/mi, respectively. It is notable that M-EPDG predicted the thermal damage to differ by orders of magnitude, indicating the critical influence of the environment. However, the relative effect of the other inputs is shown to be similar for all climates. In addition, such a vast difference between locations may be explained by the built-in uncertainty of the thermal cracking model (TCModel).

Effect of pre-existing conditions: The milled thickness (HMILL) shows visible effect on thermal cracking, which can be explained by its contribution to the total thickness of asphalt layers. On the other hand, the rating of the existing surface (PR) and its rutting (TOTRUTEXIST) appears to be irrelevant.

Effect of AC overlay inputs: The volumetric parameters of the mix (ACOLGRAD) are predicted to have a significant effect on the thermal damage output. Note that for the given range of inputs, binder properties, (given the only low grade of -22) (ACOLBIND), and overlay thickness (HAC1) appear to be less influential.

Effect of unbound layer inputs: The subgrade and base-related inputs are not expected to have any significant effect on thermal (transverse) cracking.

Effect of traffic: No traffic volume effect is anticipated as predicted by the M-EPDG transverse cracking model. This corresponds to the concept that thermal cracking is caused primarily by the environment and not traffic.

Table 5.14. ANOVA of inputs for the thermal cracking model in AC-overlaid AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	CLIMATE	1001.59	0.000	3.00	Yes	High
2	ACOLGRAD	7.2	0.002	0.86	Yes	Moderate
3	HAC1	2.19	0.123	0.34	No	Low
4	HMILL	2.06	0.138	0.31	No	Low
5	ACOLBIND	1.66	0.201	0.22	No	Low
6	AADTT	0.12	0.887	-0.92	No	Low
7	ES	0.06	0.940	-1.22	No	Low
8	HBASE	0.04	0.964	-1.40	No	Low
9	EB	0.03	0.975	-1.52	No	Low
10	ES	0.01	0.998	-2.00	No	Low
11	TOTRUTEXIST	0.01	1.000	-2.00	No	Low

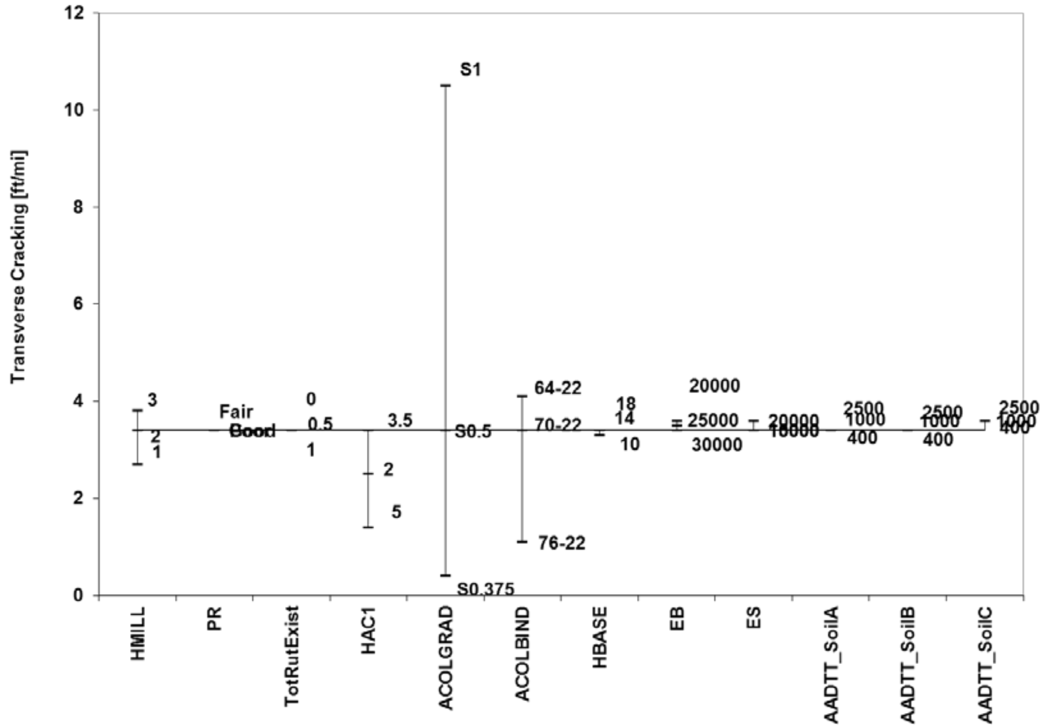

Figure 5.22. Relative effect of variables on transverse cracking in AC-overlaid AC design located in SHORE climate

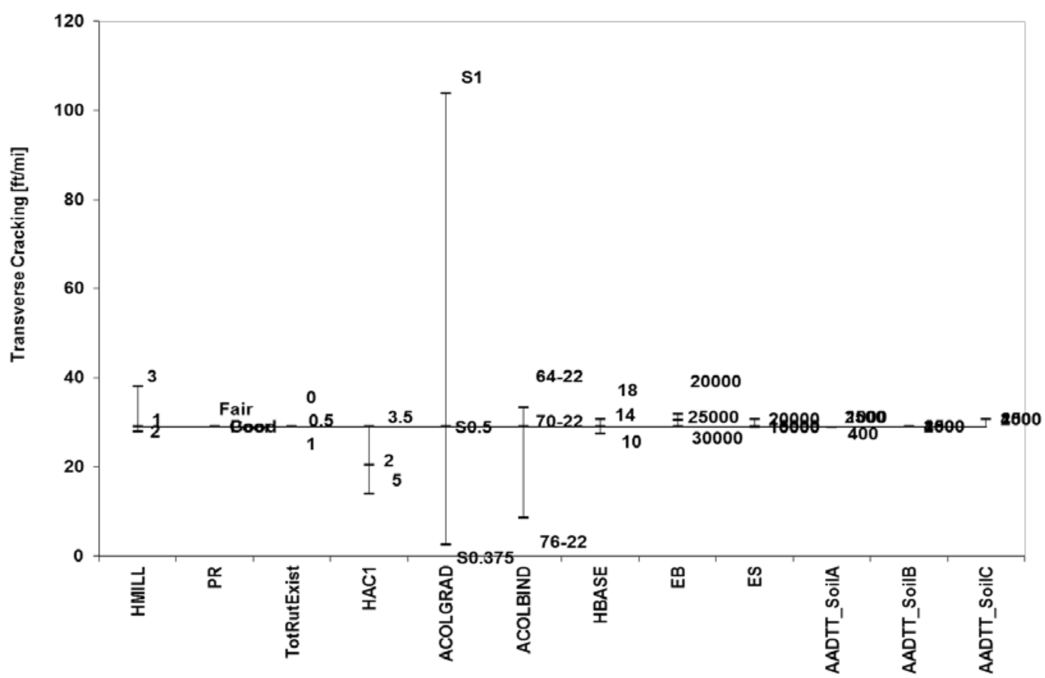

Figure 5.23. Relative effect of variables on transverse cracking in AC-overlaid AC design located in INLAND climate

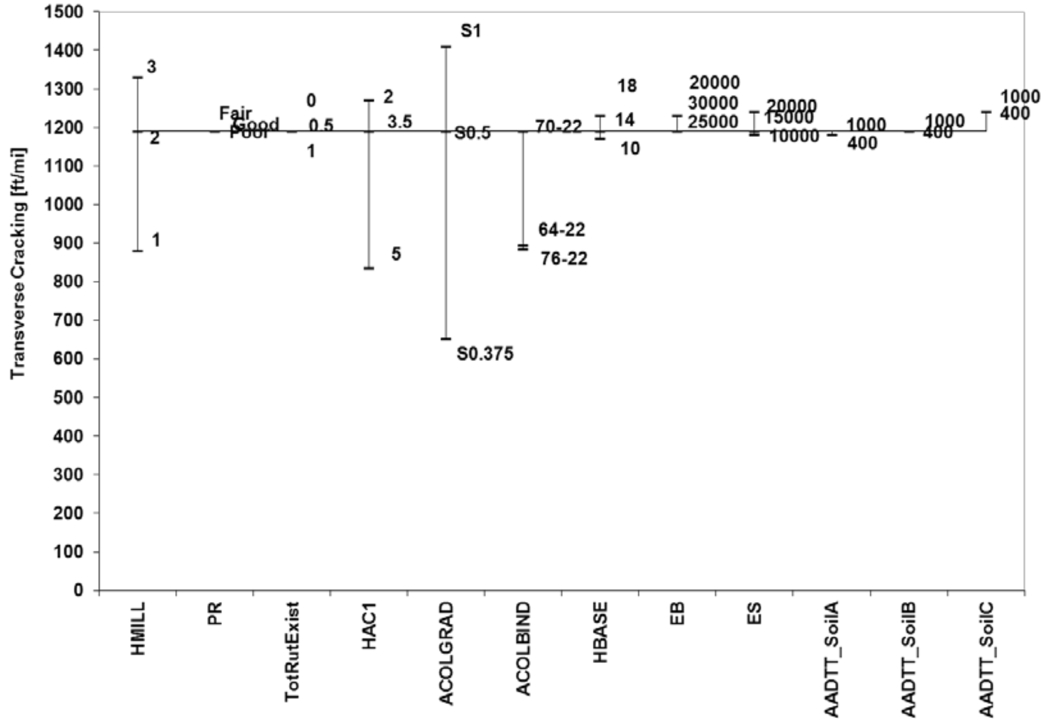

Figure 5.24. Relative effect of variables on transverse cracking in AC-overlaid AC design located in MOUNT climate

Analysis of AC Rutting Model for AC-Overlaid AC design

The sensitivity runs for the AC-overlaid AC pavement designs yielded overall negligibly low values of rutting in the AC layer after 20 years of service (average of 0.06 in with standard deviation of 0.01 in). This can be explained by the "deep strength" of the analyzed pavement structures. However, it is understood that the M-EPDG rutting models should be calibrated on the local material properties and climatic variables. Therefore, the analysis of the relative effect of the various inputs on rutting (Figure 5.25) is provided here so it could be referred to, while deciding on what inputs to use for calibration. Table 5.15 summarizes the input importance ranking for this analysis.

Effect of pre-existing conditions: The relative effect chart in Figure 5.25 indicates a visibly non-linear effect of milling thickness (HMILL), Pavement Rating (PR) and total rutting in existing pavement (TOTRUTEXIST). Thus, an increase in HMILL from 2 to 3 inches shows twice as large an increase in rutting as decrease in HMILL from 2 to 1 inch. Also, a negligible decrease in AC rutting is shown for TOTRUTEXIST greater than 0.5 in, as well as for the improvement in existing pavement rating from fair to good.

Effect of AC layer inputs: It appears that binder properties (ACOLBIND) dominate the range of AC rutting values, while variations in volumetric properties of the mix (ACOLGRAD) makes a lesser contribution. The overlay thickness (HAC1) with the analyzed range of values (2 to 5 inches) shows the most effect for thinner overlays (2 to 3.5 inches).

Effect of unbound layer inputs: As expected, only subgrade strength (ES) contributes to the extent of rutting in the top AC layer, while neither base modulus (EB) nor base thickness (HBASE) have any effect on this distress in the asphalt layer.

Effect of traffic: Clearly, for the given range of inputs, the truck traffic volume expressed in AADTT is expected to be the major factor at any location and subgrade type.

Table 5.15. ANOVA of inputs for the AC rutting model in AC-overlaid AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	AADTT	1561.68	0.000	3.19	Yes	Critical
2	CLIMATE	581.69	0.000	2.76	Yes	High
3	ACOLBIND	181.70	0.000	2.26	Yes	High
4	TOTRUTEXIST	110.19	0.000	2.04	Yes	High
5	HAC1	94.97	0.000	1.98	Yes	High
6	HMILL	65.15	0.000	1.81	Yes	High
7	ACOLGRAD	24.79	0.000	1.39	Yes	High
8	PR	23.30	0.000	1.37	No	High
9	ES	8.38	0.001	0.92	No	Moderate
10	EB	2.36	0.106	0.37	No	Low
11	HBASE	0.26	0.769	-0.59	No	Low

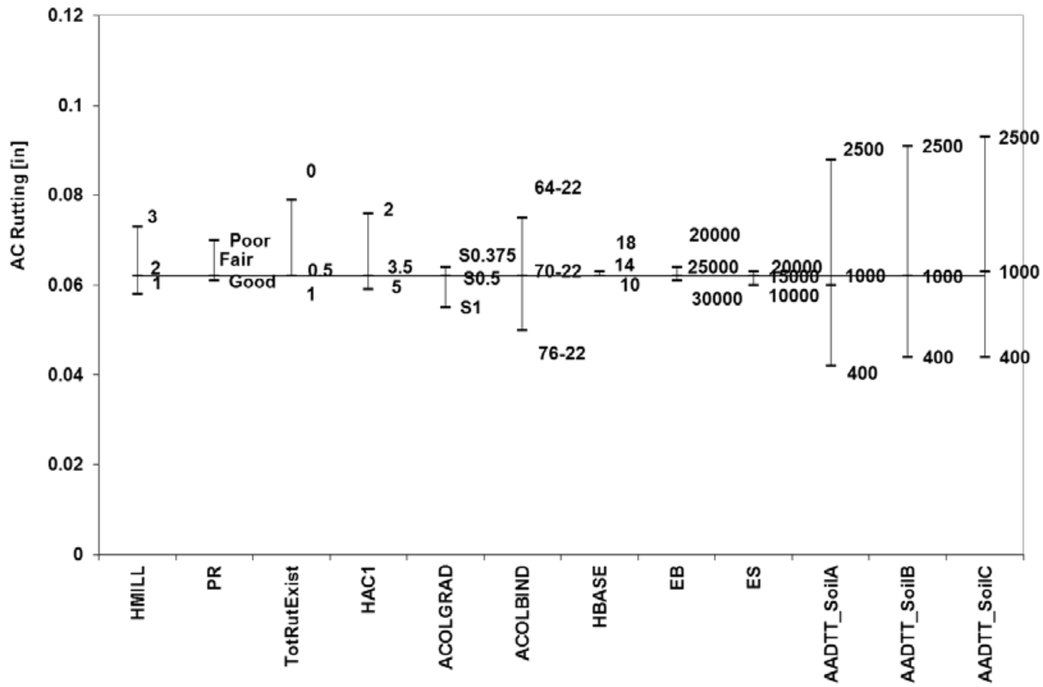

Figure 5.25. Relative effect of variables on AC rutting in AC-overlaid AC design located in Connecticut

Analysis of the Total Rutting Model for AC-Overlaid AC design

The cumulative total rutting in flexible pavement is modeled by the M-EPDG as a sum of permanent deformations in the asphalt, granular base, and subgrade layers. The M-EPDG runs of the total rutting model resulted in 0.08-in average value, which, in considering the average AC rutting of 0.06, suggested only minor or no contribution of base and subgrade deformations.

The effect of overlay thickness, and traffic appear to be similar, yet to a much lesser degree, than the pre-existing conditions. The influence of high-temperature PG of the binder (ACOLBIND) seems to be similar to that of overlay thickness. The ANOVA results in Table 5.16 support the aforementioned conclusions. Note that, although the differences in total rutting between climates were numerically small (0.01 in), they appear to be consistently significant and, therefore, CLIMATE variable ranks as highly important.

Table 5.16. ANOVA of inputs for the total rutting model in AC-overlaid AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	TOTRUTEXIST	3035.84	0.000	3.48	Yes	Critical
2	HMILL	2990.19	0.000	3.48	Yes	Critical
3	AADTT	275.28	0.000	2.44	Yes	High
4	CLIMATE	69.86	0.000	1.84	Yes	High
5	HAC1	37.54	0.000	1.57	Yes	High
6	ACOLBIND	28.99	0.000	1.46	Yes	High
7	PR	13.95	0.000	1.14	Yes	High
8	HBASE	4.58	0.015	0.66	Yes	Moderate
9	ACOLGRAD	4.27	0.020	0.63	Yes	Moderate
10	ES	1.54	0.224	0.19	No	Low
11	EB	0.02	0.979	-1.70	No	Low

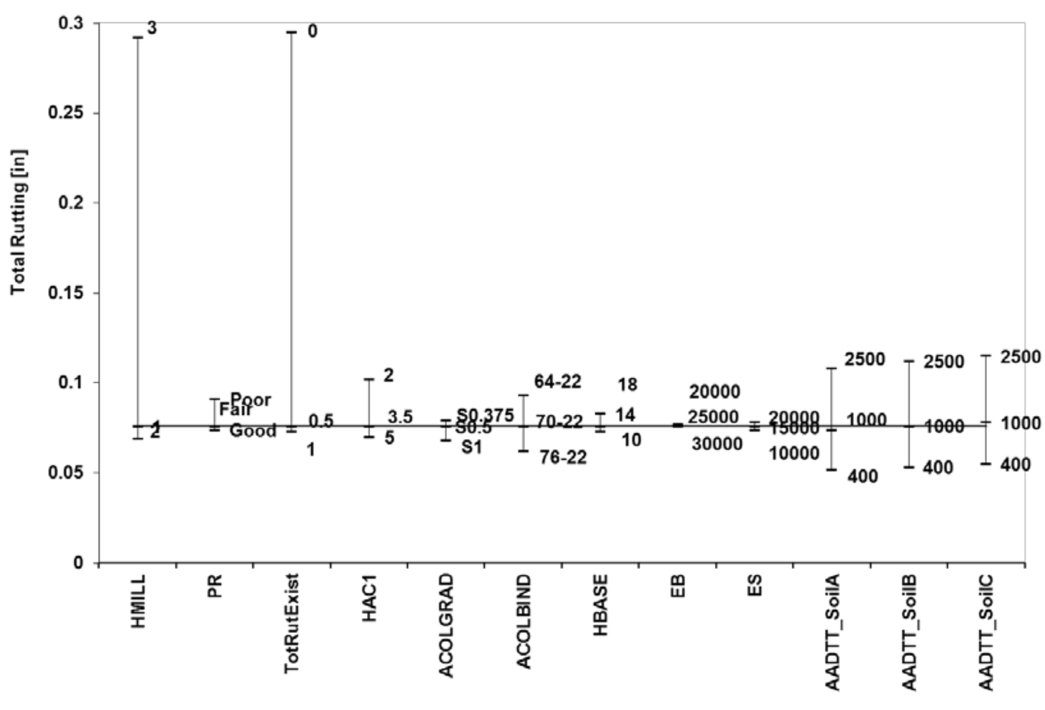

Figure 5.26. Relative effect of variables on total rutting in AC-overlaid AC design located in Connecticut

Analysis of IRI Model for AC-Overlaid AC design

As explained earlier, the IRI output in the M-EPDG predictions is primarily governed by the extent of rutting and, to a lesser degree, by fatigue cracking. Therefore, the effect of a specific input on the IRI is expected to be similar to that on the rutting and fatigue cracking. This phenomenon can be tracked again in the following discussion.

Effect of climate: The baseline M-EPDG predictions of IRI values indicated similar outcomes for SHORE and INLAND climate (93 and 94 in/mi, respectively), while MOUNT climate yielded a higher roughness value of 100 in/mi. The relative effect of the other inputs is plotted in Figure 5.27 for SHORE and INLAND climates and Figure 5.28 for MOUNT climate.

Effect of pre-existing conditions: Total existing rutting (TOTRUTEXIST) and milled thickness (HMILL) appear to be the most influential factors of IRI in a specified climate. Pavement rating (PR), on the other hand, had very little influence on the IRI.

Effect of AC layer inputs: The overlay's mix volumetrics, binder properties, and thickness show moderate effects on IRI. In addition, it is obvious that in MOUNT climate, their effect is more pronounced than it is in the milder SHORE and INLAND climates. Such a trend is most likely governed by the difference in thermal cracking between the three climates as predicted by the M-EPDG.

Effect of unbound layer inputs: Both relative effect charts and the ANOVA results (Table 5.17) indicate no influence or very little influence from the base thickness, as well as base and subgrade moduli on the predicted IRI values.

Effect of traffic: It is shown that the variation in AADTT results in a slightly higher variation in IRI values than that due to layer material properties and thicknesses. This effect, however, is much lower than that of climate and pre-existing conditions.

Table 5.17. ANOVA of inputs for the IRI model in AC-overlaid AC design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	CLIMATE	946.44	0.000	2.98	Yes	High
2	HMILL	153.08	0.000	2.18	Yes	High
3	TOTRUTEXIST	131.5	0.000	2.12	Yes	High
4	AADTT	10.44	0.000	1.02	Yes	High
5	HAC1	5.87	0.005	0.77	Yes	Moderate
6	ACOLGRAD	4.76	0.013	0.68	Yes	Moderate
7	ACOLBIND	2.89	0.065	0.46	Yes	Moderate
8	PR	0.84	0.436	-0.08	No	Low
9	HBASE	0.36	0.701	-0.44	No	Low
10	ES	0.2	0.817	-0.70	No	Low
11	EB	0.01	0.987	-2.00	No	Low

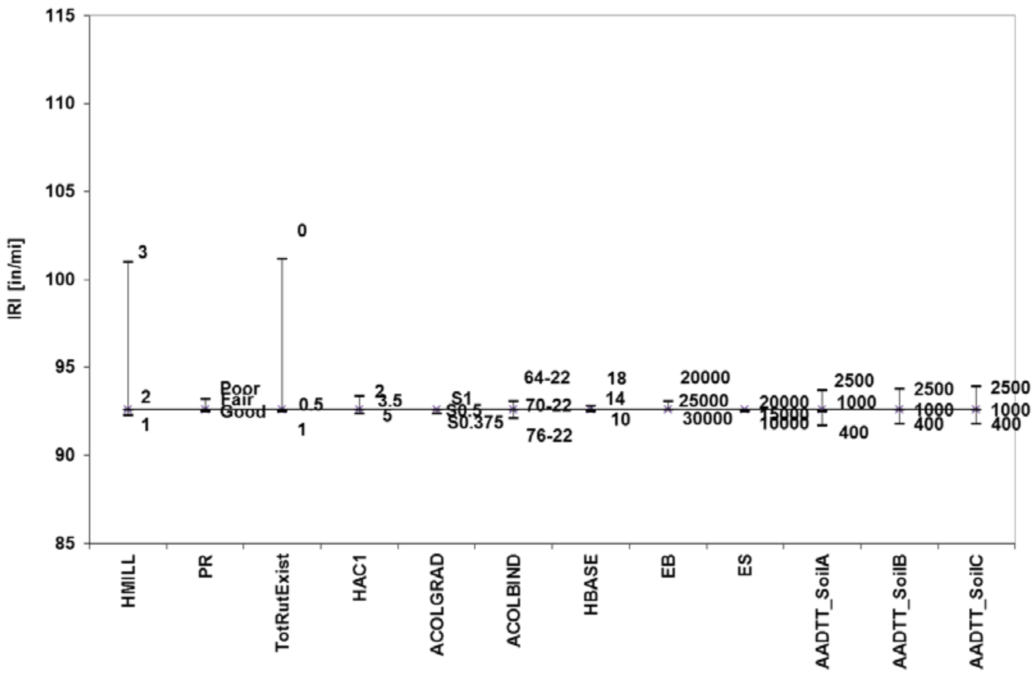

Figure 5.27. Relative effect of variables on IRI in AC-overlaid AC design located in SHORE and INLAND climates

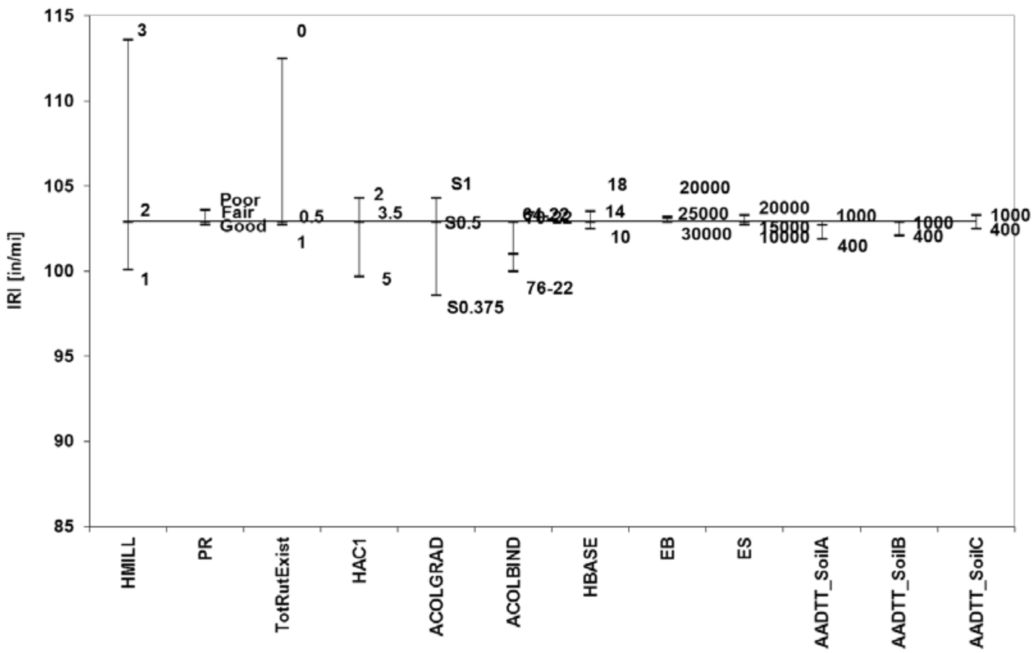

Figure 5.28. Relative effect of variables on IRI in AC-overlaid AC design located in MOUNT climate

Summary of the AC-Overlaid AC Design Sensitivity to Inputs

Similarly to the new AC design analysis, this section explains a combined effect of each input on cracking (i.e. combination of longitudinal, alligator, and thermal cracking) and rutting (combination of AC and total rutting). For that purpose, the logF values are averaged over all types of cracking and rutting separately to determine overall cracking and rutting rankings of inputs. Table 5.18 summarizes the individual importance rankings for cracking, rutting, and IRI. The sensitivity summary charts in Figures 5.29 through 5.31 illustrate those rankings.

The following conclusions are provided for the AC-overlaid AC pavement design in respect to its sensitivity to the inputs in discussion:

- Location and traffic volume appear to be important for an optimal overlay design. The traffic volume can be of lesser importance for the low-volume roads in the colder locations.

- The pre-overlay condition of the existing surface should be considered first to reduce cracking susceptibility, whereas the milled thickness is expected to affect overall performance of the overlay.

- When rutting is of a greater concern, total rutting in the existing surface is the most influencing input on the M-EPDG prediction.

- The AC overlay thickness shows to be an important factor in the cracking and rutting outputs, while a less important factor for IRI predictions.

- The overlay mix and binder properties show high influence on rutting and low influence on cracking, which makes them of moderate importance when an IRI prediction is concerned.

- For the analyzed range of unbound layer properties, the models for cracking, rutting, and IRI do not appear to be sensitive to subgrade type, base moduli, or base thickness.

It is understood that the above conclusions are only valid for the specific range of parameters evaluated in this study. It is anticipated that some of the sensitivity trends shown here may change after re-calibration of the M-EPDG distress prediction models.

Table 5.18. Summary of the combined sensitivity ranking of AC-overlaid AC design inputs

Cracking Ranking			Rutting Ranking			IRI ranking		
Predictor	Mean logF	Importance	Predictor	Mean logF	Importance	Predictor	Mean logF	Importance
PR	2.50	High	AADTT	2.33	High	CLIMATE	2.98	High
HAC1	2.23	High	TOTRUT EXIST	2.24	High	HMILL	2.18	High
HMILL	2.12	High	HAC1	1.98	High	TOTRUT EXIST	2.12	High
CLIMATE	1.44	High	HMILL	1.81	High	AADTT	1.02	High
AADTT	1.12	High	ACOL BIND	1.70	High	HAC1	0.77	Moderate
ACOL GRAD	0.08	Low	CLIMATE	1.70	High	ACOL GRAD	0.68	Moderate
SUB GRADE	0.08	Low	PR	1.37	High	ACOL BIND	0.46	Moderate
ACOL BIND	-0.10	Low	ACOL GRAD	1.03	High	PR	-0.08	Low
BASE	-0.89	Low	BASE	0.28	Low	HBASE	-0.44	Low
HBASE	-1.50	Low	SUB GRADE	-0.39	Low	ES	-0.70	Low
TOTRUT EXIST	-2.00	Low	BASE	-0.96	Low	EB	-2.00	Low

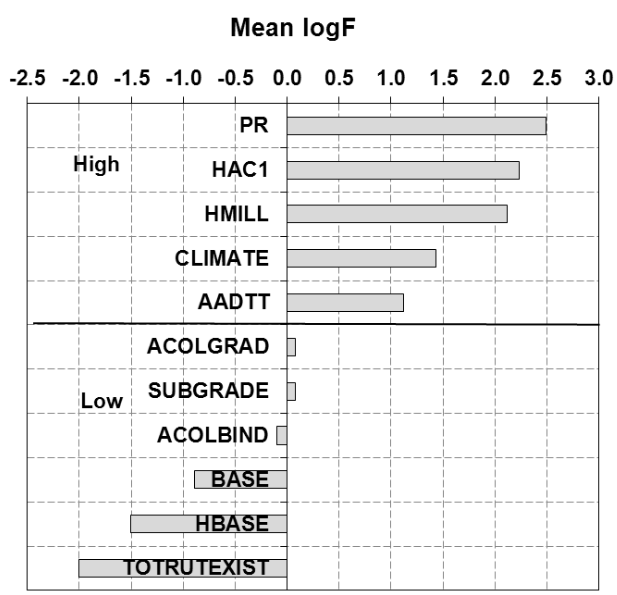

Figure 5.29. Cracking sensitivity to the AC-overlaid AC design inputs

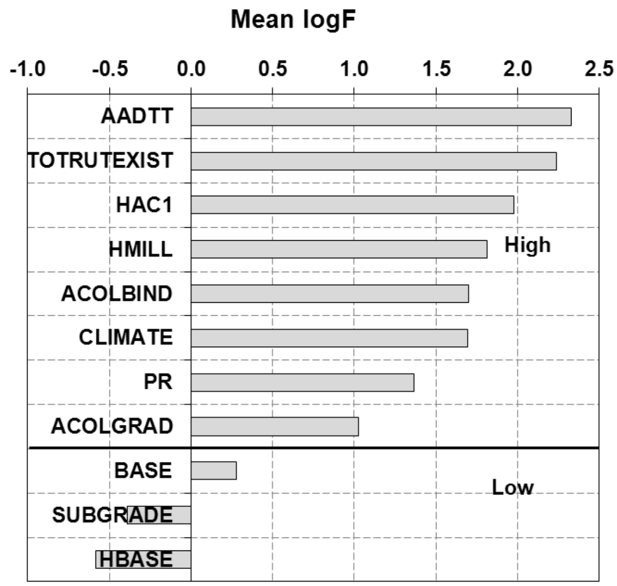

Figure 5.30. Rutting sensitivity to the AC-overlaid AC design inputs

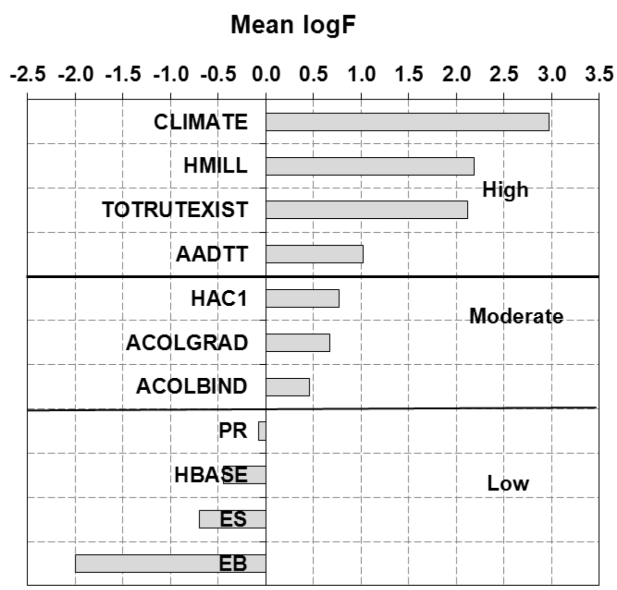

Figure 5.31. IRI sensitivity to the AC-overlaid AC design inputs

Sensitivity Results for AC-Overlaid Rubblized JPCP Pavement Design

The vast majority of PCC pavements on the interstate highways and state routes in Connecticut have been overlaid with asphalt. The remaining small percentage of concrete pavements are anticipated to be rehabilitated in the near future. Therefore, the AC-overlaid PCC Pavement Design came into consideration for the sensitivity analysis in this project.

The M-EPDG regards two options for treating the existing PCC pavements before overlay: crack & seat and rubblization. The latter option allows for use of the rubblized concrete as a high-modulus base, thus eliminating risk of reflection cracking as compared to the crack & seat method. Therefore, the AC-overlay over rubblized PCC was recommended by the team and approved for the sensitivity analysis by ConnDOT. Note that rubbilized jointed plain concrete pavement (JPCP) is analyzed here with the understanding that this is a typical type of PCC pavement in Connecticut. Effectively, the following M-EPDG prediction models were evaluated:

- Longitudinal (surface-down fatigue) cracking
- Alligator (bottom-up fatigue) cracking
- Thermal (transverse) cracking
- AC rutting
- Total rutting
- Roughness (IRI)

The sensitivity of the above prediction models for Connecticut design inputs were assessed by exploring the relative effect of the climate, traffic, layer thicknesses, and material properties on the predicted output as well as by the statistical ANOVA. A total of 66 M-EPDG simulations were run for this type of design. The description and range of input is provided in Tables 5.3 and A.11. Following is the discussion of the sensitivity results for each distress model along with the summary of input importance ranking.

Analysis of Longitudinal Cracking Model in AC-Overlaid Rubblized JPCP Design

For each of the three Connecticut climates, the M-EPDG predicted virtually zero longitudinal cracking (0 to 0.92 ft/mi) for all but one (21 out of 22 runs), where a lower PCC stiffness (200,000 psi) was considered. Nevertheless, the relative effect of some other inputs is visible (Figure 5.32) and, therefore, cannot be neglected, as supported by the ANOVA results in Table 5.19. The details are following:

Effect of climate: The baseline runs for the three climates yielded identical results for SHORE and MOUNT climates with a value of 0.08 ft/mi, while a slightly different value of 0.11 ft/mi was obtained for INLAND climate. The difference between those values is negligible.

Effect of existing JPCP inputs: It is clear from Figure 5.32 that the stiffness of the rubblized concrete (EPCC) is the major factor of the longitudinal cracking, especially if EPCC is lower than 500,000 psi. If this factor is kept fixed, the fractured slab thickness may have moderate effect on the surface-down fatigue in AC layer.

Effect of AC layer inputs: It appears that for the typical design considered here (2 to 5-in AC overlay over 9-in rubblized JPCP), any variation in thickness (HACOL), volumetrics (ACOLGRAD) and binder properties (ACOLBIND) would result in a low level of longitudinal cracking.

Effect of unbound layers: The relative effect of subgrade modulus (ES) is visible, while the base inputs (BASE and HBASE) only show moderate to no effects, respectively.

Effect of traffic: Both the relative effect chart in Figure 5.32 and ANOVA ranking in Table 5.19 indicate the AADTT variable as the most influential factor if the EPCC input is kept constant.

Table 5.19. ANOVA of inputs for the longitudinal cracking model in AC-overlaid JPCP design.

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	EPCC	6754.1	0.000	3.83	Yes	Critical
2	AADTT	31.99	0.000	1.51	Yes	High
3	SUBGRADE	18.37	0.000	1.26	Yes	High
4	HPCC	10.91	0.000	1.04	Yes	High
5	BASE	5.6	0.007	0.75	Yes	Moderate
6	CLIMATE	1.76	0.184	0.25	Yes	Low
7	HACOL	1.09	0.344	0.04	Yes	Low
8	ACOLGRAD	0.21	0.808	-0.68	No	Low
9	ACOLBIND	0.15	0.864	-0.82	No	Low
10	HBASE	0.01	0.989	-2.00	No	Low

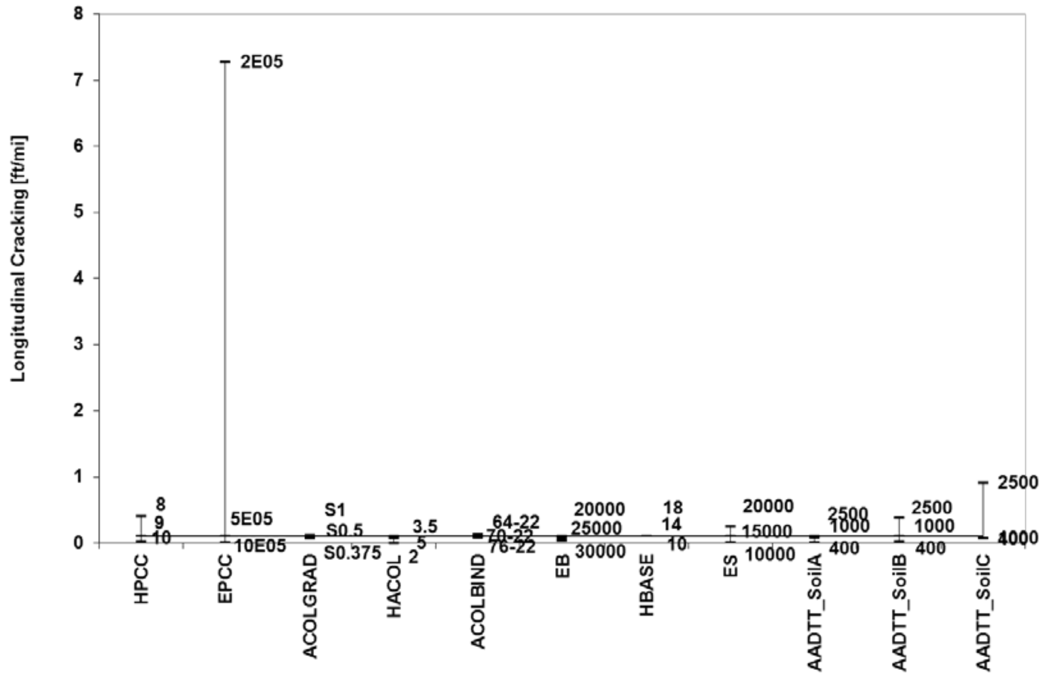

Figure 5.32. Relative effect of variables on longitudinal cracking in AC-overlaid rubblized JPCP design located in Connecticut

Analysis of the Alligator Cracking Model in AC-Overlaid Rubblized JPCP Design

The alligator (bottom-up fatigue) cracking for the pavement type AC-Overlaid Rubblized JPCP did not appear to be an issue as predicted by the M-EPDG model. Note that the maximum 0.26 percent alligator cracking area covered was only predicted when the low concrete stiffness (EPCC=200,000 psi) is considered (Figure 5.33). Nevertheless, the relative effect of some other factors appears to be statistically significant if EPCC is kept at the default M-EPDG value of 500,000 psi (Figure 5.34 and Table 5.20). Although alligator cracking is not an issue for the Connecticut Interstate highways, the state routes and local arterials can experience this type of distress. Therefore, in order to develop the recommendations for the calibration of the alligator cracking model for the design in discussion, the sensitivity of other inputs is explored in this report.

Effect of climate: As mentioned above, only a very small extent of alligator cracking was predicted for these types of roads. However, the variability in the baseline project output due to climate was moderately significant, as shown in Table 5.20

Effect of existing JPCP inputs: The stiffness of the rubblized concrete (EPCC) appears to be the major factor in the alligator cracking. Nevertheless, the fractured slab thickness (HPCC) shows relatively high effects as compared with base and subgrade inputs, for example (see Figure 5.33).

Effect of AC layer inputs: Both thickness (HACOL) and volumetrics (ACOLGRAD) show a much higher importance for the alligator cracking predictions than binder PG (ACOLBIND) does.

Effect of unbound layers: The variations in base stiffness (EB) and subgrade modulus (ES) yielded statistically high and moderate influence, respectively, on the alligator cracking predictions for the given range of inputs, while PCEPCC is kept at 500,000 psi. Change in base thickness, however, had no effect on the prediction.

Effect of traffic: As expected, AADTT input is the major factor of the load-related fatigue, providing relatively strong support from the fractured PCC slab (EPCC=500,000 psi).

Table 5.20. ANOVA of inputs for the alligator cracking model in AC-overlaid rubblized JPCP design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	EPCC	261403.81	0.000	5.42	Yes	Critical
2	AADTT	9922.08	0.000	4.00	Yes	Critical
3	HACOL	896.36	0.000	2.95	Yes	High
4	ACOLGRAD	307.85	0.000	2.49	Yes	High
5	HPCC	132.05	0.000	2.12	Yes	High
6	BASE	19.55	0.000	1.29	Yes	High
7	CLIMATE	14.32	0.000	1.16	Yes	High
8	SUBGRADE	6.38	0.004	0.80	No	Moderate
9	ACOLBIND	1.94	0.156	0.29	No	Low
10	HBASE	0.31	0.735	-0.51	No	Low

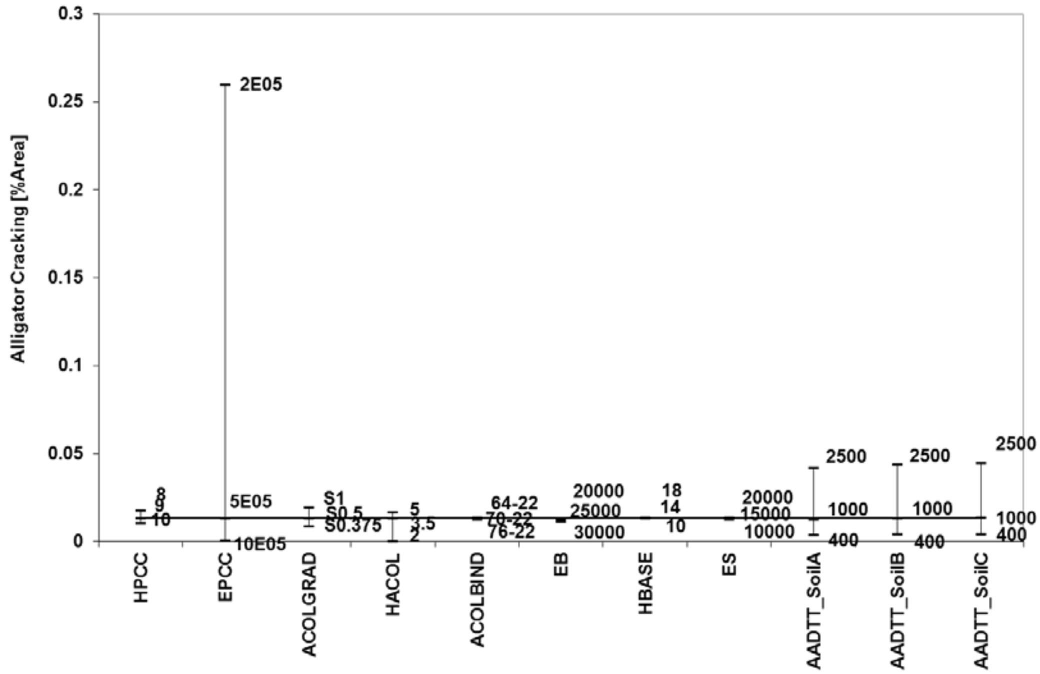

Figure 5.33. Relative effect of variables on alligator cracking in AC-overlaid rubblized JPCP design located in Connecticut (all inputs)

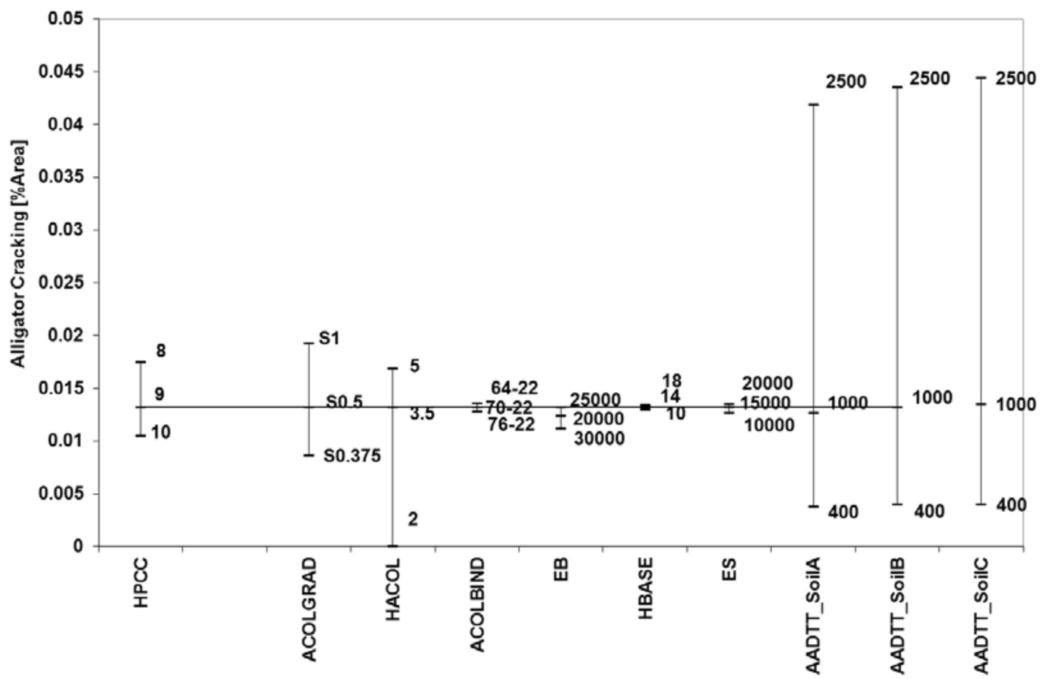

Figure 5.34. Relative effect of variables on alligator cracking in AC-overlaid rubblized JPCP design located in Connecticut (EPCC excluded)

Analysis of Thermal (Transverse) Cracking Model in AC-Overlaid Rubblized JPCP Design
The most notable observation from the relative effect charts in Figures 5.35 and 5.36 is that the baseline design simulations predicted thermal failure (1610 ft/mi and 1630 ft/mi, respectively) in SHORE and MOUNT climates versus much less thermal cracking of 137 ft/ mi for INLAND climate. The failure for a pavement located in a mild SHORE climate can only be explained by a relatively thin AC layer. On the other hand, the previous studies in Minnesota and South Dakota reported crashing of the thermal cracking model due to bugs in the software code. Therefore, it is recommended to expend additional effort to investigate the unexplained trends when calibrating the TCMODEL during the M-EPDG implementation. For the moment, since not all the values depicted in Figure 5.35 reached the failure level, it was decided to take a closer look at the difference between climates in the relative effect of AC layer inputs.

Effect of AC overlay thickness (HACOL): The M-EPDG thermal cracking model predicted failure or almost failure for 2 to 3.5-in overlays, while 5-in thickness resulted in virtually no cracking (13 ft/mi). This trend makes the HACOL variable the most influential input for a given location.

Effect of AC volumetric properties (ACOLGRAD): The relative effect chart in Figure 5.35 clearly indicates that risk of thermal failure increases with an increase in Nominal Maximum Aggregate Size [NMAS] from 0.375 inch to 0.5 inch. and 1 inch.

Effect of binder properties (ACOLBIND): It is notable that binder PG 64-22 and 70-22 yield very similar thermal cracking values that decrease substantially when PG 76-22 is used in simulation. In understanding that all three binders have the same low-temperature PG of -22, it can be implied that thermal cracking may occur also at intermediate temperatures in stiffer mixes and the higher anticipated elasticity of the PG 76-22 may slow down the cracking.

As expected, all the other factors show no effect on thermal cracking.

Table 5.21. ANOVA of inputs for the thermal cracking model in AC-overlaid rubblized JPCP design

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	CLIMATE	181.43	0.000	2.26	Yes	High
2	HACOL	5.07	0.010	0.71	Yes	Moderate
3	ACOLGRAD	4.52	0.016	0.66	Yes	Moderate
4	SUBGRADE	1.19	0.315	0.08	Yes	Low
5	AADTT	0.84	0.440	-0.08	Yes	Low
6	ACOLBIND	0.28	0.755	-0.55	Yes	Low
7	BASE	0.12	0.889	-0.92	Yes	Low
8	HPCC	0.05	0.955	-1.30	No	Low
9	EPCC	0.04	0.959	-1.40	No	Low
10	HBASE	0.04	0.961	-1.40	No	Low

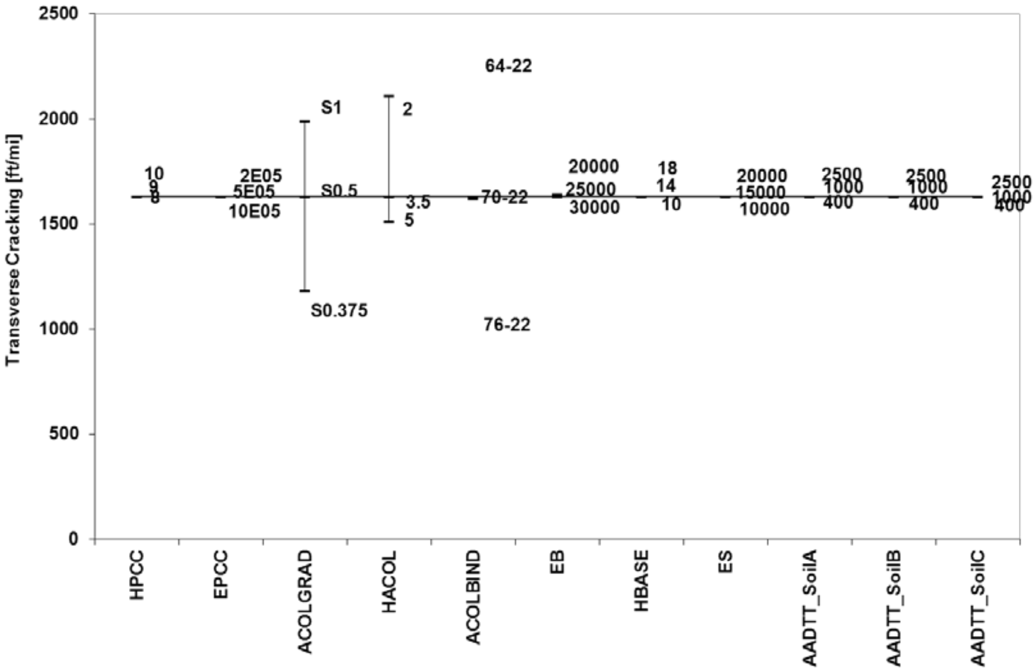

Figure 5.35. Relative effect of variables on thermal cracking in AC-overlaid rubblized JPCP design located in MOUNT climate

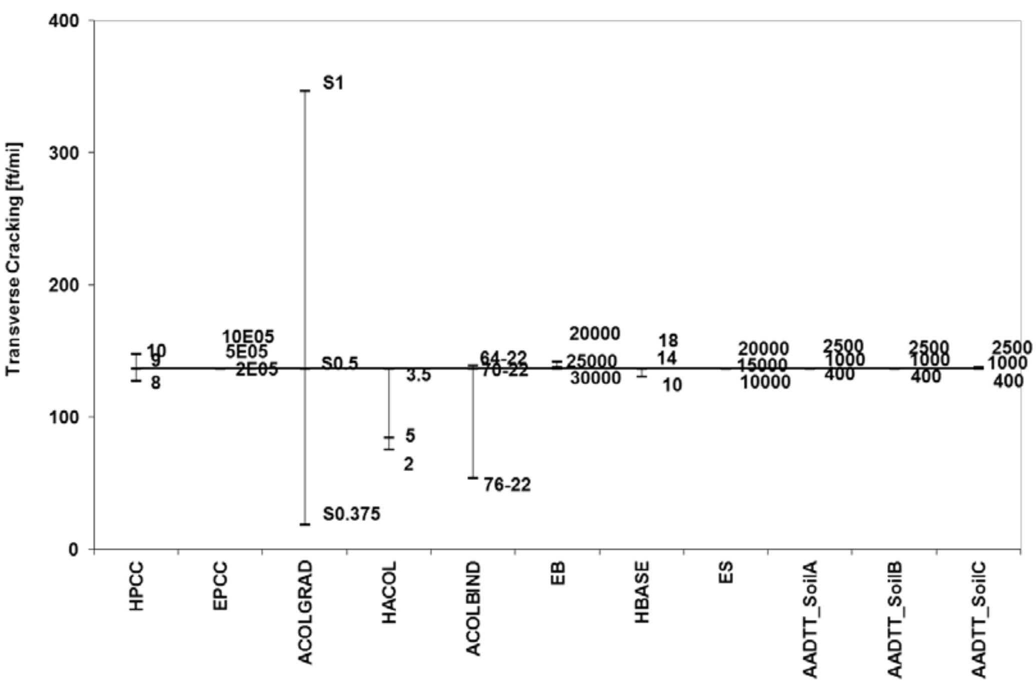

Figure 5.36. Relative effect of variables on thermal cracking in AC-overlaid rubblized JPCP design located in INLAND climate

Analysis of AC Rutting Model in AC-Overlaid Rubblized JPCP Design

Similarly low values of AC rutting, with a maximum of 0.14 in., were predicted for all 66 simulation runs for pavements with 20 years in service. Nevertheless, the small yet consistent differences were observed in the response of the AC rutting model to the variation in input values. This sensitivity is explained next and is supported by the relative effect charts in Figures 5.37 and 5.38, as well as by the ANOVA results and importance rankings in Table 5.22

Effect of climate: The AC rutting model predicted, on average, a higher rutting in the AC layer for INLAND climate (0.07 inches with standard deviation of 0.01) as compared with SHORE and MOUNT climate (0.11 inches with standard deviation of 0.02 for both). Therefore, the ANOVA attributes the second highest importance ranking to the CLIMATE variable.

Effect of existing PCC inputs: Neither relative effect charts nor ANOVA results suggest such an effect. Note that a significant decrease in PCC modulus (EPCC) can be apparently neglected due to very small differences in the results.

Effect of AC layer inputs: As expected, both the thickness of asphalt (HACOL) and binder PG grade (ACOLBIND) show a high influence on AC rutting values. The AC volumetrics (ACOLGRAD) also demonstrated a significant effect, although to a lesser degree.

Effect of unbound layer inputs: The subgrade modulus (ES) shows consistent yet moderate influence on the AC rutting, while granular base inputs (EB and HBASE) appear to be less important for the given range of input values.

Effect of traffic: Ultimately, truck traffic volume is the major external factor of AC rutting at any given combination of climate and subgrade.

Table 5.22. ANOVA of inputs for the AC rutting model in AC-overlaid rubblized JPCP design.

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	AADTT	418.31	0.000	2.62	Yes	High
2	CLIMATE	218.94	0.000	2.34	Yes	High
3	HACOL	137.28	0.000	2.14	Yes	High
4	ACOLBIND	37.34	0.000	1.57	Yes	High
5	ACOLGRAD	7.68	0.001	0.89	Yes	Moderate
6	SUBGRADE	3.59	0.036	0.56	Yes	Moderate
7	EPCC	1.82	0.175	0.26	No	Low
8	BASE	0.45	0.641	-0.35	No	Low
9	HPCC	0.44	0.647	-0.36	No	Low
10	HBASE	0.1	0.904	-1.00	No	Low

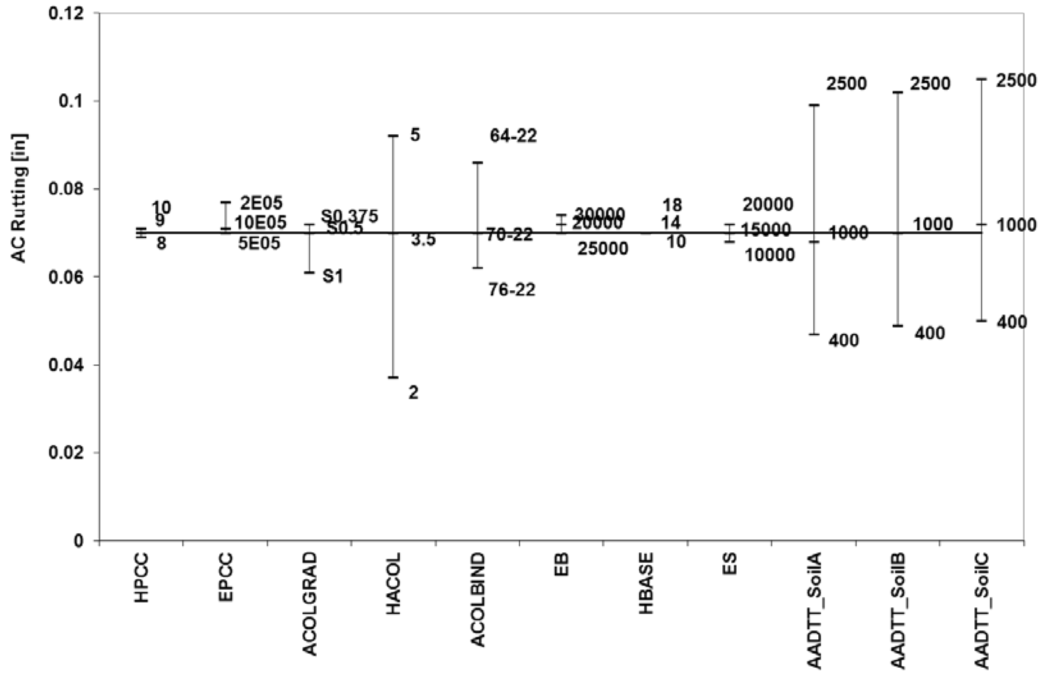

Figure 5.37. Relative effect of variables on AC rutting in AC-overlaid rubblized JPCP design located in SHORE and MOUNT climates

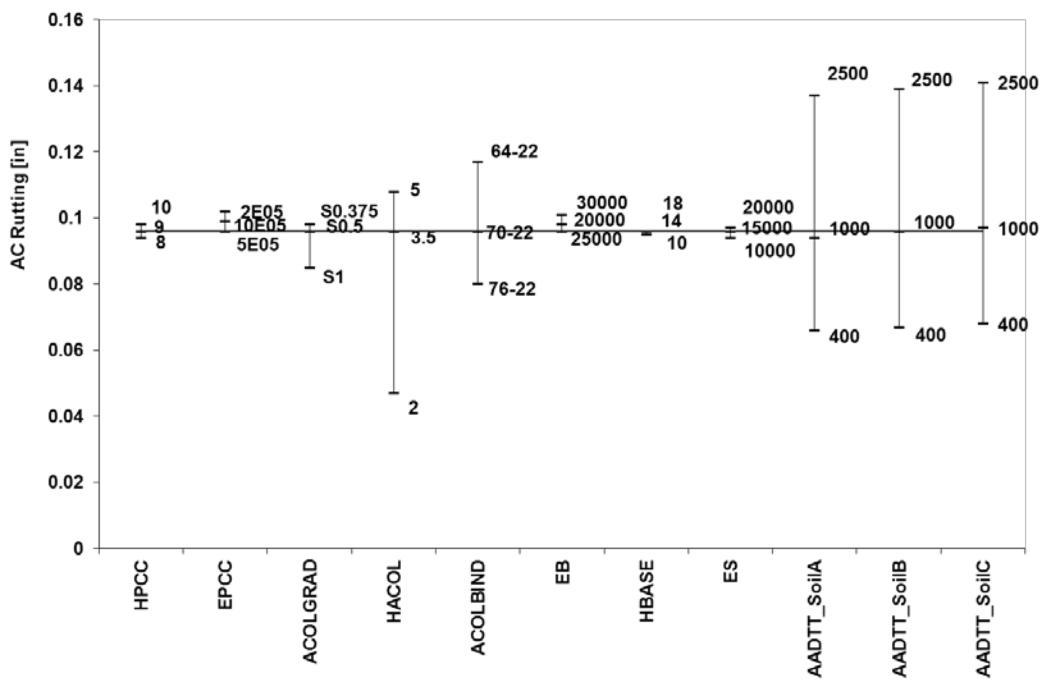

Figure 5.38. Relative effect of variables on AC rutting in AC-overlaid rubblized JPCP design located in INLAND climate

Analysis of Total Rutting Model in AC-Overlaid Rubblized JPCP Design

Overall, total rutting predictions by the M-EPDG ranged between 0.2 and 0.4 inches, which was well below the default threshold of 0.75 inches, yet making the variation sufficiently noticeable for the sensitivity analysis. In consideration of a maximum average of 0.14 inches of AC rutting, the contribution of the underlying layers to the total rutting for the AC overlay on JPCP appears to be higher than for the other types of pavement design discussed in this report. Figures 5.39 and 5.40 illustrate the relative effect of the design inputs, whereas Table 5.23 summarizes their importance ranks.

Effect of climate: For all runs in SHORE climate, the total rutting predictions were lower when all other inputs were fixed (0.27 in). Note that practically no difference in total rutting output was predicted for INLAND and MOUNT climate (0.29 and 0.30 inches, respectively). The statistical analysis ranks CLIMATE as the second most influential input after the traffic (AADTT).

Effect of existing PCC inputs: As expected, stiffer and thicker fractured PCC slabs are predicted to yield lower total rutting due to the high modulus (~ 500 Kpsi) giving better protection to unbound layers.

Effect of AC layer inputs: The binder properties (ACOLBIND) show the highest effect among this group of inputs, whereas asphalt thickness (HACOL) and volumetrics (ACOLGRAD) show a lesser degree of influence.

Effect of unbound layers: It is shown in relative effective charts, with the support of ANOVA results, that variation in subgrade modulus (ES) has a relatively high effect as compared with granular base stiffness (EB). The thickness of base apparently does not contribute to the variation in total rutting in this analysis.

Effect of traffic: As indicated by both relative charts and ANOVA results, the change in AADTT results in the biggest change in total rutting, as compared with all the other inputs.

Table 5.23. ANOVA of inputs for the total rutting model in AC-overlaid rubblized JPCP design.

Order No.	Predictor Index	F	p-value	logF	Statistical Significance	Assigned Importance
1	AADTT	833.5	0.000	2.92	Yes	High
2	CLIMATE	352.8	0.000	2.55	Yes	High
3	SUBGRADE	133.35	0.000	2.13	Yes	High
4	EPCC	95.86	0.000	1.98	Yes	High
5	ACOLBIND	31.47	0.000	1.50	Yes	High
6	BASE	14.42	0.000	1.16	Yes	High
7	HPCC	11.96	0.000	1.08	No	High
8	HACOL	9.84	0.000	0.99	No	Moderate
9	ACOLGRAD	7.12	0.002	0.85	No	Moderate
10	HBASE	1.39	0.261	0.14	No	Low

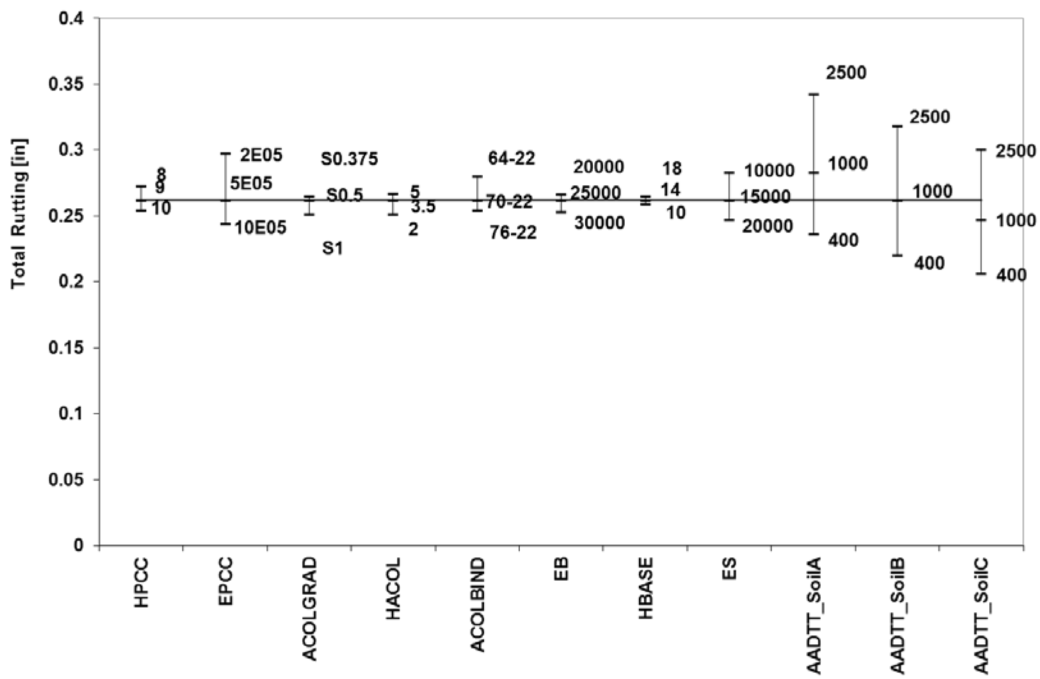

Figure 5.39. Relative effect of variables on total rutting in AC-overlaid rubblized JPCP design located in SHORE climate

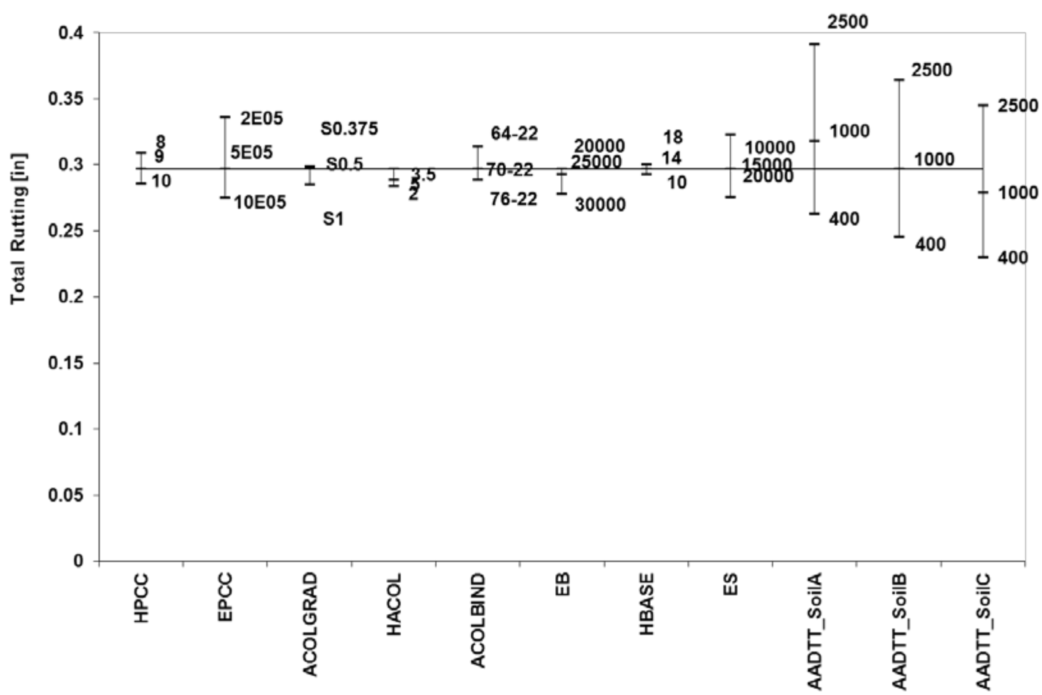

Figure 5.40. Relative effect of variables on total rutting in AC-overlaid rubblized JPCP design located in INLAND and MOUNT climates

Analysis of IRI Model in AC-Overlaid Rubblized JPCP Design

Reasonably, the sensitivity of the IRI predictions to these discussed inputs is governed by the sensitivity of the IRI model to the contributing distresses, the major contributors being rutting, fatigue, and thermal damage (see Figure 3.14). Thus, the inputs showing high influence on those distresses will also have a significant effect on the predicted IRI values.
The sensitivity of the IRI model to the specific groups of inputs is discussed next with the support of relative effect charts (Figures 5.41 and 5.42) and ANOVA rankings (Table 5.24).

Effect of climate: The M-EPDG produces significantly higher values for the SHORE and MOUNT climates (116 in/mi) as compared with INLAND climate (103 in/mi) which can be explained by the contribution of thermal failure predicted by the M-EPDG. Since all the other inputs only yield smaller changes in IRI, the CLIMATE variable shows the highest ranking in the ANOVA results.

Effect of AC layer inputs: The thickness and volumetric properties of the AC layer (HACOL and ACOLGRAD) have a moderate effect on the predicted IRI values. The low statistical significance of the ACOLBIND input can be explained by the thermal cracking issues of both PG 64-22 and PG 70-22 binders in two out of three climates, as explained above. Another reason for such a trend may be the low AC rutting predicted for the given range of inputs.

Effect of unbound layers: The subgrade stiffness is expected to mainly contribute to the total rutting in the discussed design. Therefore, the ES variable appears to rank as a moderately influencing factor. Obviously, neither one of the granular base-related inputs (EB and HBASE) significantly affect the change in IRI.

Effect of traffic: The AADTT variable is the most influential factor in total rutting and fatigue. Nevertheless, it's low impact on thermal failure, as well as the very low AC rutting and fatigue predictions results in the overall moderate impact of this factor on the IRI in this analysis.

Table 5.24. ANOVA of inputs for the IRI model in AC-overlaid rubblized JPCP design

Order No.	Predictor Index	F-ratio	p-value	logF	Statistical Significance	Assigned Importance
1	CLIMATE	167.18	0.000	2.22	Yes	High
2	HACOL	4.75	0.014	0.68	Yes	Moderate
3	SUBGRADE	4.05	0.024	0.61	Yes	Moderate
4	AADTT	3.65	0.034	0.56	Yes	Moderate
5	ACOLGRAD	3.42	0.041	0.53	Yes	Moderate
6	EPCC	1.12	0.335	0.05	Yes	Low
7	ACOLBIND	0.89	0.416	-0.05	No	Low
8	BASE	0.33	0.723	-0.48	No	Low
9	HPCC	0.13	0.881	-0.89	No	Low
10	HBASE	0.05	0.951	-1.30	No	Low

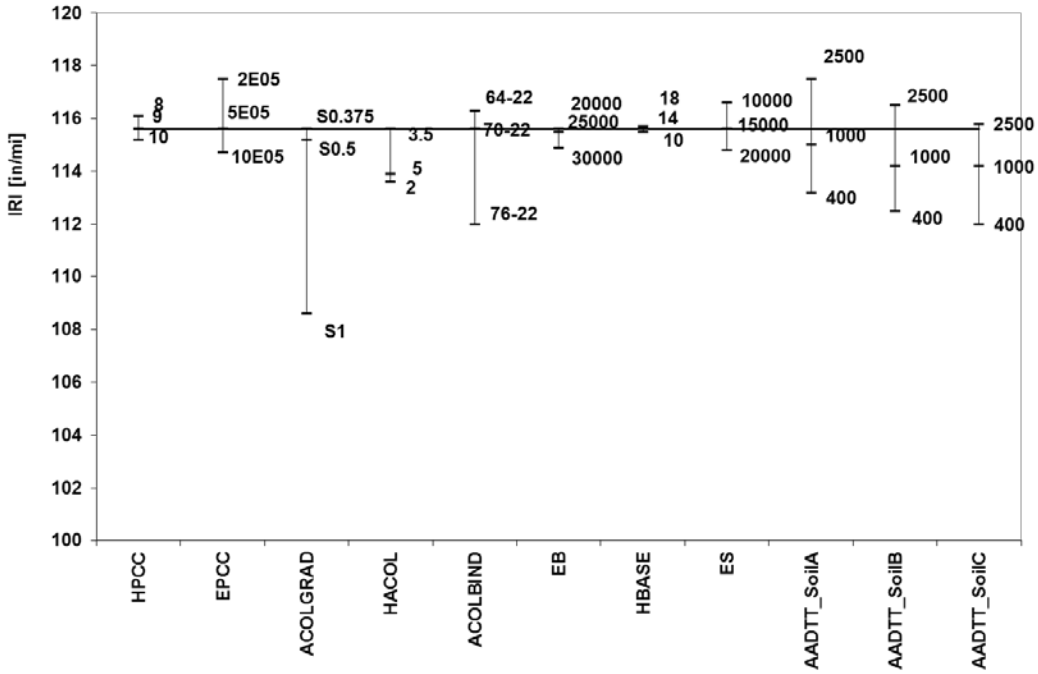

Figure 5.41. Relative effect of variables on IRI in AC-overlaid rubblized JPCP design located in SHORE and MOUNT climate

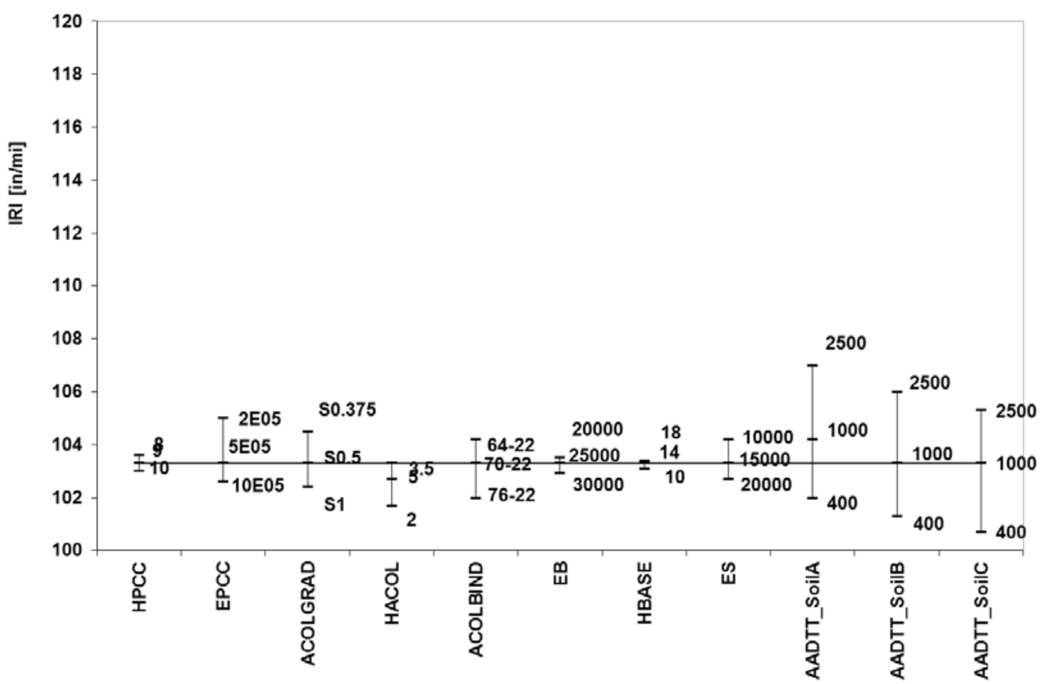

Figure 5.42. Relative effect of variables on IRI in AC-overlaid rubblized JPCP design located in INLAND climate

Summary of the AC-Overlaid JPCP Design Sensitivity to Inputs

This section discusses a combined effect of each input on cracking (i.e. combination of longitudinal, alligator, and thermal cracking) and rutting (combination of AC and total rutting). Table 5.25 summarizes the average logF values that determine overall cracking and rutting rankings of inputs. Those rankings are compared in Figures 5.43 through 5.45.

The following conclusions are provided for the AC-overlaid rubblized JPCP pavement design in respect to its sensitivity to the inputs in discussion:

- Overall, the M-EPDG cracking predictions show the highest sensitivity to anticipated traffic load, project location, fractured PCC slab support, and thickness of the overlay. Volumetric properties of the asphalt mix, subgrade stiffness, and thickness of the fractured PCC layer show moderate influence on the predicted cracking values.

- The M-EPDG rutting prediction models are highly sensitive to all site factors (AADTT, CLIMATE, and SUBGRADE), AC layer thickness, and AC binder properties. The volumetrics of the asphalt mix and the stiffness of fractured PCC affect rutting predictions to a moderate degree.

- The IRI output appears to be mostly controlled by location of the project (i.e., climate), whereas the AC layer inputs, subgrade, and traffic volume show lesser influence on IRI.

- In general, base modulus does not show any significant influence on either of the distresses considered in this analysis.

- It should be noted that a "moderate" ranking of some inputs in Table 5.25 does not diminish their importance for the design. The ranking is to be used for further recommendations on data collection to meet the required level of hierarchy (See Chapter 6).

Table 5.25. Summary of the combined sensitivity ranking of AC-overlaid rubblized JPCP design inputs

Cracking Ranking			**Rutting Ranking**			**IRI ranking**		
Predictor	Mean logF	Importance	Predictor	Mean logF	Importance	Predictor	Mean logF	Importance
EPCC	2.62	High	AADTT	2.77	High	CLIMATE	2.22	High
AADTT	1.81	High	CLIMATE	2.44	High	HACOL	0.68	Moderate
HACOL	1.23	High	HACOL	1.57	High	SUBGRADE	0.61	Moderate
CLIMATE	1.22	High	ACOLBIND	1.54	High	AADTT	0.56	Moderate
ACOLGRAD	0.82	Moderate	SUBGRADE	1.34	High	ACOLGRAD	0.53	Moderate
SUBGRADE	0.71	Moderate	EPCC	1.12	High	EPCC	0.05	Low
HPCC	0.62	Moderate	ACOLGRAD	0.87	Moderate	ACOLBIND	-0.05	Low
BASE	0.37	Low	BASE	0.41	Low	BASE	-0.48	Low
ACOLBIND	-0.36	Low	HPCC	0.36	Low	HPCC	-0.89	Low
HBASE	-1.30	Low	HBASE	-0.43	Low	HBASE	-1.30	Low

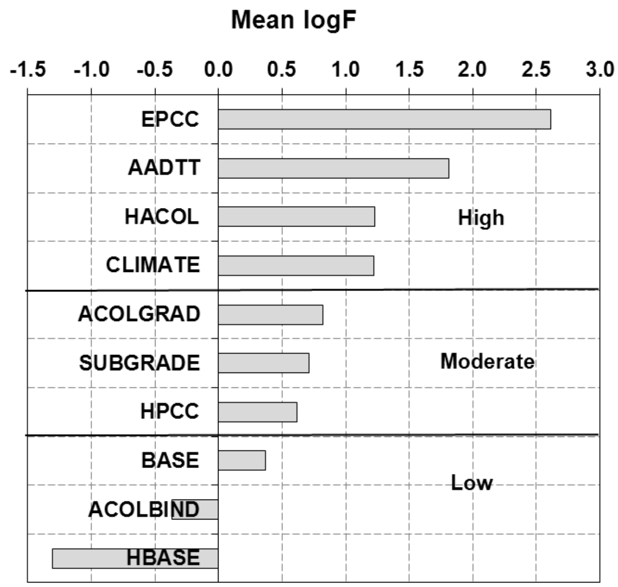

Figure 5.43. Cracking sensitivity to the AC-overlaid rubblized JPCP design inputs

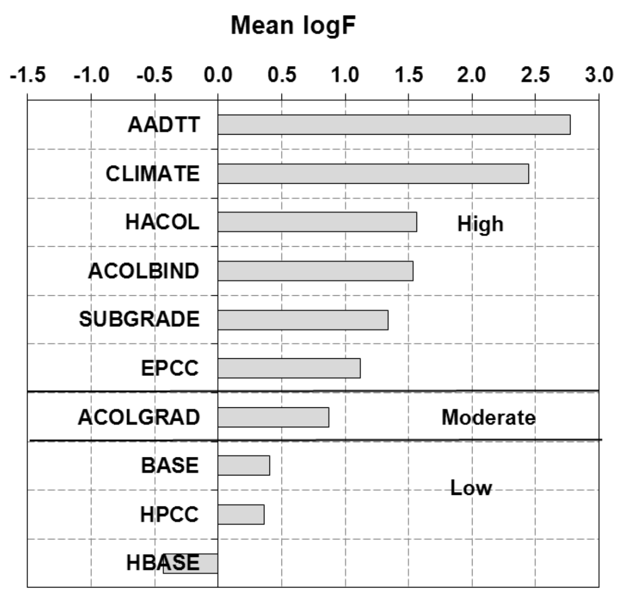

Figure 5.44. Rutting sensitivity to the AC-overlaid rubblized JPCP design inputs

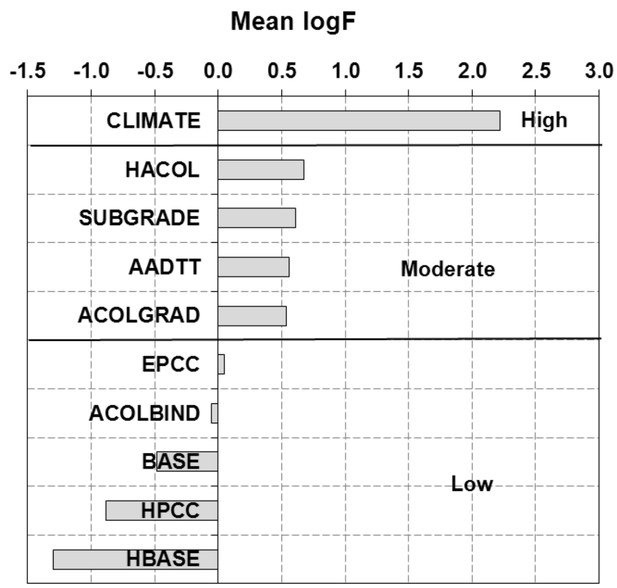

Figure 5.45. IRI sensitivity to the AC-overlaid rubblized JPCP design inputs

CHAPTER 6 Identification of Additional Data Collection Protocols to Meet M-EPDG Requirements

The sensitivity analysis of the M-EPDG prediction models identified the degree of impact for input variables on the predicted ouput value of a particular distress. The summary of the sensitivity results is provided in Chapter 5. Based on a ranking of an input for a targeted design (New AC, AC-overlaid AC, or AC-overlaid PCC pavement), the recommended level of hierarchy and a corresponding scope of testing required to meet that level should be established as part of the M-EPDG implementation process. The description of the hierarchical levels is provided in Chapter 3. The current chapter discusses assignment of hierarchical levels to inputs based on their importance ranking and provides recommendations on data collection and testing protocols to meet the M-EPDG requirements for that level.

Summary of M-EPDG Hierarchical Levels for Typical Connecticut Inputs

The importance of ranking each analyzed input was established with respect to the effect of that input on pavement performance expressed in cracking, rutting, or roughness. The importance rankings with respect to each of the three designs in consideration are summarized in Tables 6.1 through 6.3. Note that the overall ranking is assigned as an aggregate of the three sub-rankings (i.e. cracking, rutting, and IRI). It may be changed by ConnDOT based on the future calibration of the M-EPDG prediction models using actual pavement management data. The rankings in Tables 6.1 through 6.3 are superimposed with each of the levels of hierarchy and the corresponding requirements on data collection and testing.

CHAPTER 7 Evaluation of Need for Calibration of MEPDG Distress Prediction Models for Connecticut

As explained previously in Chapter 3, the distress prediction models incorporated in the M-EPDG software have been calibrated based on the data collected from the LTPP test sections spread across the continental U.S. and Eastern Canada. Due to wide variability in climatic conditions, subgrade types, and the local material properties, the nationally calibrated prediction equations may produce large errors for a particular project or even for the whole region such as New England. Therefore, the calibration and validation of the M-EPDG models are strongly recommended by AASHTO.

To evaluate the calibration needs for Connecticut, the research team identified the currently terminated SPS-9A project located on the Connecticut State Route 2 as a viable source of information. A well-documented construction history, pavement performance, and laboratory testing data provided real values for climatic, traffic, and material-related inputs that were used in the M-EPDG trial runs. Once the predicted deterioration curves were obtained, they were superimposed with the field trends to evaluate the errors. Based on the magnitude of prediction errors, recommendations on the calibration were made for each of the prediction models included in the sensitivity analysis. It should be noted that this study only provides an example based on a limited dataset, whereas the statewide calibration and validation study has to be implemented as a part of the M-EPDG implementation process outlined in Chapter 8. The next few sections describe input datasets, the trial validation results, and provide recommendations on calibration and validation for the M-EPDG models in Connecticut.

Description of the LTPP SPS-9A Sections on Route 2

The six Connecticut LTPP sections constructed in 1997 to serve as SPS-9A sites on the characterization of Superpave asphalt mixtures are aligned along a 10-km stretch of Route 2, between the towns of Lebanon and Bozrah. Route 2 is a four-lane, median-divided highway, functionally classified as a principal arterial. It is also a part of the National Highway System (Non-Interstate) of the U.S. According to the Connecticut Department of Transportation (ConnDOT), the average daily traffic on those LTPP sections increased from 20,000 AADT in 1998 to about 27,000 AADT in 2007, with approximately 10% trucks. (Larsen 1997) The cumulative traffic for 1997-2008 was 73.9 and 71.8 million vehicles for westbound and eastbound sections, respectively.

The original pavement structure of Route 2, constructed in 1970, consisted of a 10-in (250-mm) subbase, a 4-in (100-mm) calcium chloride stabilized base, a 6-in (150-mm) plant mix HMA base, and a 4-in (100-mm) surface course containing ConnDOT Class 1 HMA (NMAS of 12.5 mm) (Larsen 1997). It was overlaid in 1986 with 2-in (50-mm) HMA Class 114 course without milling. In 1997, the top 50 mm of pavement were replaced with a 25-mm ConnDOT Class 2 leveling course, overlaid with a 62.5-mm surface course of various HMA designs (Larsen 1997). In summary, two mix designs combined with three binder grades and two RAP contents (0 and 25 ± 5 percent) were designated for research purposes (Larsen 1997).

M-EPDG Inputs for the LTTPP SPS-9A Sections

Tables 7.1 and 7.2 summarize the traffic and material-related inputs for the six LTPP SPS-9A sections analyzed by the M-EPDG trial runs. The vast majority of input values, as well as the pavement performance data, were extracted from the construction and five-year evaluation reports (Larsen 1997. Larsen 2003). The traffic inputs were back calculated based on the WIM data provided by ConnDOT. The temperature, wind, and precipitation data for the Route 2 locations were interpolated from the nearest weather stations located in Windsor Locks, Willimantic, and Groton, CT. The location coordinates and elevations were found through Google Earth ® 2013, whereas the groundwater table was provided in the construction report (Larsen 1997). The M-EPDG simulations were performed for a 12-year design life period encompassing years 1997 through 2009.

Table 7.1. Traffic Inputs for SPS-9A Sections

Input Parameter	**Section 090901**	**Section 090902**	**Section 090903**	**Section 090960**	**Section 090961**	**Section 090962**
AADTT* [trucks]	580	580	580	597	597	597
Operational Speed [mi/hr]	55	55	55	55	55	55
Traffic Growth Rate	1.6%	1.6%	1.6%	1.9%	1.9%	1.9%
Vehicle Class Distribution	Eastbound			Westbound		
	Class 4	3.0%		Class 4	4.6%	
	Class 5	44.8%		Class 5	42.8%	
	Class 6	6.4%		Class 6	4.1%	
	Class 7	0.5%		Class 7	2.7%	
	Class 8	13.7%		Class 8	13.4%	
	Class 9	29.3%		Class 9	29.5%	
	Class 10	0.4%		Class 10	0.7%	
	Class 11	1.5%		Class 11	1.9%	
	Class 12	0.2%		Class 12	0.2%	
	Class 13	0.2%		Class 13	0.1%	

Table 7.2. Pavement Structure Inputs for SPS-9A Sections

Input Parameter	Section 090901	Section 090902	Section 090903	Section 090960	Section 090961	Section 090962
AC Surface Layer 1 Inputs						
Thickness [in]	2.3	2.4	2.1	2.4	2.2	2.2
Effective Binder content [%]	4.52	5	5	4.9	4.6	4.8
Air Voids [%]	3.3	4.8	4.1	3.5	5.0	5.2
Asphalt Mix Gradation						
% Retained ¾”	0	0	0	5	0	0
% Retained 3/8”	20	16	16	26	23	23
% Retained #4	45	44	44	45	55.3	55.3
% Passing #200	5	3.5	3.5	5	3.1	3.1
Asphalt Binder	AC-20	PG 64-28	PG 64-22	AC-20	PG 64-28	PG 76-22
Tensile Strength@14F [psi] (calculated)	738.3	894.5	765.6	738.3	988.1	946.7
Leveling AC Layer 2 Inputs						
Thickness [in]	1.1	1.5	1.4	1.0	1.1	1.6
Effective Binder content [%]	6.1					
Air Voids	3.5					
Asphalt Mix Gradation						
% Retained ¾”	0					
% Retained 3/8”	5					
% Retained #4	30					
% Passing #200	5					
Asphalt Binder	AC-20					
Existing AC Layer 3 Inputs						
Thickness [in]	4					
Effective Binder content [%]	5.8					
Air Voids [%]	4.5					
Asphalt Mix Gradation						
% Retained ¾”	5					
% Retained 3/8”	30					
% Retained #4	50					
% Passing #200	6					
Asphalt Binder	AC-20					

Table 7.2 Pavement Structure Inputs for SPS-9A Sections (Continued)

Input Parameter	**Section 090901**	**Section 090902**	**Section 090903**	**Section 090960**	**Section 090961**	**Section 090962**
Existing AC Premixed Base Layer 4 Inputs						
Thickness [in]	6					
Effective Binder Content [%]	5					
Air Voids [%]	2					
Asphalt Mix Gradation						
% Retained ¾”	30					
% Retained 3/8”	46					
% Retained #4	58					
% Passing #200	3					
Asphalt Binder	AC-20					
Granular Base Layer 5Inputs						
Material	A-1a					
Thickness	4					
Modulus [psi]	42,000					
Base Gradation						
% Passing #200	8.7					
% Passing #80	12.9					
% Passing #40	20					
% Passing #10	33.8					
% Passing #4	44.7					
% Passing 3/8”	57.2					
% Passing 1/2”	63.1					
% Passing 1 ½”	85.8					
% Passing 3”	97.8					
Subbase (Selected Borrow) Layer 6 Inputs						
Material	A-1-b					
Thickness	10					
Modulus [psi]	25,000					
Base Gradation						
% Passing #200	0-5					
% Passing #80	0-10					
% Passing #40	5-25					
% Passing #8	15-45					
% Passing #4	20-42					
% Passing 3/4”	45-80					
% Passing 1 ½”	100					

Table 7.2 Pavement Structure Inputs for SPS-9A Sections (Continued)

Input Parameter	Section 090901	Section 090902	Section 090903	Section 090960	Section 090961	Section 090962
Subgrade Layer 7 Inputs						
Material	A-3					
Thickness	Semi-infinite					
Modulus [psi]	18,500					
Base Gradation						
% Passing #200	5.2					
% Passing #80	33					
% Passing #40	76.8					
% Passing #10	93.4					
% Passing #4	95.3					
% Passing 3/4”	98					
% Passing 1 ½”	99					

M-EPDG Simulation Results for the LTPP SPS-9A Sections

Table 7.3 compares performance indicator values predicted by the M-EPDG with those reported after 12 years of service. Note that the prediction values are reported at 50 percent reliability (deterministic approach). In addition, to facilitate direct comparison with the M-EPDG units of measure (ft/mi), the field values are normalized to the mile length. It is obvious that none of the predicted values exactly match the measured ones, except for alligator cracking. In order to evaluate the errors and feasibility of calibration, predicted values are superimposed on the measured values to evaluate accuracy of fit by a linear regression. R-squared is used as a measure of the association between two datasets. The trendlines with regression equations along with R-squared values, are depicted in Figures 7.1 through 7.4 for longitudinal (wheelpath) and transverse cracking, total rutting, and IRI.

Table 7.3. M-EPDG-predicted versus measured distress in the LTPP SPS-9A sections

Section	Longitudinal Cracking (ft/mi)		Alligator Cracking (%)		Transverse Cracking (ft/mi)		Total Rutting (in)		IRI (in/mi)	
	M-EPDG	Field	M-EPDG	Field	M-EPDG	Field	M-EPDG	Field	M-EPDG	Field
090901	0	84	0.0000	0.0	149.0	222	0.039	0.144	85.1	74.0
090902	0	42	0.0000	0.0	0.0	63	0.043	0.152	84.0	80.1
090903	0	0	0.0000	0.0	4.8	0	0.041	0.170	84.0	77.7
090960	0	0	0.0001	0.0	187.0	84	0.039	0.170	85.4	64.4
090961	0.03	0	0.0000	0.0	0.2	53	0.052	0.203	84.4	74.7
090962	0	0	0.0001	0.0	2.2	0	0.034	0.146	83.7	78.0

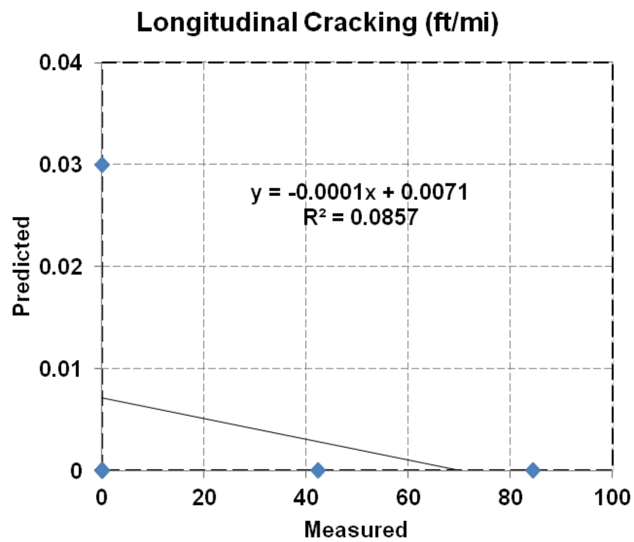

Figure 7.1. Predicted versus measured longitudinal (wheelpath) cracking

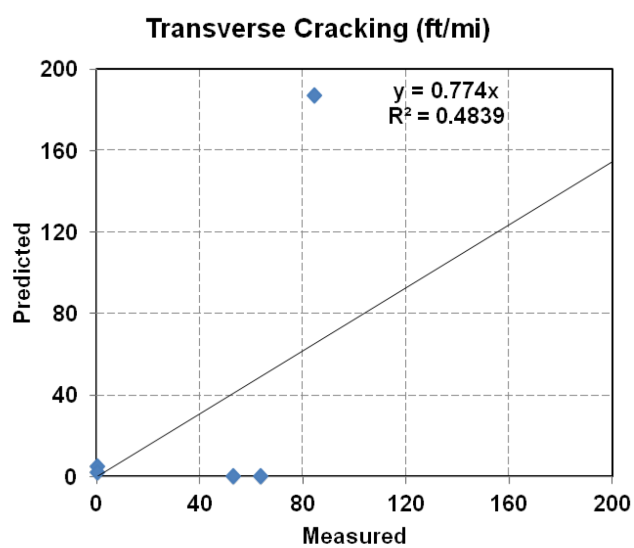

Figure 7.2. Predicted versus measured thermal (transverse) cracking.

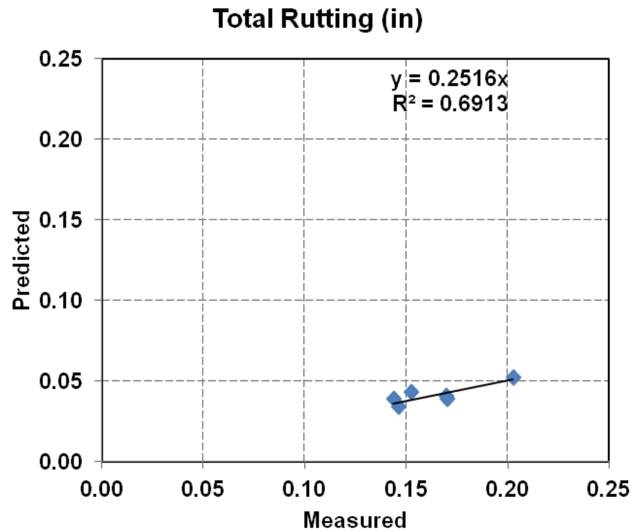

Figure 7.3. Predicted versus measured total rutting

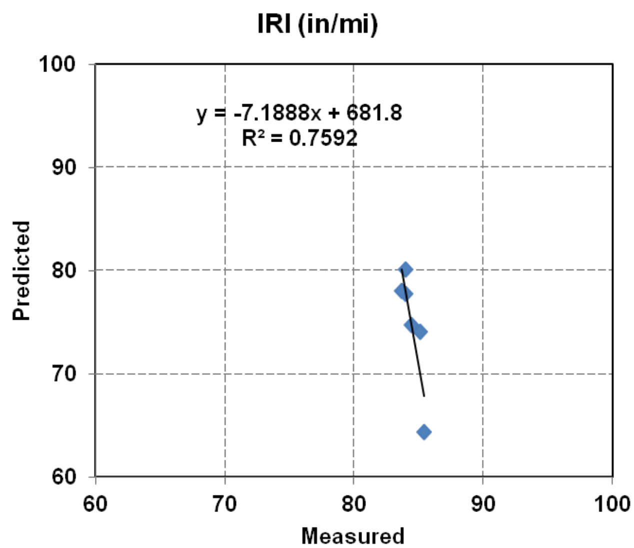

Figure 7.4. Predicted versus measured IRI

The following is a summary of the preliminary validation results for the chosen set of sections:

- **Longitudinal Cracking:** The M-EPDG predicted zero top-down fatigue for all sections at a reliability of 50 percent, whereas the condition survey revealed very few low severity cracks in the wheel path of sections 090901 and 090902. Because of prevalent zero values, it is impossible to correlate predicted output with the field measurements (consider R-squared=0.086 in Figure 7.1). However, as explained in Chapter 3, the 80 percent reliability predictions reach as high as 200 ft/mi. The calibration of a model with such a high built-in error appears impractical in general. Ultimately, more research should be done with use of better suited candidate sections where the extent of longitudinal cracking in the wheelpath is noticeable.

- **Alligator Cracking:** Due to a relatively low truck traffic volume (580-600 AADTT in one direction), as well as a "deep-strength" nature of the pavement structure on the Route 2, no alligator cracking has developed during the 12 years of service. The zero-values by the M-EPDG should not be, however, attributed to a good quality of predictions. Instead, it is recommended to choose a different set of pavement sections for the calibration of the fatigue cracking models because this type of distress is not typical for Connecticut State routes.

- **Thermal (Transverse) Cracking:** The results in Table 7.3 indicate the noticeable extent of transverse cracking in the sections paved with standard Class 1 (Marshall) mixes with lower air void content (sections 090901 and 090960), as compared with Superpave sections. It is notable that the M-EPDG model yielded similar predictions, albeit while underestimating the extent of thermal failure on average (77 percent of measured) at moderate goodness of fit (R-squared=0.48), as shown in Figure 7.2. In knowing that thermal cracking is one of the main distresses on asphalt surfaces in Connecticut, it is strongly recommended to consider this model for calibration.

- **Total Rutting:** It is impossible to distinguish the contribution of the AC layer to total rutting from the contribution of the unbound materials without coring the pavement structure. It is especially true for such low levels of rutting as detected on the Route 2 sections. Therefore, only total rutting model predictions are discussed in this report. In general, the M-EPDG underestimated total rutting for the given dataset by 25 percent of the measured rutting. However, the relatively high accuracy of fit for the linear trends depicted in Figure 7.3 (R-squared=0.69) suggest that a scaling factor can be applied to rutting predictions to adapt the model to the Connecticut environments. Ultimately, a larger dataset involving a wide range of traffic volumes and layer thicknesses should be utilized during the calibration.

- **IRI:** The verticality of the linear trend in Figure 7.4 clearly indicates no association between predicted and measured values for the given set of sections, regardless of the high R-squared value of 0.76. This outcome may be a result of the combined low predictability demonstrated by the fatigue cracking models. On the other hand, it may be a result of discrepancy in IRI measurements. At any rate, the calibration of the IRI model is possible if the field measurements are consistent with growth in roughness with pavement age.

In summary, it appears that all the M-EPDG models should be calibrated to local conditions. Special care should be taken with the fatigue (longitudinal and alligator) predictions where very low values were predicted for the thick pavements. It is recommended that ConnDOT allocate the resources for calibration and validation of all the M-EPDG models to facilitate creation of the design catalog, which in turn will save time and lower costs in the future pavement design activities.

CHAPTER 8 Implementation Plan and Recommendations for Future Research

The concluding task of this project is to develop a roadmap for the implementation of the M-EPDG by the Connecticut DOT. This roadmap includes a step-by-step outline of the activities and processes that should be undertaken to facilitate a change in design philosophy and adoption of the mechanistic-empirical approach to pavement design. The outline consists of 10 general steps, some of which have been or can be completed concurrently. It should be noted that this chapter only describes tentative activities proposed by the UConn research team, which should be finalized and approved by ConnDOT's M-EPDG Implementation Team.

1. Conduct sensitivity analysis of M-EPDG inputs.
   Note: Include soil typical of upper CT-river valley (clayey soil); include a thinner pavement structure (4" bound material over 6" of Processed Aggregate granular base on 10" Subbase); consider also other rehabilitation alternative(s) of AC over repaired PCC pavement and/or over AC/PCC pavement.
2. Recommend M-EPDG input levels and required resources to obtain those inputs.
3. Assemble a ConnDOT M-EPDG Implementation Team to develop and implement a communication plan.
4. Conduct staff training.
5. Develop formal ConnDOT-specific M-EPDG-related documentation.
6. Develop and populate a central database(s) with required M-EPDG input values.
7. Align distress data collection in Connecticut with the M-EPDG defined performance indicators.
8. Calibrate and validate M-EPDG performance prediction models to local conditions.
9. Define the long-term plan for adopting the M-EPDG design procedure as the official ConnDOT pavement design method.
10. Develop a design catalog.

The list of the above activities necessary for a successful implementation was developed based on previous work (Saeed 2003, Yut et al. 2007, Hoerner et al. 2007) and customized to address ConnDOT specific needs. Following is the explanation of each implementation step.

<u>Step 1. Conduct sensitivity analysis of M-EPDG inputs.</u>

This step has been completed as described in Chapters 4 and 5. The sensitivity analysis allowed the ability to differentiate the degree of influence of the individual inputs and specific input categories on the predicted extent of distress in a particular pavement structure. The following was done in this implementation step:

- The typical pavement designs along with representative input ranges were selected.
- The M-EPDG simulation runs were performed to establish the variation in the distress output.
- A comprehensive analysis of significance was conducted to rank the investigated inputs in order of their influence on the predicted overall performance of the typical pavement designs.

It should be noted that this study did not target large numbers of input as in the South Dakota study (Hoerner et al., 2007), nor the full factorial of input interactions as in Minnesota (Yut et al. 2007, Velasquez et al. 2009). Instead, the research team focused on the pavement features and material properties within the range that is typical for Connecticut. Due to relatively small variations in some inputs prescribed by current ConnDOT specifications, such as: unbound material moduli; base thickness; and mix design parameters; , the sensitivity of the M-EPDG software to those inputs appeared to be low, in some cases. In the event ConnDOT decides to pursue optimization of design by changing the range of inputs, a sensitivity analysis of the new input range is recommended.

<u>Step 2. Recommend M-EPDG input levels and required resources to obtain those inputs</u>

Hierarchical levels, as prescribed by the M-EPDG, were assigned to each input, based on the degree of influence of the investigated inputs on the predicted pavement performance. . Furthermore, based on the hierarchical level, a scope of data collection and material testing was recommended. Note that for the moment, those recommendations are tentative, while pending discussion and approval by ConnDOT. To finalize those recommendations, the following tasks should be completed:

- Determination of gaps between the current ConnDOT data collection/testing protocols and the required data and testing for the recommended M-EPDG input levels.
- Assessment of ConnDOT data sources for new sampling or testing procedures that are required to close these identified gaps.

<u>Step 3. Assemble a ConnDOT M-EPDG Implementation Team and develop and implement a communication plan</u>

In understanding the complexity of the new Design Guide and the challenges presented by the need for its calibration to produce reliable solutions, it is recommended that ConnDOT assemble an Implementation Team to champion the transition from the AASHTO1986(1993) design procedure to the M-EPDG. The team would include both overseeing and technical committees. The overseeing committee is expected to have representatives from the major stakeholders; that is, office of ConnDOT Commissioner, and the asphalt industry. The technical committee would consist of ConnDOT personnel who specialize in the following areas:

- Traffic data collection and analysis
- Asphalt binder and mix characterization, sampling, and testing
- Unbound materials (aggregates) characterization, sampling, and testing
- Pavement management, including maintenance and rehabilitation
- Climatic data (weather, precipitation, depth of groundwater table)
- M-EPDG performance model calibration and validation
- Personnel training

The Implementation Team is recommended to undertake the following activities:

a) Assign the specific responsibilities for the ConnDOT personnel involved with M-EPDG implementation and future use.
b) Develop an approach and establish a schedule for completing the implementation steps in hand.
c) Deliver the necessary training to all personnel involved with M-EPDG implementation and future use.
d) Hold regular meetings to keep all informed of the progress in all M-EDPG-related activities.

Step 4. Conduct staff training.

The M-EPDG requires using a sophisticated software package as well as the need for an innovative approach to data collection and interpretation of testing results. Therefore, the training of the personnel involved with M-EPDG is critical for the success of its implementation. The recommendations on the training approach are based on previously published work (Coree et al. 2005, Hoerner et al. 2007), and are described in more detail in Chapter 9.

Step 5. Develop formal ConnDOT-specific M-EPDG-related documentation.

There is a large number of data inputs required for an M-EPDG analysis, which is also designed to be customized for a given agency. Therefore, it seems reasonable for ConnDOT to develop some formal guidelines for the personnel involved in the use and calibration of the M-EPDG, and also for the third parties conducting design for ConnDOT. Those documents may include but not be limited to:

- **M-EPDG Pavement Design Procedural Manual** (Baus and Stires 2010, Bayomy et al. 2012) to outline a step-by-step procedure that could be easily implemented by a pavement designer.
- **M-EPDG Material Characterization Guidelines** to document; 1) the different acceptable M-EPDG input levels associated with each material-related input; 2) the recommended M-EPDG input level for each input; 3) the MEPDG level-specific laboratory and field testing protocols (if applicable), and 4) acceptable default values for some inputs.

The above guidelines can be based on the M-EPDG documentation available elsewhere (www.trb.org/mepdg, AASHTO 2008) as well as on the information provided in this report.

Step 6. Develop and populate a central database(s) with required M-EPDG input values.

To unify the process of input collection during the design, it may be necessary to create a central database where some global (non-project specific) inputs would be stored. The examples of such inputs are provided:

- **Climatic data**: Ideally, the M-EPDG-generated or -interpolated weather station data for DOT districts may be stored in climatic files.

- **Traffic data:** The default values for some traffic-related inputs such as vehicle class distribution and hourly truck distribution (not discussed here – refer to Hoerner et al. 2007) can be centrally stored.

- **Default material property data**: Such data can include asphalt binder PG for the districts, volumetrics of the mix, aggregate gradation and other properties included, for example, in FORM 816, Section M.04 specifications.

<u>Step 7. Align distress data collection in Connecticut with the M-EPDG defined performance indicators.</u>

The calibration of the M-EPDG distress prediction models requires the use of pavement management data, specifically, construction history, and pavement performance trends. Pavement performance should be defined and expressed in units of measure compatible with the default M-EPDG performance indicators (e.g. longitudinal, reflection, and alligator cracking). The Connecticut Pavement Preservation Manual defines the distresses in accordance with the LTPP Distress Manual (ConnDOT 2011), which is in full compliance with the M-EPDG. The units of measure for transverse and longitudinal cracking, however, are expressed in full-width and full-length equivalents, correspondingly, rather than in ft/mi as prescribed by the M-EPDG. Therefore, it is envisioned that for the purpose of pavement design, the appropriate units of measure will be used.

<u>Step 8. Calibrate and validate MEPDG performance prediction models to local conditions.</u>

In order to produce a reliable design, the M-EPDG distress prediction models should be calibrated on the historical performance trends obtained from the pavement management system. Next, the adequacy of the predictions should be validated on an independent set of data. The calibration term refers to the mathematical process through which the total error or difference between observed and predicted values of distress is minimized (NCHRP 2003b). The validation is performed to confirm that the calibrated model can produce robust and accurate predictions for cases other than those used for model calibration (NCHRP 2003b). The NCHRP Research Digest No. 284 recommends the split-sample jackknifing approach that uses a single database to both calibrate and validate a given model. This method is instrumental in saving time and expenditures on distress data collection (NCHRP 2003b). More detailed information on the use of the split-sample jackknifing approach is available elsewhere (NCHRP 2003a, NCHRP 2003b, and AASHTO 2010).

<u>Step 9. Define the long-term plan for adopting the M-EPDG design procedure as the official ConnDOT pavement design method</u>.

AASHTO officially recognized the M-EPDG procedure in 2008. However, in order to achieve full acceptance of the M-EPDG as the official design procedure in Connecticut, more experience with the calibrated/validated models must be obtained. Therefore, it is recommended that ConnDOT begin evaluating the accuracy and consistency of the M-EPDG output as soon as possible. Tentatively, the proposed long-term schedule for the implementation requires three years (Table 8.1). Throughout this 3-year period, it is recommended that both the AASHTO 1986 and M-EPDG analyses should be conducted for every pavement design. The primary goal of this exercise is to produce and review expected performance data for given pavement designs, with the ultimate goal of gaining confidence in the MEPDG predicted performance. All selected M-EPDG inputs and collected performance data should be recorded and stored so they can be used in future calibration and validation efforts. The decision to adopt the M-EPDG for pavement design is a decision that should not be made until the implementation team members have great confidence that the calibrated and validated M-EPDG performance models are predicting distress values that are reasonable and considered to be acceptably accurate for Connecticut conditions.

Step 10. Develop a design catalog

It is understood that running multiple M-EPDG simulations to achieve an optimal design for every ConnDOT project is time consuming and hence impractical. Instead, a design catalog can be developed after gaining confidence with the calibrated M-EPDG distress prediction models. The concept of the design catalog employs multiple design alternatives to achieve an optimal design. Such a design will employ a particular range of inputs (layer thicknesses and material properties) that would yield an overall satisfactory performance in specified site conditions (climate, traffic and subgrade). For example, in the development of such a catalog, M-EPDG runs representing different combinations of site conditions (climate, traffic, and subgrade) and design features (layer thickness, slab geometry, dowel diameter, and so on) would be conducted ahead of time. Based on selected performance limits (e.g., 15 percent of fatigue cracking and 0.2 inches of rutting), an expected pavement life would be computed for each hypothetical design. By compiling results associated with enough combinations of typical design inputs, it is envisioned that eventually, a pavement design engineer could use the information recorded in the design catalog to select a given design, rather than have to use the software to simulate a given scenario.

Table 8.1. Projected timeline for the M-EPDG implementation

Implementation Step	Complete	Year1	Year 2	Year 3	Future Activity
1. Conduct sensitivity analysis of M-EPDG inputs.	**X**				
2. Recommend M-EPDG input levels and required resources to obtain those inputs.	**X**				
3. Assemble a ConnDOT M-EPDG Implementation Team and develop and communication plan.		**X**			
4. Conduct staff training.		**X**			
5. Develop formal ConnDOT specific MEPDG-related documentation.			**X**	**X**	
6. Develop and populate a central database(s) with required M-EPDG input values.		**X**	**X**	**X**	
7. Align distress data Collection in Connecticut with the M-EPDG definitions			**X**		
8. Calibrate and validate M-EPDG performance prediction models to local conditions.			**X**	**X**	
9. Define the long-term plan for adopting the M-EPDG design procedure as the official ConnDOT Pavement Design Method		**X**			
10 Develop design catalog.					X

CHAPTER 9 Development of M-EPDG Training Course for ConnDOT Designated Personnel

The M-EPDG approach associated with the new 2008 AASHTO Pavement Design Guide is markedly different from that in the previous 1993 AASHTO procedure. Therefore, it is anticipated that ConnDOT will allocate resources for training all the personnel involved in the M-EPDG implementation process. In line with the recommendations by previous reviewed work (Hoerner et al. 2007), the research team envisions the training will not only involve the to-be assembled ConnDOT M-EPDG Implementation Team, but also include ConnDOT pavement designers, laboratory personnel, and pavement management specialists. In addition, some external personnel who conduct business with the ConnDOT may be involved in training. Figure 9.1 illustrates the flow of the training process.

The training materials will arrive from the variety of sources included but not limited to those provided on the federal level, web resources, college courses and publications. The training can involve on-line and posted handouts, classroom delivery, webinars, and workshops. The FHWA training courses related to the M-EPDG are listed in Table 9.1. The formal M-EPDG documentation is provided with the M-EPDG software (also on www.trb.org/mepdg) as well as summarized in the 2008 AASHTO Pavement Design Guide (AASHTO 2008). Lastly, at this time, the research team has volunteered the course materials developed for the Pavement Design class taught at the Civil Engineering Department at UConn. Ultimately, those materials can be delivered during a workshop at the chosen ConnDOT location. Appendix B includes the copies of the PowerPoint presentations delivered during the academic year 2012-2013.

Table 9.1, FHWA Training Courses Recommended for the Implementation Team

Course No.	Course Title	Note
NHI #131064	Introduction to Mechanistic Design for New and Rehabilitated Pavements	The general framework of the mechanistic-empirical design procedure and the individual components are discussed in detail. The course includes several hands-on exercises pertaining to materials characterization, structural response calculations, pavement performance prediction, and mechanistic-empirical pavement design..
NHI #132040	Geotechnical Aspects of Pavements	The course content includes geotechnical exploration and characterization of in-place and constructed subgrades; design and construction of subgrades and unbound layers for paved and unpaved roads, with emphasis on the American Association of State Highway Transportation Officials (AASHTO) 1993 empirical design procedure and on the new Mechanistic-Empirical Pavement Design Guide (M-EPDG)
NHI #151018	Application of the Traffic Monitoring Guide	This training covers the application of procedures used as published in the FHWA's "Traffic Monitoring Guide" (TMG) and other recent developments in traffic monitoring, including an overview of the application of the TMG procedures to develop data and information needed to support state and national programs including the Highway Performance Monitoring System (HPMS), pavement management, safety management, congestion management, and environmental management

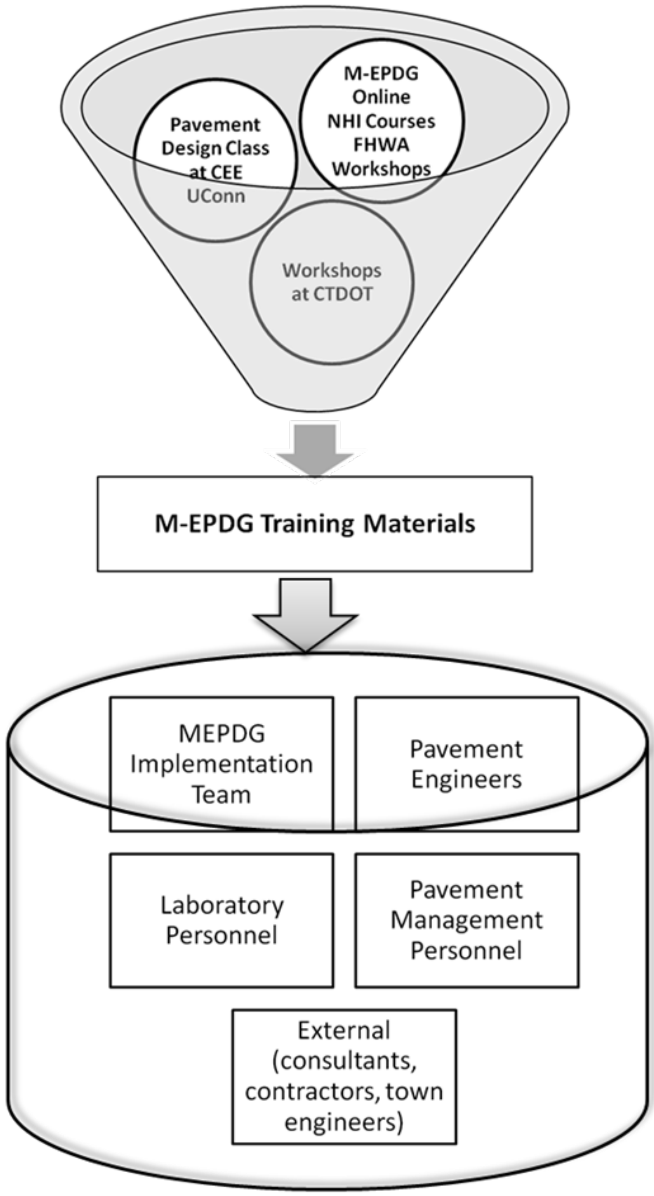

Figure 9.1. M-EPDG training flow chart

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Appendix A. Summary of the M-EPDG Inputs for Sensitivity Analysis

This Appendix presents summary of typical traffic-, structure-, material-, and climate-related inputs for newly constructed HMA, HMA-overlaid HMA, and HMA-overlaid rubblized JPCP pavement designs. The series of tables are organized by the category of input in the order of appearance on the M-EPDG User Interface.

Table A.1. General information inputs

Input Name	Units	Input Type	Value(s)	Notes
Design Life	years	Fixed	20	Default value
Base/Subgrade Construction Month	month/ year	Fixed	June 2006	Default value
Pavement Construction Month	month/ year	Fixed	August 2006	Default value
Traffic Open Month	month/ year	Fixed	October2 006	Default value
Type of Design		Variable	• New Flexible Pavement • Asphalt Concrete Overlay/ AC over AC • Asphalt Concrete Overlay/ AC over JPCP (fractured)	Chosen after consultations with CTDOT

Table A.2. Site/Project information inputs

Input Name	Units	Input Type	Value(s)	Notes
Location	n/a	Info only	Not used	The information only inputs are used for documentation purposes only
Project ID	n/a	Info only	Not used	
Section ID	n/a	Info only	Not used	
Date	n/a	Info only	Not used	
Station/milepost format	n/a	Info only	Not used	
Station/milepost begin	n/a	Info only	Not used	
Station/milepost end	n/a	Info only	Not used	
Traffic Direction	n/a	Info only	Not used	

Table A.3. Analysis parameter inputs

Input Name	Units	Input Type	Limit Value	Reliability	Notes
Initial IRI	in/mile	Fixed	62	90%	Default Value
Terminal IRI	in/mile	Fixed	178	90%	Default Value
AC Surface-Down Cracking (Longitudinal Cracking)	ft/mi	Fixed	1000	90%	Default Value
AC Bottom-Up cracking (Alligator Cracking)	%	Fixed	25	90%	Default Value
AC Thermal Fracture	ft/mi	Fixed	1000	90%	Default Value
Chemically Stabilized Layer Fatigue Fracture	%	Fixed	Not used	Not used	Not used
Permanent Deformation – Total Pavement	in	Fixed	0.43	90%	Default Value
Permanent Deformation - AC	in	Fixed	0.43	90%	Default Value

Table A.4. Traffic inputs

Input Name	Units	Input Type	Value(s)	Notes
Main Traffic Inputs				
Initial two-way AADTT	trucks	Variable	400 (low) 1000 (base) 2500 (high)	1.9 mln ESALs (Level 2[1)]) 4.2 mln ESALs (Level 3 Medium volume[1]) 12.1 mln ESALs (Level 3 High volume[1])
Number of lanes in design direction	units	Fixed	1	
Percent of trucks in design direction	%	Fixed	55	Assumed value[2]
Percent of trucks in design lane	%	Fixed	100	Assumed value[2]
Operational speed	mph	Fixed to functional class	40 55 70	Level 2 Level 3 Medium volume Level 3 High volume
Traffic Volume Adjustment Factors				
Monthly adjustment dactor (MAF)	N/A	Fixed	1	Level 3 M-EPDG default for all months and vehicle classes
Vehicle class distribution	%	Fixed to functional class and traffic level	Level 3 M-EPDG default	Principal Arterials-Interstate – Level 3 high Principle arterial (others) – Level 3 medium Minor Arterial – Level 2
Truck hourly distribution	%	Fixed	Level 3 M-EPDG default	Level 3 M-EPDG default table used for all runs
Axle Load Distribution Factors				
Axle factors by axle type	units	Fixed	Level 3 M-EPDG default	Level 3 M-EPDG default table used for all runs
General Traffic Inputs				
Mean wheel location	in	Fixed	18	MEPDG default value
Traffic wander standard deviation	in	Fixed	10	MEPDG default value
Design lane width	ft	Fixed	12	MEPDG default value
Number of axle types per truck class	units	Fixed	Level 3 M-EPDG default	Level 3 M-EPDG default table used for all runs

[1]Table M.04.03-4 (CTDOT)
[2]Assumed to arrive at designated ESALs

Table A.5. Climatic inputs

Input Name	Units	Input Type	Value(s)	Notes
Climate I (SHORE) Inputs				
Latitude	degrees	Fixed	41.10	Generated from the M-EPDG climatic data for Bridgeport, CT
Longitude	degrees	Fixed	-73.09	
Elevation	ft	Fixed	11	
Depth of water table	ft	Fixed	10	Assumed
Climate II (INLAND) Inputs				
Latitude	degrees	Fixed	41.44	Generated from the M-EPDG climatic data for Hartford CT
Longitude	degrees	Fixed	-72.39	
Elevation	ft	Fixed	18	
Depth of water table	ft	Fixed	10	Assumed
Climate III (MOUNT) Inputs				
Latitude	degrees	Fixed	41.92	Location: Putnam, CT Interpolated from the M-EPDG climatic data for Worchester, MA
Longitude	degrees	Fixed	-71.89	
Elevation	ft	Fixed	415	
Depth of water table	ft	Fixed	100	Assumed

Table A.6. Structure inputs

Input Name	Units	Input Type	Value(s)	Notes
General Structure Inputs				
Surface short-wave absorptivity	units	Fixed	0.9	M-EPDG default value – single input for the whole structure
Interface	units	Fixed	1 - for AC and granular base; n/a - for subgrade	M-EPDG default value – single input for each layer
Rehabilitation Level	units	Fixed	3	For AC-overlaid AC design only
Milled Thickness	in	Variable	1-Low 2-Baseline 3-High	For AC-overlaid AC design only
Pavement rating	N/A	Variable	Poor-Low Fair-Baseline Good-High	For AC-overlaid AC design only
Total Rutting (existing pavement)	in	Variable	0-Low 0.5-Baseline 1-High	For AC-overlaid AC design only

Table A.7. HMA Design Properties inputs

HMA E* Predictive Model
NCHRP 1-37A Viscosity based model (nationally calibrated)
HMA Rutting Model Coefficients
NCHRP 1-37A coefficients (nationally calibrated)
Fatigue Endurance Limit
Not set
Reflective Cracking Analysis
Included

Table A.8. Asphalt Concrete material inputs

Input Name	Units	Input Type	Value(s)	Notes
Asphalt Mix inputs (Aggregate Gradation)				
Cumulative % Retained 3/4“ sieve	%	Fixed	0	S0.375 CTDOT mix S0.5 CTDOT mix
Cumulative % Retained 3/8“ sieve	%	Fixed	5 – Layer 1 20-Layer 2 20 – Overlay, Layer 3	S0.375 CTDOT mix S0.5 CTDOT mix S1.0 CTDOT mix
Cumulative % Retained #4 sieve	%	Fixed	25 – Layer 1 37 – Layer 2 36 – Overlay, Layer 3	S0.375 CTDOT mix S0.5 CTDOT mix S1.0 CTDOT mix
% Passing #200 sieve	%	Fixed	6	S0.375 CTDOT mix S0.5 CTDOT mix S1.0 CTDOT mix
Asphalt Binder inputs				
Option	n/a	Fixed	PG XX-XX	Superpave binder grading
PG grade	n/a	Variable	64-22 70-22 (Base) 76-22	A=10.98; VTS=-3.68 (generated by M-EPDG A=10.299; VTS=-3.426 (generated by M-EPDG A=9.71; VTS=-3.208 (generated by M-EPDG
Asphalt General inputs				
Reference temperature	°F	Fixed	70	M-EPDG default value
Poisson’s Ratio	units	Fixed	0.35	M-EPDG default value
Effective binder content	%	Fixed	5.5 5.4 4.9 4.8 4.5 4.4	Surface S0.375, Traffic Level 2 Surface S0.375, Traffic Level 3 Surface S0.5, Traffic Level 2 Surface S0.5 Traffic Level 3; HMA Base Surface S1.0 Traffic Level 2 Surface S1.0 Traffic Level 3
Air voids	%	Fixed	4	CTDOT req. for all AC layers
Total unit weight	pcf	Fixed	148 150	Surface and binder courses HMA base course
Thermal conductivity	BTU/hr-ft-F°	Fixed	0.67	M-EPDG default for all AC layers
Heat capacity	BTU/lb-F°	Fixed	0.23	M-EPDG default for all AC layers

Table A9. Thermal Cracking Inputs (Generated by the M-EPDG siftware based on binder PG and aggregate gradation)

Binder Grade	Loading Time [sec]	Creep Compliance [1/psi]			Tensile Strength at 14°F [psi]
		-4°F	14°F	32°F	
PG64-22	1	8.65888e-008	1.5345e-007	2.20157e-007	701.16
	2	9.3778e-008	1.76916e-007	2.77215e-007	
	3	1.04206e-007	2.13532e-007	3.75944e-007	
	10	1.12857e-007	2.46186e-007	4.73378e-007	
	20	1.22228e-007	2.83834e-007	5.96065e-007	
	50	1.35819e-007	3.42578e-007	8.08349e-007	
	100	1.47095e-007	3.94966e-007	1.01785e-006	
PG 70-22	1	1.10534e-007	1.85932e-007	2.65104e-007	715.76
	2	1.18949e-007	2.12863e-007	3.2899e-007	
	3	1.31064e-007	2.54543e-007	4.37659e-007	
	10	1.41042e-007	2.91412e-007	5.43129e-007	
	20	1.5178e-007	3.33623e-007	6.74015e-007	
	50	1.67239e-007	3.98947e-007	8.96648e-007	
	100	1.79971e-007	4.56733e-007	1.11273e-006	
PG 76-22	1	1.38092e-007	2.21502e-007	3.14035e-007	728.89
	2	1.47917e-007	2.52293e-007	3.85617e-007	
	3	1.61985e-007	2.99662e-007	5.05874e-007	
	10	1.73509e-007	3.41318e-007	6.21184e-007	
	20	1.85854e-007	3.88765e-007	7.62777e-007	
	50	2.0353e-007	4.61757e-007	1.00065e-006	
	100	2.1801e-007	5.25947e-007	1.22875e-006	

Table A. 10. Unbound layer material inputs

Input Name	Units	Input Type	Value(s)	Notes
Granular base inputs: Strength Properties				
Unbound material type	n/a	Fixed	A-1-a	For all base types
Poisson's ratio	units	Fixed	0.35	M-EPDG default value
Coefficient of lateral pressure, K_0	units	Fixed	0.5	M-EPDG default value
Modulus (Representative value)	psi	Variable	20,000 25,000 30,000	Assumed for CTDOT Grading A Assumed for CTDOT Grading B Assumed for CTDOT Grading C
Granular base inputs: ICM inputs				
Gradation			#200: 0-5 #100: 0-10 #40: 5-25 #10: 15-45 #4: 20-52 1 ½": 55-100 3 ½": 90-100	CTDOT Grading A
	%Pass. Range	Variable	#200: 0-5 #100: 0-10 #40: 5-25 #10: 15-45 #4: 20-52 1 ½": 55-95 3 ½": 90-100	CTDOT Grading B
			200: 0-5 #100: 0-10 #40: 5-25 #10: 15-45 #4: 20-52 3/4": 45-80 1 ½": 100	CTDOT Grading C
Plasticity Index (PI)	units	Fixed	1	For all base types
Liquid Limit (LL)	units	Fixed	6	For all base types
Compacted Layer	Y/N	Fixed	Yes	For all base types
Index Properties from Sieve Analysis	n/a	Computed	Computed	Computed by the ICM from gradation, PI, and LL.
User Overridable Index Properties	n/a	Computed	Computed	Computed by the ICM from gradation, PI, and LL.
User Ovrridable Soil Water Characteristic Curve Parameters	n/a	Computed	Computed	Computed by the ICM from gradation, PI, and LL.
Subgrade inputs: Strength Properties				
Unbound material type	n/a	Fixed	A-1-b	For all soil types
Poisson's ratio	units	Fixed	0.35	M-EPDG default value
Coefficient of lateral pressure, K_0	units	Fixed	0.5	M-EPDG default value

Modulus (Representative value)	psi	Variable	10,000 15,000 20,000	Soil A Soil B Soil C
Subgrade inputs: ICM inputs				
Gradation	%Pass. Mean	Fixed	#200: 13.4 #80: 20.8 #40: 37.6 #10: 64 #4: 75 3/8“: 82.3 1/2“: 85.8 3/4”: 90.8 1”: 93.6 1 ½”: 96.7 2”: 98.4 3 ½”: 99.4	Soil A:
	%Pass. Mean	Variable	#200: 13.4 #80: 20.8 #40: 37.6 #10: 64 #4: 75 3/8“: 82.3 1/2“: 85.8 3/4”: 90.8 1”: 93.6 1 ½”: 96.7 2”: 98.4 3 ½”: 99.40	Soil B
	%Pass. Mean		#200: 13.4 #80: 20.8 #40: 37.6 #10: 50 #4: 74.2 3/8“: 82.3 1/2“: 85.8 3/4”: 90.8 1”: 93.6 1 ½”: 96.7 2”: 98.4 3 ½”: 99.4	Soil C
Plasticity Index (PI)	units	Fixed	1	For all base types
Liquid Limit (LL)	units	Fixed	11	For all base types
Compacted Layer	Y/N	Fixed	Yes	For all base types
Index Properties from Sieve Analysis	n/a	Computed	Computed	Computed by the ICM from gradation, PI, and LL.
User Overridable Index Properties	n/a	Computed	Computed	Computed by the ICM from gradation, PI, and LL.
User Ovrridable Soil Water Characteristic Curve Parameters	n/a	Computed	Computed	Computed by the ICM from gradation, PI, and LL.

Table A.11. Existing Fractured JPCP Inputs

Input Name	Units	Input Type	Value(s)	Notes
Layer Thickness	in	Variable	8-Low 9-Baseline 10-High	For AC-overlaid Rubblized JPCP only
Unit Weight	pcf	Fixed	150	For AC-overlaid Rubblized JPCP only
Poisson's Ratio	unitless	Fixed	0.2	For AC-overlaid Rubblized JPCP only
Elastic Resilient Modulus	pci		200,000-Low 500,000-Baseline 1,000,000 - High	For AC-overlaid Rubblized JPCP only
Fracture Type	N/A	Fixed	Rubblization	For AC-overlaid Rubblized JPCP only
Thermal Conductivity	BTU/hr-ft-F°	Fixed	1.25	For AC-overlaid Rubblized JPCP only
Heat Capacity	BTU/lb-F°	Fixed	0.28	For AC-overlaid Rubblized JPCP only

Appendix B Training Materials

The reader will find the training resources hosted by the University of Connecticut at the following URL:

http://www.cti.uconn.edu/caplab/wp-content/uploads/sites/2/2014/09/Appendix_B_MEPDG_Training_UConn.pdf

Appendix B. M-EPDG Training Materials (UConn)

Connecticut Department of Transportation

M-E PDG Training Module I

Overview

Prepared by Dr. Iliya Yut

Department of Civil Engineering, UConn

May2013

Evolution of Flexible Pavement Design

Empirical

WSDOT 1948	AASHTO 1961	AASHTO 1972	AASHTO 1986-1993
(h=f[CBR, Traffic])	(SN=f[Mr, h, Traffic, Reliability]		

Mechanistic-Empirical

AI
1982

(Cracking/Rutting Damage = f[ε, E, Traffic, T, seasonal Mr])

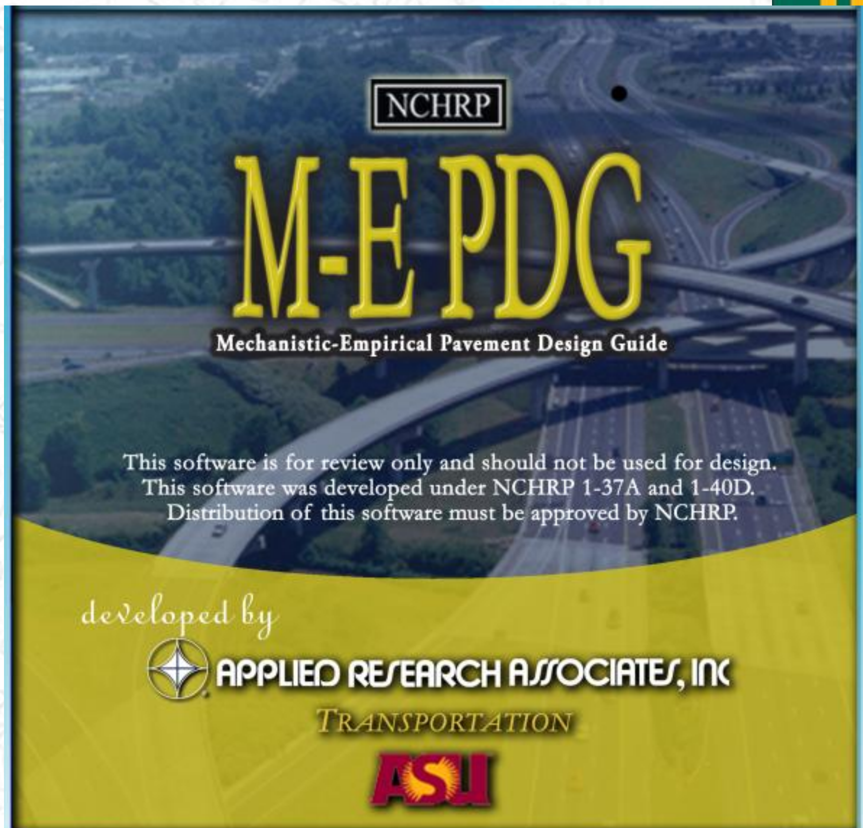
AASHTO
2004 (NCHRP 1-37)

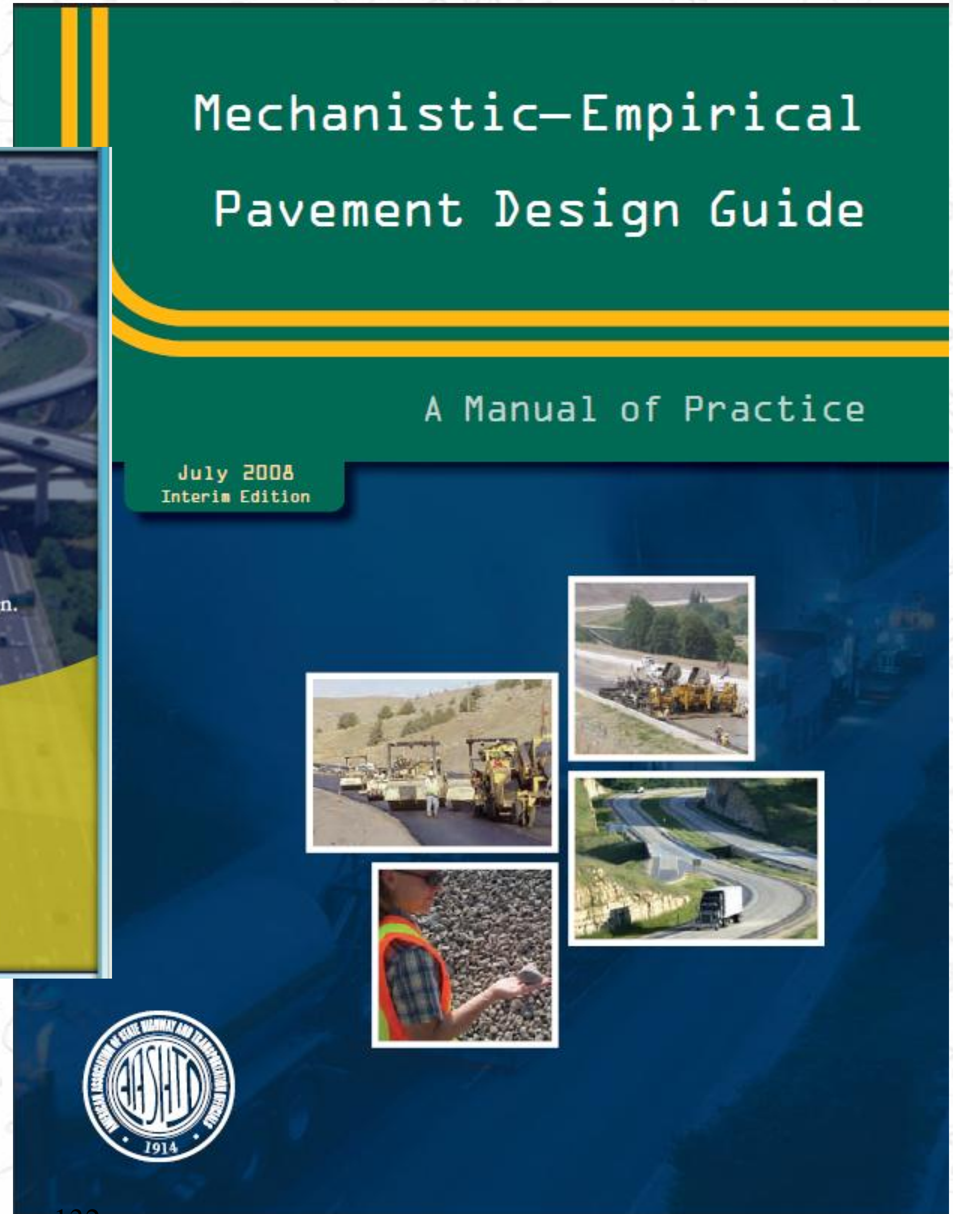
MEPDG
2008

(Distress = f[traffic parameters, pavement thickness, material properties, temperature, moisture)

Outline

- Overview of the M-E PDG
- Design Inputs
  - Traffic
  - Subgrade
  - Material Characterization
  - Reliability
  - Environmental effects


NCHRP
M-E PDG
Mechanistic-Empirical Pavement Design Guide
This software is for review only and should not be used for design.
This software was developed under NCHRP 1-37A and 1-40D.
Distribution of this software must be approved by NCHRP.
developed by
APPLIED RESEARCH ASSOCIATES, INC
TRANSPORTATION
ASU

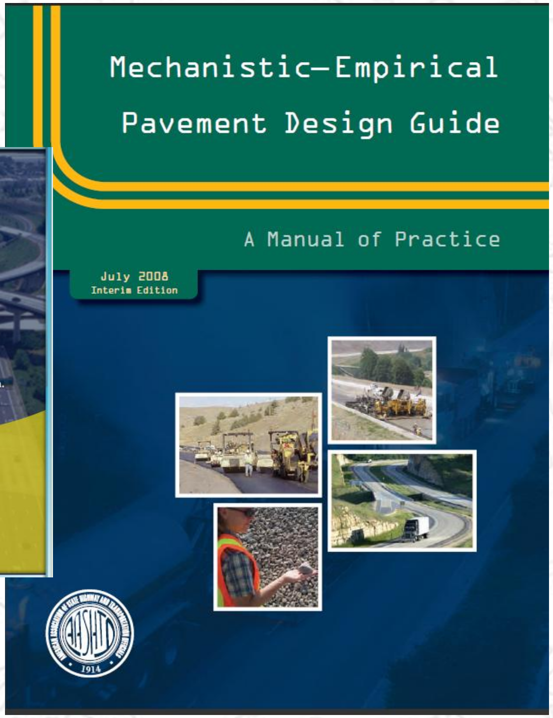

Mechanistic–Empirical
Pavement Design Guide
A Manual of Practice
July 2008
Interim Edition
AASHTO
1914

M-EPDG Objective

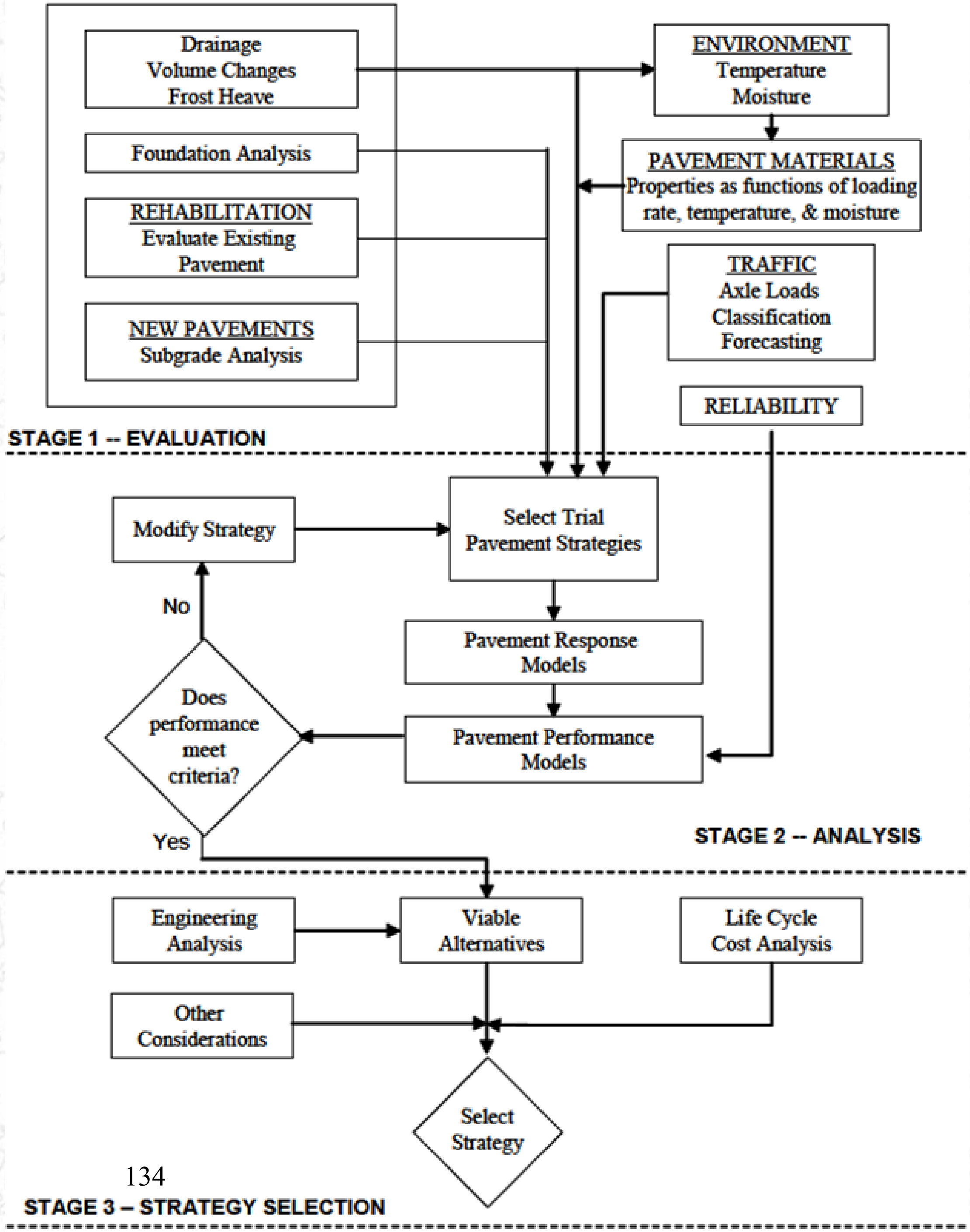
- ⇨ *To provide the highway community with a state-of-the-practice tool for the design of new and rehabilitated pavement structures, based on mechanistic-empirical procedures.*

M-EPDG Content

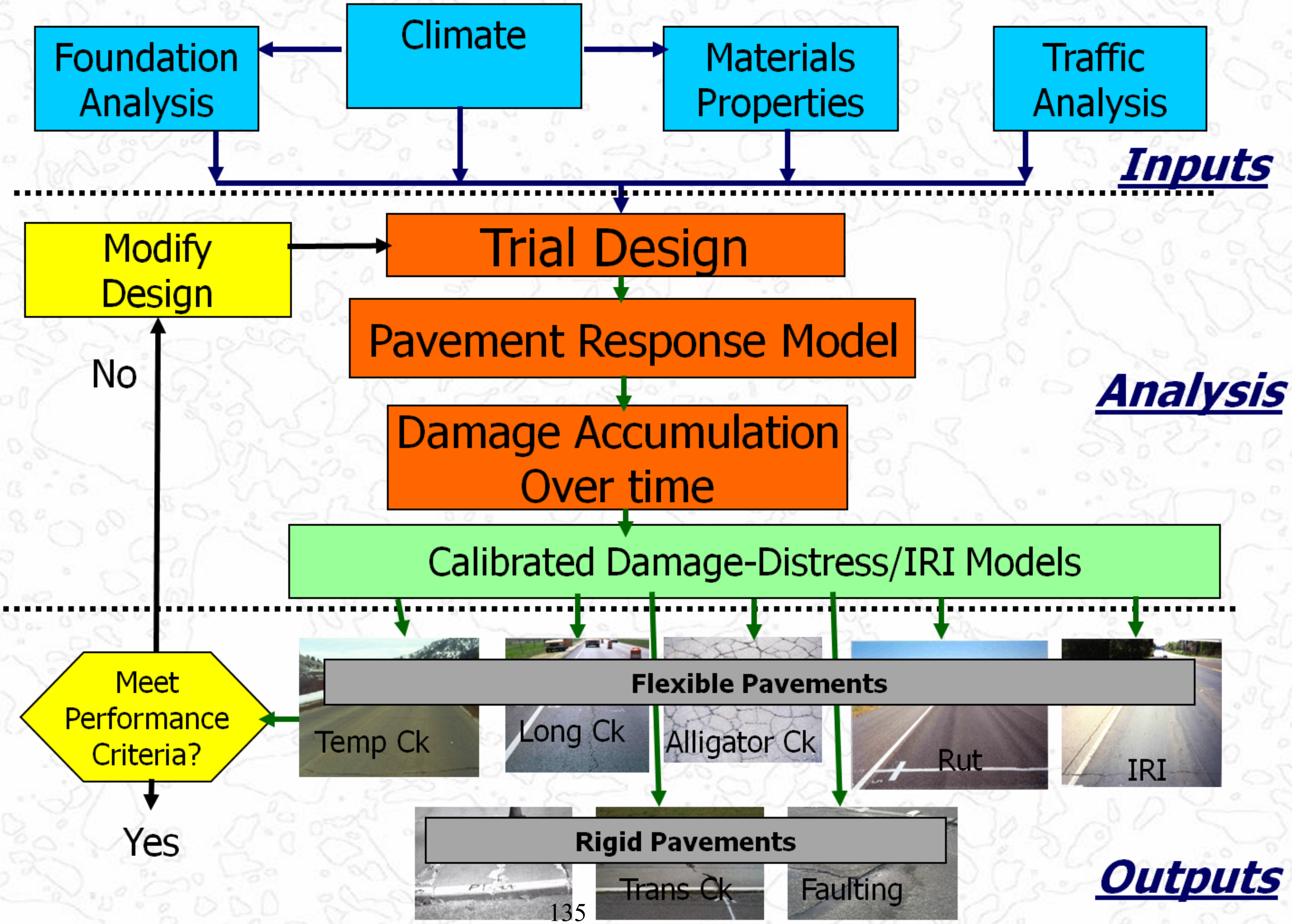
- ⇨ *Manual of Practice*
- ⇨ *Software*



Three-Stage Design Approach



M-E PDG Software Design Process (Stage 2)



Hierarchical Design Inputs

- **Level 1**
  - Highest level of accuracy (lowest level of uncertainty)
  - For heavy trafficked pavements or dire safety/economic consequences of early failure
  - Require material testing data
  - Time and resource consuming
- **Level 2**
  - Intermediate level of accuracy (closest to earlier versions of AASHTO procedure (AASHTO1986-1993)
  - Uses agency databases and empirical correlations to provide material inputs

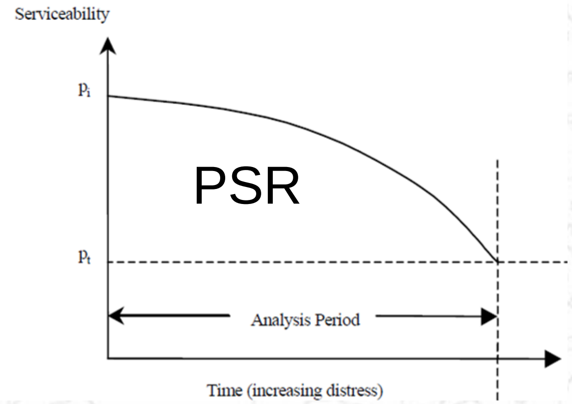
Hierarchical Design Inputs

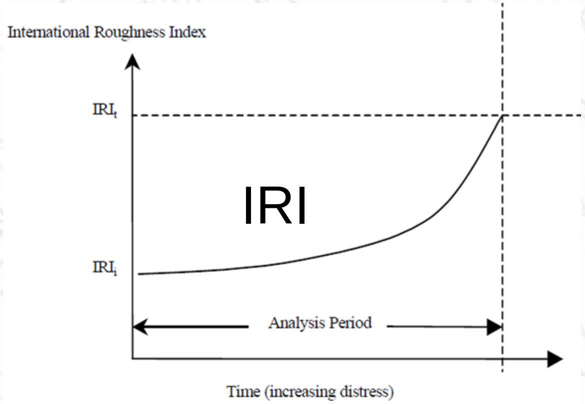
- **Level 3**
  - Lowest level of accuracy
  - For pavements with minimal consequences of early failure (e.g., low-volume roads)
  - Typical regional average values are used

  - **Note:** *Regardless the input level, the same models and procedures are used to predict distress and smoothness.*

Pavement Performance Concept

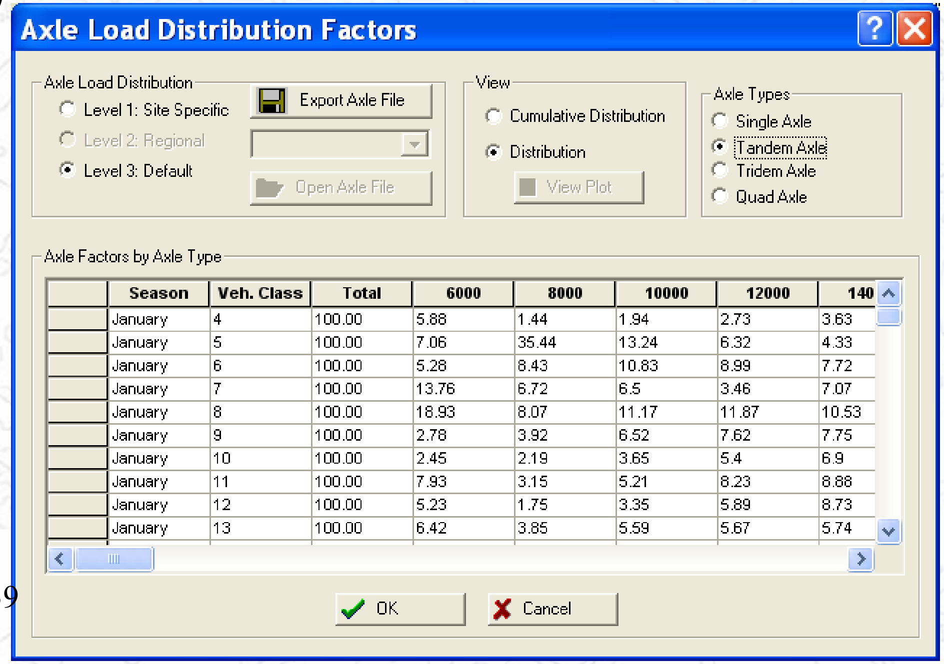
- **Structural Performance**
  - Related to the physical condition
  - Measured by predicted distresses in pavements:
    - Fatigue/Thermal cracking, and Rutting for flexible
    - Cracking and faulting for rigid pavements
- **Functional Performance**
  - Related to serviceability level/riding comfort
  - Measured by predicted IRI





Traffic Characterization

- **Truck traffic loadings (Class 4- Class13)**
  - Full axle load spectra for 4 axle types:
  - Single (3000-41000 Lbf)
  - Tandem (6000-82000 Lbf)
  - Tridem (12000-102000 Lbf)
  - Quad (12000-102000 Lbf)



Season	Veh. Class	Total	6000	8000	10000	12000	140
January	4	100.00	5.88	1.44	1.94	2.73	3.63
January	5	100.00	7.06	35.44	13.24	6.32	4.33
January	6	100.00	5.28	8.43	10.83	8.99	7.72
January	7	100.00	13.76	6.72	6.5	3.46	7.07
January	8	100.00	18.93	8.07	11.17	11.87	10.53
January	9	100.00	2.78	3.92	6.52	7.62	7.75
January	10	100.00	2.45	2.19	3.65	5.4	6.9
January	11	100.00	7.93	3.15	5.21	8.23	8.88
January	12	100.00	5.23	1.75	3.35	5.89	8.73
January	13	100.00	6.42	3.85	5.59	5.67	5.74

Traffic Characterization

- **Hierarchical levels**
  - Level 1
    - Requires site-specific data (vehicle count by class, direction, and lane
    - Incorporates axle weight data on project level
    - May use default tire pressure, spacing and axle spacing
  - Level 2
    - May use State or regional axle load spectra
  - Level 3
    - Provide default load spectrum data for a specific functional class of highway

Material Characterization

- **Three major groups of material parameters**
  - Pavement response model inputs
    - Modulus (E), Poisson's ratio (v) for each layer
  - Material-related pavement distress criteria
    - Measure of material strength (shear strength, compressive strength, modulus of rupture)
  - Other material properties
    - Special properties (C.T.E of PCC and HMA)

Material Characterization

- **Classes of Materials**
  - Dense-graded, hot-mix asphalt concrete (HMAC)
  - Open-graded, asphalt-treated permeable base (ATPB)
  - Cold mix asphalt (CMA)
  - Portland cement concrete (PCC)
  - Cement treated base (CTB) and lean concrete base (LCB)
  - Open-graded, cement-treated permeable base (CTPB)
  - Granular bases (aggregate base [AB], granular agg. base [GAB], coarse agg. [CA])
  - Lime-stabilized layers
  - Stabilized soils
  - Bedrock

Pavement Structure Modeling

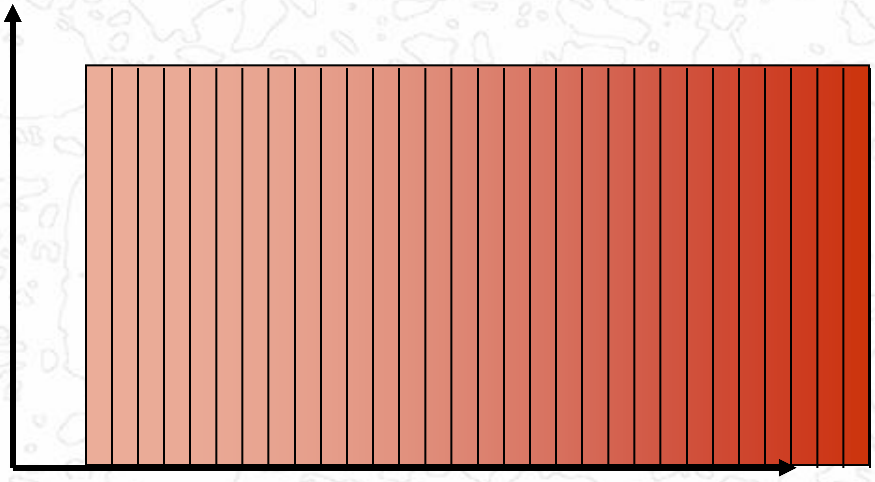
- **Structural Response Models**
  - Compute σ,ε, and δ due to traffic and climatic loading at critical locations
  - For flexible pavements
    - Multi-layer elastic analysis by JULEA (J. Usan et al.) for Level 2 and 3 (nationally calibrated on LTPP data)
    - Finite element analysis (FEA) by DSC2D for Level 1 (not calibrated)
  - For rigid pavements
    - 2-D finite element program ISLAB2000 (L. Khazanovich et al.)
    - Calibrated using Artificial Neural Networks (ANN)

Pavement Structure Modeling

- **Structural Response Model Inputs (Monthly)**
  - Traffic Loading
  - Pavement Cross-Section
  - Poisson’s ratio (for each layer)
  - Elastic modulus (for each layer)
  - Thickness(for each layer)
  - Inter-layer friction (for PCC to base)
  - C.T.E. for PCC (C.T.C. for HMA)
  - Layer temperature for HMA materials
  - Temperature/moisture gradient for PCC slab

Pavement Structure Modeling

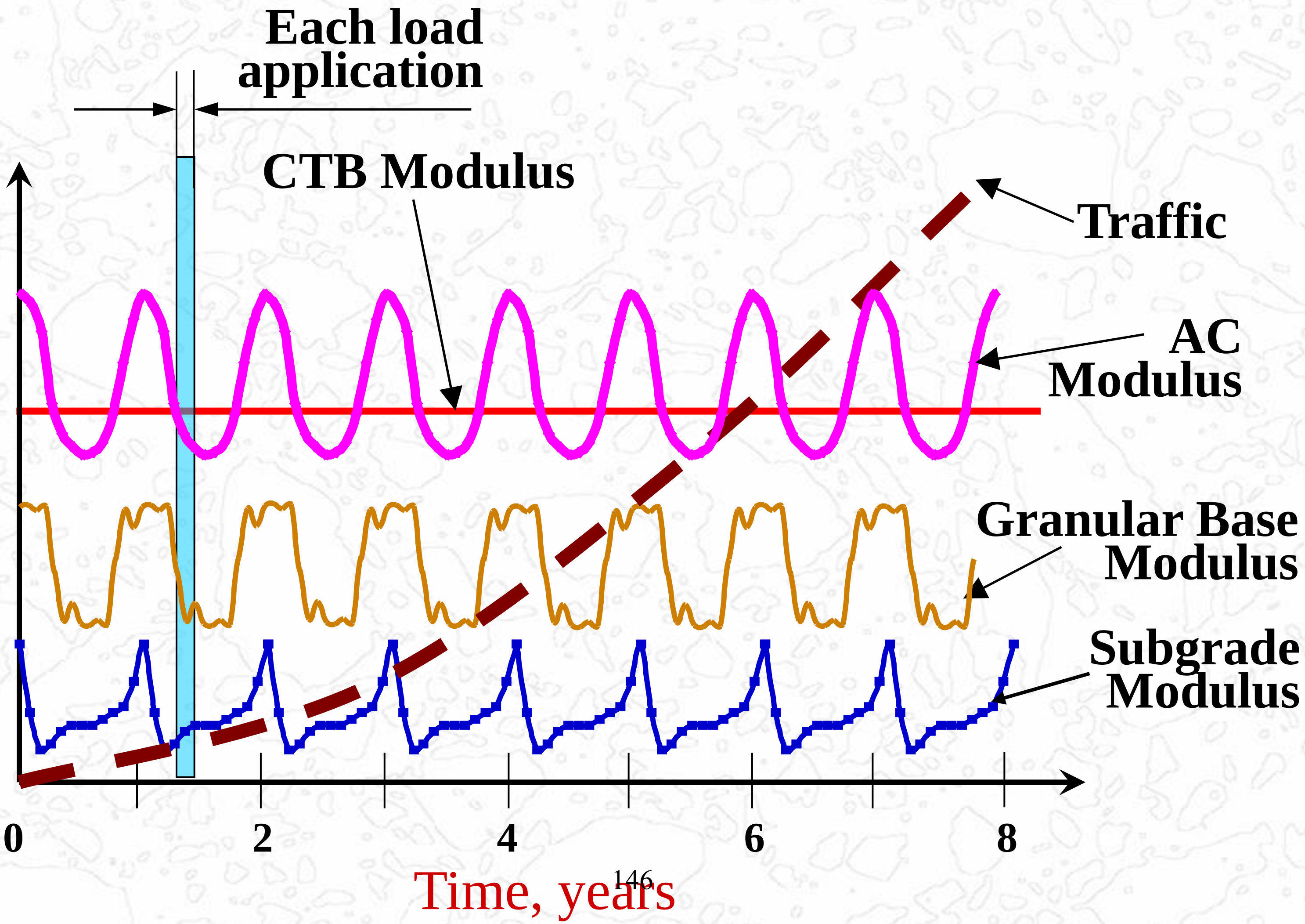
- **Incremental Damage Accumulation**
  - Design life is divided into time increments of:
    - 1 month for rigid pavements
    - 15 days for flexible pavements



Design life

Pavement Structure Modeling

⇨ **Incremental Damage Accumulation**



Pavement Structure Modeling

⇨ **Incremental Damage Accumulation (Miner's Law)**

$$\sum \frac{n_i}{N_i} = 1$$

n_i – applied traffic repetitions and i-th strain level

N_i – allowable repetitions at i-th strain level

Pavement Structure Modeling

➪ Incremental Damage Accumulation

$$\textit{Fatigue Damage} = \sum_{i}\sum_{j}\sum_{k}\sum_{l}\sum_{m}\sum_{n} \frac{n_{ijklmn}}{N_{ijklmn}}$$

where:

n_{ijklmn} = Applied number of load applications at condition i,j,k,.

N_{ijklmn} = Allowable number of load applications at condition i,j,

i = Age

j = Season

k = Axle combination

l = Load level

m = Temperature gradient

n = Traffic path

Damage Distress Models

- ⇨ Accumulated "damage" related to key distress types through calibrated prediction models

– Note: Models calibrated with LTPP database

Rehabilitation Design of Existing Pavements

- **Input Data**
  - Existing traffic lane condition (e.g., distress, smoothness, surface friction, deflections)
  - Pavement-shoulder interface
  - Pavement design features (e.g., layer thickness, structural parameters, construction requirements)
  - Material properties
  - Traffic parameters
  - Climatic conditions
  - Drainage
  - Other factors (e.g., bridge clearance, safety, utilities etc.)

Rehabilitation Design of Existing Pavements

- **Identification of Feasible Rehab Strategies**
  - Reconstruction without lane additions
  - Reconstruction with lane additions
  - Structural overlay (with or without milling the existing layer)
  - Non-structural overlay (thin HMA layer)
  - Restoration without overlays (PCC pavements)

Design Reliability

- **Everything associated with pavement design is variable or uncertain in nature**
- **Sources of variability**: traffic, materials, construction, performance
- Design Reliability for Distresses:

R=P[Distress over Design period < Critical Distress Level]

- Design Reliability for smoothness (IRI):

R=P[IRI over Design period < Critical IRI Level]

Design Reliability

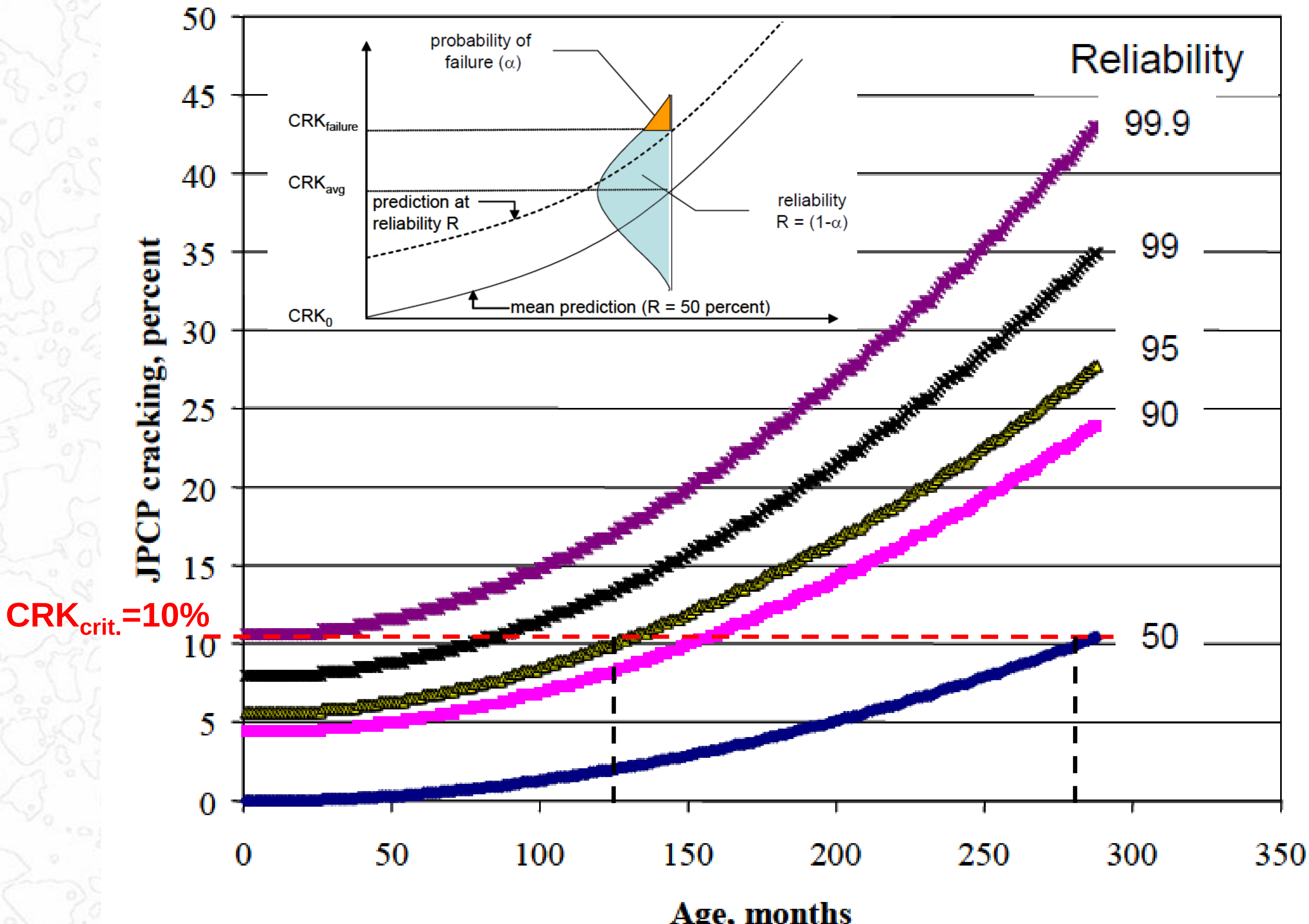
- ***AASHTO1993 has different definition***

R=P(N<n)

where N=predicted ESALs; n=actual ESALs

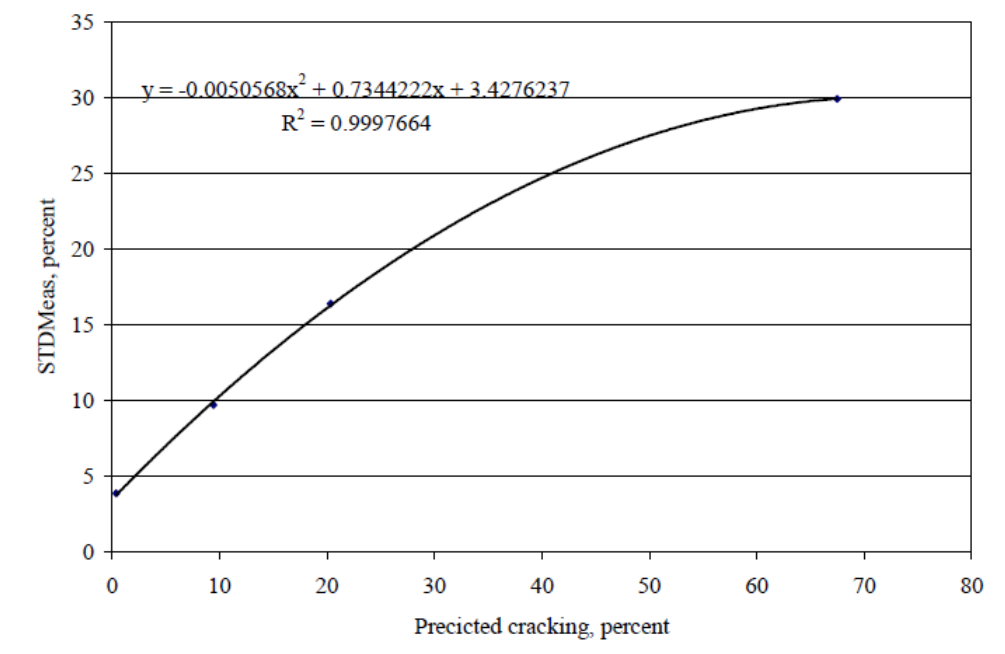
- AASHTO approach: thicker pavement => higher R
- MEPDG approach: other design features can be considered to improve R (e.g., HMA mix design, dowel bars, subgrade improvement)

Design Reliability



Design Reliability

- ⇨ ***Prediction of variability (Standard Deviation):***



- ⇨ ***Calculation of design reliability***

Design Reliability

⇨ *Calculation of design reliability*

1. Using the Design Guide cracking model, predict the cracking level over the design period using mean inputs to the model. This corresponds approximately to a "mean" slab cracking due to symmetry of residuals.

2. Estimate cracking at the desired relabilty level using the following relationship:

$$CRACK\_P = CRACK\_mean + STDmeas * Zp \quad (1.1.9)$$

where,

CRACK_P = cracking level corresponing to the reliability level p.
CRACK_mean = cracking predicted using the deterministic model with mean inputs (corresponing to 50 percent reliabality).
STDmeas = standard deviation of cracking corresponding to cracking predicted using the deterministic model with mean inputs
Zp = standardized normal deviate (mean 0 and standard deviation 1) corresponding to reliability level p.

Design Reliability

➪ ***Recommended levels of reliability***

Functional Classification	Recommended Level of Reliability	
	Urban	Rural
Interstate/Freeways	85 . – . 97	80 . – . 95
Principal Arterials	80 . – . 95	75 . – . 90
Collectors	75 – 85	70 . – . 80
Local	50 . – . 75	50 . – . 75

Enhanced Integrated Climate Model (EICM)

EICM Module predicts:

- ⇨ Environmental effects adjustment factors for unbound Resilient modulus

Finite Element/Linear Elastic Analysis Modules

- ⇨ Hourly temperature profile through AC layers

Thermal Cracking Module

- ⇨ Temperature Frequency Distribution at mid-depth of bound sublayers

Fatigue/Permanent Deformation Modules

- ⇨ Average moisture content for unbound materials

Unbound Permanent Deformation Module

EICM Analysis

- Records the user supplied resilient modulus, *MR*, of all unbound layer materials
- Evaluates equilibrium moisture condition and the seasonal changes in moisture contents.
- Evaluates the effect of changes in soil moisture the user entered resilient modulus, *MR*.
- Evaluates the effect of freezing on the layer *MR*.
- Evaluates the effect of thawing and recovery from the frozen *MR* condition.
- Evaluates changes in temperature as a function of time for all asphalt bound layers.

Environmental Effects Adjustment Factors

EICM computes climatic adjustment factors for the Resilient modulus for:

- ⇨ Frozen material
- ⇨ Recovering material
- ⇨ Unfrozen or fully recovered material
- ⇨ Environmental effect through composite adjustment factor

$$M_R = F_{env} \cdot M_{Ropt}$$

Soil Moisture Adjustment

$$\log \frac{M_R}{M_{Ropt}} = a + \frac{b-a}{1+EXP\left(\ln\frac{-b}{a} + k_m \cdot (S - S_{opt})\right)}$$

M_R/M_{Ropt}	=	Resilient modulus ratio; M_R is the resilient modulus at a given time and M_{Ropt} is the resilient modulus at a reference condition.
a	=	Minimum of $\log(M_R/M_{Ropt})$.
b	=	Maximum of $\log(M_R/M_{Ropt})$.
k_m	=	Regression parameter.
$(S - S_{opt})$	=	Variation in degree of saturation expressed in decimal.

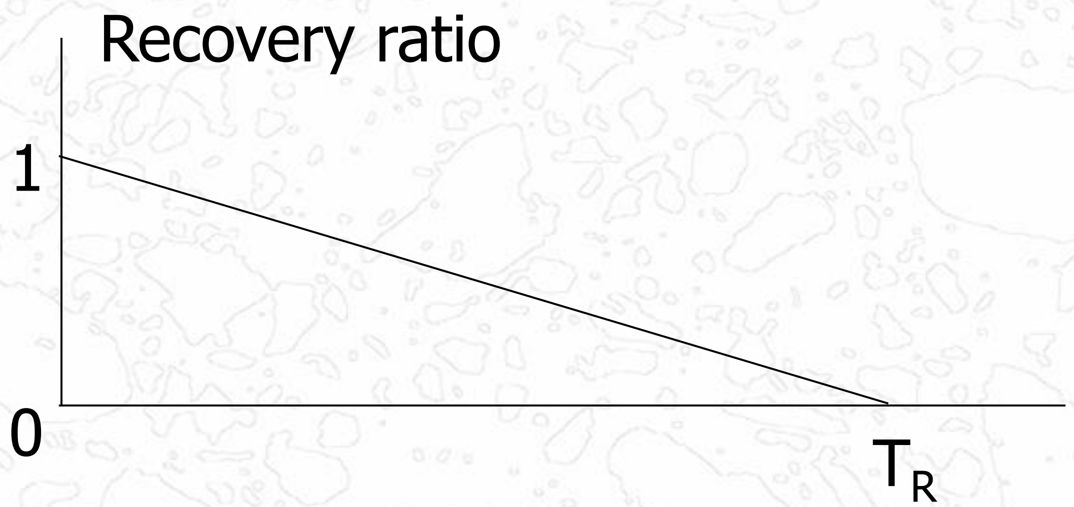
Resilient Moduli for Thawed Unbound Materials

RF = modulus reduction factor = $MR_{min}/\min(MR_{unfrz}, MR_{opt})$

Recommended values of *RF* for fine-grained materials ($P_{200} > 50\%$).

P_{200} (%)	***PI* < 12%**	***PI* = 12% - 35%**	***PI* > 35%**
50 – 85	0.45	0.55	0.60
> 85	0.40	0.50	0.55

Resilient Moduli for Recovering Unbound Materials



- T_R = 90 days for sands/gravels with $P_{200}PI$ < 0.1.
- T_R = 120 days for silts/clays with 0.1 < $P_{200}PI$ < 10.
- T_R = 150 days for clays with $P_{200}PI$ > 10.

Time-depth diagram and matrix of adjustment coefficients

LEGEND:

- FROZEN
- RECOVERING
- UNFROZEN

Time (days)

Nodes	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*	*12*	*13*	*14*	
1															AC
2															
3	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	BASE
4	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	
5	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
6	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
7	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
8	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
9	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	SUBBASE
10	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
11	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
12	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
13	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
14	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_U	F_U	F_U	
15	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_U	F_U	F_U	F_U	
16	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_U	F_U	F_U	F_U	F_U	F_U	
17	F_R	F_R	F_R	F_R	F_R	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	SUBGRADE
18	F_R	F_R	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
19	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
20	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
21	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
22	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
23	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
24	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	

Matrix of adjustment coefficients

LEGEND:

- FROZEN
- RECOVERING
- UNFROZEN

Time (days)

Nodes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
1															AC
2															
3	50	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	BASE
4	50	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	
5	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
6	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
7	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
8	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
9	75	75	75	75	75	75	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	SUBBASE
10	75	75	75	75	75	75	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	
11	75	75	75	75	75	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	
12	75	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	
13	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	
14	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	1	1	1	
15	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	1	1	1	1	
16	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	1	1	1	1	1	1	
17	0.8	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	SUBGRADE
18	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
19	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
20	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
21	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
22	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
23	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
24	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	

Quintile temperature distribution

If the mean monthly temperature (μ) reported is 50^{o}F and has a standard deviation (σ) of 15^{o}F

Sub-Season	z-value	Temperature, ^{o}F $= \mu + z(\sigma)$
1	-1.2816	30.8
2	-0.5244	44.8
3	0	50.0
4	0.5244	55.2
5	1.2816	69.2

Connecticut Department of Transportation

M-E PDG Training Module II

Flexible Pavement Design

Prepared by Dr. Iliya Yut

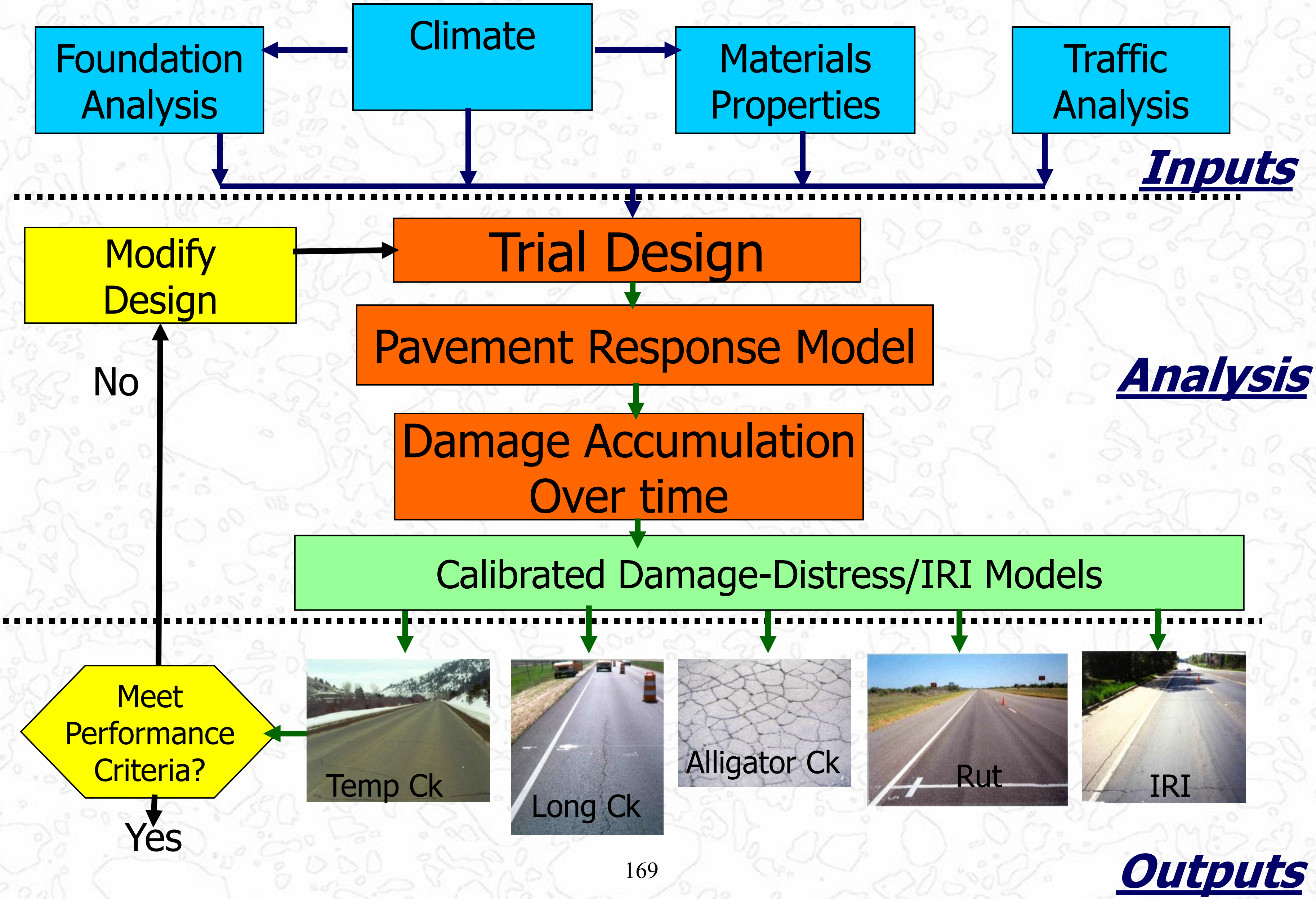
Department of Civil Engineering, UConn

May2013

M-E PDG models for flexible pavements

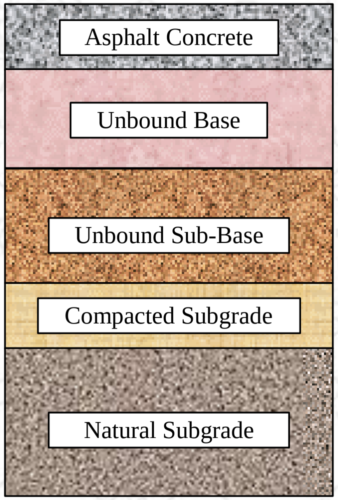
- Overview of the M-E PDG
- Load Related Cracking
- Rutting Models
- Thermal Cracking
- Roughness models

M-E PDG Design Process

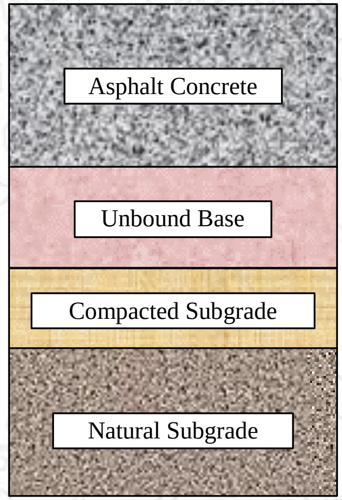


Possible Asphalt Pavement Systems

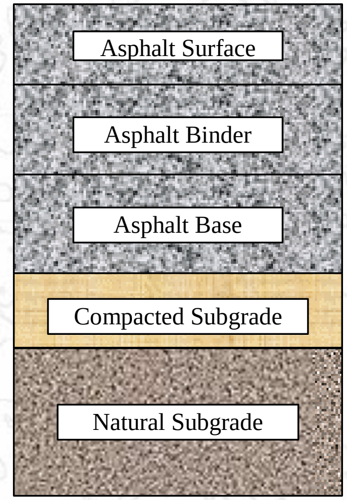
Conventional

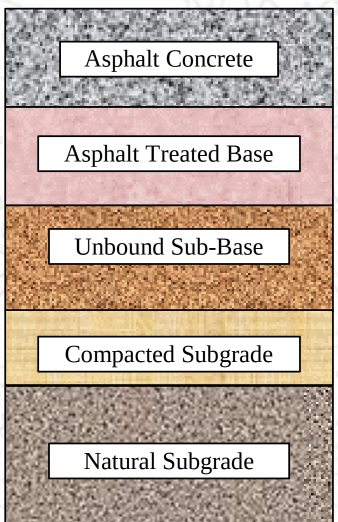


Deep Strength

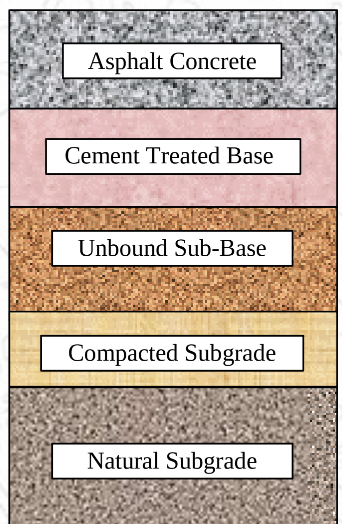


Full Depth

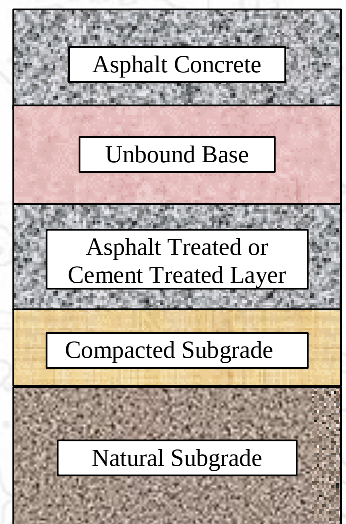




Semi-Rigid with ATB



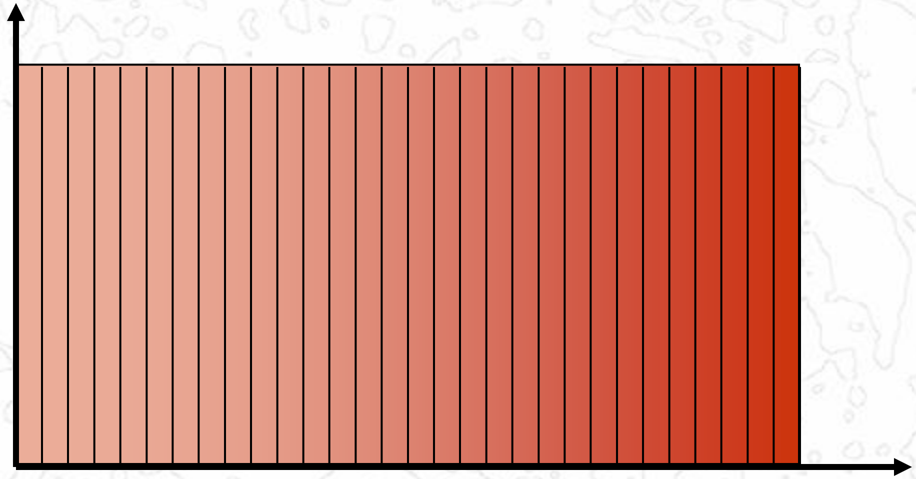
Semi-Rigid with CTB



Inverted Section

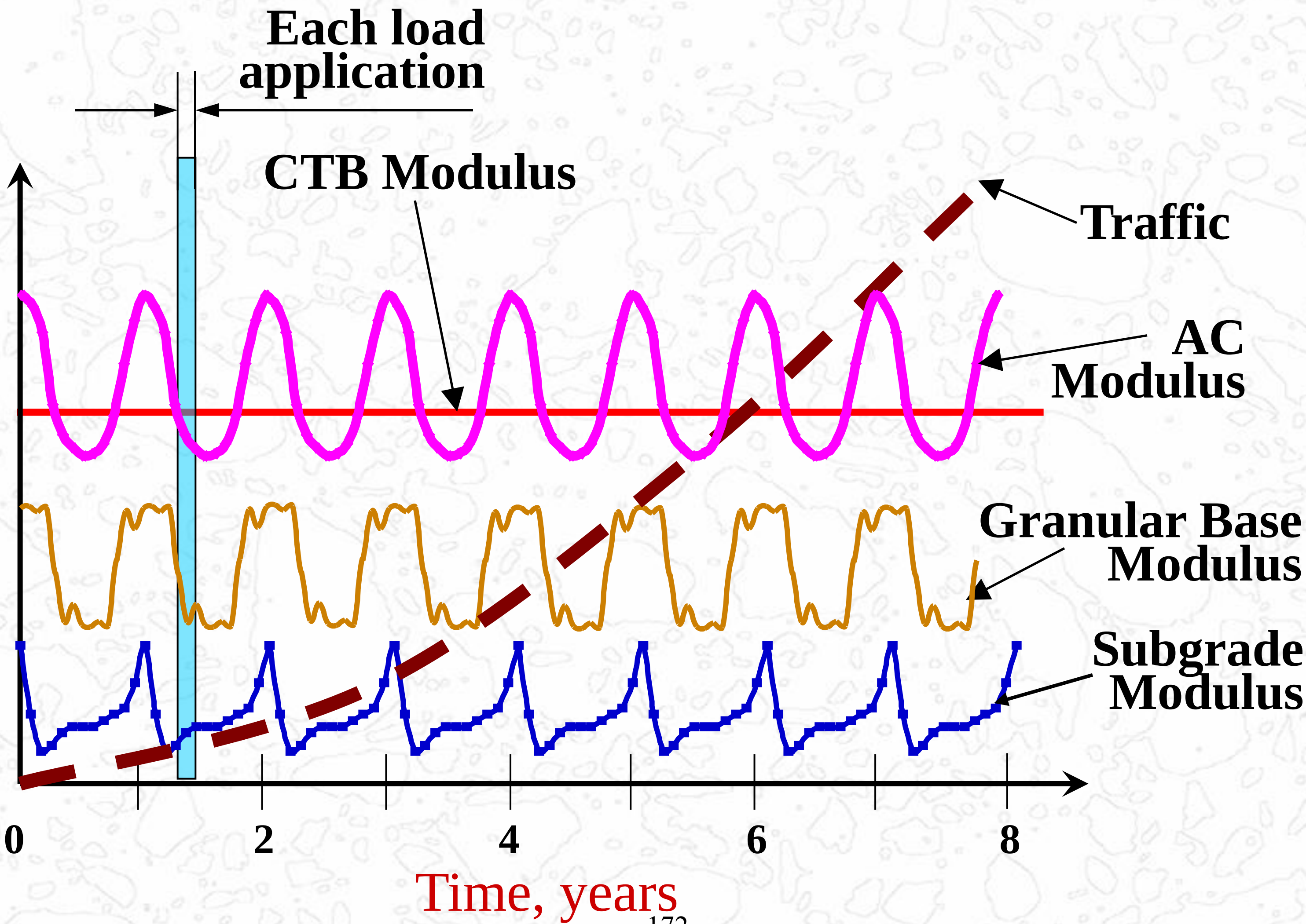
Damage Accumulation - Incremental Damage Concept

- Design life is divided into time increments of:
  - 1 month for rigid pavements
  - 15 days for flexible pavements



Design life

Incremental Changes Over Pavement Life



Rules of Simulation

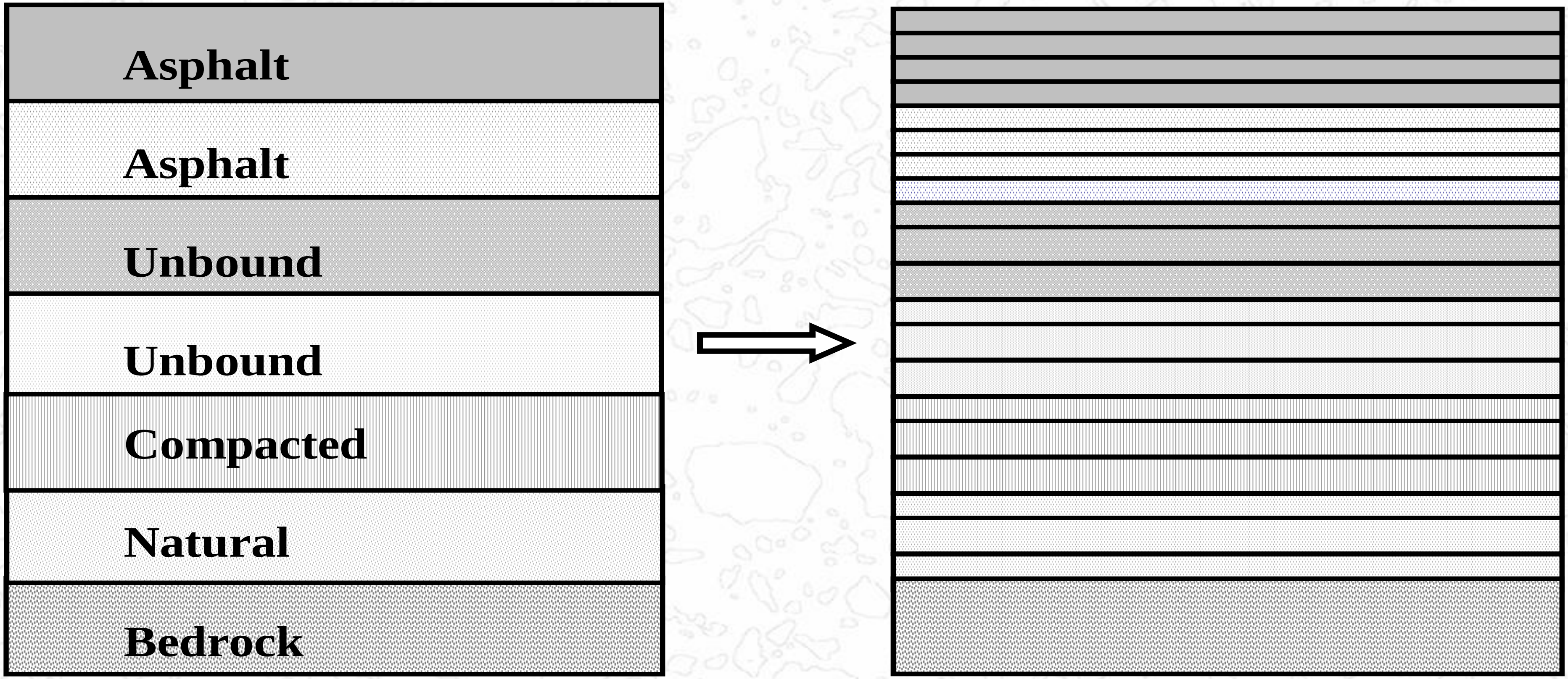
- Simulate the pavement structure and foundation as detailed as possible; divide the subgrade or foundation soils into two layers especially when bedrock and other hard soils are not encountered.
- Combine layers as needed
  - Try to combine the lower layers first and treat the upper layers in more detail, if at all possible.
  - Thin non-structural layers should be combined with other layers
  - Any layer that is less than 1-inch in thickness should be combined with the supporting layer
  - Similar materials of adjacent layers should be combined into one layer
  - Filter fabrics used for drainage purposes between a fine-grained soil and aggregate base material should be ignored

Sub-Layering for Structural Analysis (cont)

- AC surface layer
  - 0.5 in top sub-layer
  - Remaining parts: from 1 in
- AC binder – no sublayering
- AC base – no sublayering
- CTB – no sublayering
- AGG base (1-st unbound layer)
  - no sublayering if <4”
  - 4” top sub-layer and remaining are >4”
- AGG subbase
  - Sublayers >4”
- Subgrade
  - 12” first 8’, infinite subgrade after that
- Bedrock – no sublayering

- **Maximum number of sublayers – 20**
- **Maximum number of evaluation points - 26**

Sub-Layering for Structural Analysis



Global Aging System

- ⇨ Original to mix/lay-down model.
- ⇨ Surface aging model.
- ⇨ Air void adjustment.
- ⇨ Viscosity-depth model

Surface Aging Model

$$\log\log(\eta_{aged}) = \frac{\log\log(\eta_{t=0}) + At}{1 + Bt}$$

A depends on mean annual temperature and reduced time

B depends on reduced time

Air Void Adjustment

$$\log\log(\eta_{aged})' = F_v \log\log(\eta_{aged})$$

$$F_v = \frac{1 + 1.0367\times 10^{-4}(VA)(t)}{1 + 6.1798\times 10^{-4}(t)}$$

$$VA = \frac{VA_{orig} + 0.011(t) - 2}{1 + 4.24\times 10^{-4}(t)(Maat) + 1.169\times 10^{-3}\left(\dfrac{t}{\eta_{orig,77}}\right)} + 2$$

Viscosity-Depth Model

$$\eta_{t,z} = \frac{\eta_t(4+E) - E(\eta_{t=0})(1-4z)}{4(1+Ez)}$$

$\eta_{t,z}$	=	Aged viscosity at time t, and depth z
η_t	=	Aged surface viscosity
z	=	Depth, in
E	=	$23.83e^{(-0.0308 \text{ Maat})}$
Maat	=	Mean annual air temperature, °F

Enhanced Integrated Climate Model (EICM)

EICM Module predicts:

- ⇨ Environmental effects adjustment factors for unbound Resilient modulus

Finite Element/Linear Elastic Analysis Modules

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- ⇨ Records the user supplied resilient modulus, *MR*, of all unbound layer materials
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Environmental Effects Adjustment Factors

EICM computes climatic adjustment factors for the Resilient modulus for:

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- ⇨ Environmental effect through composite adjustment factor

$$M_R = F_{env} \cdot M_{Ropt}$$

Soil Moisture Adjustment

$$\log\frac{M_R}{M_{Ropt}} = a + \frac{b-a}{1+EXP\left(\ln\frac{-b}{a}+k_m\cdot(S-S_{opt})\right)}$$

M_R/M_{Ropt} = Resilient modulus ratio; M_R is the resilient modulus at a given time and M_{Ropt} is the resilient modulus at a reference condition.

a = Minimum of $\log(M_R/M_{Ropt})$.

b = Maximum of $\log(M_R/M_{Ropt})$.

k_m = Regression parameter.

$(S - S_{opt})$ = Variation in degree of saturation expressed in decimal.

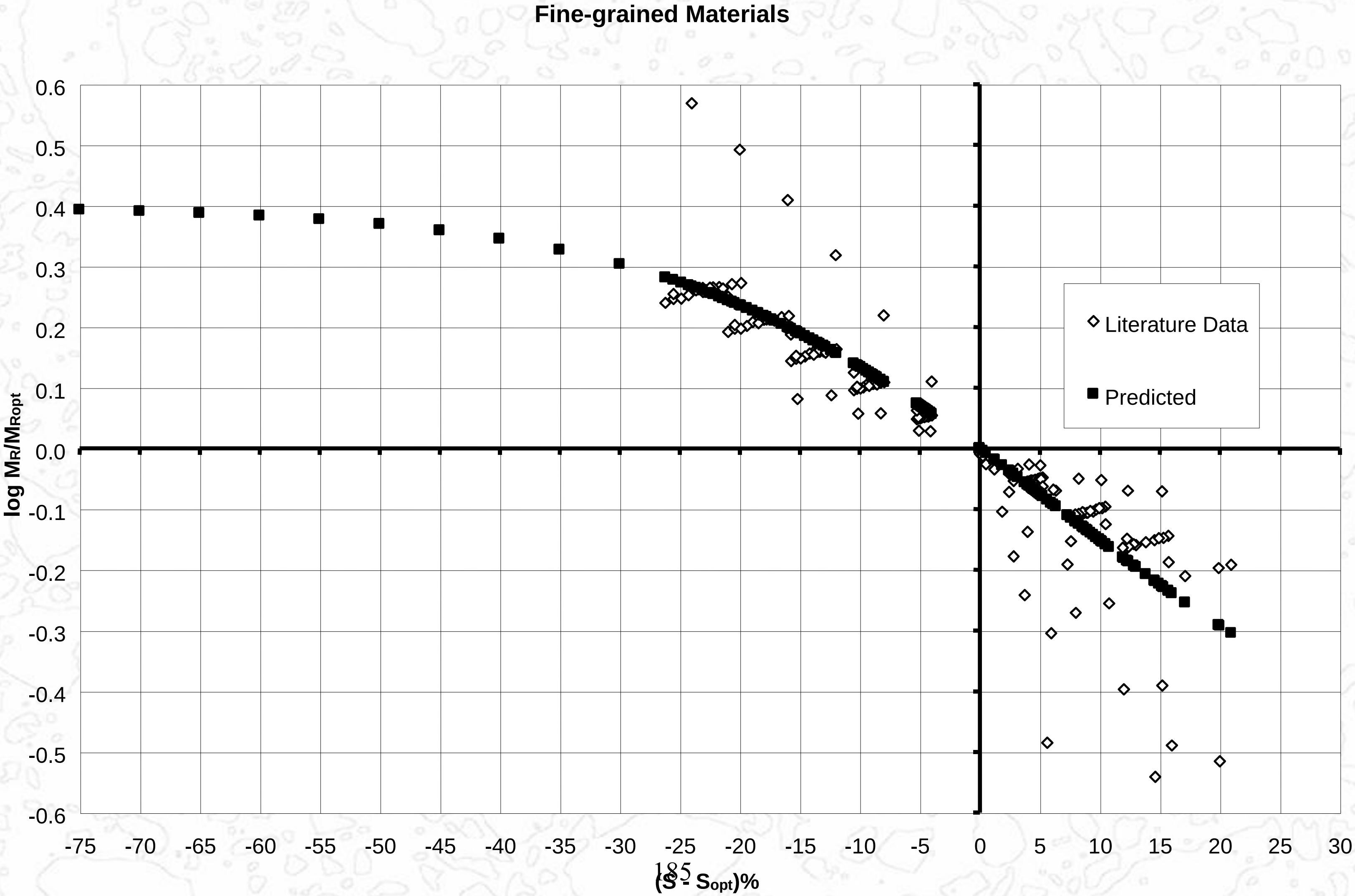
Soil Moisture Adjustment

$$\log \frac{M_R}{M_{Ropt}} = a + \frac{b-a}{1 + EXP\left(\ln \frac{-b}{a} + k_m \cdot (S - S_{opt}) \right)}$$

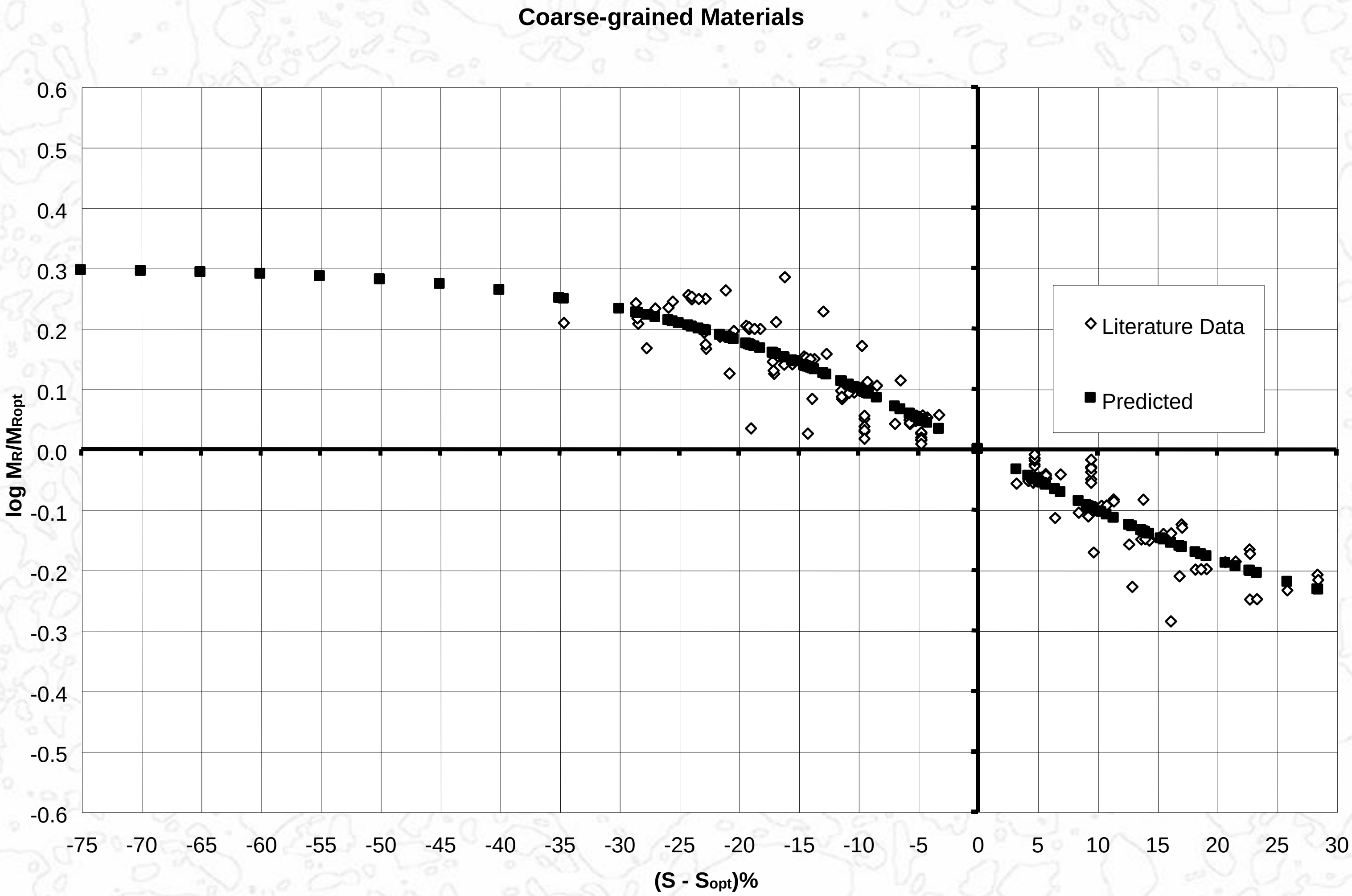
Values of a, b, and k_m for coarse-grained and fine-grained materials.

Parameter	**Coarse-Grained Materials**	**Fine-Grained Materials**	**Comments**
a	- 0.3123	-0.5934	Regression parameter.
b	0.3	0.4	Conservatively assumed, corresponding to modulus ratios of 2 and 2.5, respectively.
k_m	6.8157	6.1324	Regression parameter.

Resilient modulus - moisture model for fine-grained materials



Resilient modulus - moisture model for coarse-grained materials



Resilient Moduli for Thawed Unbound Materials

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Recommended values of *RF* for fine-grained materials (P_{200} > 50%).

P_{200} (%)	***PI* < 12%**	***PI* = 12% - 35%**	***PI* > 35%**
50 – 85	0.45	0.55	0.60
> 85	0.40	0.50	0.55

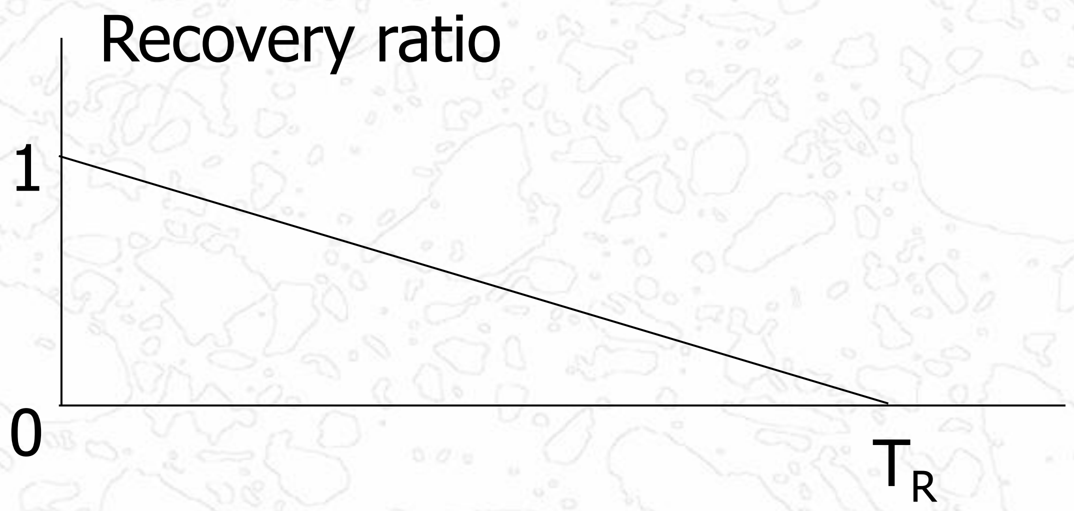
Resilient Moduli for Thawed Unbound Materials

RF = modulus reduction factor = $MR_{min}/\min(MR_{unfrz}, MR_{opt})$

Recommended values of *RF* for coarse-grained materials ($P_{200} < 50\%$).

Distribution of Coarse Fraction*	P_{200} (%)	$PI < 12\%$	$PI = 12\% - 35\%$	$PI > 35\%$
Mostly Gravel $P_4 < 50\%$	< 6	0.85	-	-
	6 – 12	0.65	0.70	0.75
	> 12	0.60	0.65	0.70
Mostly Sand $P_4 > 50\%$	< 6	0.75	-	-
	6 – 12	0.60	0.65	0.70
	> 12	0.50	0.55	0.60

Resilient Moduli for Recovering Unbound Materials



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- T_R = 120 days for silts/clays with $0.1 < P_{200}PI < 10$.
- T_R = 150 days for clays with $P_{200}PI > 10$.

Time-depth diagram and matrix of adjustment coefficients

LEGEND:

- FROZEN
- RECOVERING
- UNFROZEN

Time (days)

Nodes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
1															AC
2															
3	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	BASE
4	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	
5	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
6	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
7	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
8	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
9	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	SUBBASE
10	F_F	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
11	F_F	F_F	F_F	F_F	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
12	F_F	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
13	F_F	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	
14	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_U	F_U	F_U	
15	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_U	F_U	F_U	F_U	
16	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_R	F_U	F_U	F_U	F_U	F_U	F_U	
17	F_R	F_R	F_R	F_R	F_R	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	SUBGRADE
18	F_R	F_R	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
19	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
20	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
21	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
22	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
23	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	
24	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	F_U	

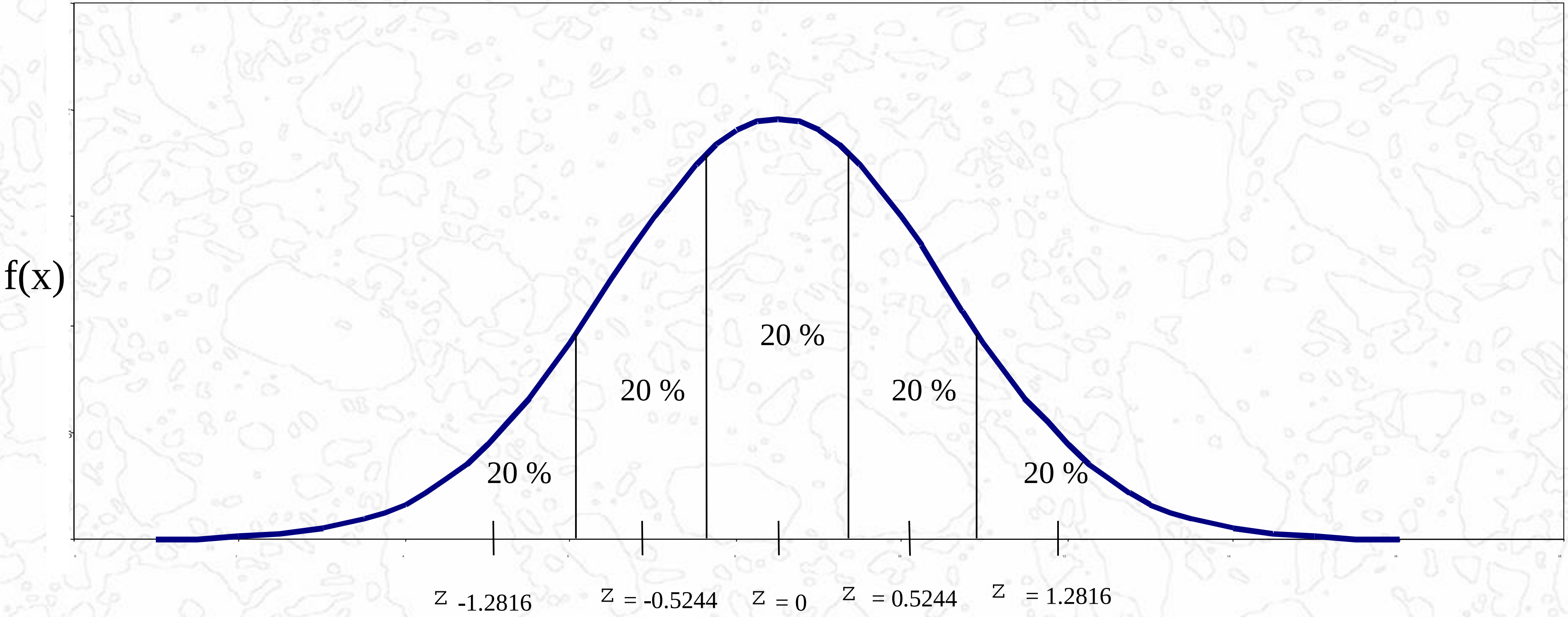
Matrix of adjustment coefficients

LEGEND:
- FROZEN
- RECOVERING
- UNFROZEN

Time (days)

Nodes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
1															AC
2															
3	50	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	BASE
4	50	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	
5	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
6	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
7	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
8	50	50	50	50	50	50	50	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
9	75	75	75	75	75	75	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	SUBBASE
10	75	75	75	75	75	75	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	
11	75	75	75	75	75	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	
12	75	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	
13	75	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	
14	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	1	1	1	
15	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	1	1	1	1	
16	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	1	1	1	1	1	1	
17	0.8	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	SUBGRADE
18	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
19	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
20	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
21	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
22	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
23	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	
24	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	

Temperature Analysis for AC cracking and rutting

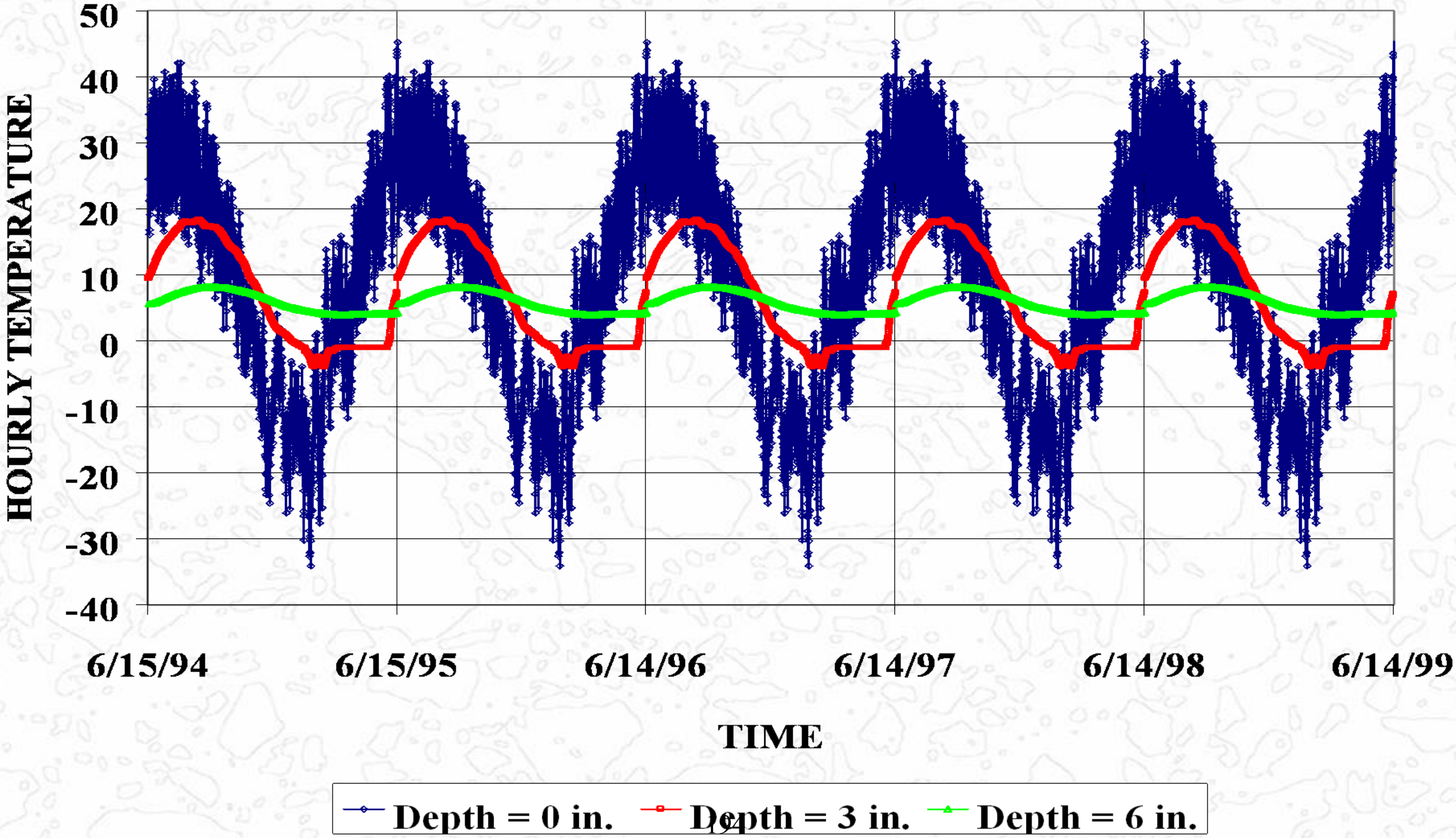


Quintile temperature distribution

If the mean monthly temperature (μ) reported is 50°F and has a standard deviation (σ) of 15°F

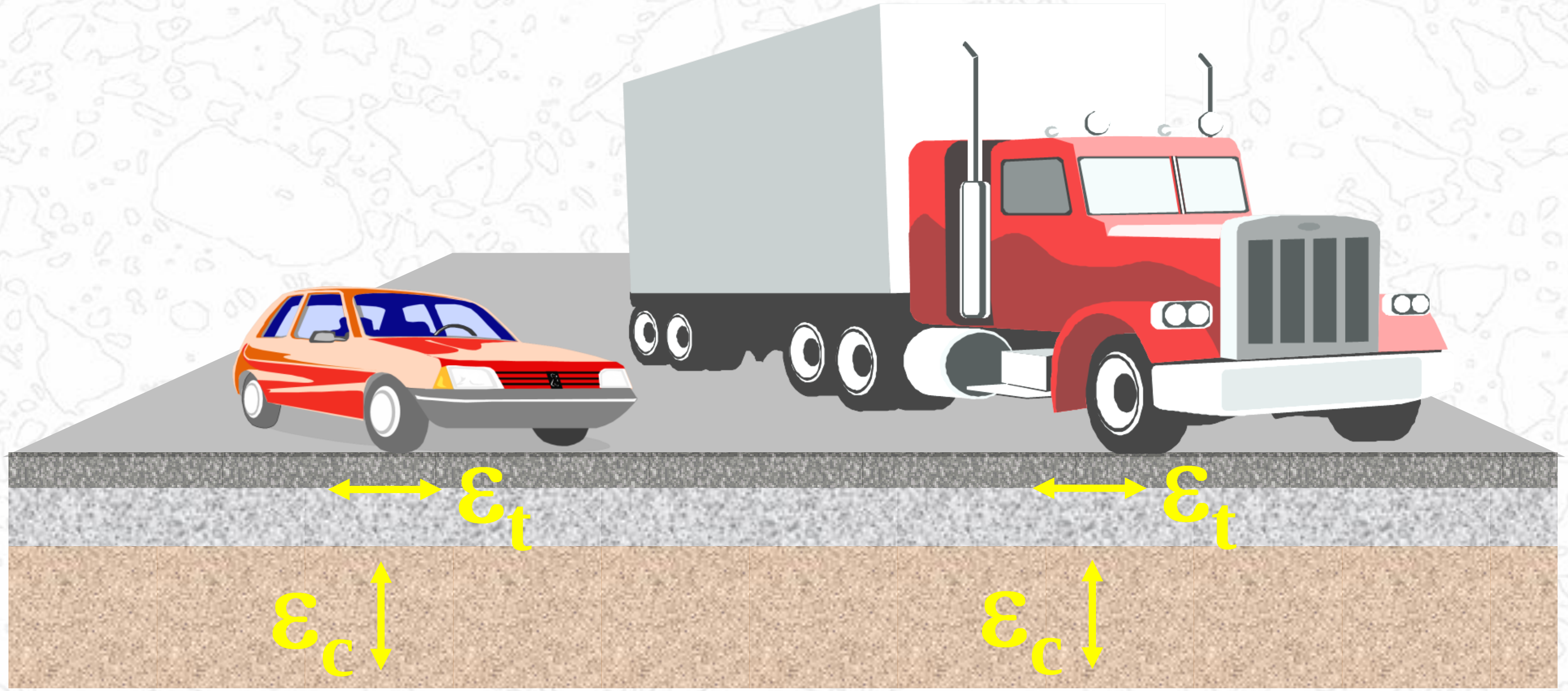
Sub-Season	z-value	Temperature, °F $= \mu + z(\sigma)$
1	-1.2816	30.8
2	-0.5244	44.8
3	0	50.0
4	0.5244	55.2
5	1.2816	69.2

Hourly Temperature Profile for AC Layers for Thermo-cracking



Critical Response Values

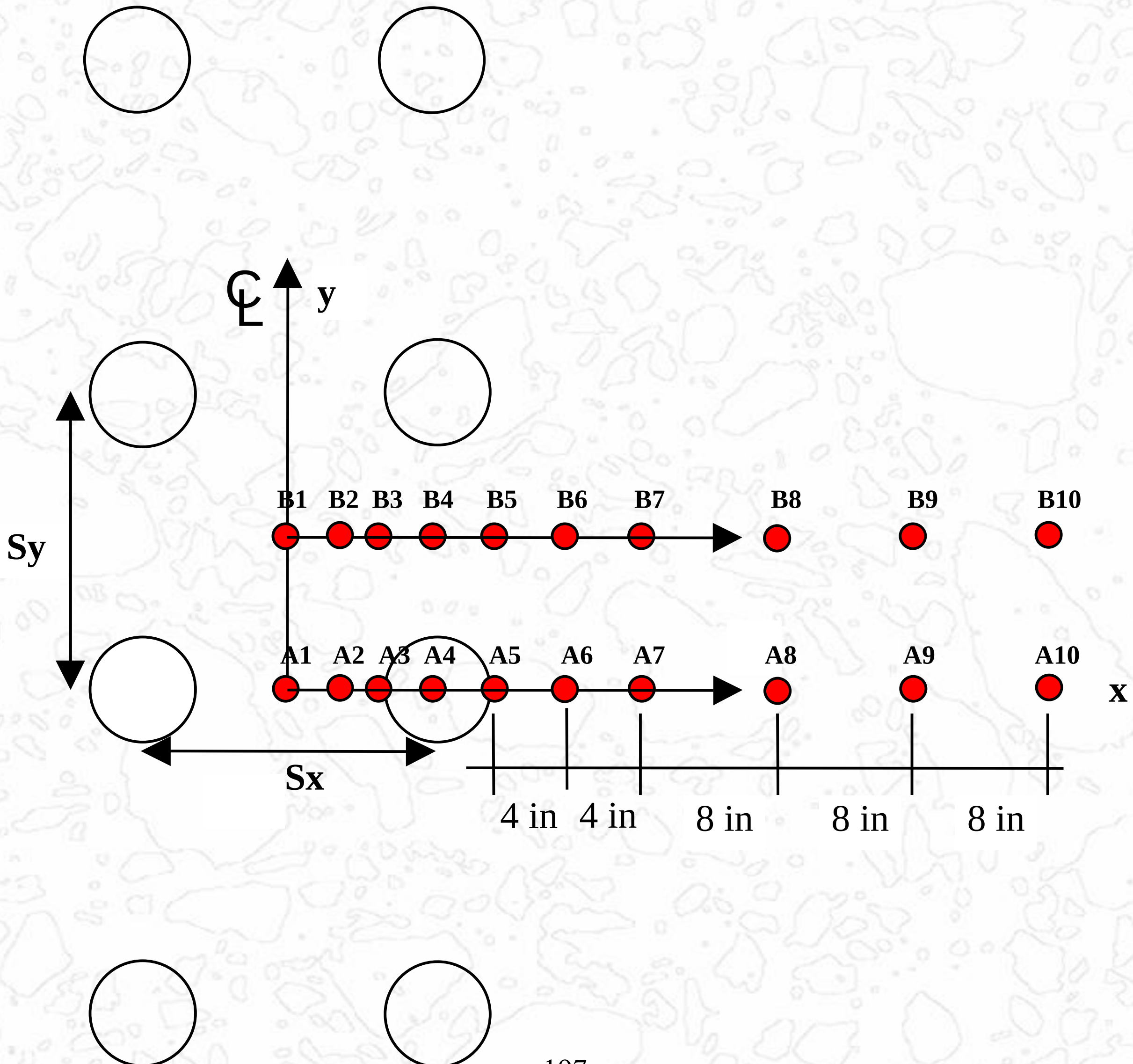
- ⇨ Cracking: ε_t at surface + bottom of all bound layers
- ⇨ Rutting: ε_c at midthickness of all layers + top of subgrade



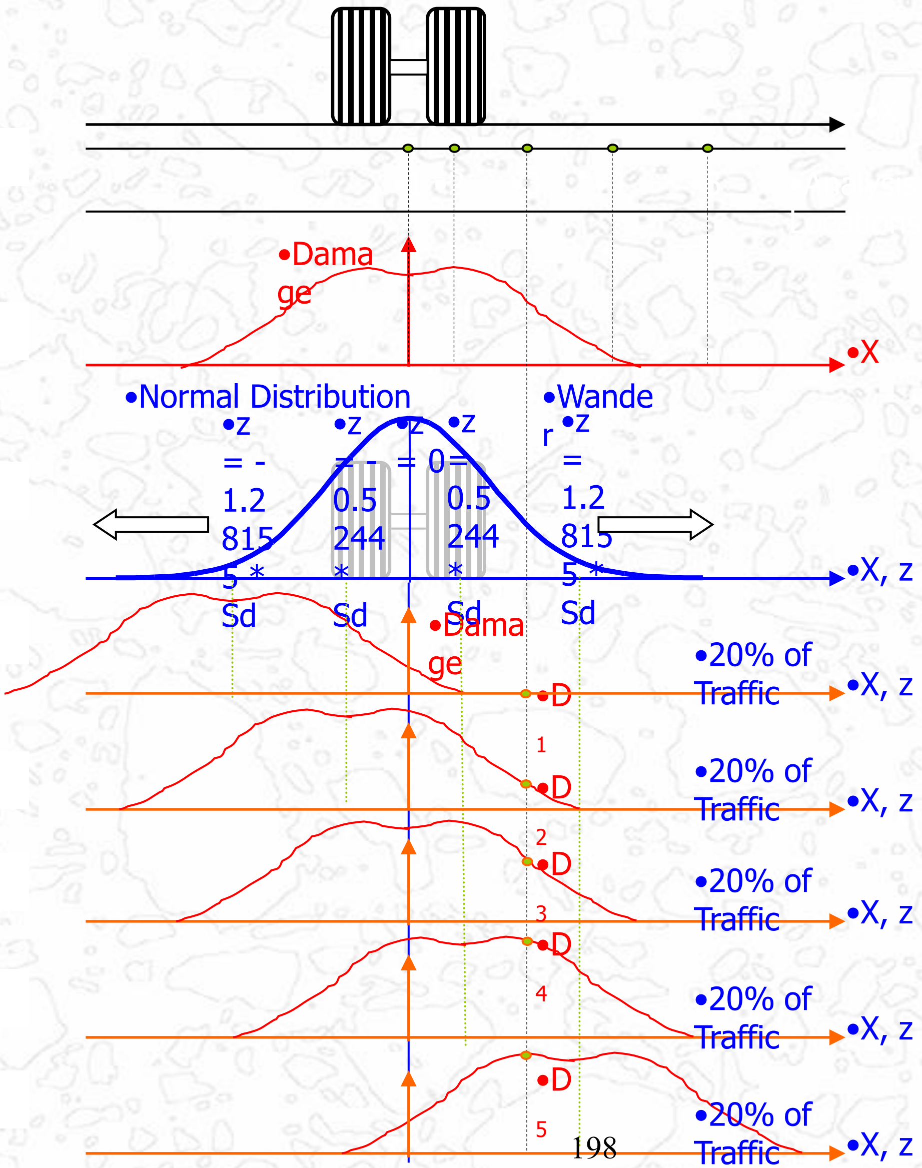
Critical Response Locations

- Fatigue Depth Locations:
  - Surface of the pavement (z=0),
  - 0.5 inches from the surface (z=0.5),
  - Bottom of each bound or stabilized layer.
- Rutting Depth Locations:
  - Mid-depth of each layer/sub-layer,
  - Top of the subgrade,
  - Six inches below the top of the subgrade.

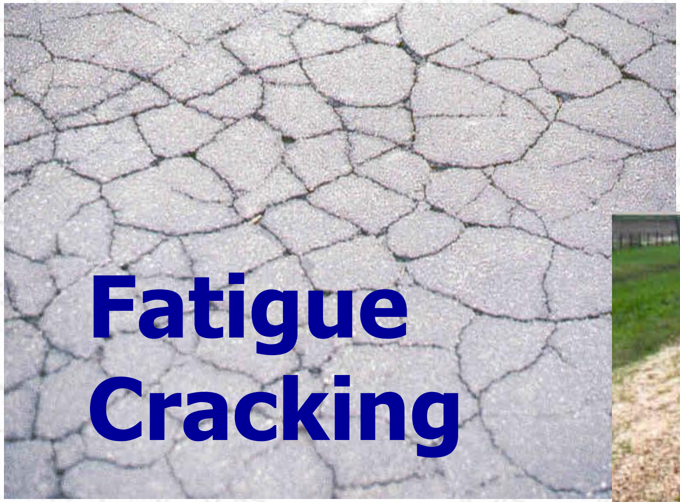
Critical Response Locations

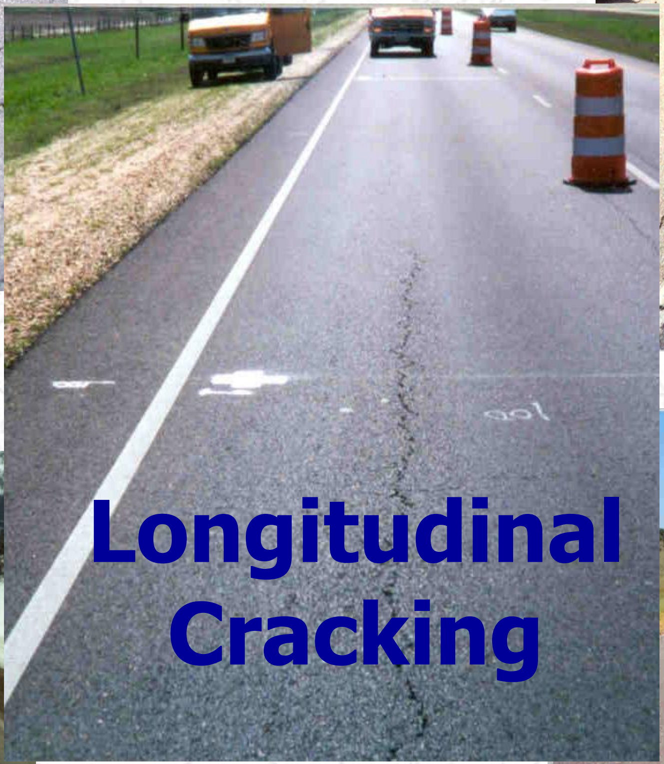


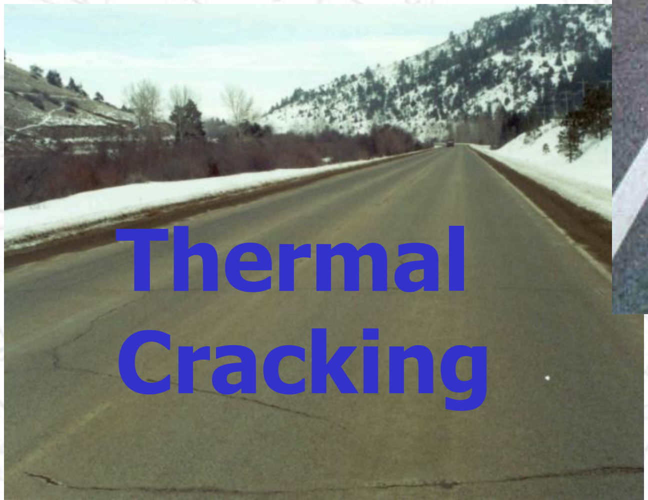
Fatigue Analysis Wander Approach

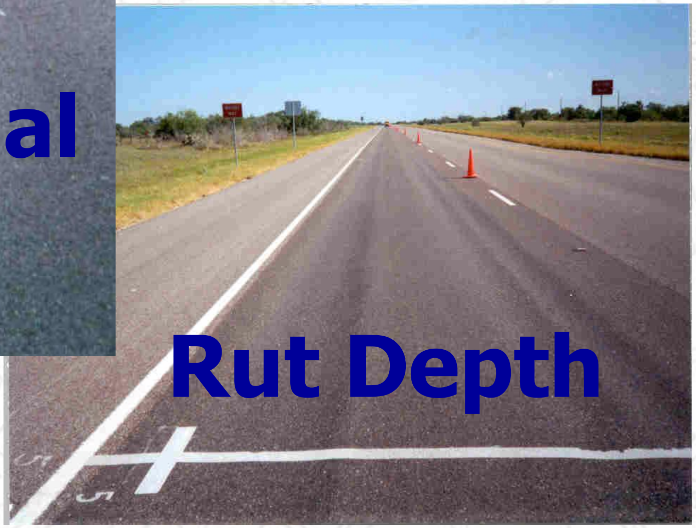


Flexible Pavement Performance



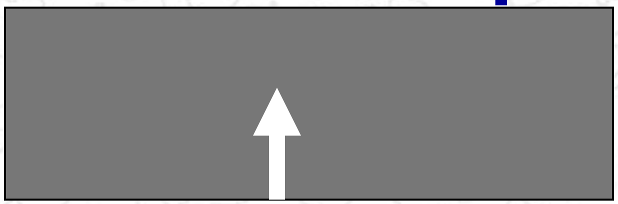






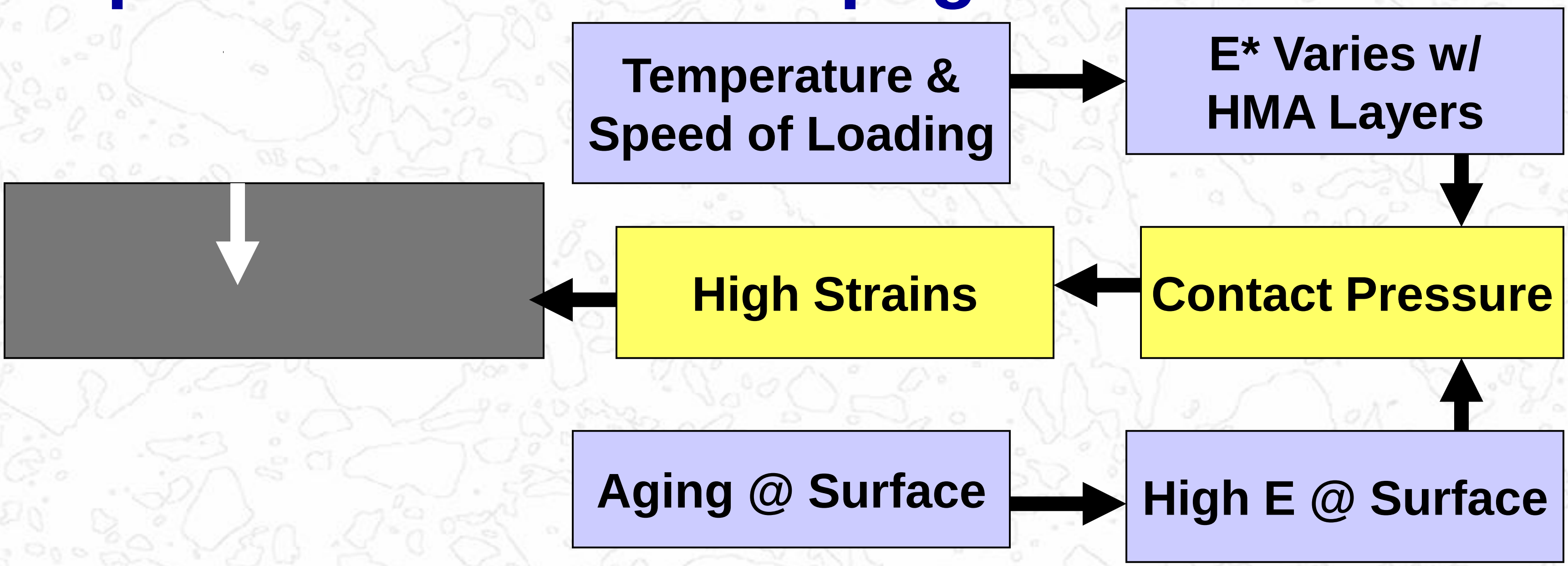
HMA Fatigue Modeling

- **Bottom – Up Crack Propagation:**

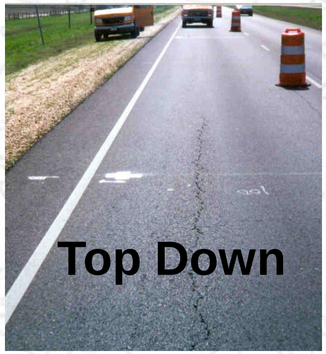


(Classical Fatigue Mechanism)

- **Top – Down Crack Propagation**

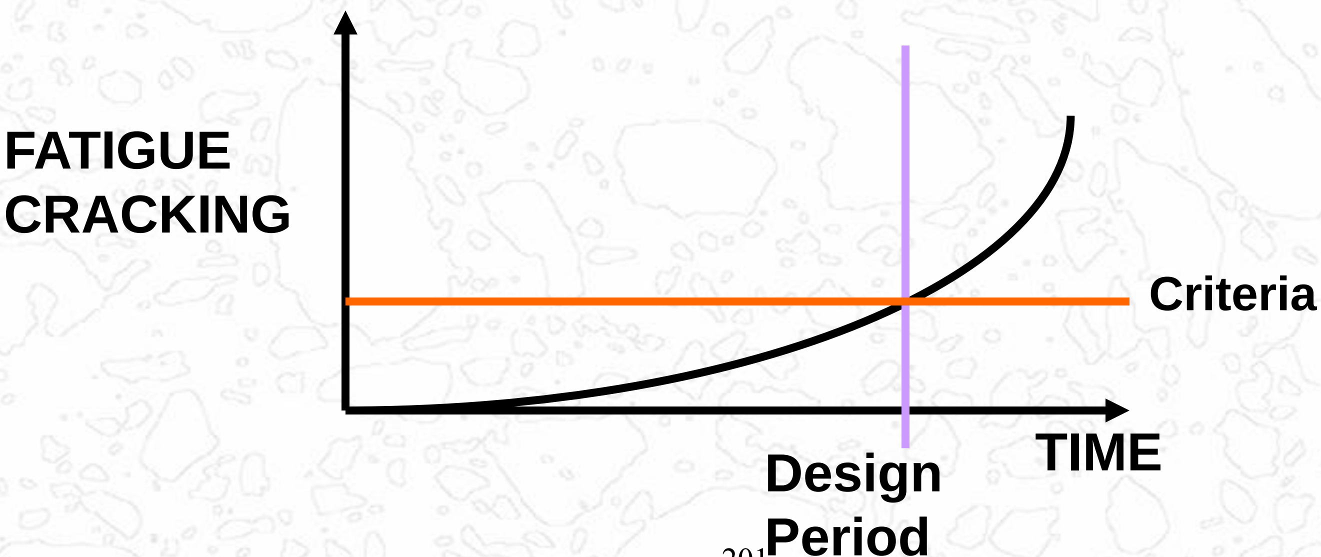


Fatigue Damage Accumulates Over Time



$$\Delta DI = \sum_{k=1}^{m} \sum_{i=1}^{j} \left[\frac{n_i}{N(\varepsilon_t)_i} \right]_k$$

Load Season



Allowable Number of Load Applications

$$N_f = k_{f1}\left(C\right)\left(\beta_{f1}\right)\left(\varepsilon_t\right)^{k_{f2}\beta_{f2}}\left(E_{HMA}\right)^{k_{f3}\beta_{f3}}$$

N_f = Allowable number of axle load applications
ε_t = Tensile strain at critical locations
E_{HMA} = Dynamic modulus of the HMA, psi
k_{f1}, k_{f2}, k_{f3} = Global field calibration parameters
β_{f1}, β_{f2}, β_{f3} = Local calibration constants;
=1.0 by default

Allowable Number of Load Applications (cont.)

$$N_f = k_{f1}\left(C\right)\beta_{f1}\left(\varepsilon_t\right)^{k_{f2}\beta_{f2}}\left(E_{HMA}\right)^{k_{f3}\beta_{f3}}$$

$$C = 10^M \qquad M = 4.84\left(\frac{V_{be}}{V_a + V_{be}} - 0.69\right)$$

V_{be}= Effective asphalt content by volume, percent
V_a = Percent air voids in the HMA mixture

Bottom-Up Cracking

$$FC_{bottom} = \left(\frac{6000}{1 + e^{(C_1 * C'_1 + C_2 * C'_2 * \log10(D*100))}} \right) * \left(\frac{1}{60} \right)$$

where:

FC_{bottom} = bottom-up fatigue cracking, percent lane area

D = bottom-up fatigue damage

C_1 = 1.0

$$C_1' = -2C_2' \qquad C_2 = 1$$

$$C'_2 = -2.40874 - 39.748 * (1 + hac)^{-2.856}$$

Top-Down Cracking

$$FC_{Top} = 10.56\left(\frac{C_4}{1+e^{\left(C_1 - C_2 Log\left(DI_{Top}\right)\right)}}\right)$$

where:

FC_{top} = top-down fatigue cracking, ft/mile

D = top-down fatigue damage

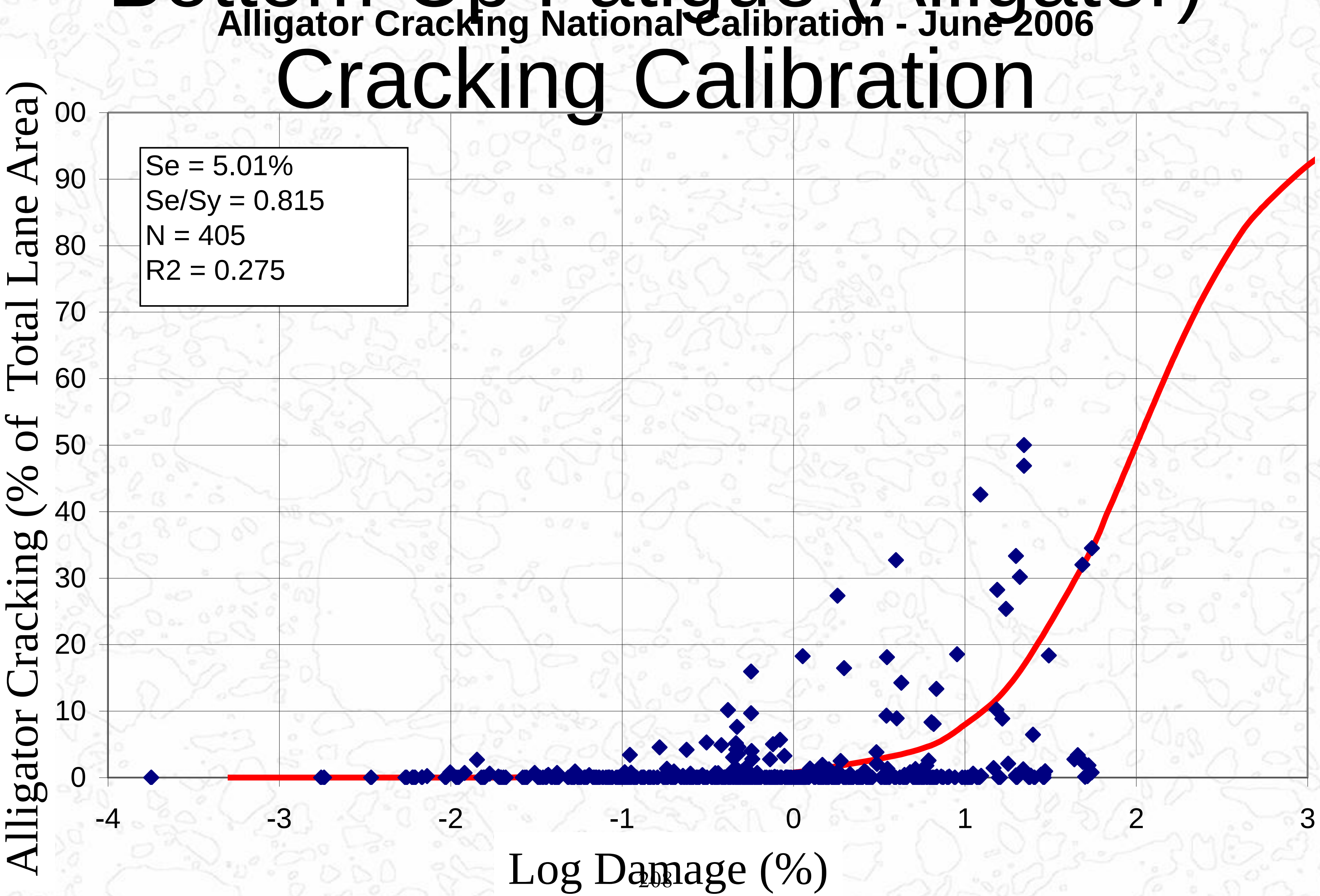
Factors Affecting Fatigue Cracking in Flexible Pavements

- ⇨HMA layer thickness.
- ⇨HMA layer dynamic modulus.
- ⇨Binder grade in the HMA mixture.
- ⇨Air voids in the asphalt layers.
- ⇨Effective binder content in the asphalt layers.

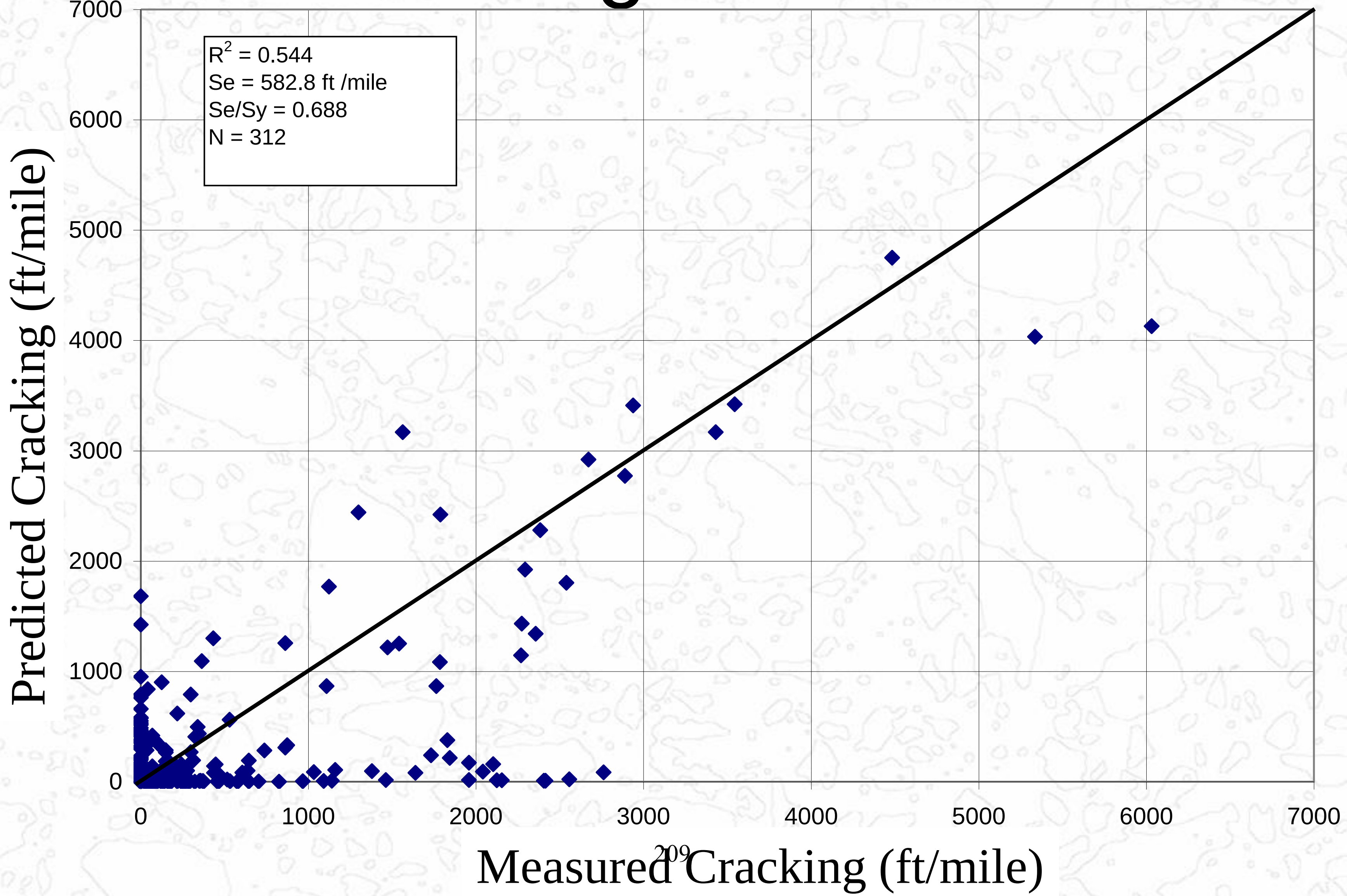
Factors Affecting Fatigue Cracking in Flexible Pavements

- ⇨Base thickness.
- ⇨Subgrade modulus.
- ⇨Traffic load configuration.
- ⇨Traffic load, contact area and tire pressure.
- ⇨Traffic load repetitions.
- ⇨Temperature and environmental conditions.

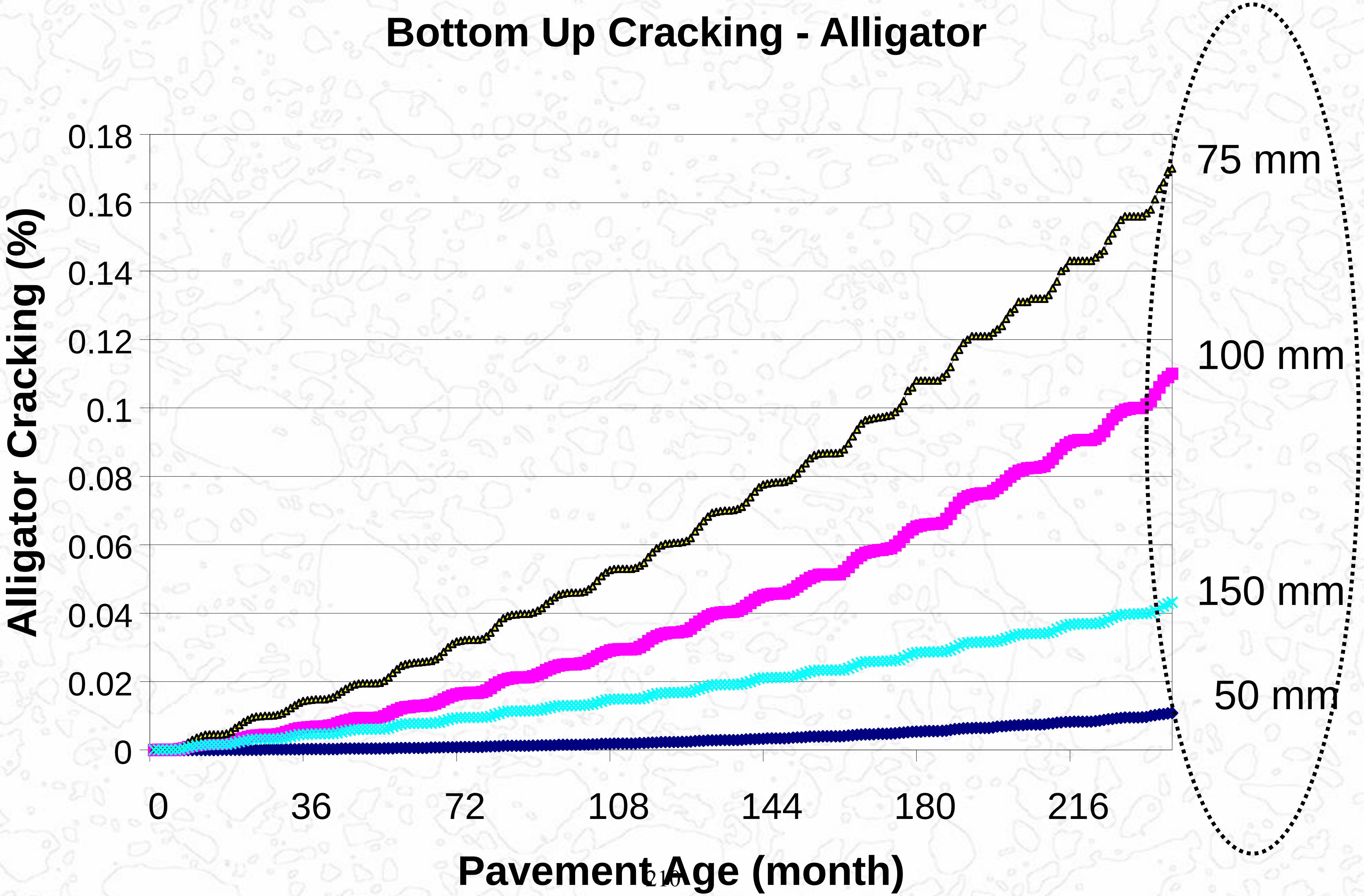
Bottom-Up Fatigue (Alligator) Cracking Calibration



Top-Down Fatigue (Longitudinal) Cracking Calibration



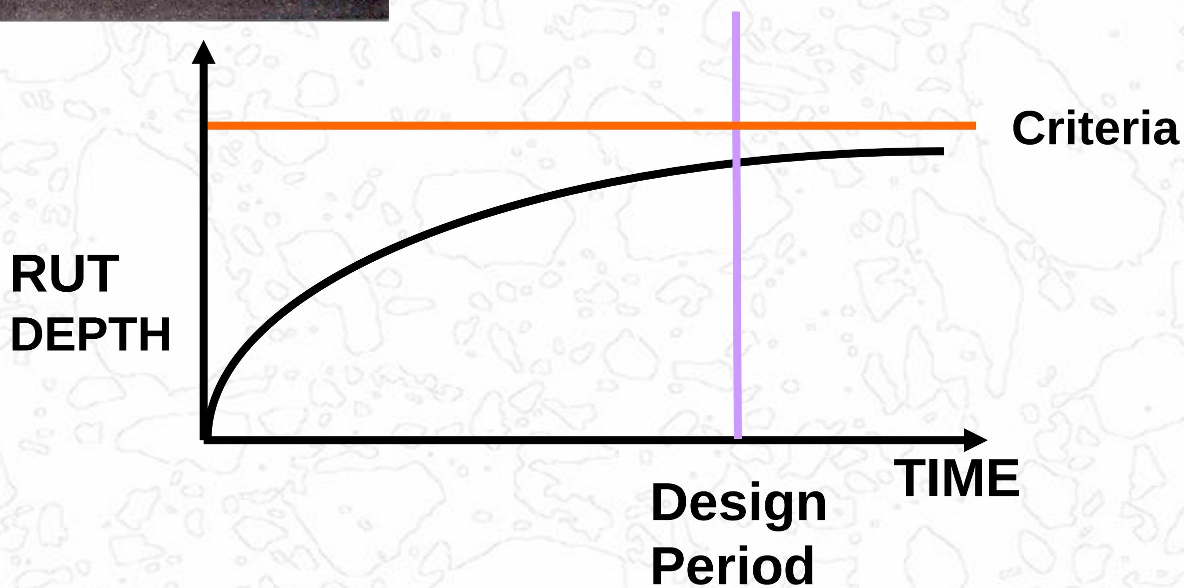
Effect of AC Thickness



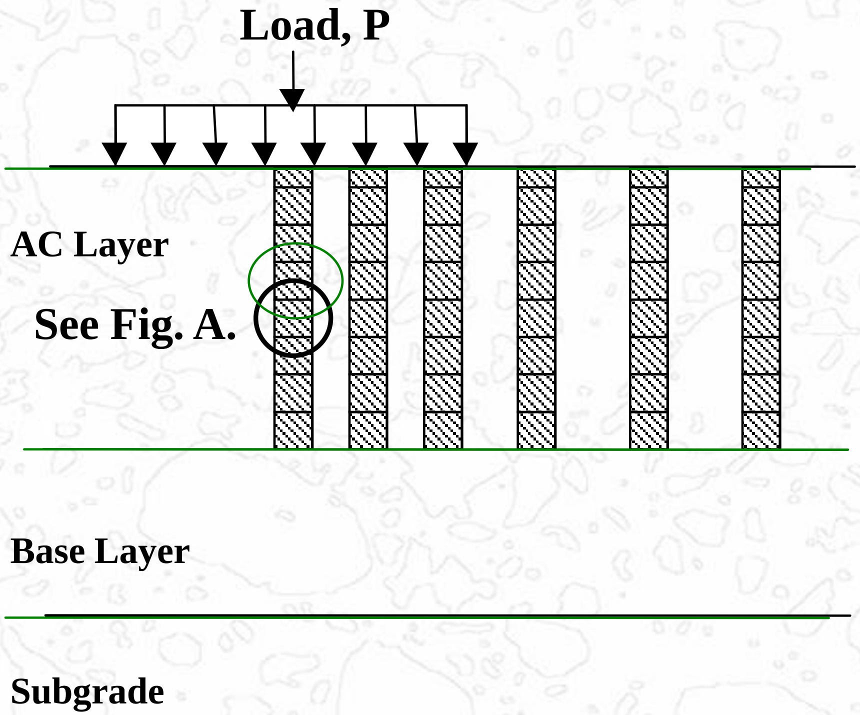
Permanent Deformation Accumulates Over Time

$$\Delta RD = \sum_{k=1}^{m} \sum_{i=1}^{j} \sum_{d=1}^{l} \left[\varepsilon_P(d)\, h_d \right]_{k,i}$$

Load Month Depth



Accumulation of Rutting



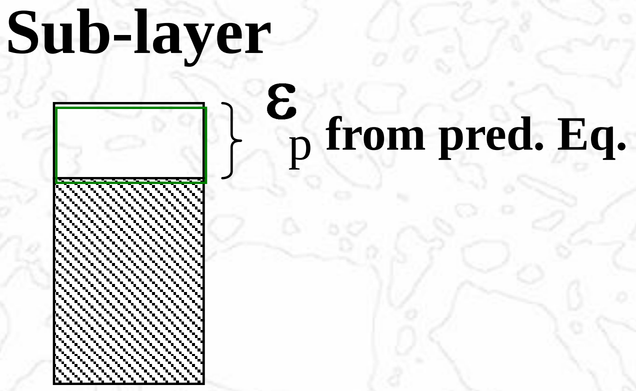


Fig. A

$$PD = \sum_{i=1}^{N\ sub\text{-}layers} \varepsilon_p^i \times h^i$$

Similar for unbound layers

Permanent Deformation in AC Layer

$$\frac{\Delta_{p(HMA)}}{\varepsilon_{p(HMA)}} = h_{HMA} = \beta_{r1} k_z \varepsilon_{r(HMA)} 10^{-3.35412} N^{0.4791*\beta_{2r}} T^{1.5606*\beta_{3r}}$$

where:

ε_p =Accumulated plastic strain at N repetitions of load (in/in)

ε_r = Resilient strain of the asphalt material as a function of mix properties, temperature and time rate of loading (in/in)

N = Number of load repetitions

T = Temperature (deg F)

a_i = Non-linear regression coefficients

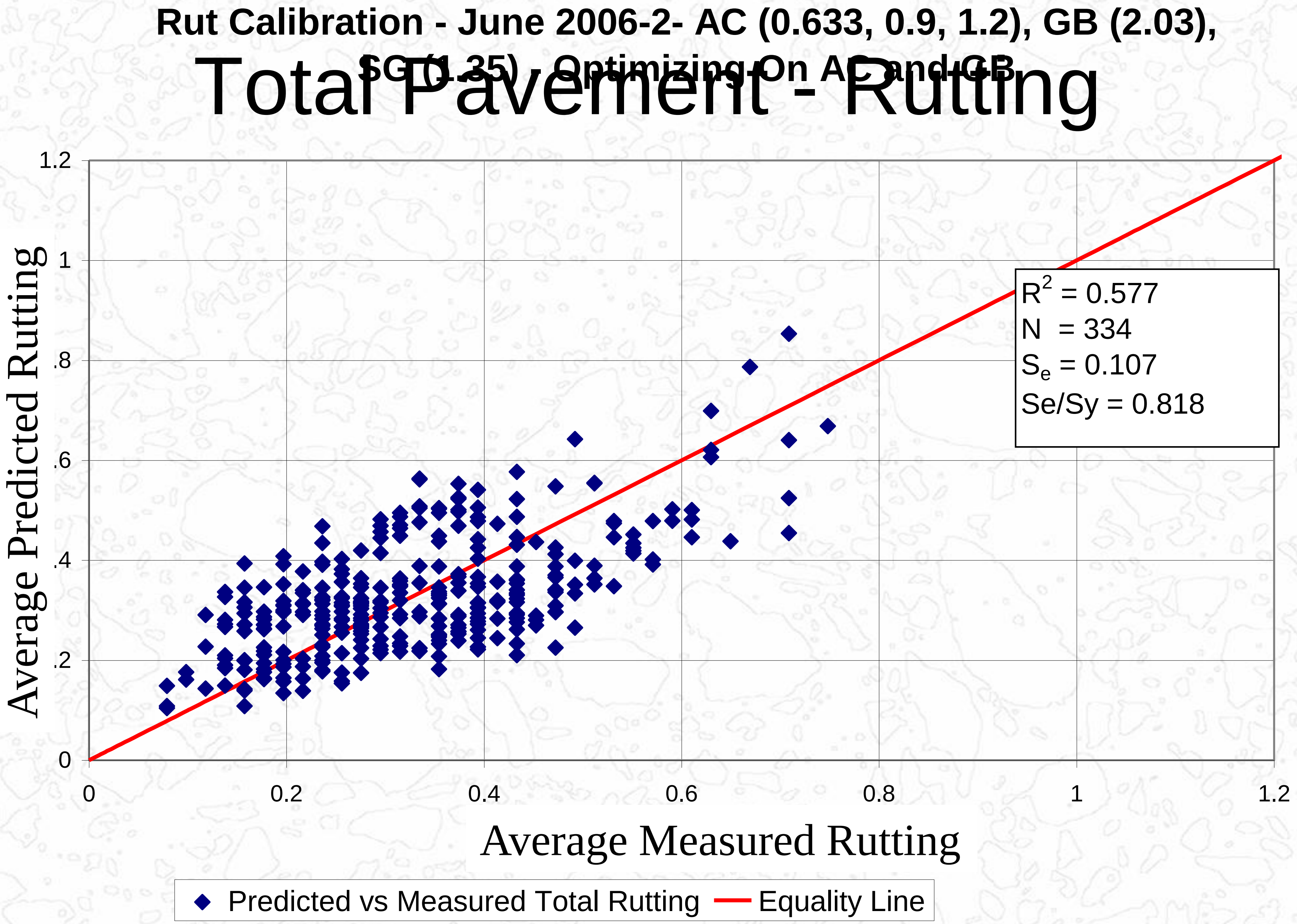
β_i = field calibration factors

Permanent Deformation in Unbound Layer (Tseng and Lytton Model)

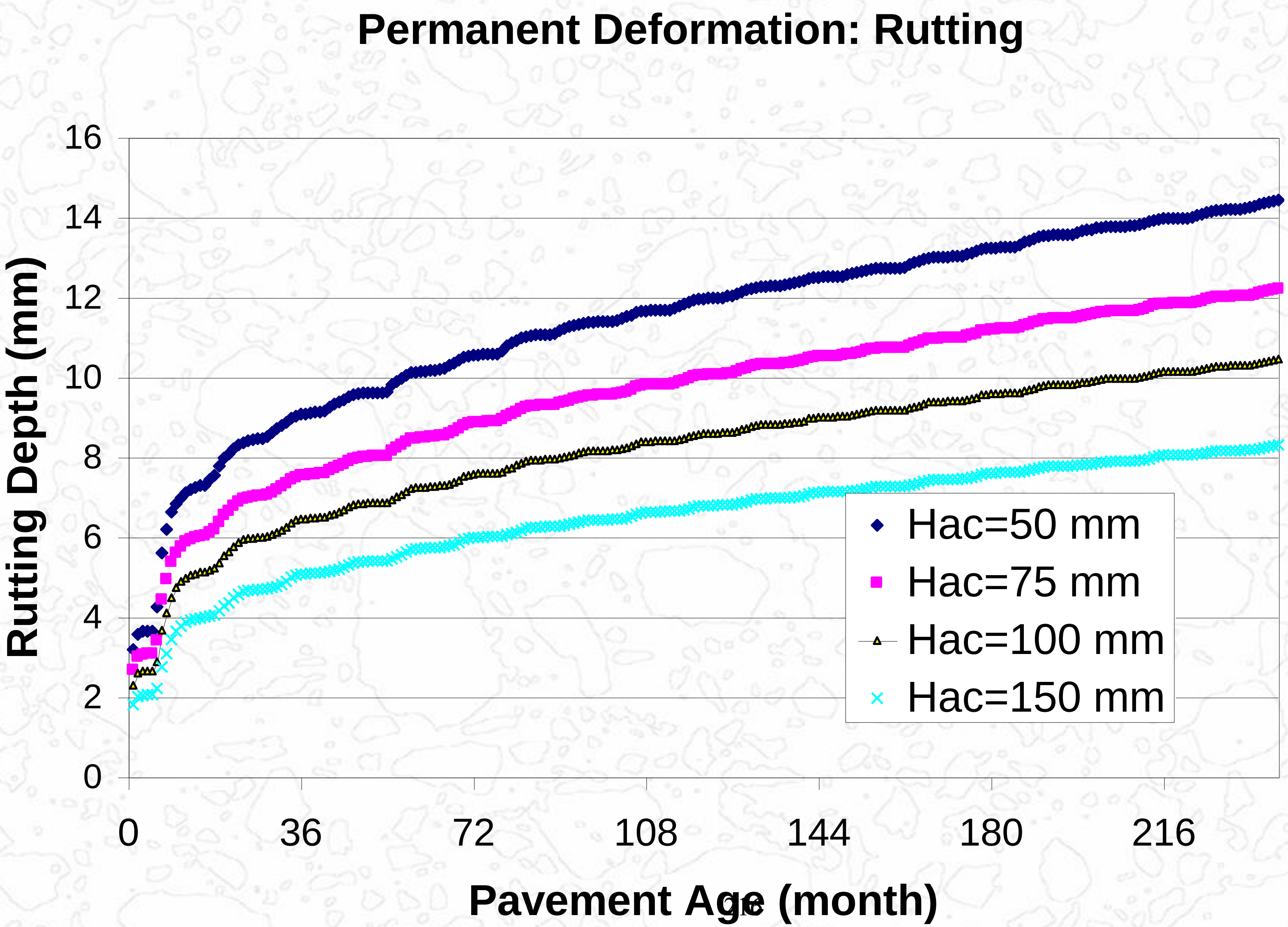
$$\Delta_{p(soil)} = \beta_{s1} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_o}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{N}\right)^{\beta}}$$

$\Delta_{p(Soil)}$ = Permanent or plastic deformation for the layer/sublayer
N = Number of axle load applications
ε_o, β, and ρ = material properties obtained for the resilient strain ε_r
ε_v = Average vertical resilient or elastic strain in the layer/sublayer
h_{Soil} = Thickness of the unbound layer/sublayer, inches
k_{s1} = Global calibration coefficients;
=1.673 for granular materials
=1.35 for fine-grained materials
β_{s1} = Local calibration constant

Total Pavement - Rutting



Effect of AC Thickness

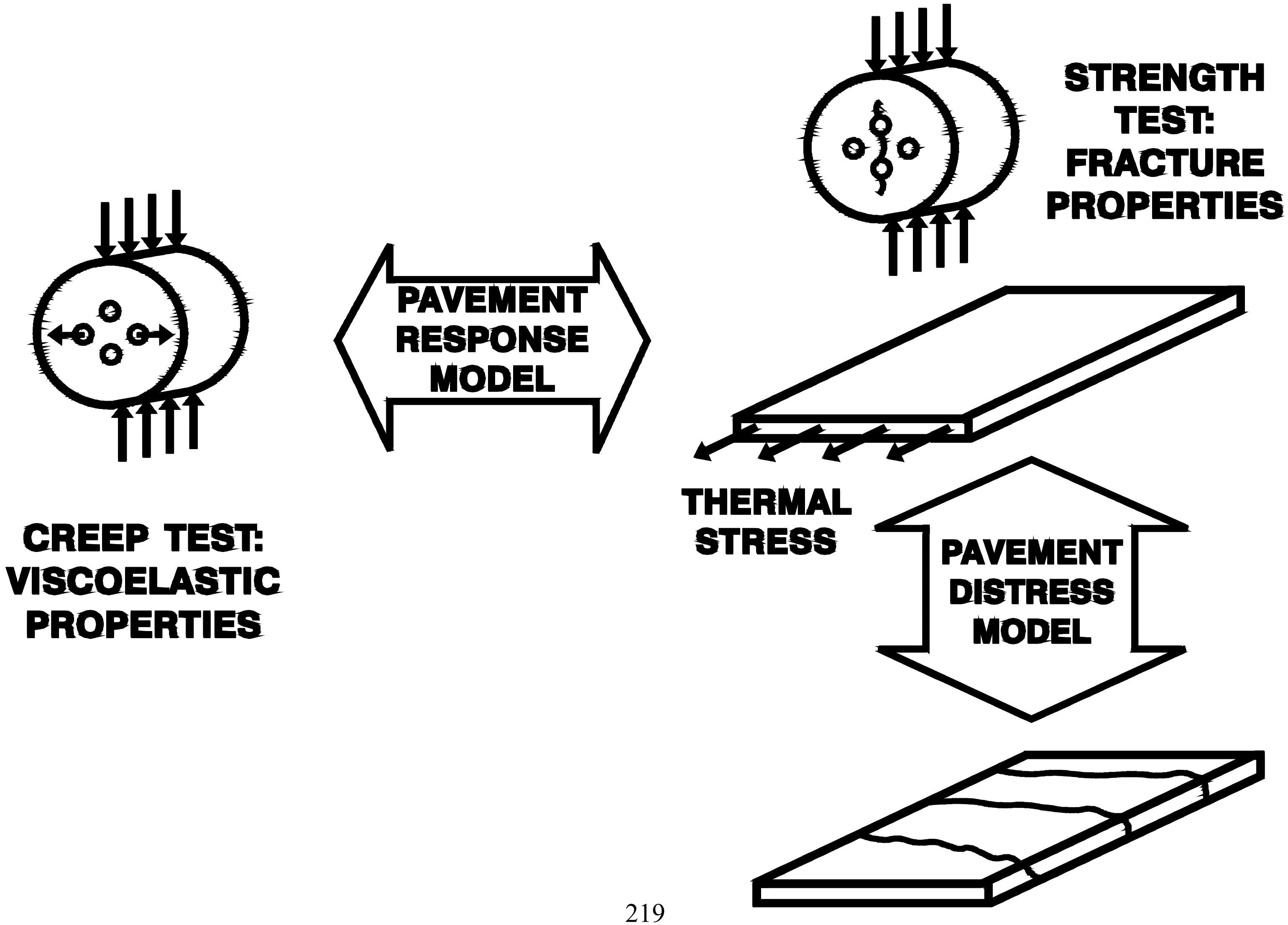


Thermal Cracking

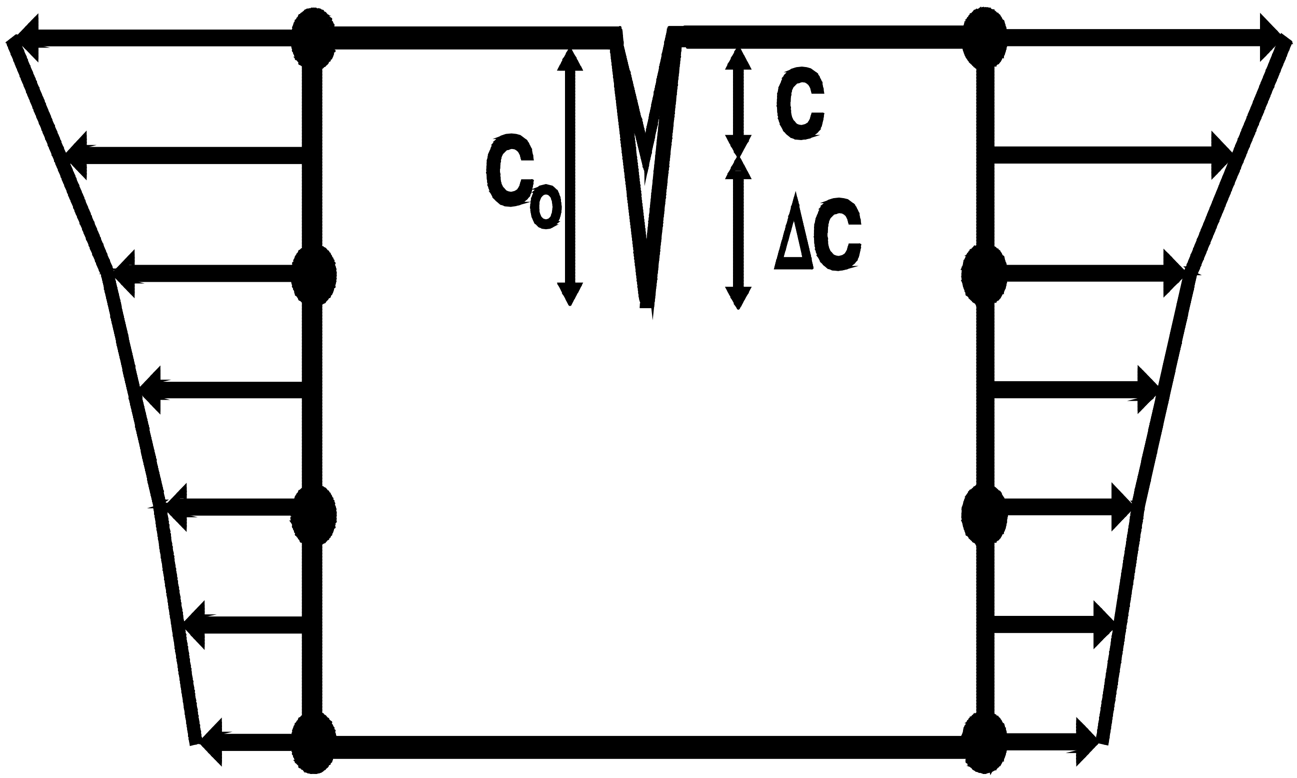
HMA-Thermal Fracture

- **Uses SHRP Thermal Fracture Model**
  - Recalibrated Using Approximately 30 Sections in NCHRP Project 9-19
- **Thermal Fatigue (cyclic)**
  - Propagation of Cracks Through the Asphalt Layer
- **Thermal Stresses**
  - Very Low Temperature
  - Mixture Properties
  - Friction
- **Mixture Fracture Properties**

Materials Characterization (IDT)



Schematic of Crack Depth Fracture Model



Amount of Crack Propagation in a Cooling Cycle

$$\Delta C = A \Delta K^n$$

ΔC	=	Change in the crack depth due to a cooling cycle.
ΔK	=	Change in the stress intensity factor
A, n	=	Fracture parameters for the asphalt mixture

Stress Intensity Factor Approximation

$$K = \sigma(0.45 + 1.99C_o^{0.56})$$

K = stress intensity factor
σ= far-field stress from pavement response model at depth of crack tip
C_o= current crack length

Schapery-Molenaar-Lytton Model

$$n = 0.8\left(1 + \frac{1}{m}\right)$$

$$A = 10^{\left(\beta * (4.389 - 2.52 * \log(E * \sigma_m * n)\right)}$$

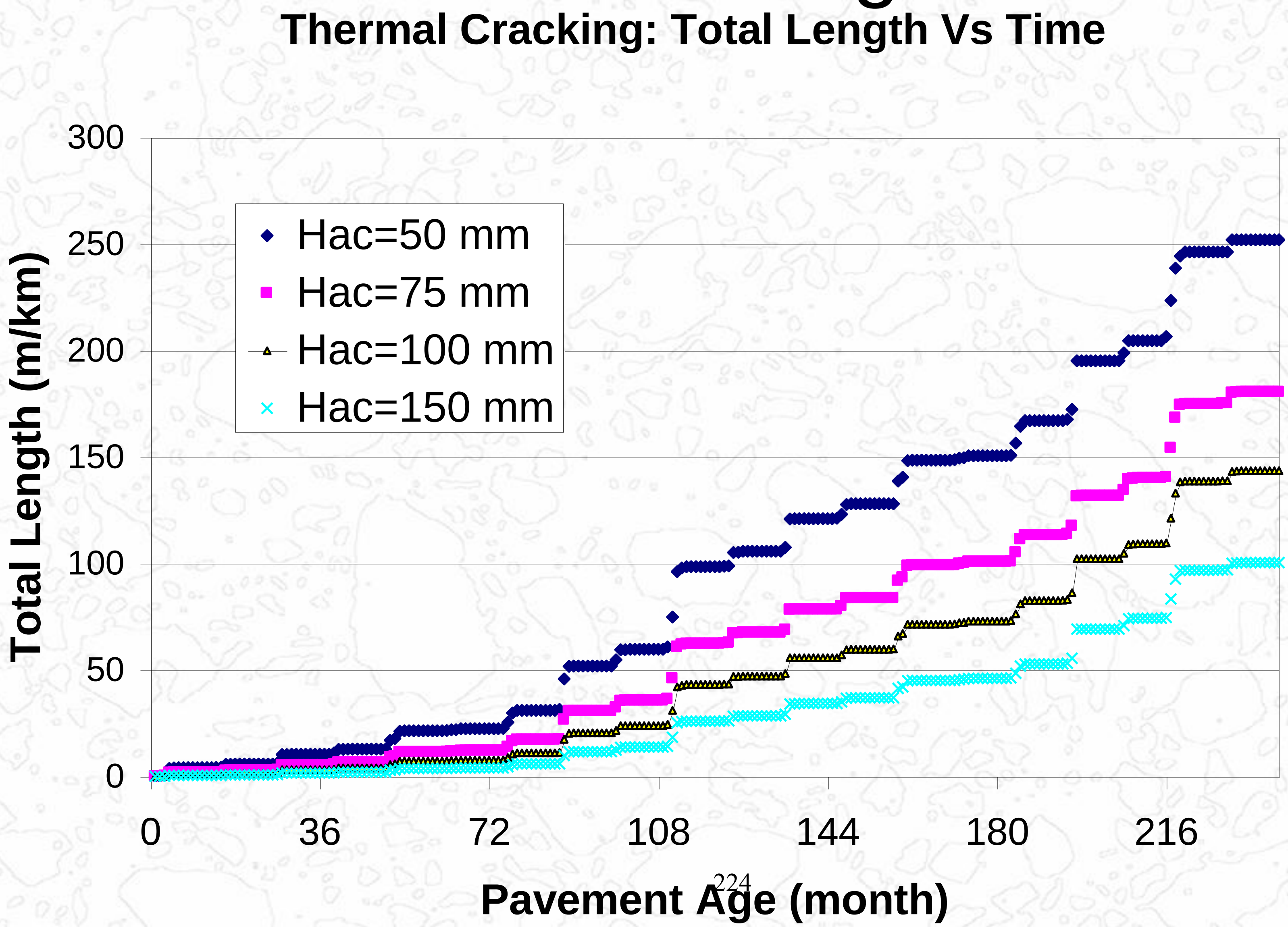
where:

E =Mixture stiffness.

σ_m =Undamaged mixture tensile strength.

b =Calibration parameter.

Effect of AC Thickness on Thermal Cracking



Pavement Smoothness – IRI

Generalized Smoothness Model

$$\mathbf{IRI = IRI_i + \Delta IRI_D + \Delta\ IRI_{SF}}$$

IRI_i = Initial IRI at construction

ΔIRI_D = Change in IRI due to distress

ΔIRI_{SF} = Change in IRI due to site factors

(age, subgrade properties, non-load distress)

Site Factor

$$SF = Age\left(0.02\left(PI+1\right)+0.008\left(Precip+1\right)+0.00064\left(FI+1\right)\right)$$

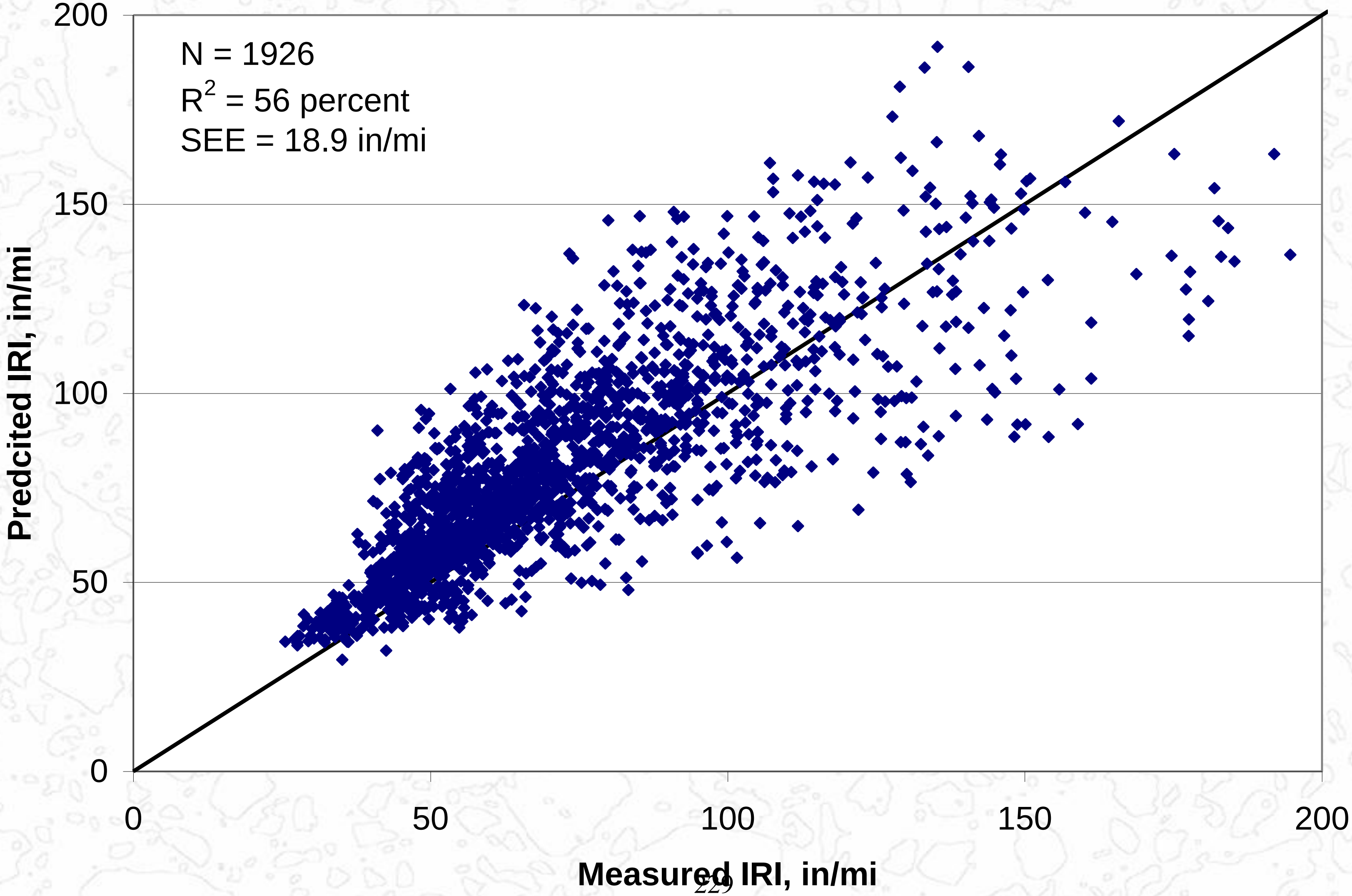
Age = Pavement age, years
PI = Percent plasticity index of the soil
FI = Average annual freezing index, degree F days
Precip= Average annual precipitation or rainfall, inches

Generalized Smoothness Model

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

IRI_o = Initial IRI after construction, in./mi.
SF = Site factor
FC_{Total} = Area of fatigue cracking ft^2/mi
TC = Length of transverse cracking ft./mi.
RD = Average rut depth, inches

IRI Model Calibration



M-E PDG for flexible pavements

<u>Summary</u>

- ⇨ Incremental Damage Approach
- ⇨ Sub-layering for structural analysis
- ⇨ Aging model (surface, air void adjustment, depth model)
- ⇨ Enhanced Integrated Climate Model (EICM)
  - ⇨ Temperature
  - ⇨ Moisture

M-E PDG for flexible pavements
Summary

- The M-E PDG incorporated the following performance prediction models
  - Load Related Cracking
  - Rutting Models
  - Thermal Cracking
  - Roughness
- The models are calibrated based on the performance data from the LTPP sections located throughout the US and Canada.
- Local calibration of the models is recommended

More Information

www.trb.org/M-E PDG

⇨Guide Documentation

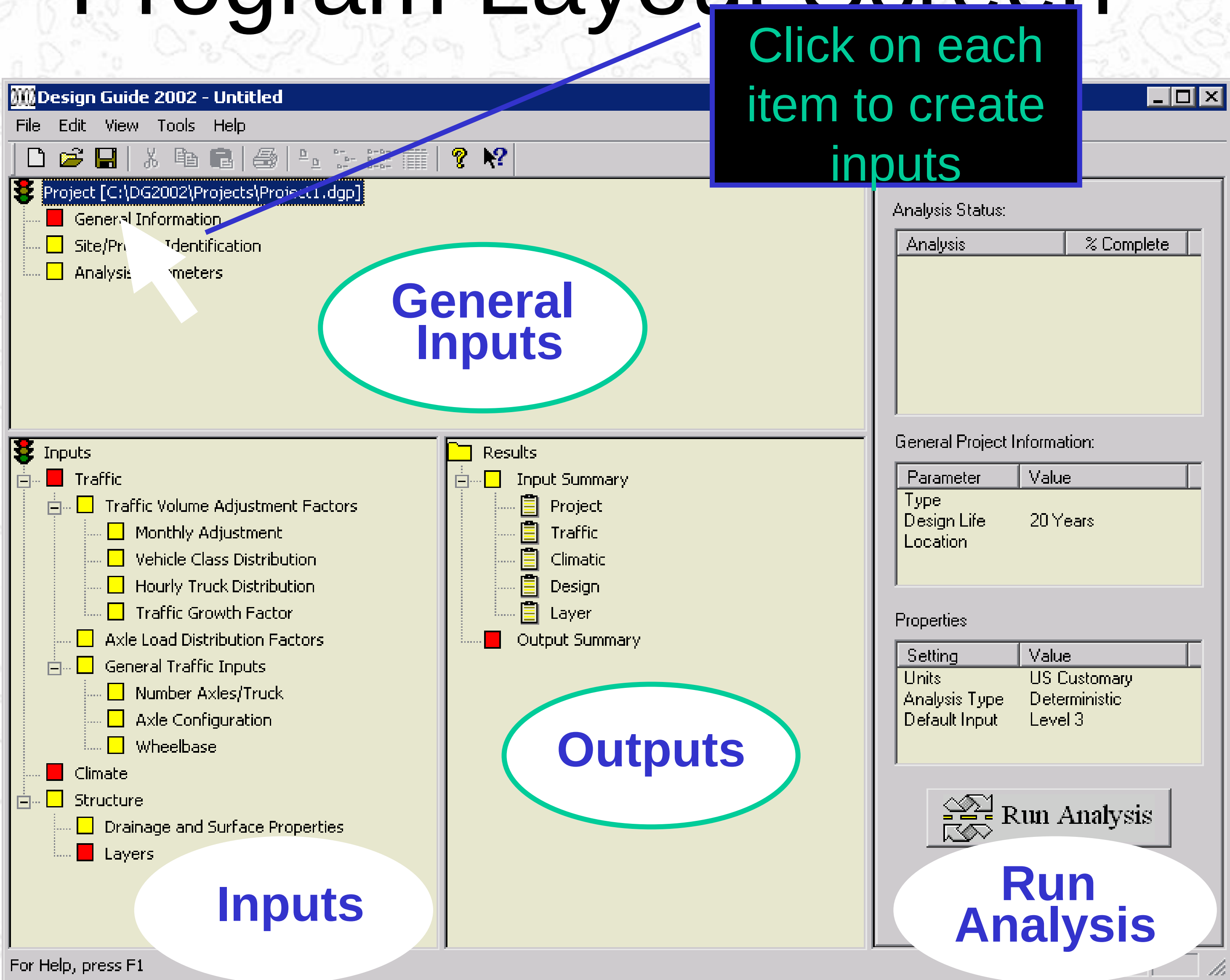
⇨Software

⇨Climatic database

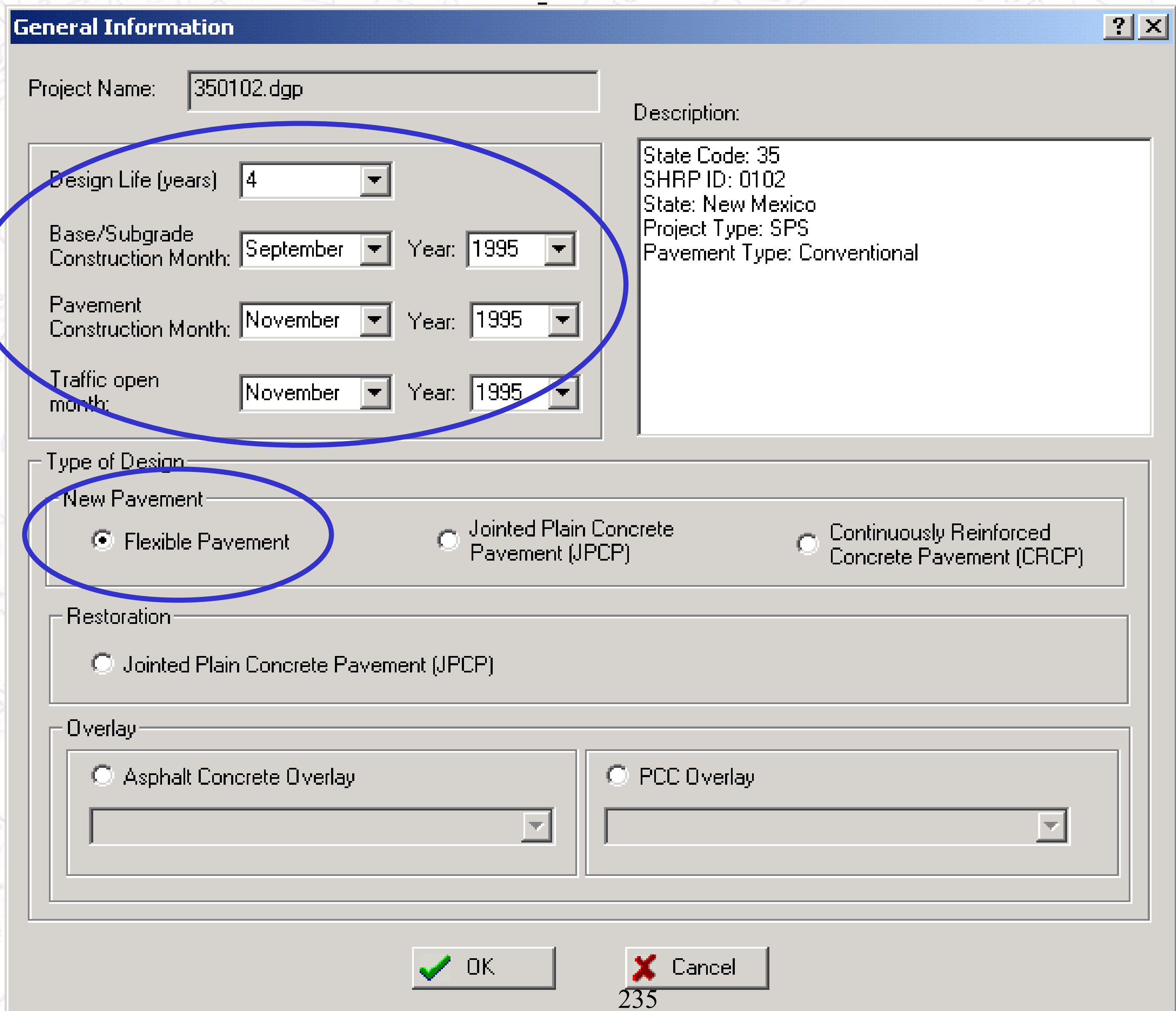
M-EPDG Software WorkShop

Step-by-step procedure

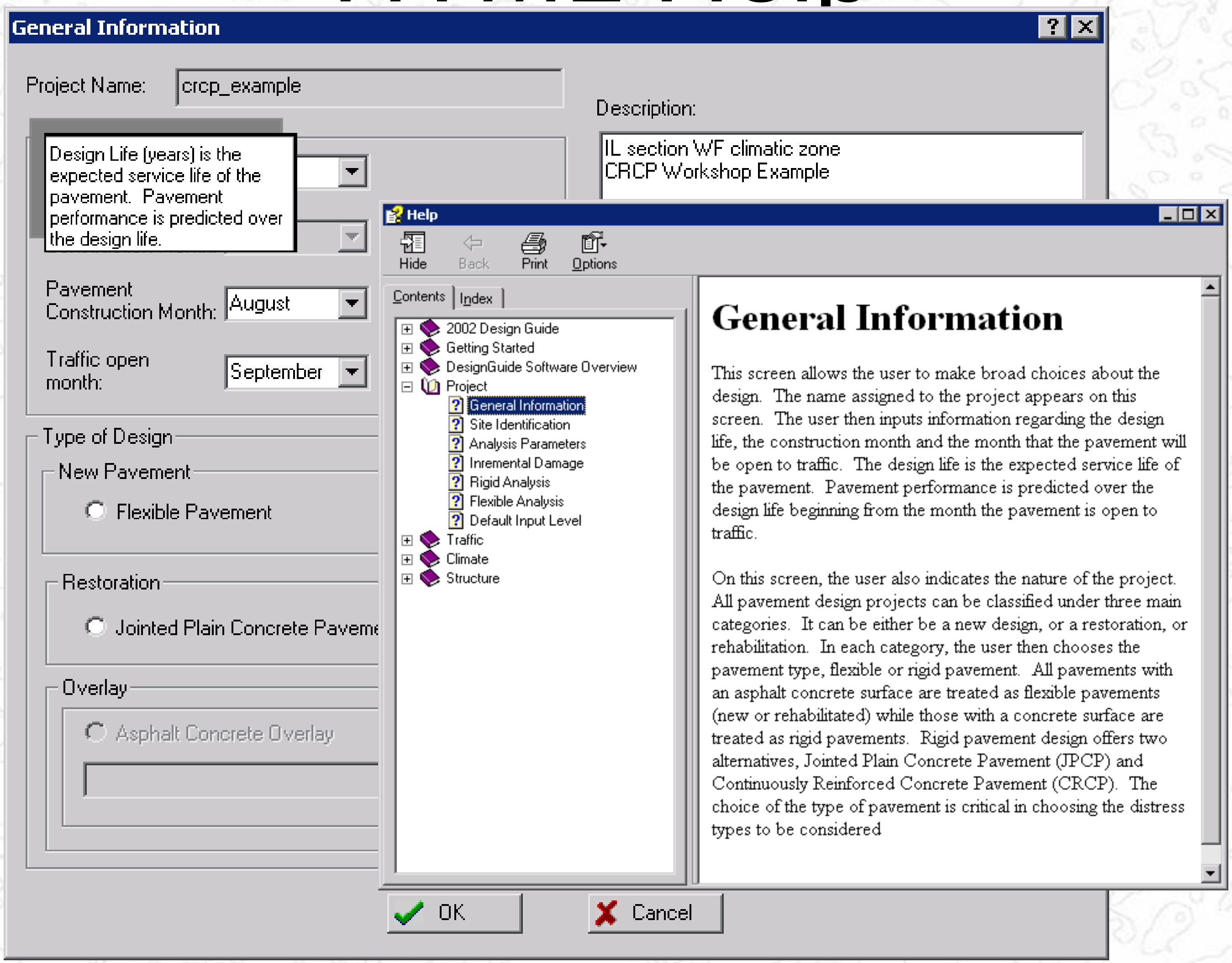
Program Layout Screen



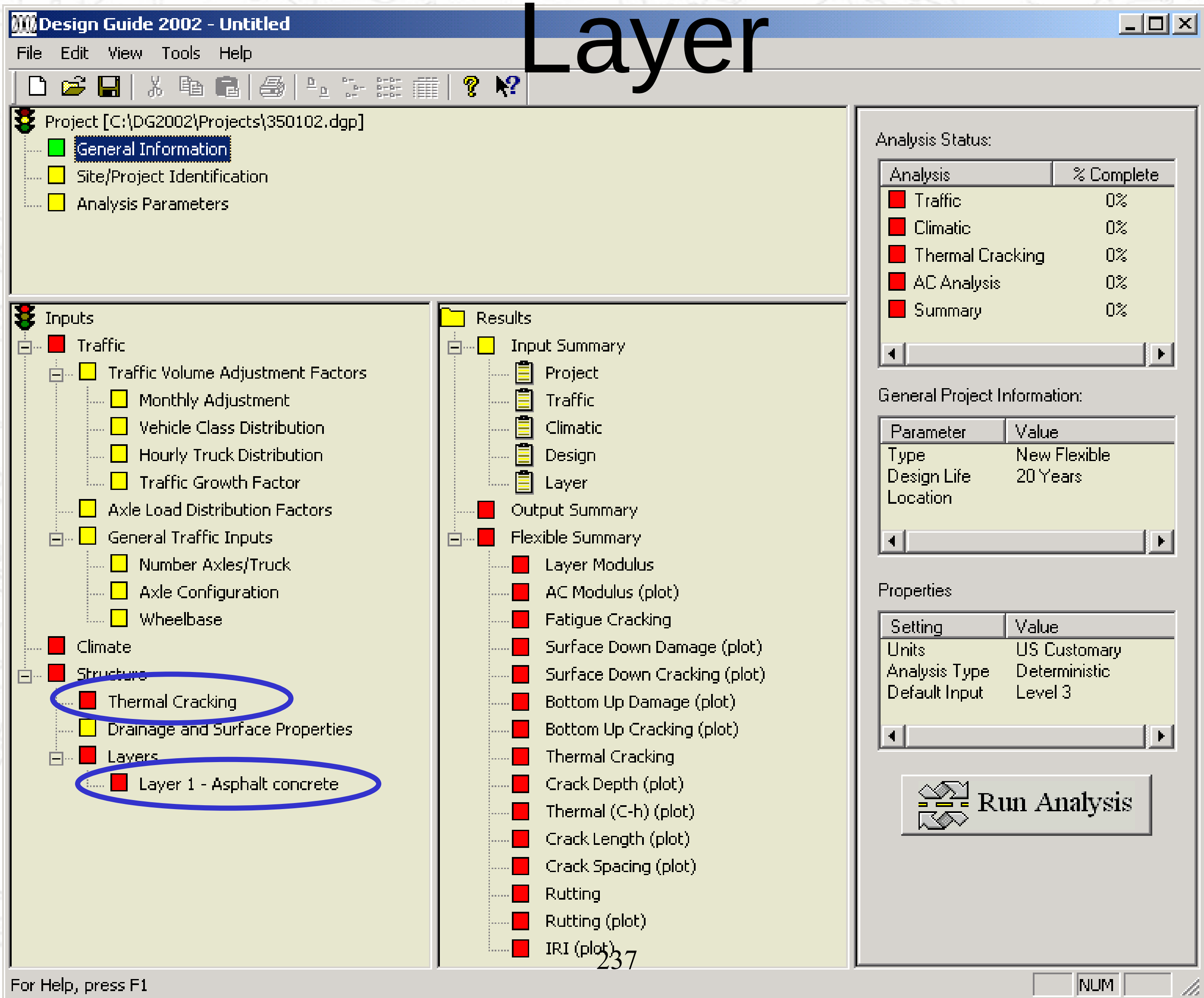
General Information Screen -



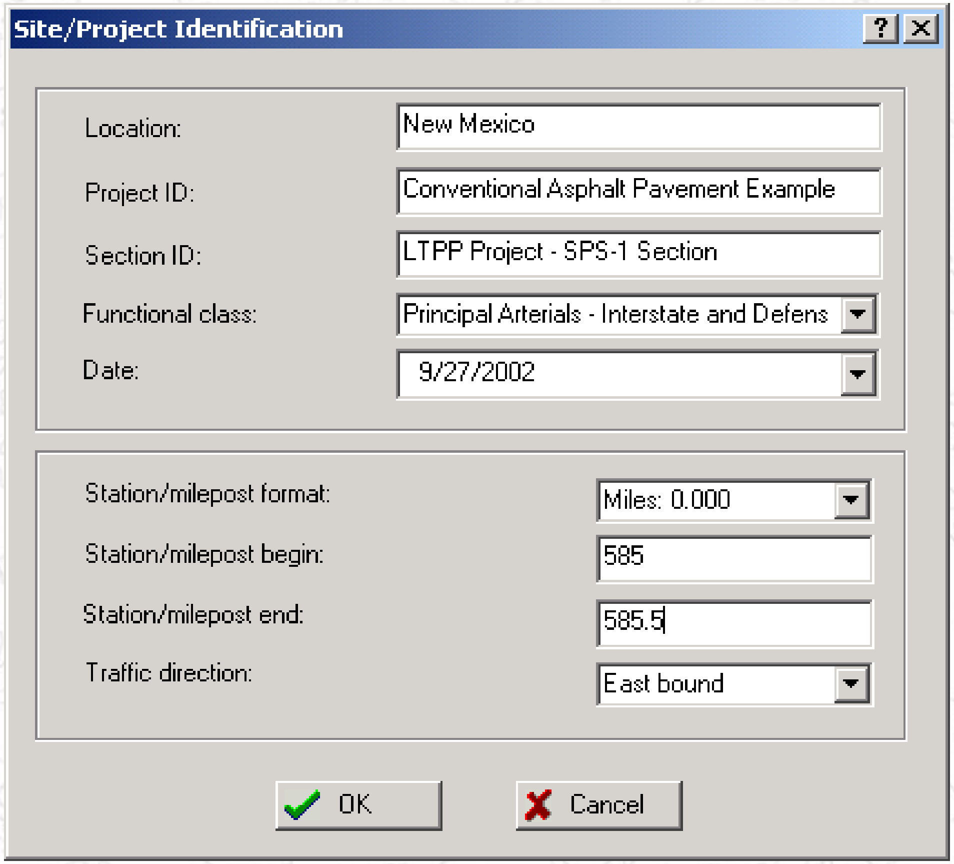
Help Options – CSH and HTML Help



Software Inserts the Thermal Cracking Screens and an AC Layer

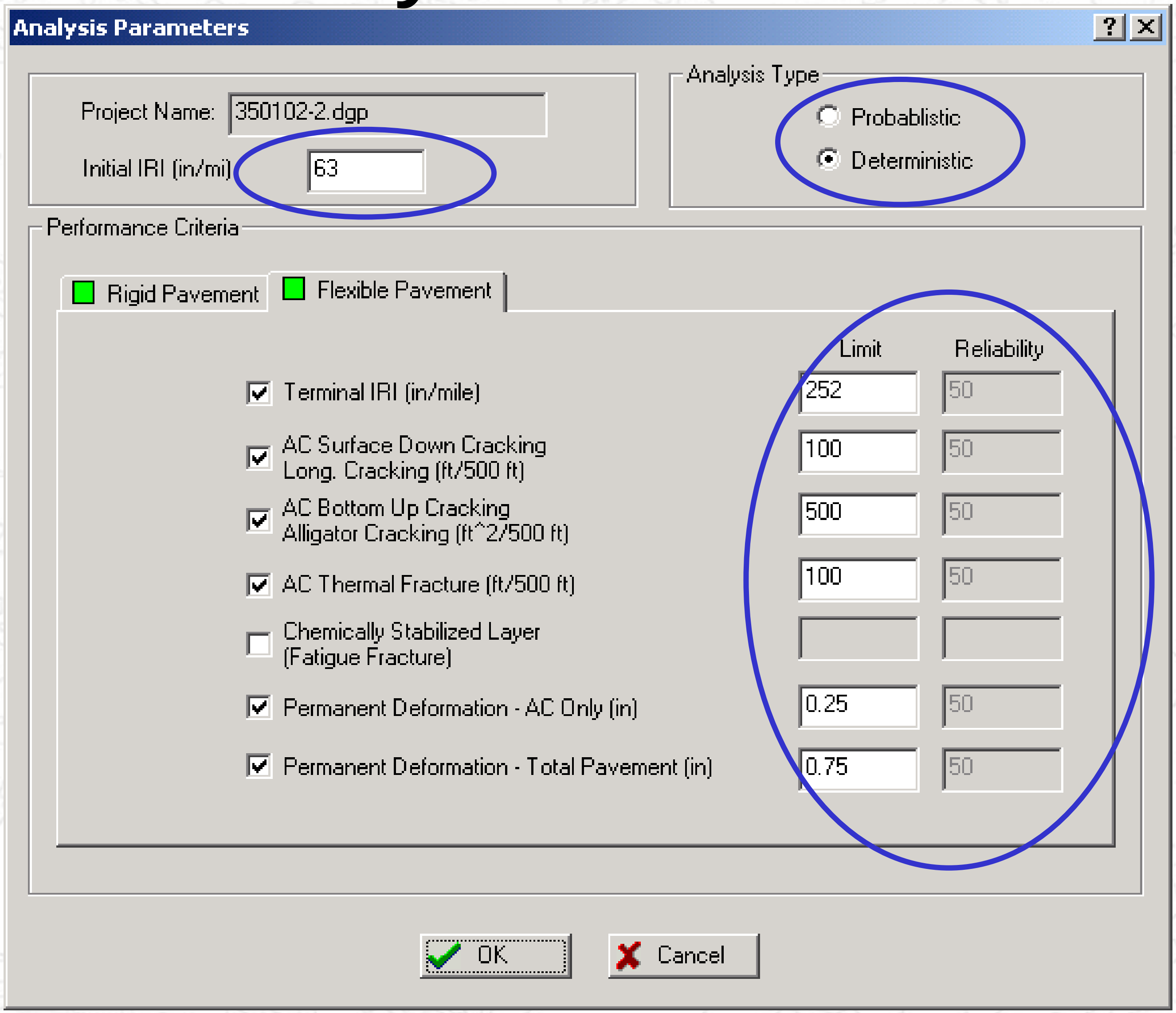


Site/Project Identification

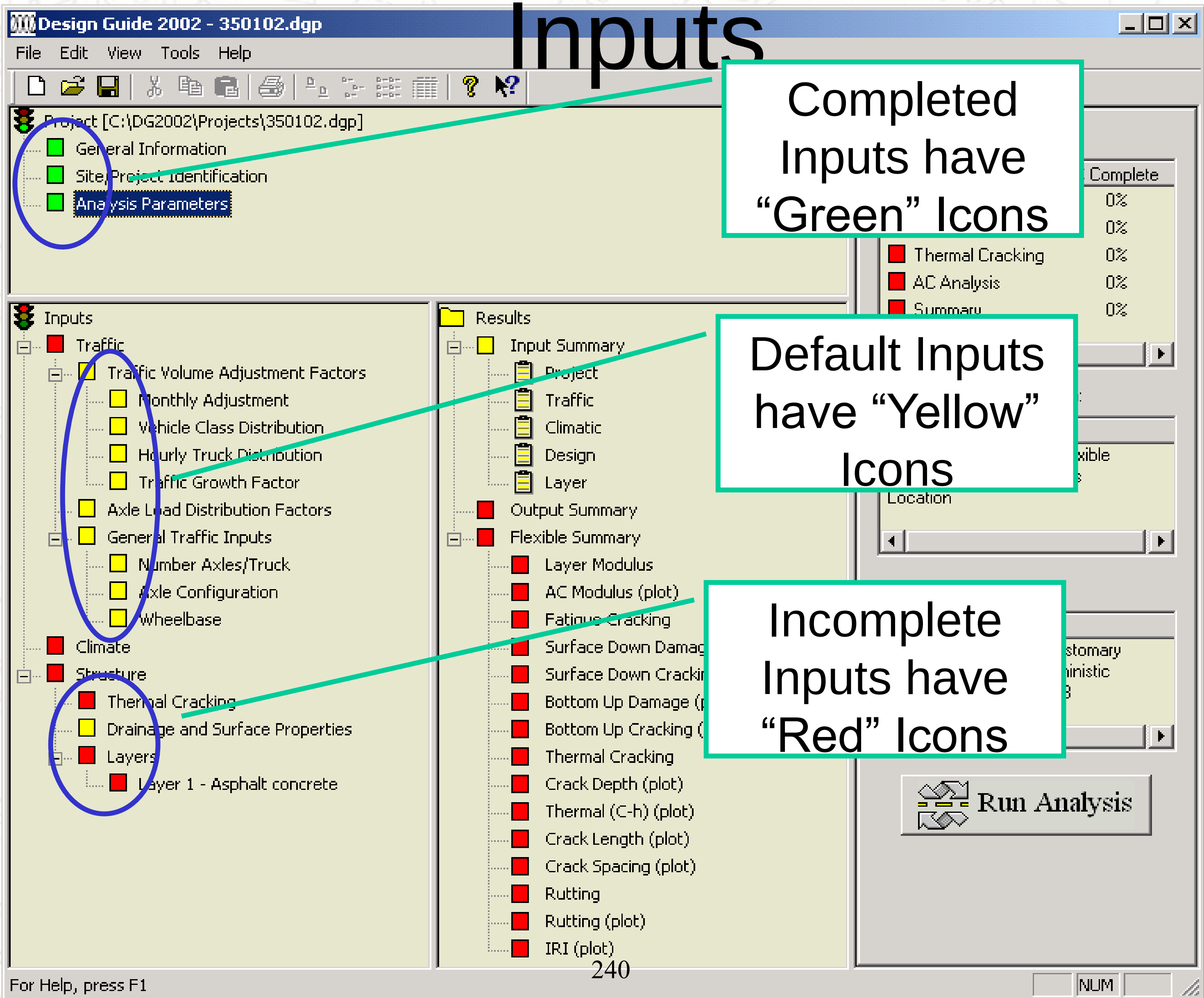


Information provided on this screen is only for the purpose of identification. These inputs will not affect the design in any way.

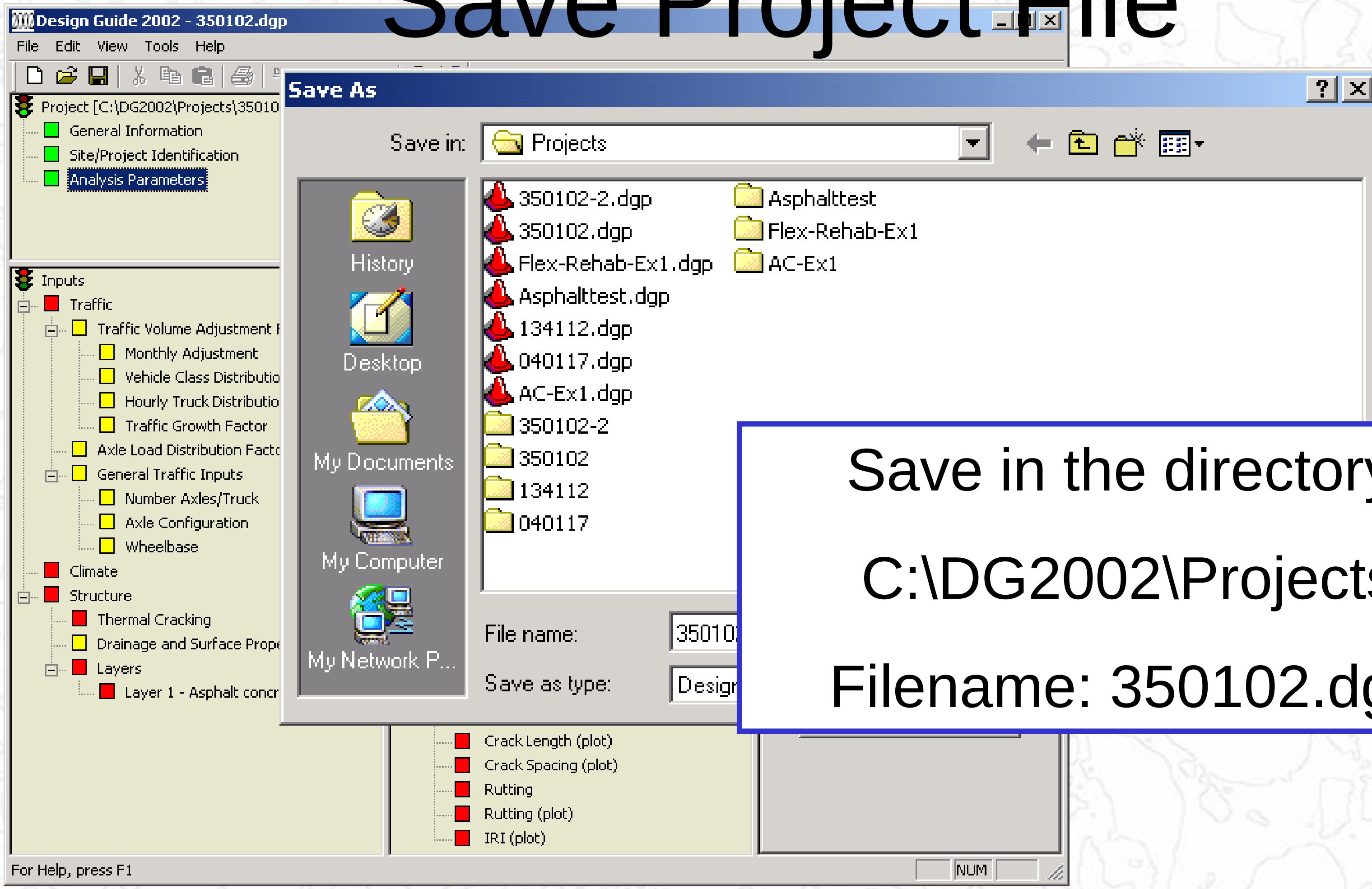
Analysis Parameters



Program Indicates Status of Inputs



Save Project File



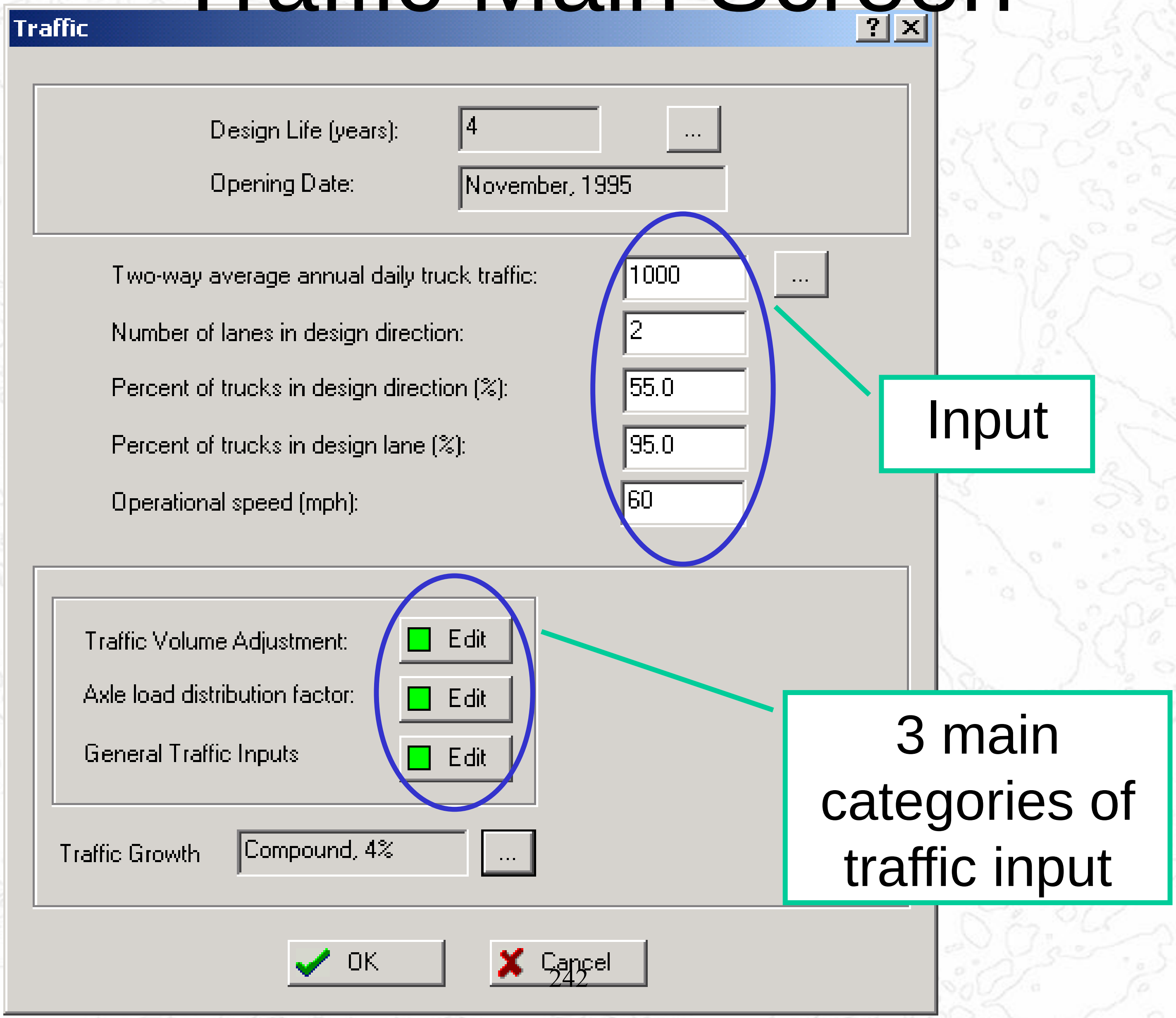
Save in the directory:

C:\DG2002\Projects

Filename: 350102.dgp

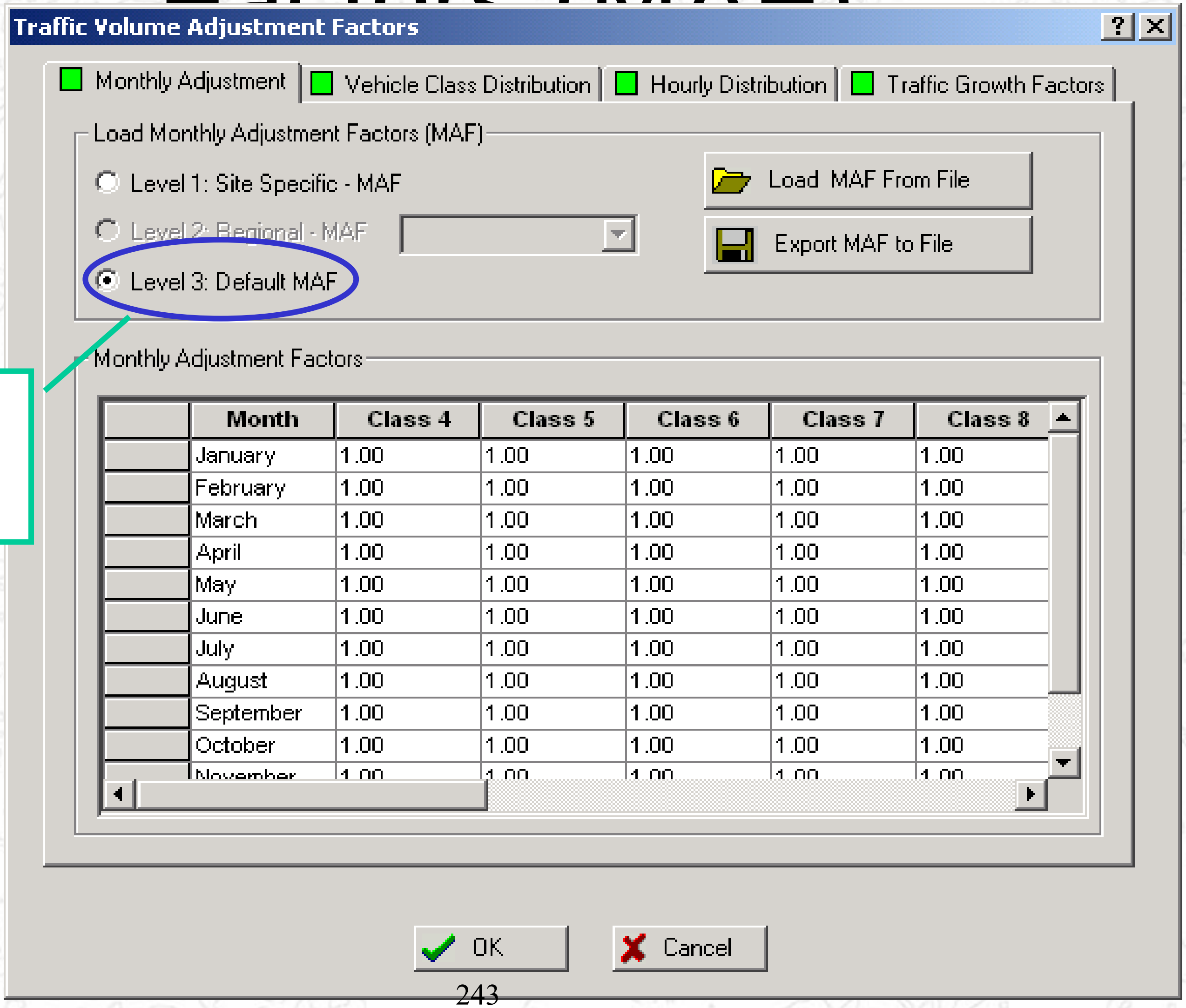
Program automatically creates a file called "350102" in C:\DG2002\Projects\ to store all project files

Traffic Main Screen



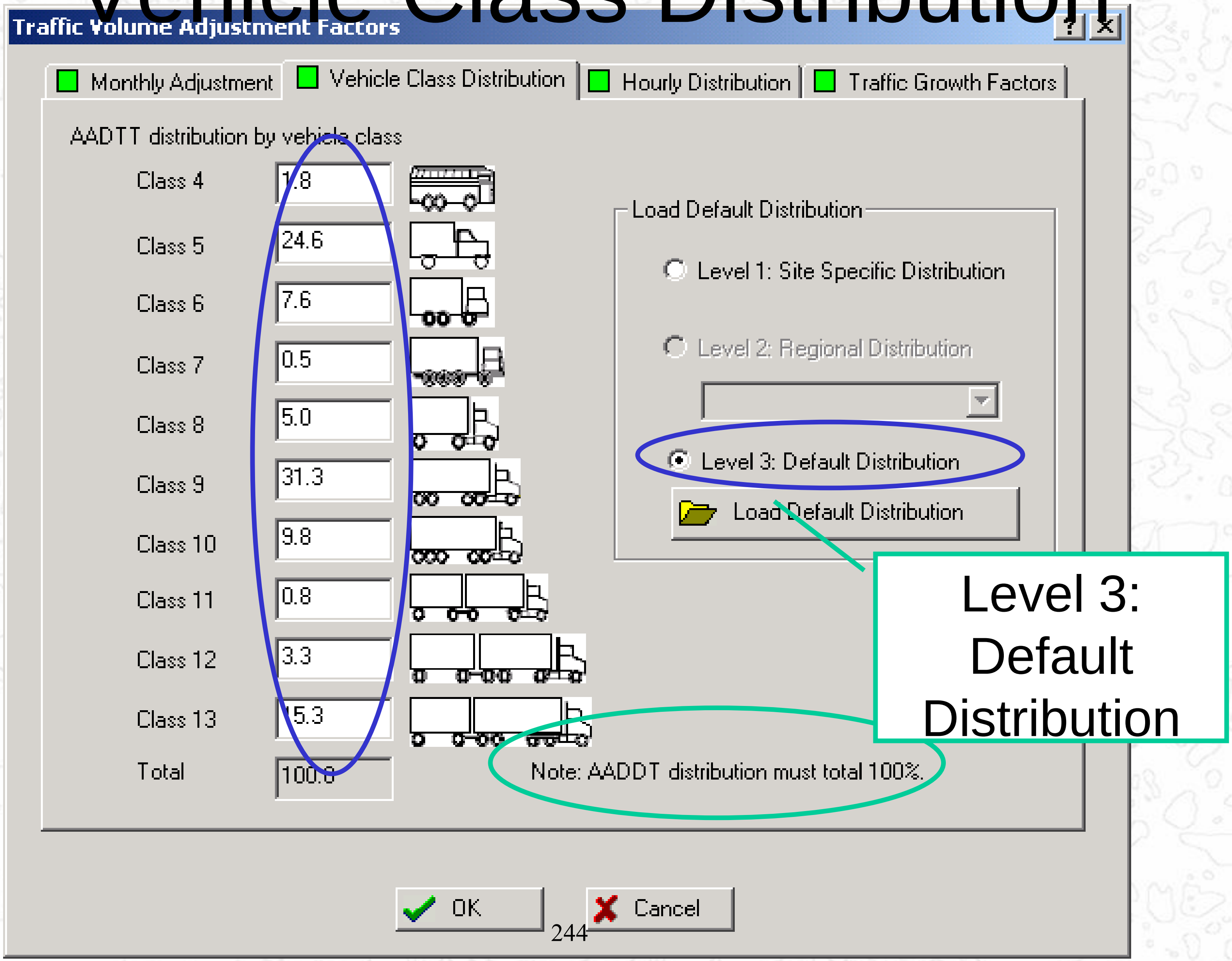
Traffic Volume Adjustment Factors Monthly Adjustment Factors (MAF)

Level 3: Default MAF

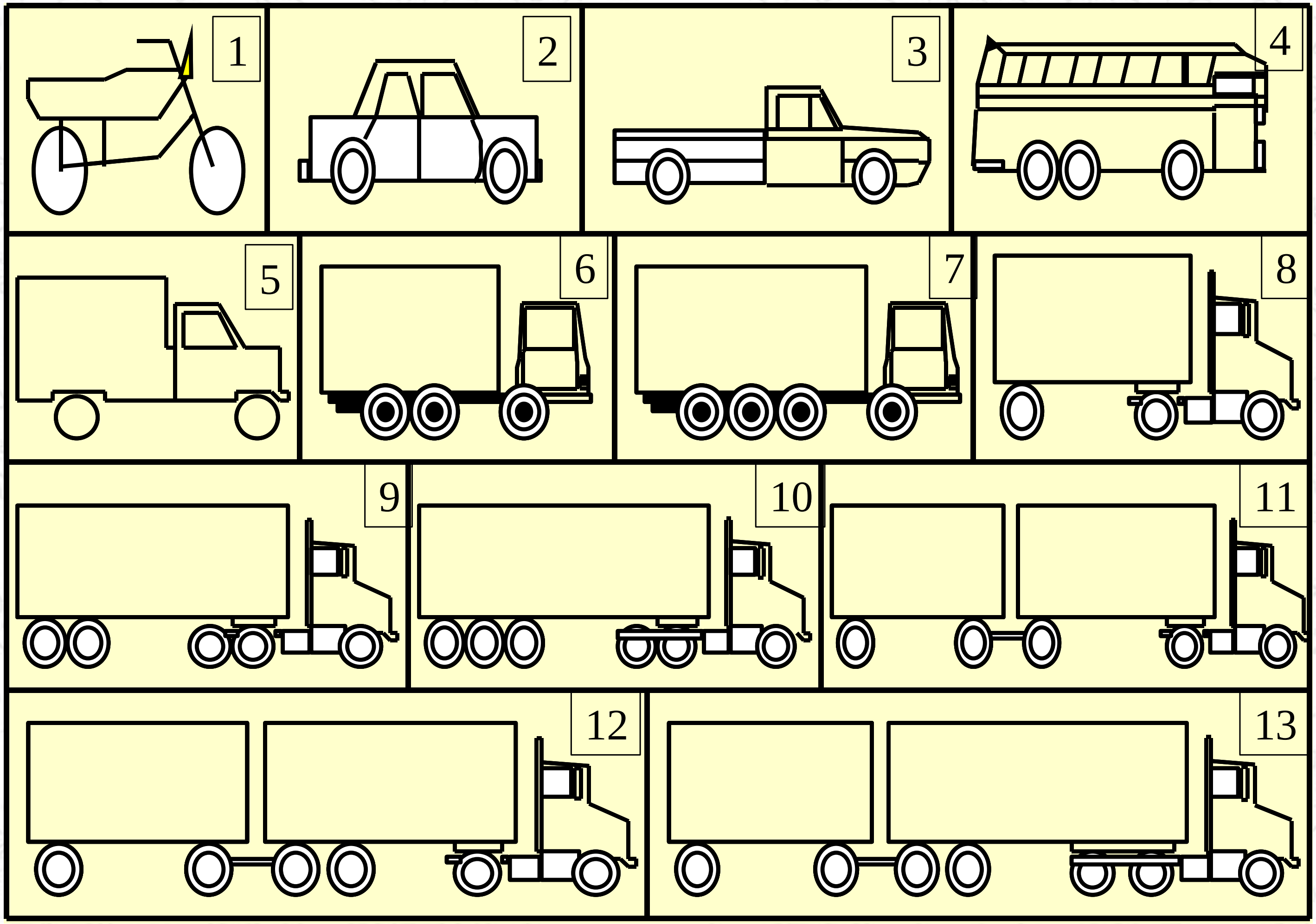


	Month	Class 4	Class 5	Class 6	Class 7	Class 8
	January	1.00	1.00	1.00	1.00	1.00
	February	1.00	1.00	1.00	1.00	1.00
	March	1.00	1.00	1.00	1.00	1.00
	April	1.00	1.00	1.00	1.00	1.00
	May	1.00	1.00	1.00	1.00	1.00
	June	1.00	1.00	1.00	1.00	1.00
	July	1.00	1.00	1.00	1.00	1.00
	August	1.00	1.00	1.00	1.00	1.00
	September	1.00	1.00	1.00	1.00	1.00
	October	1.00	1.00	1.00	1.00	1.00
	November	1.00	1.00	1.00	1.00	1.00

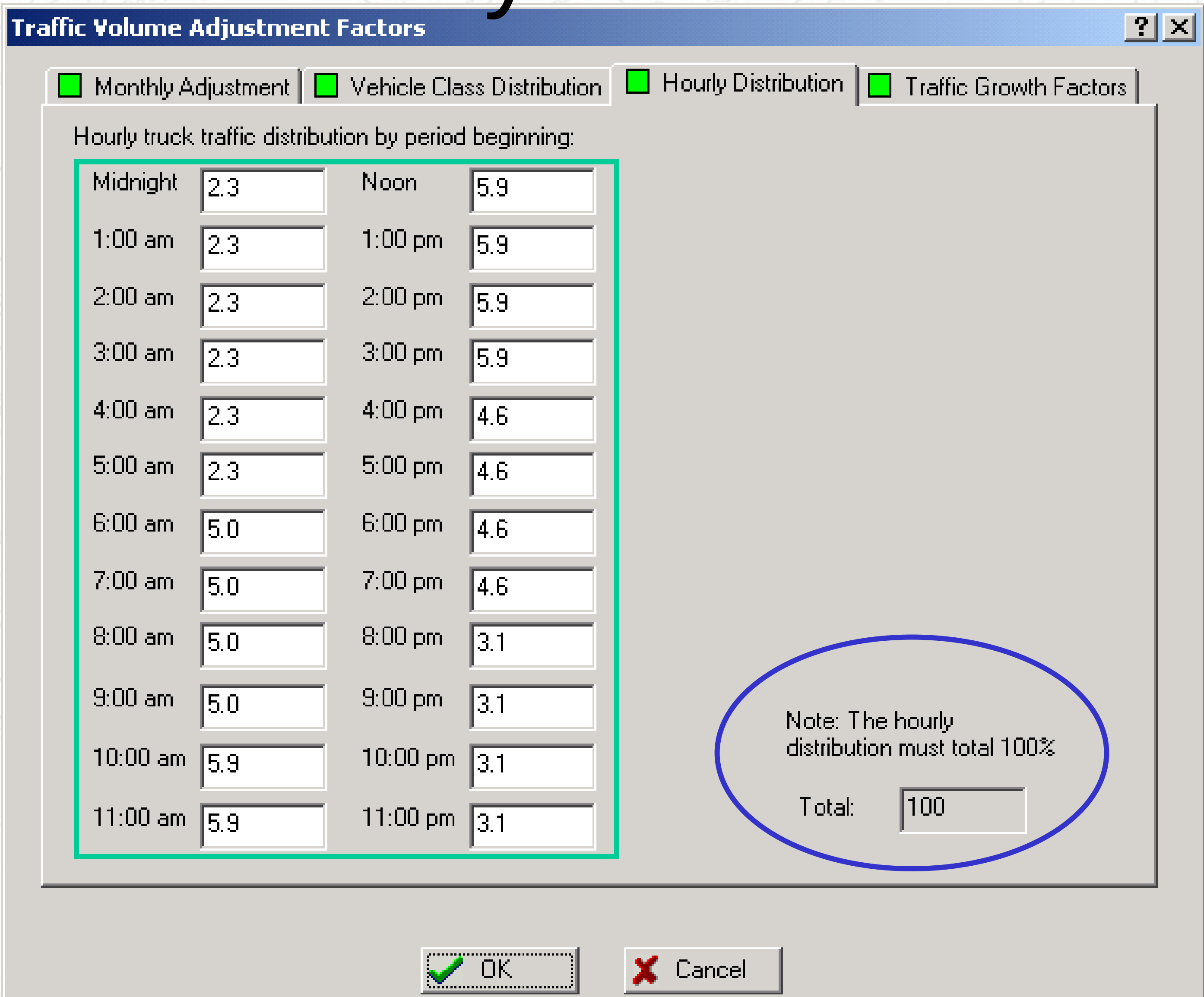
Vehicle Class Distribution



Current Traffic Data Requirements —FHWA Vehicle Classification



Hourly Distribution



Traffic Volume Adjustment Factors

Monthly Adjustment | Vehicle Class Distribution | Hourly Distribution | Traffic Growth Factors

Hourly truck traffic distribution by period beginning:

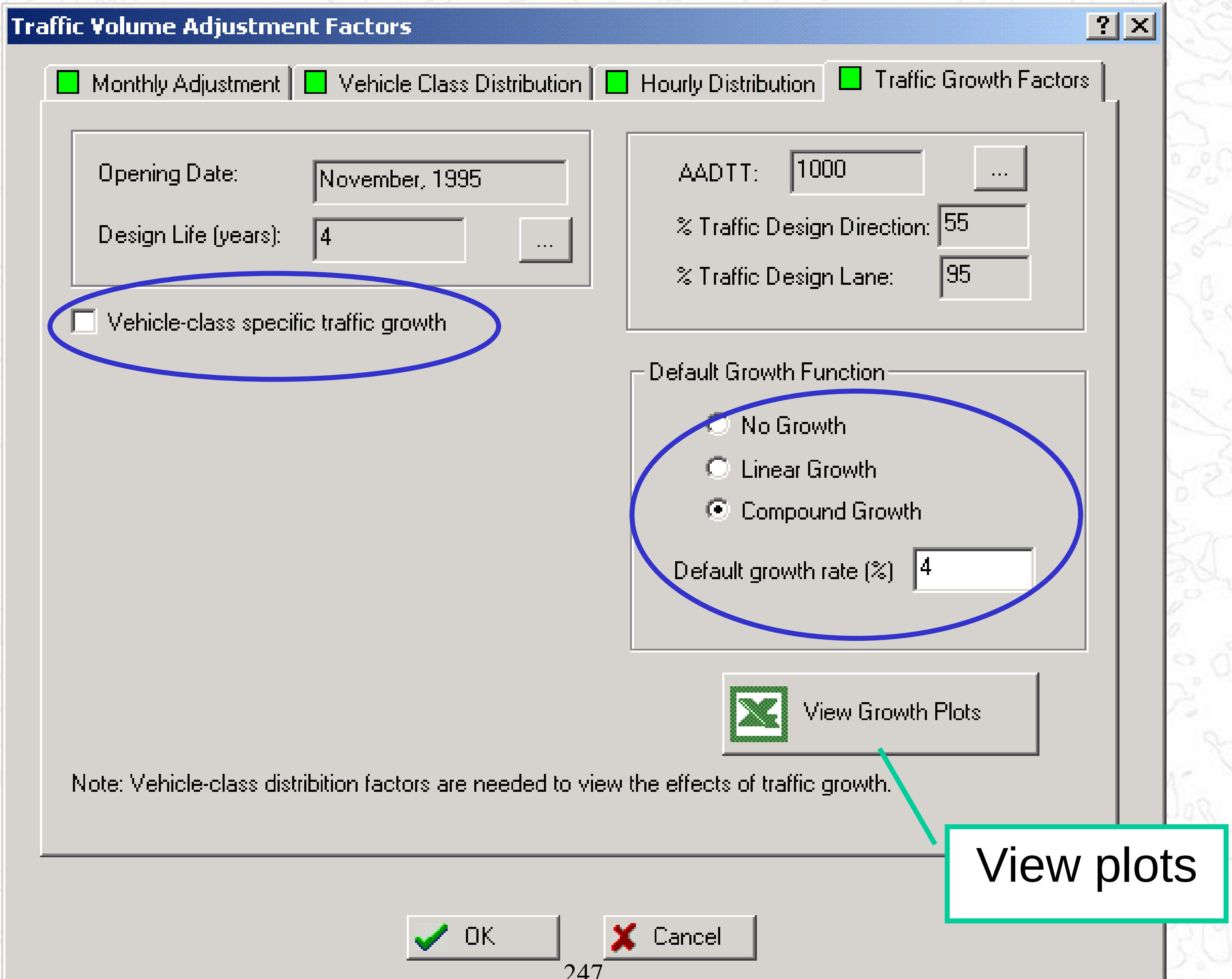
Midnight	2.3	Noon	5.9
1:00 am	2.3	1:00 pm	5.9
2:00 am	2.3	2:00 pm	5.9
3:00 am	2.3	3:00 pm	5.9
4:00 am	2.3	4:00 pm	4.6
5:00 am	2.3	5:00 pm	4.6
6:00 am	5.0	6:00 pm	4.6
7:00 am	5.0	7:00 pm	4.6
8:00 am	5.0	8:00 pm	3.1
9:00 am	5.0	9:00 pm	3.1
10:00 am	5.9	10:00 pm	3.1
11:00 am	5.9	11:00 pm	3.1

Note: The hourly distribution must total 100%

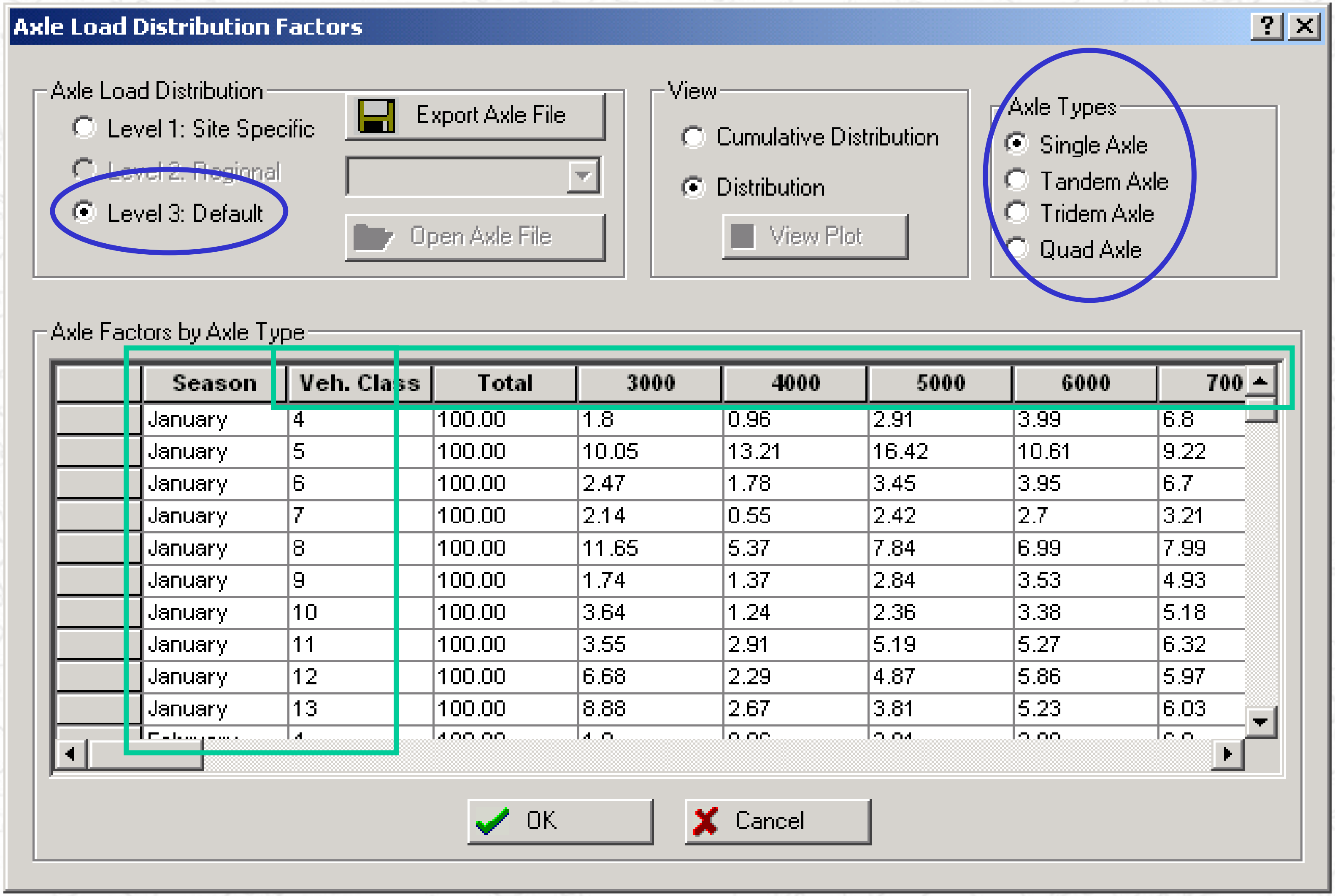
Total: 100

OK Cancel

Traffic Growth Factors

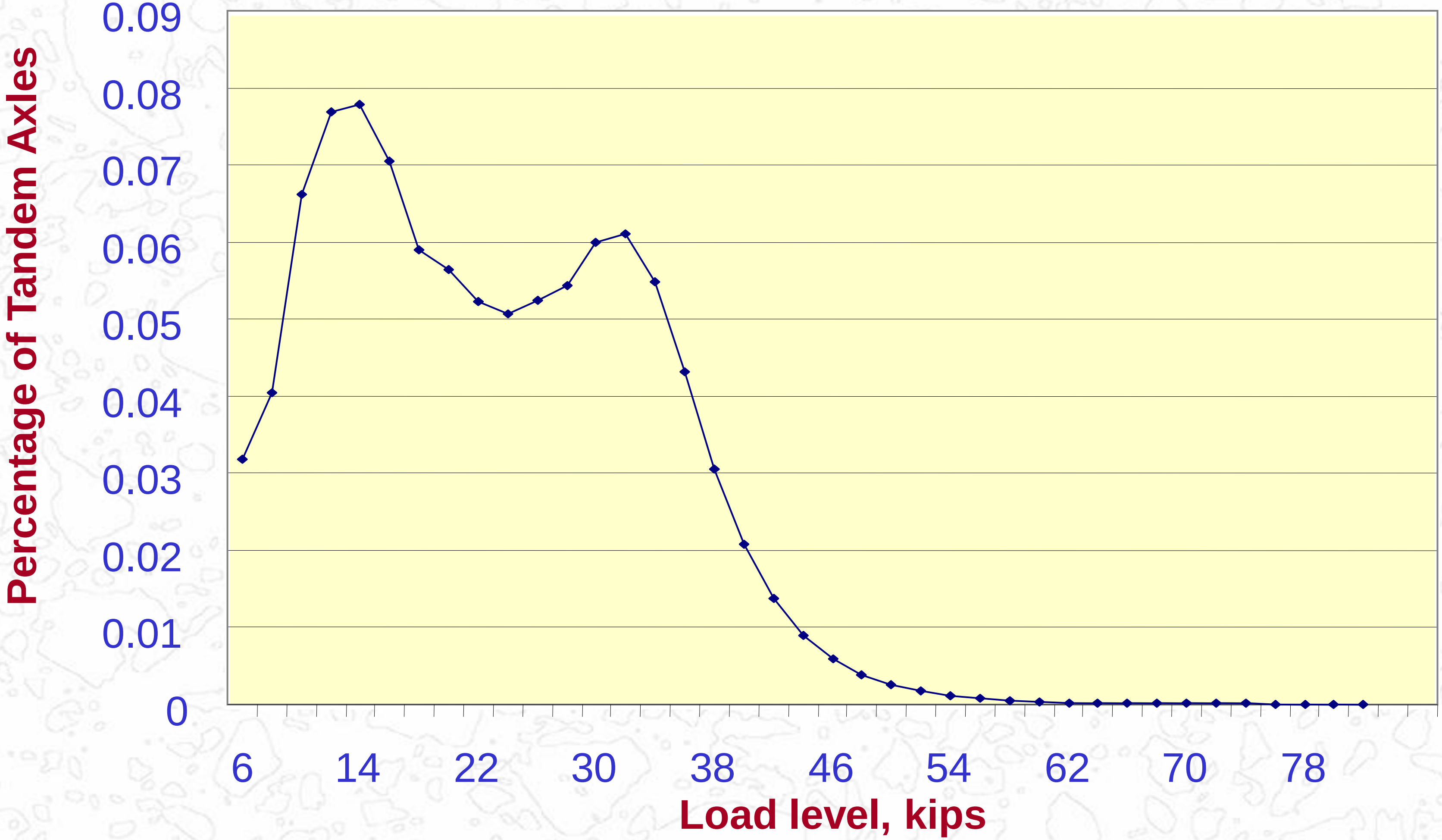


Axle Load Distribution Factors



Season	Veh. Class	Total	3000	4000	5000	6000	700
January	4	100.00	1.8	0.96	2.91	3.99	6.8
January	5	100.00	10.05	13.21	16.42	10.61	9.22
January	6	100.00	2.47	1.78	3.45	3.95	6.7
January	7	100.00	2.14	0.55	2.42	2.7	3.21
January	8	100.00	11.65	5.37	7.84	6.99	7.99
January	9	100.00	1.74	1.37	2.84	3.53	4.93
January	10	100.00	3.64	1.24	2.36	3.38	5.18
January	11	100.00	3.55	2.91	5.19	5.27	6.32
January	12	100.00	6.68	2.29	4.87	5.86	5.97
January	13	100.00	8.88	2.67	3.81	5.23	6.03

Example – Tandem Axle Distribution for the First Month of Traffic



General Traffic Inputs – Traffic Wander and Number of Axles/Truck

General Traffic Inputs

Lateral Traffic Wander

Mean wheel location (inches from the lane marking): 20

Traffic wander standard deviation (in): 10

Design lane width (ft): (Note: This is not slab width) 12

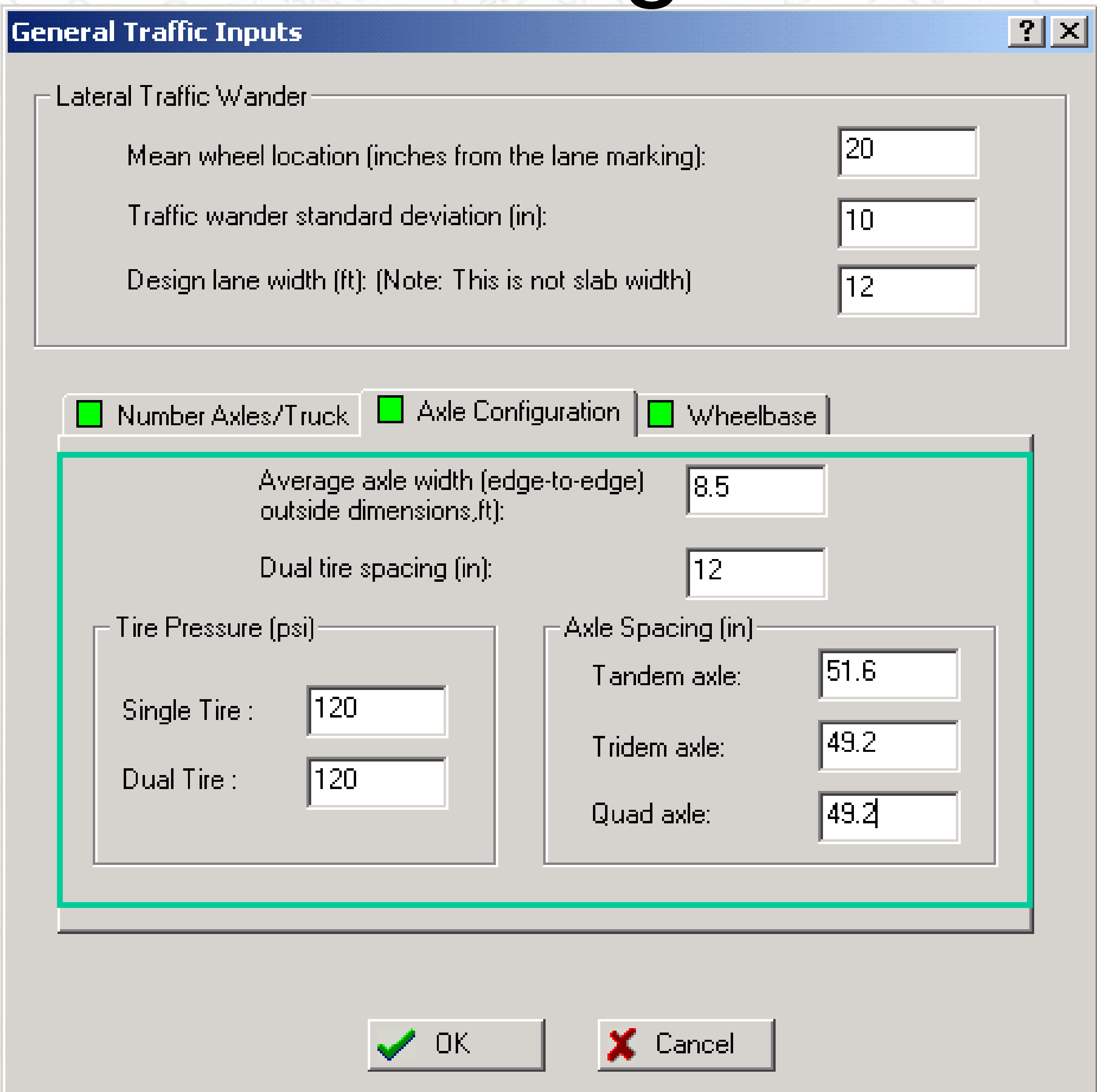
Number Axles/Truck | Axle Configuration | Wheelbase

	Single	Tandem	Tridem	Quad
Class 4	1.62	0.39	0	0
Class 5	2	0	0	0
Class 6	1.02	0.99	0	0
Class 7	1	0.26	0.83	0
Class 8	2.38	0.67	0	0
Class 9	1.13	1.93	0	0
Class 10	1.19	1.09	0.89	0
Class 11	4.29	0.26	0.06	0
Class 12	3.52	1.14	0.06	0
Class 13	2.15	2.13	0.35	0

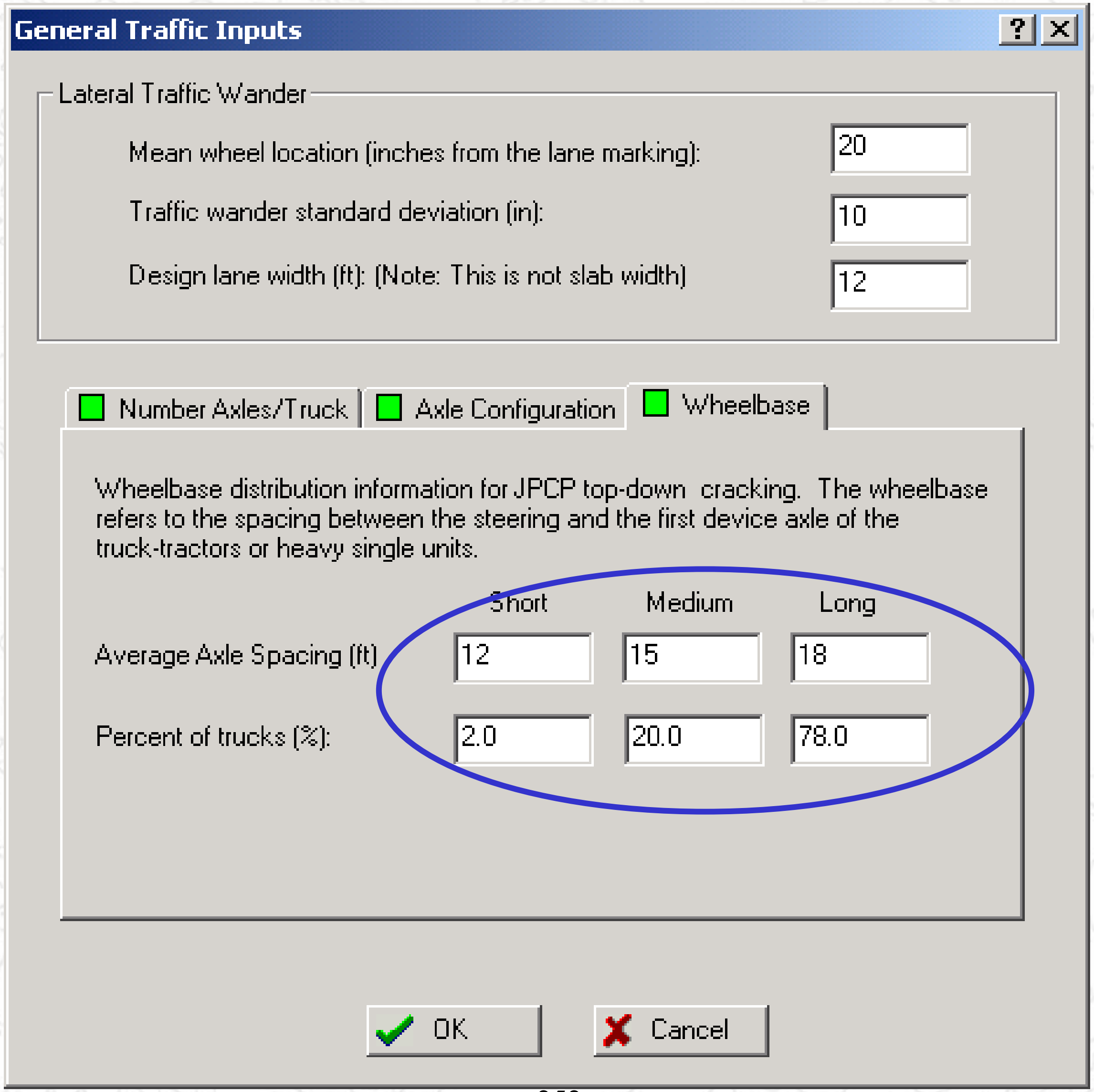
Default values

OK Cancel

Axle Configuration



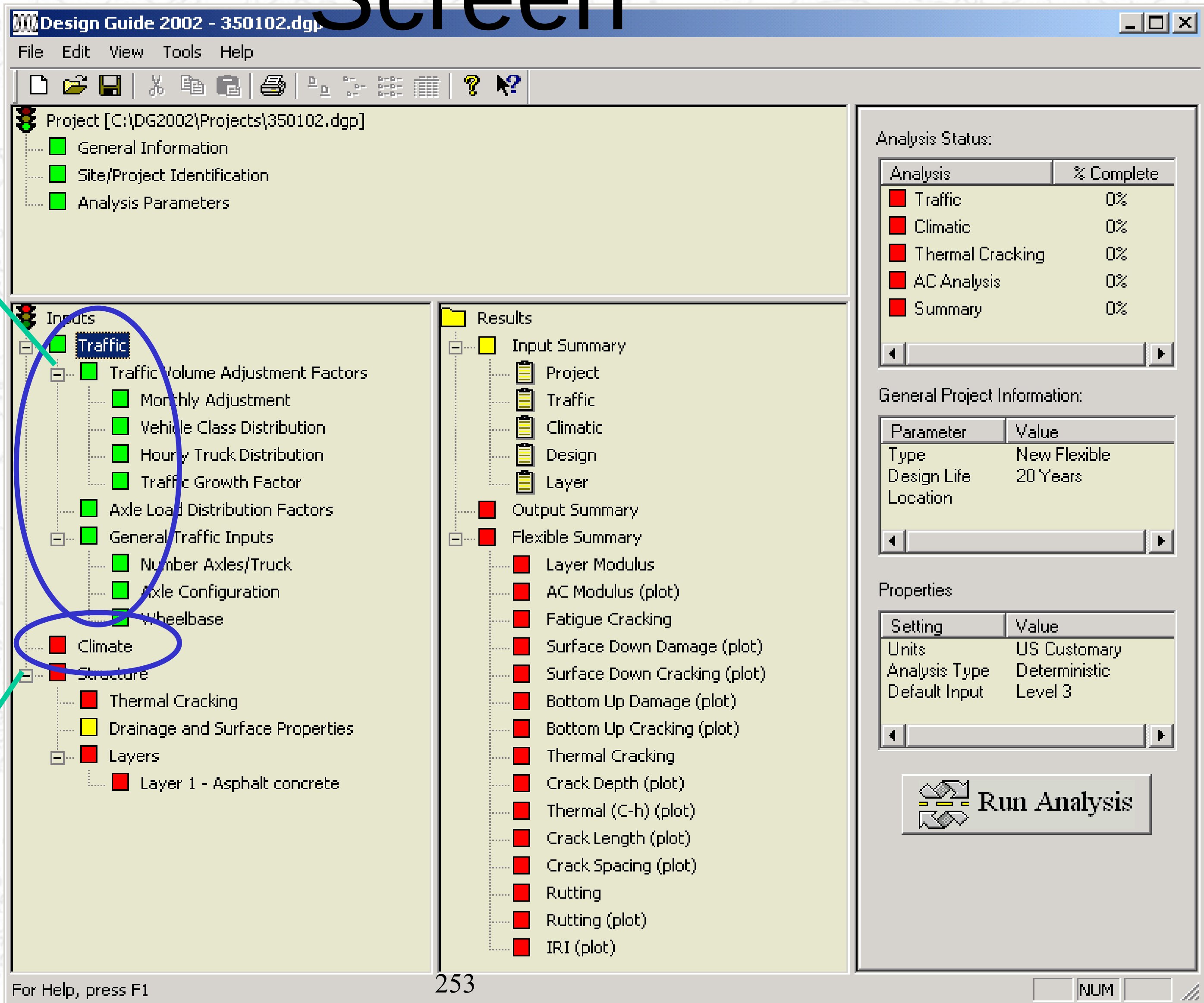
Wheelbase


General Traffic Inputs
Lateral Traffic Wander
Mean wheel location (inches from the lane marking):
20
Traffic wander standard deviation (in):
10
Design lane width (ft): (Note: This is not slab width)
12
Number Axles/Truck
Axle Configuration
Wheelbase
Wheelbase distribution information for JPCP top-down cracking. The wheelbase refers to the spacing between the steering and the first device axle of the truck-tractors or heavy single units.
Short
Medium
Long
Average Axle Spacing (ft)
12
15
18
Percent of trucks (%):
2.0
20.0
78.0
OK
Cancel

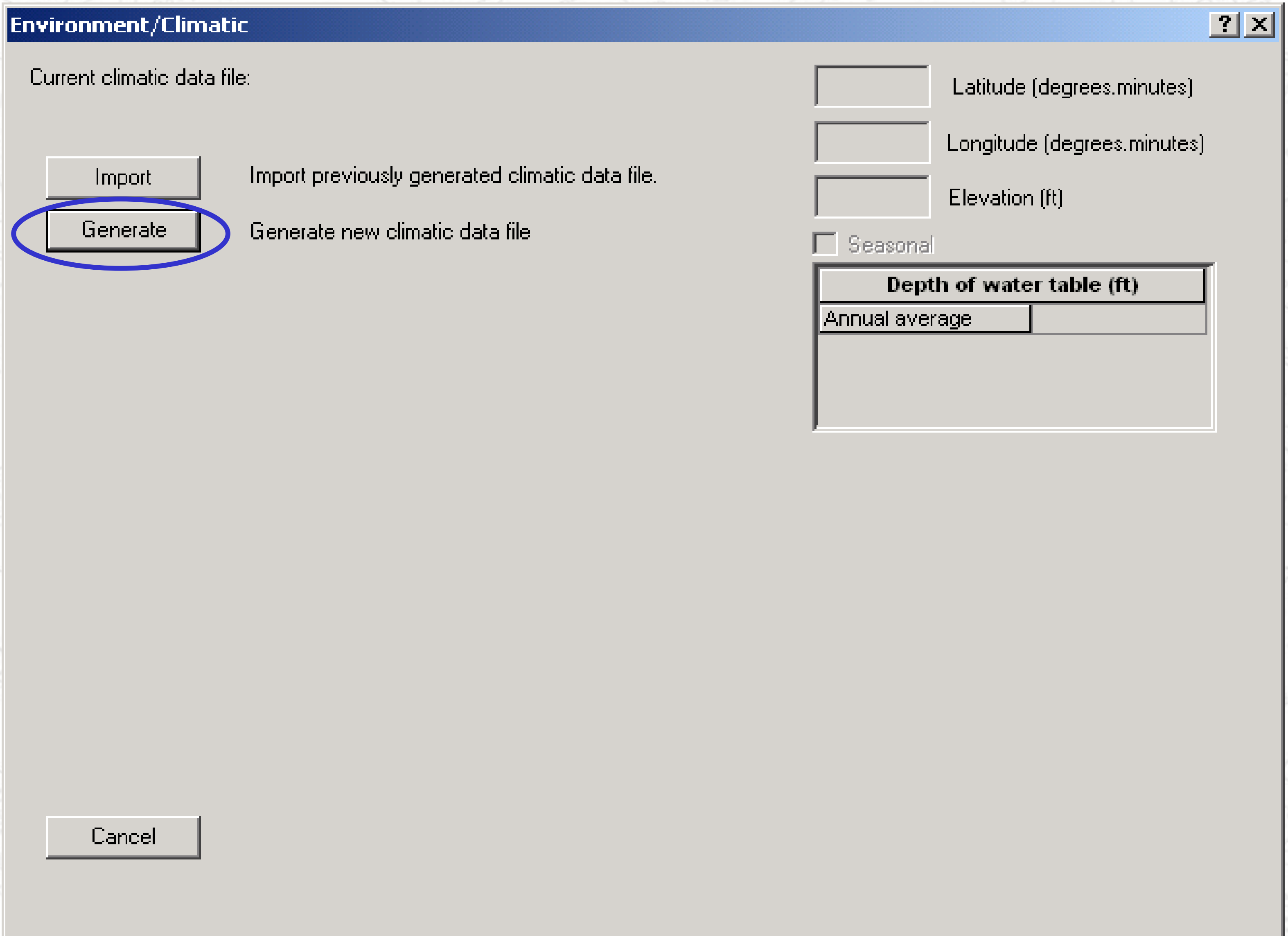
Check Status of Inputs on Layout Screen

Traffic Input Completed

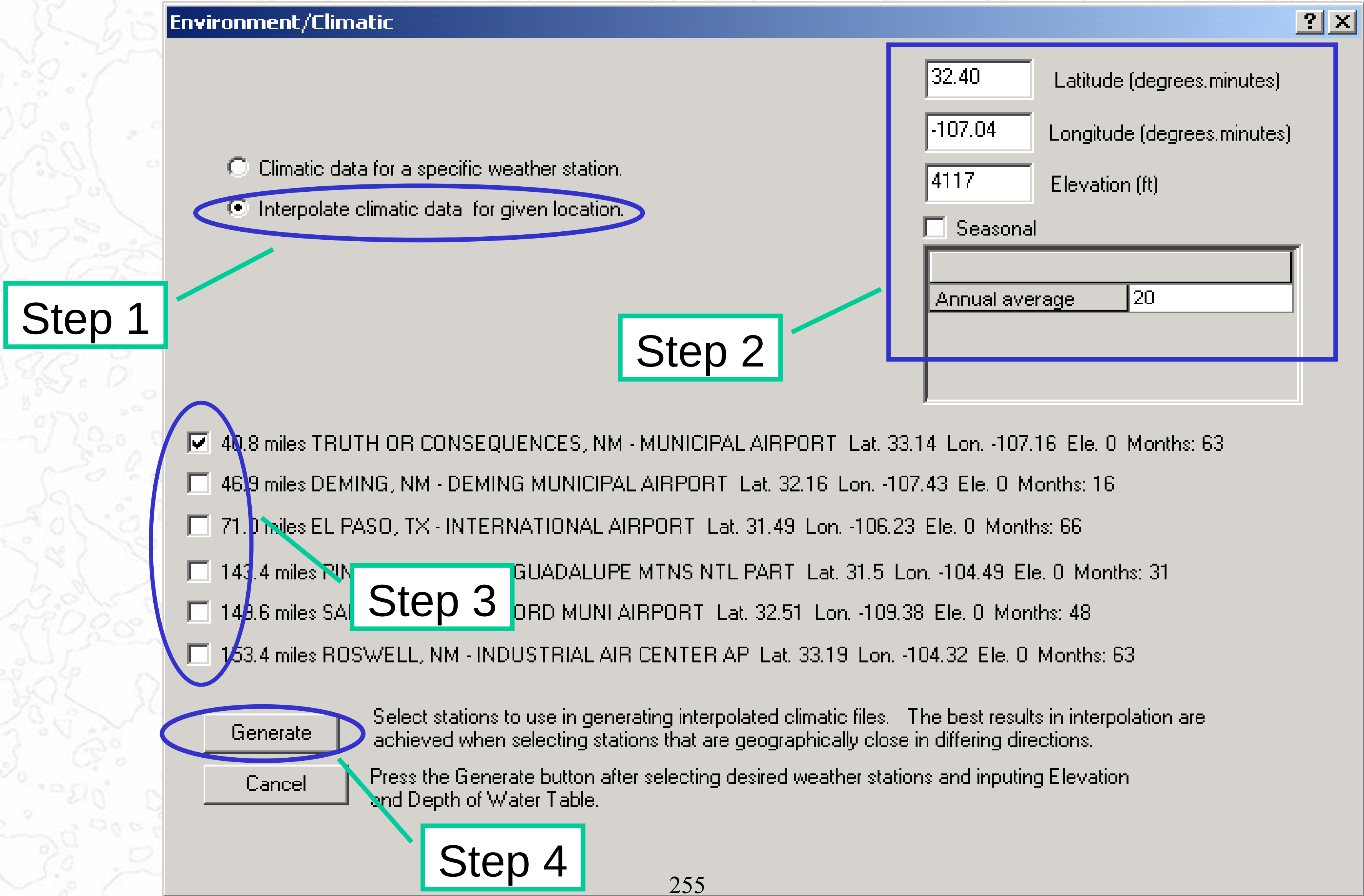
Start Climate



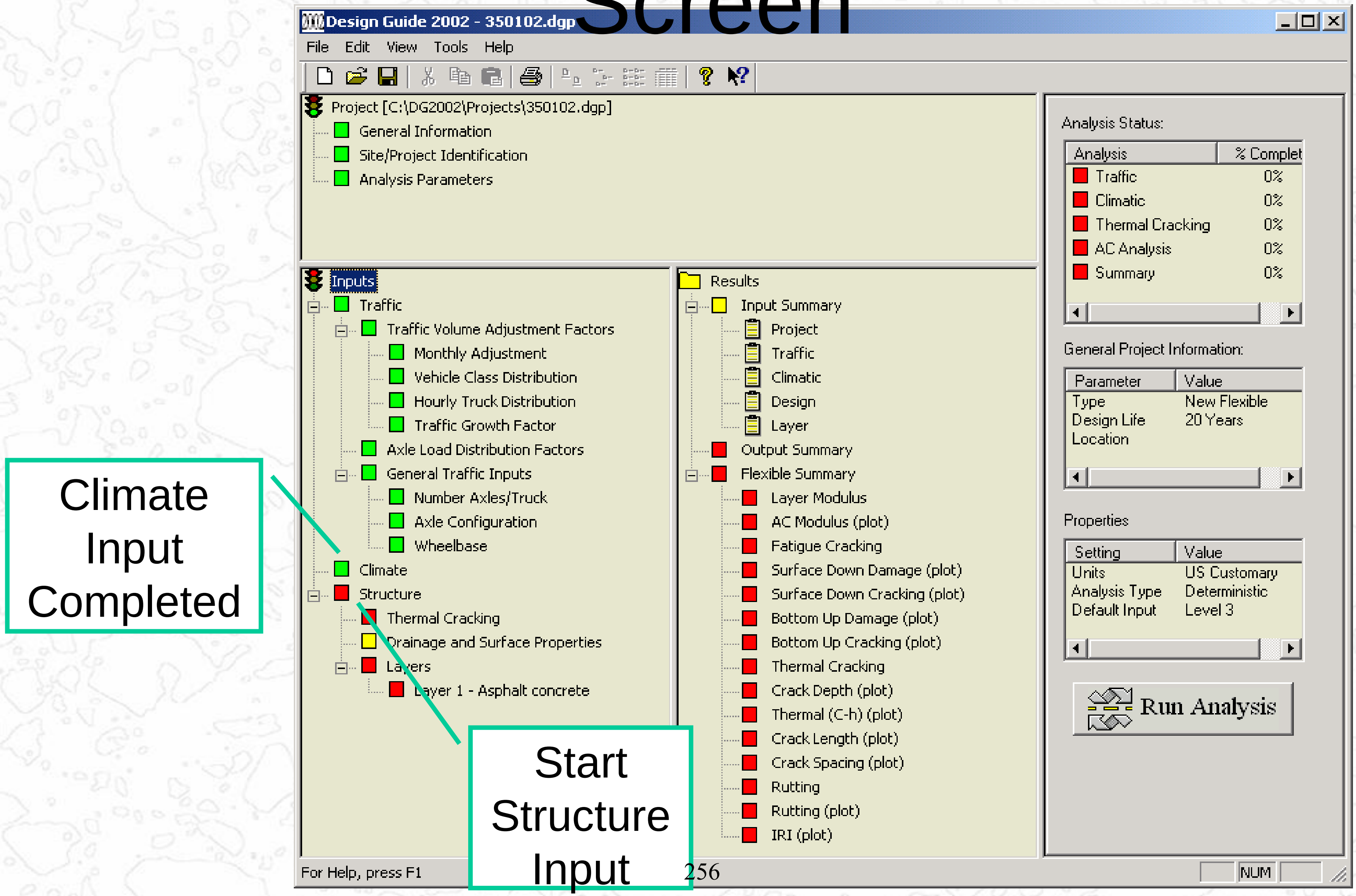
Generate Climatic File



Create "Virtual" Weather Station



Check Status of Inputs on Layout Screen



Structure Inputs

- ⇨ User needs to choose layers and the trial design
- ⇨ Example 1: Conventional AC design:

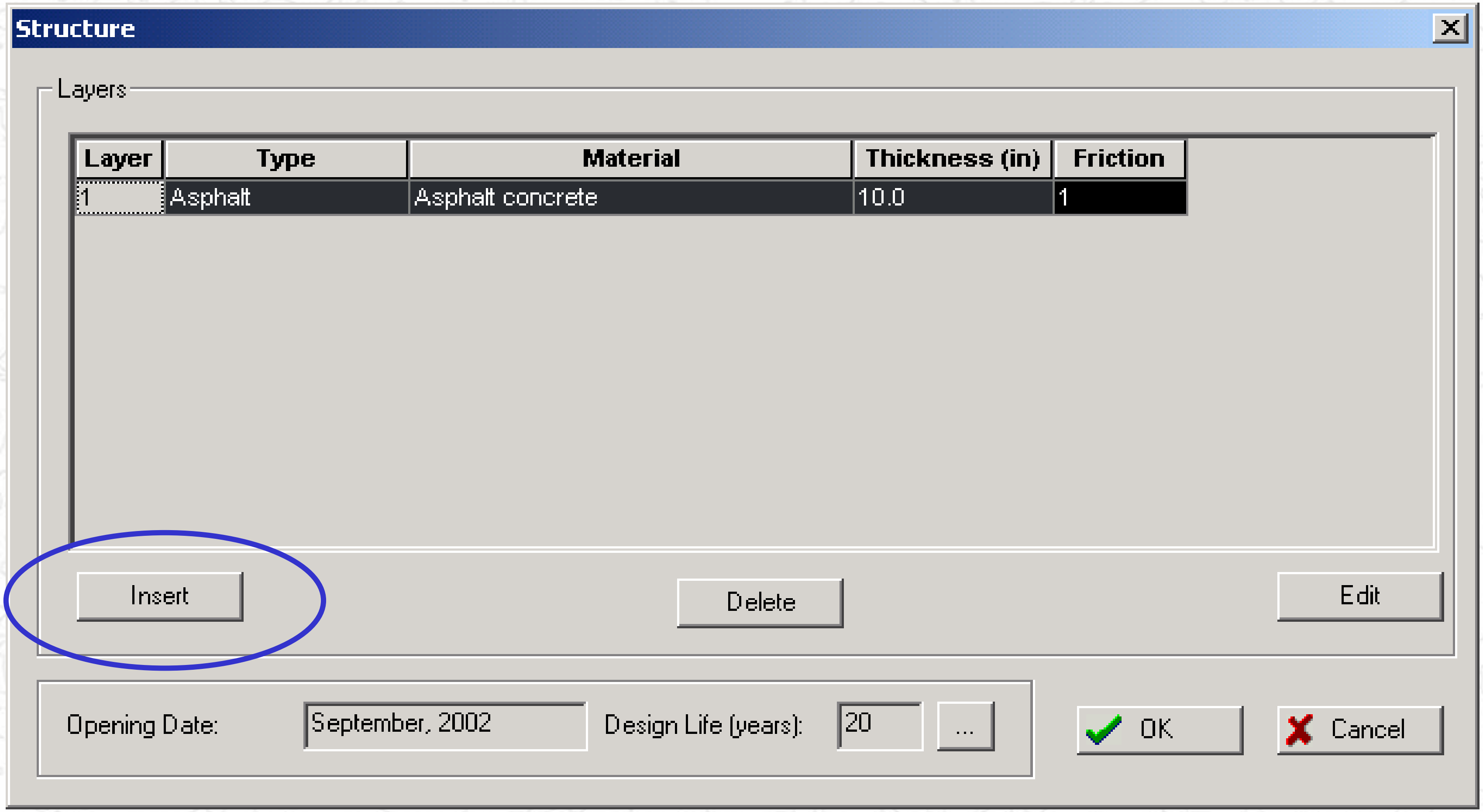
  4.8-inch Asphalt Concrete layer

  12.2-inch Granular Base layer (A-1-a)

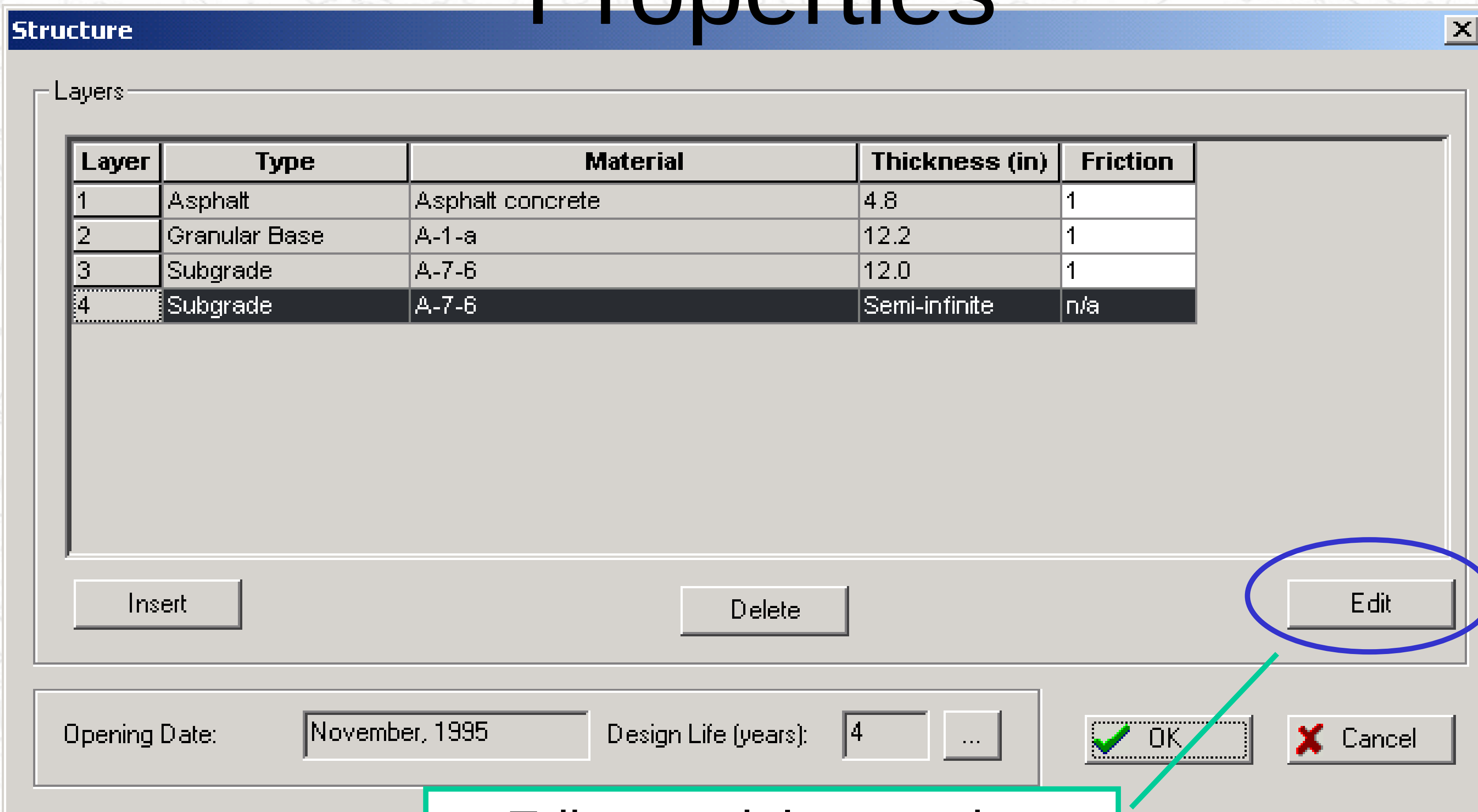
  12-inch Compacted Subgrade (A-7-6)

  Natural subgrade (A-7-6)

Insert Layers

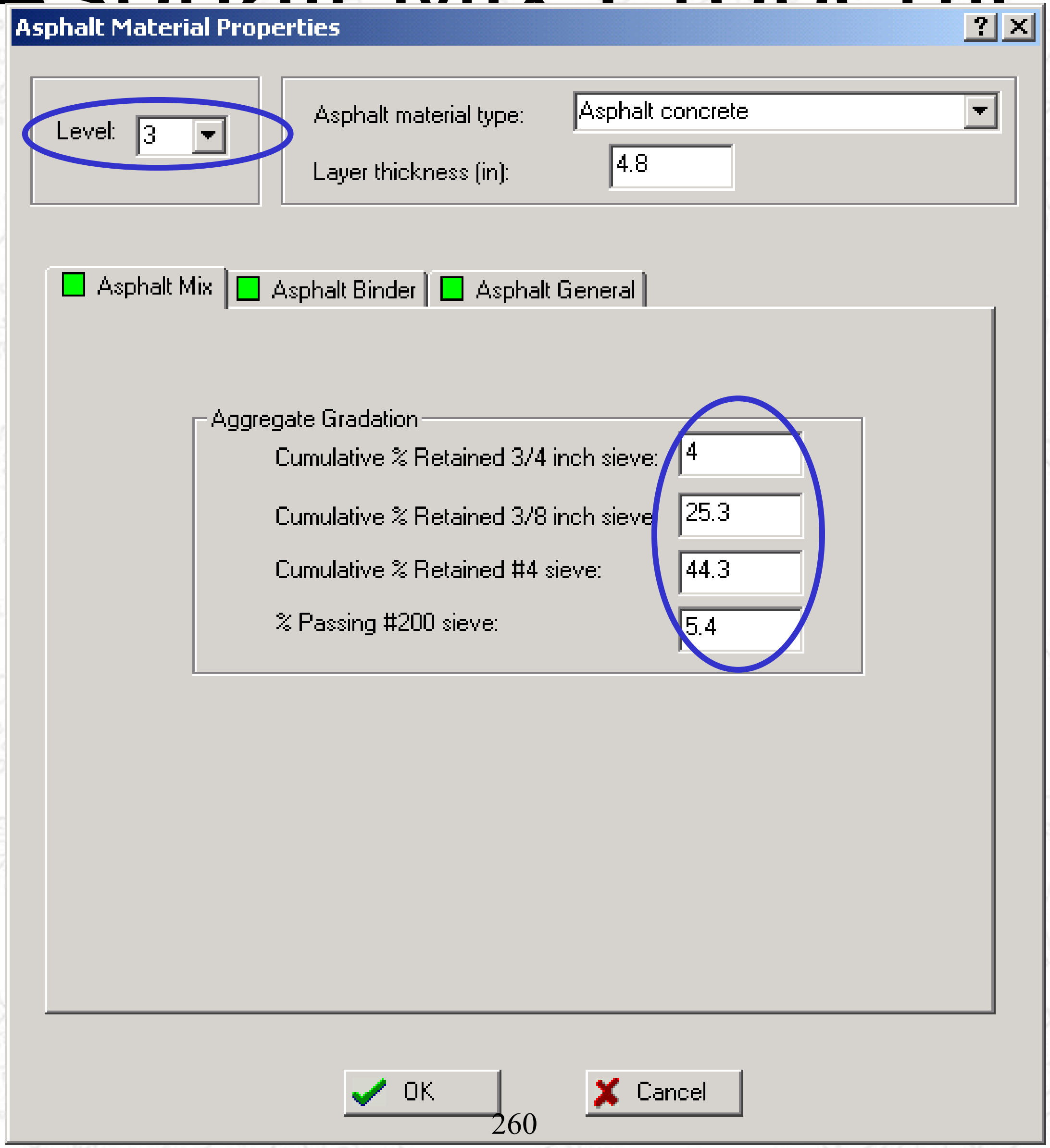


Add Layers and Edit Layer Properties

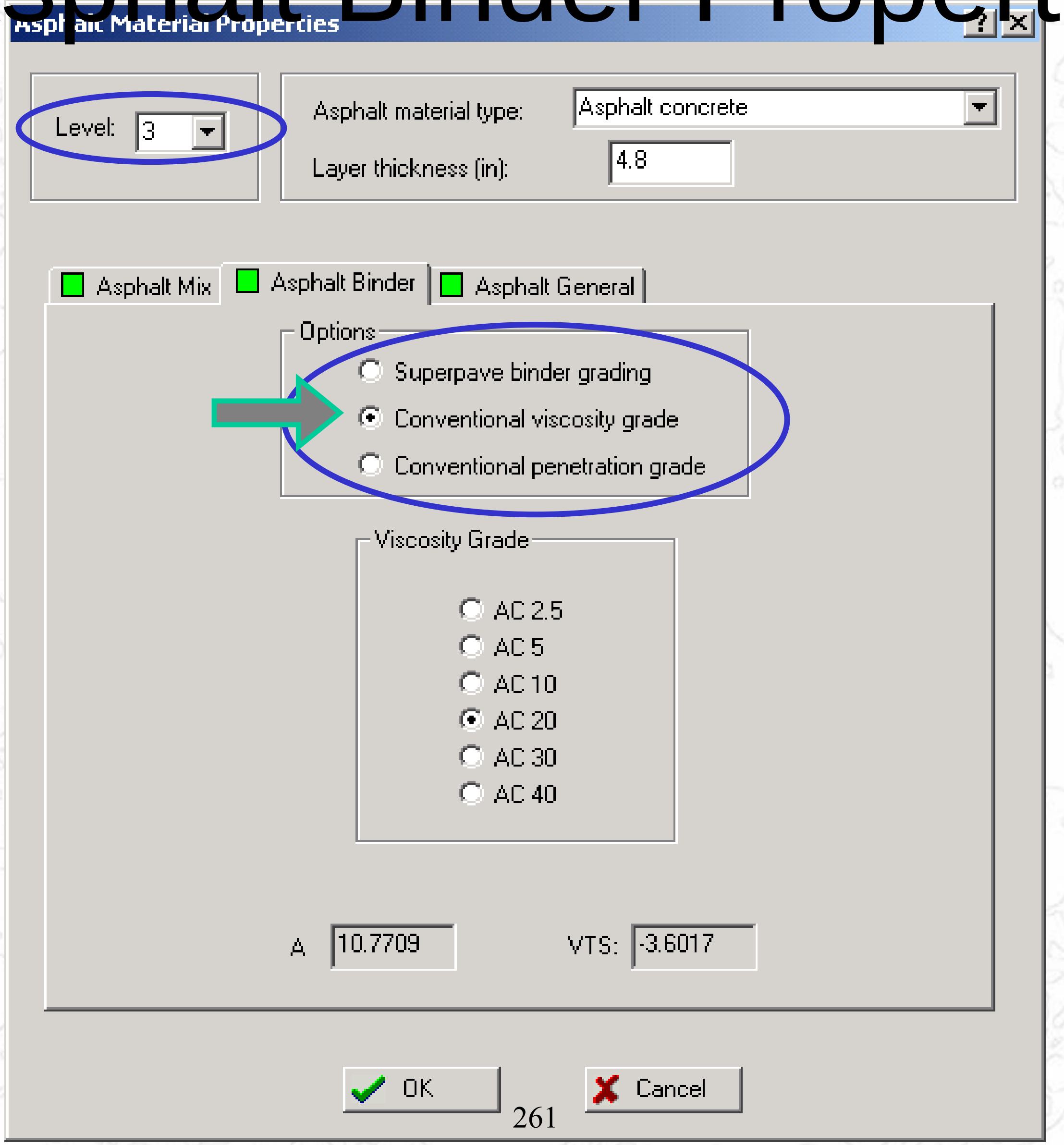


Edit material properties either from this screen or from the main screen

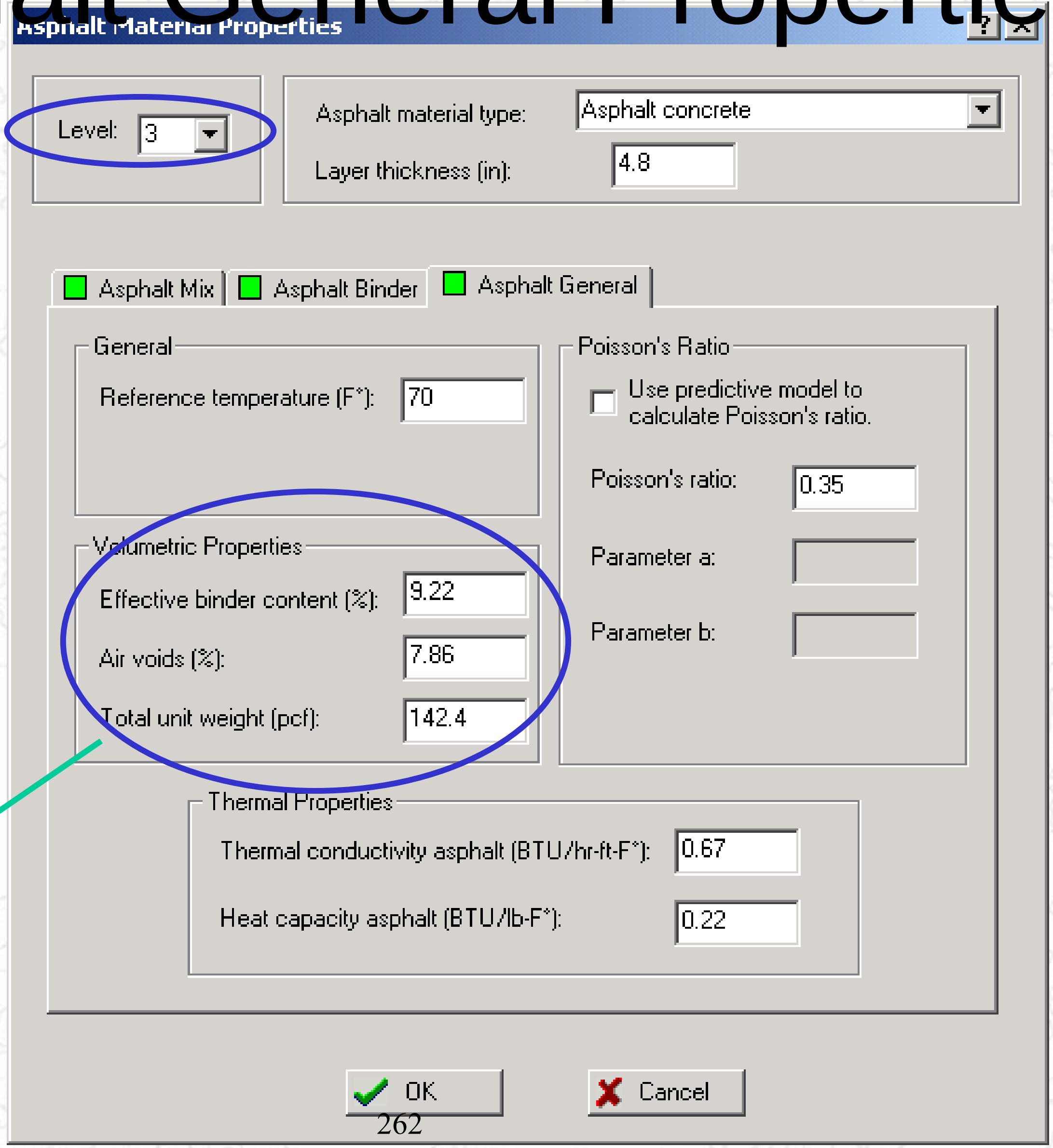
Asphalt Mix Properties



Asphalt Binder Properties

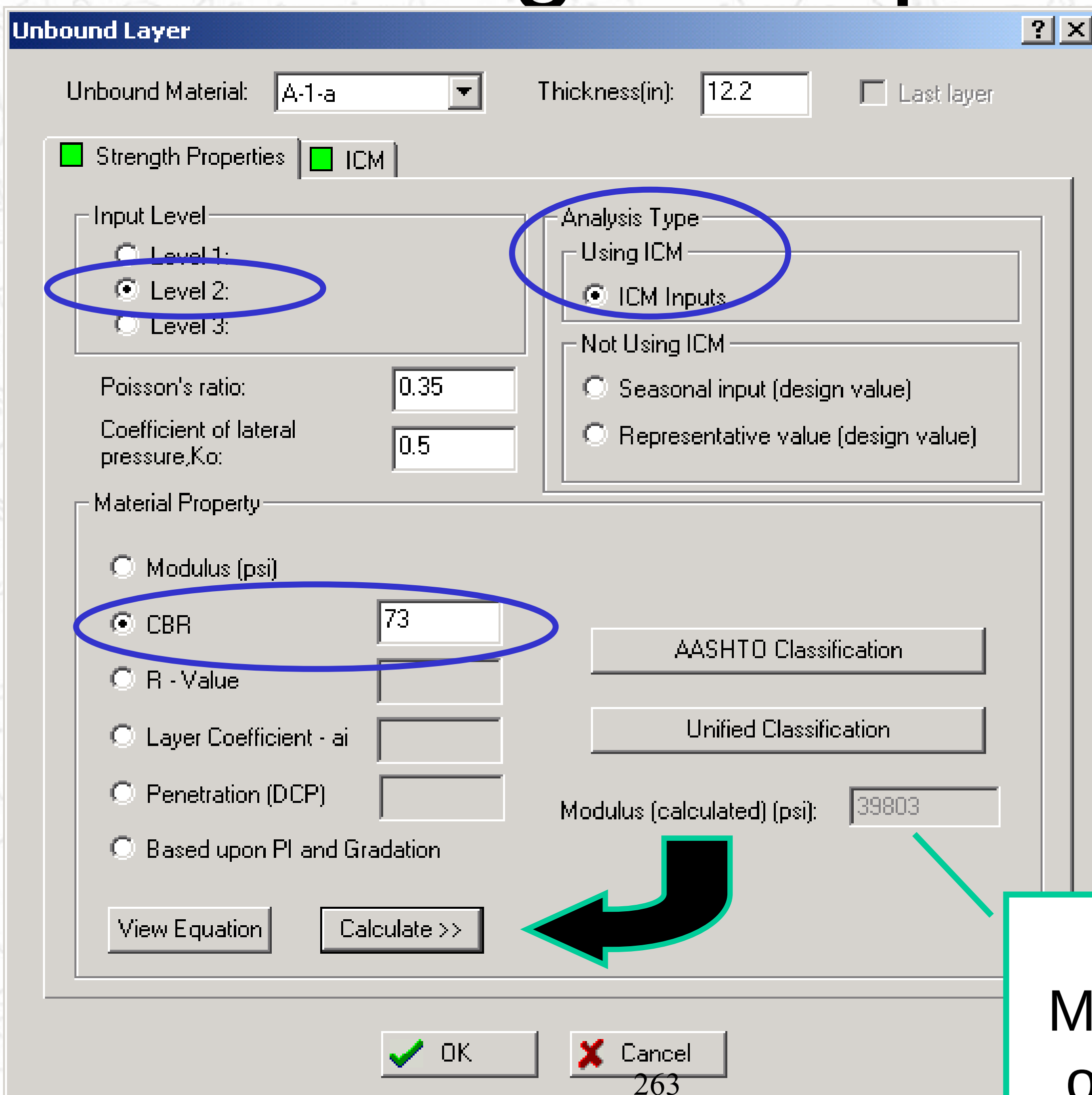


Asphalt General Properties



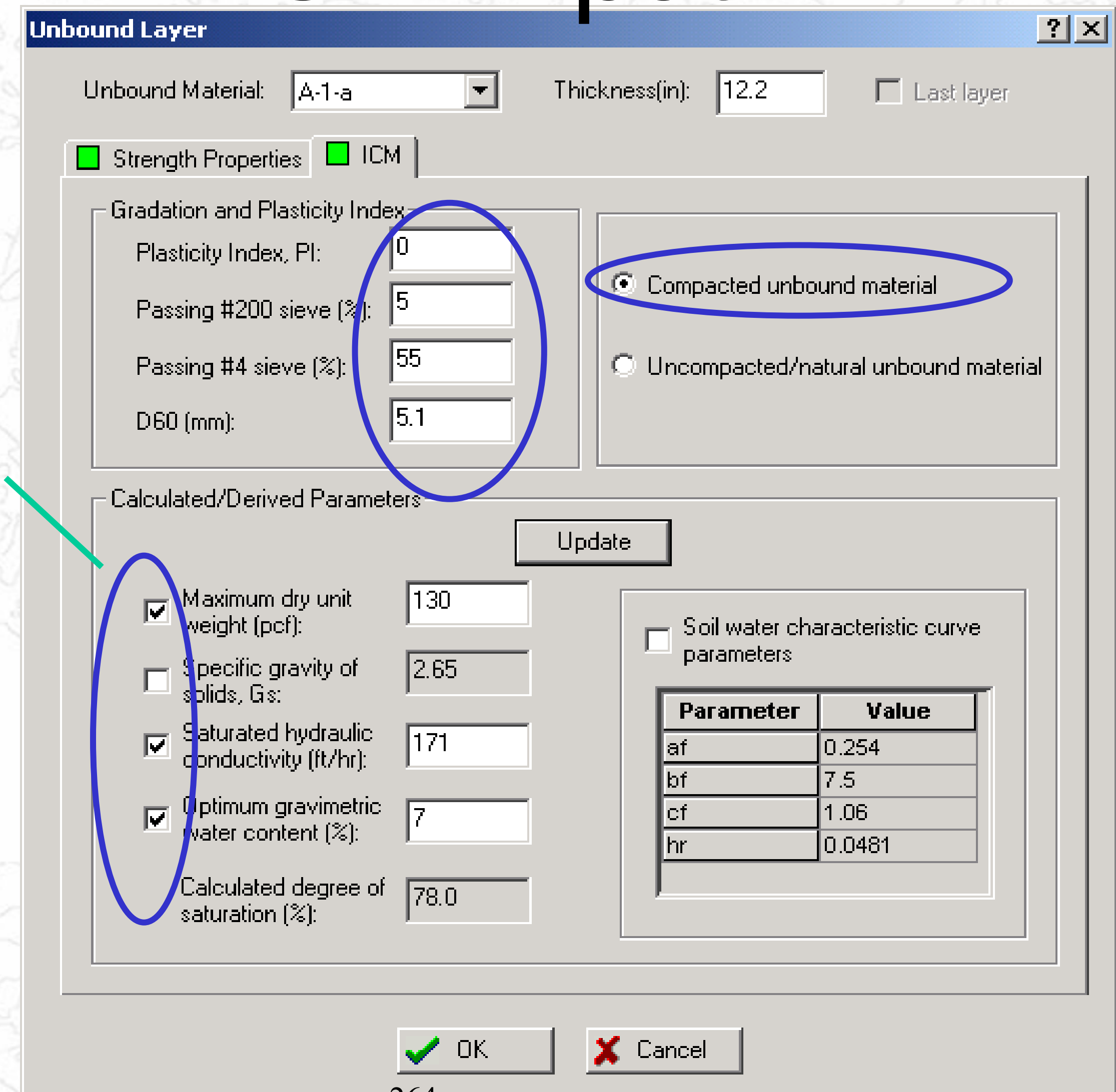
Input volumetric properties

Granular Base Layer – Strength Properties



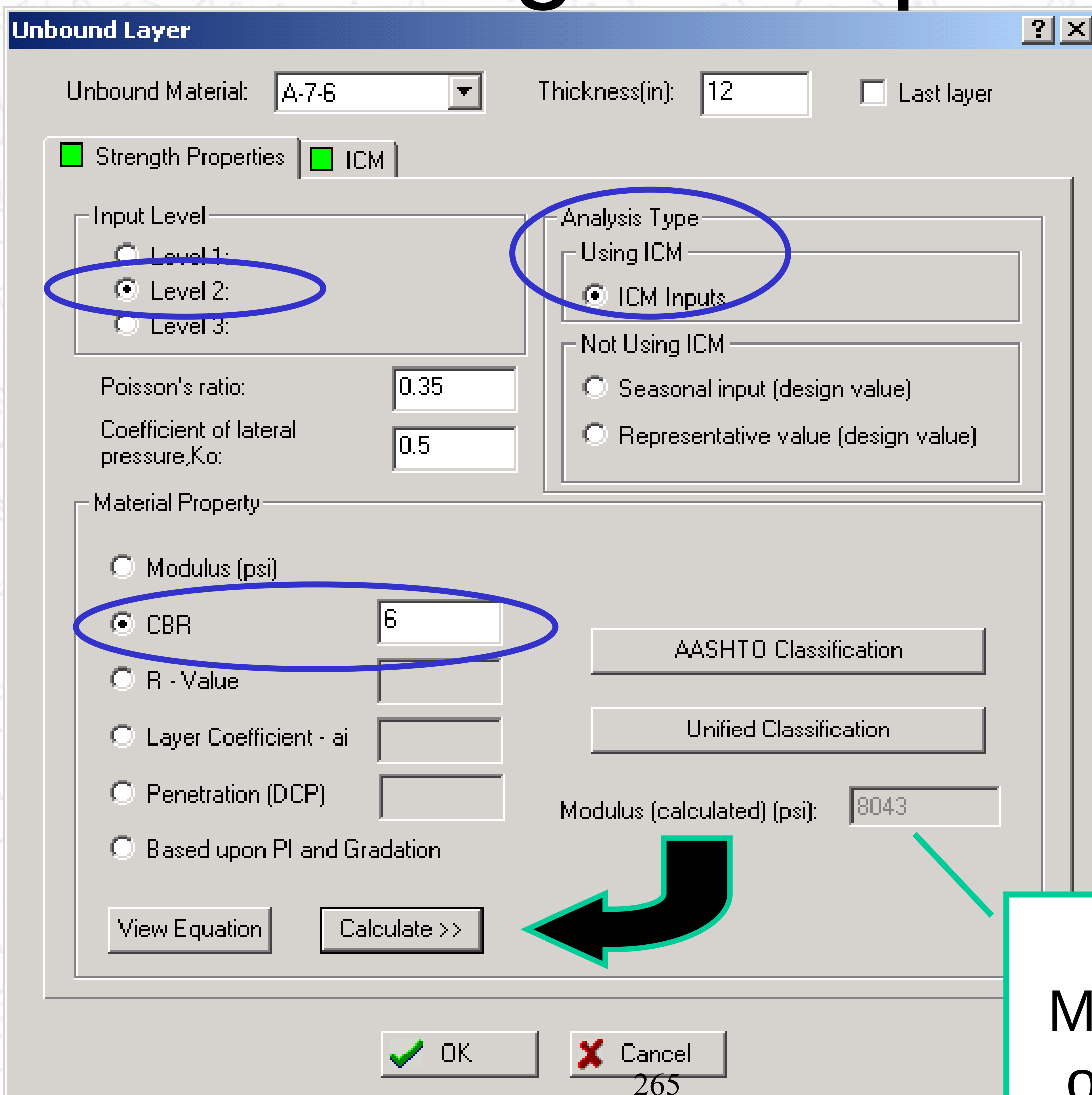
Calculated Modulus based on CBR value

Granular Base Layer - ICM Input



Level 2 analysis: Input measured properties

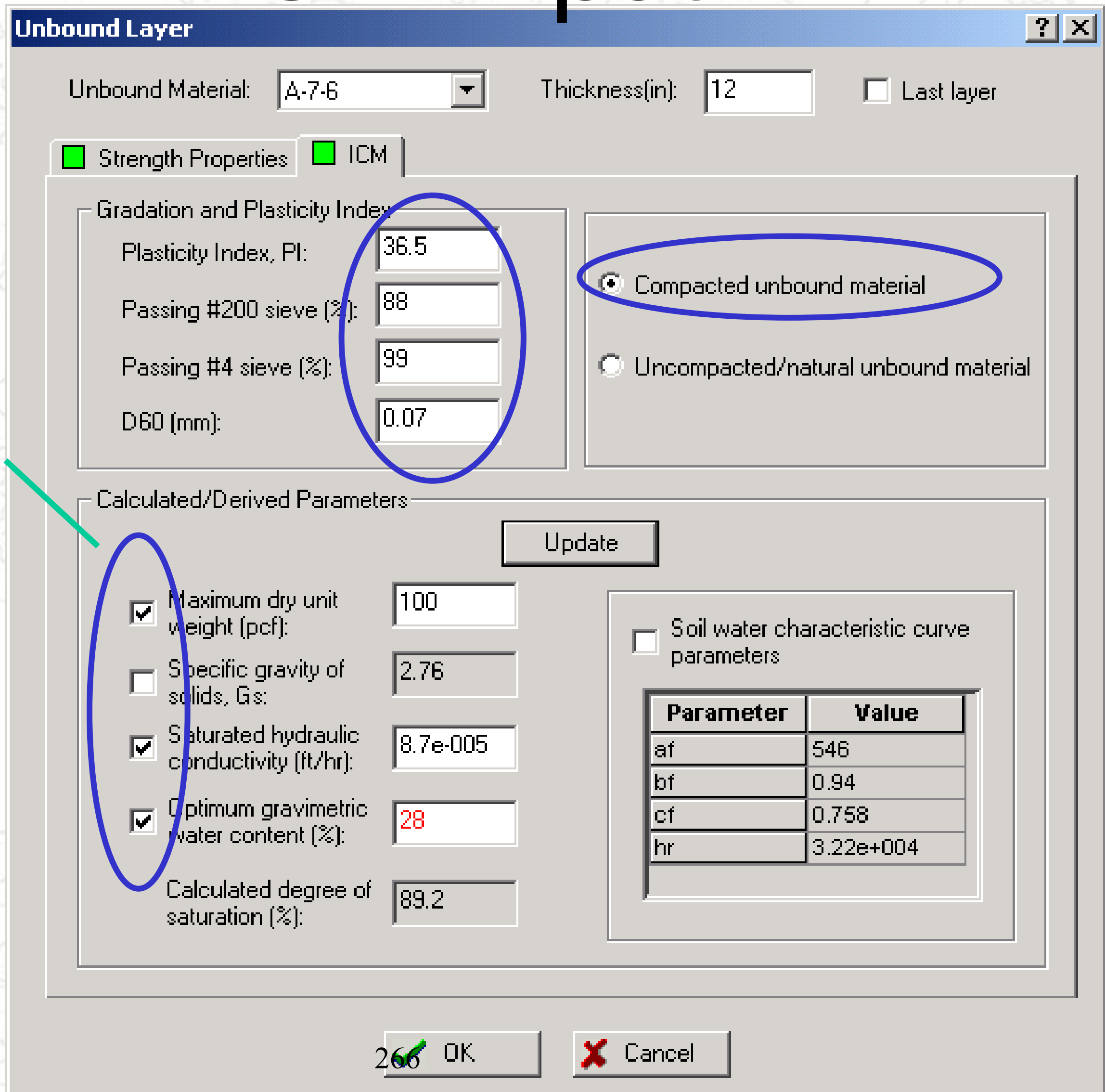
Compacted Subgrade Layer – Strength Properties



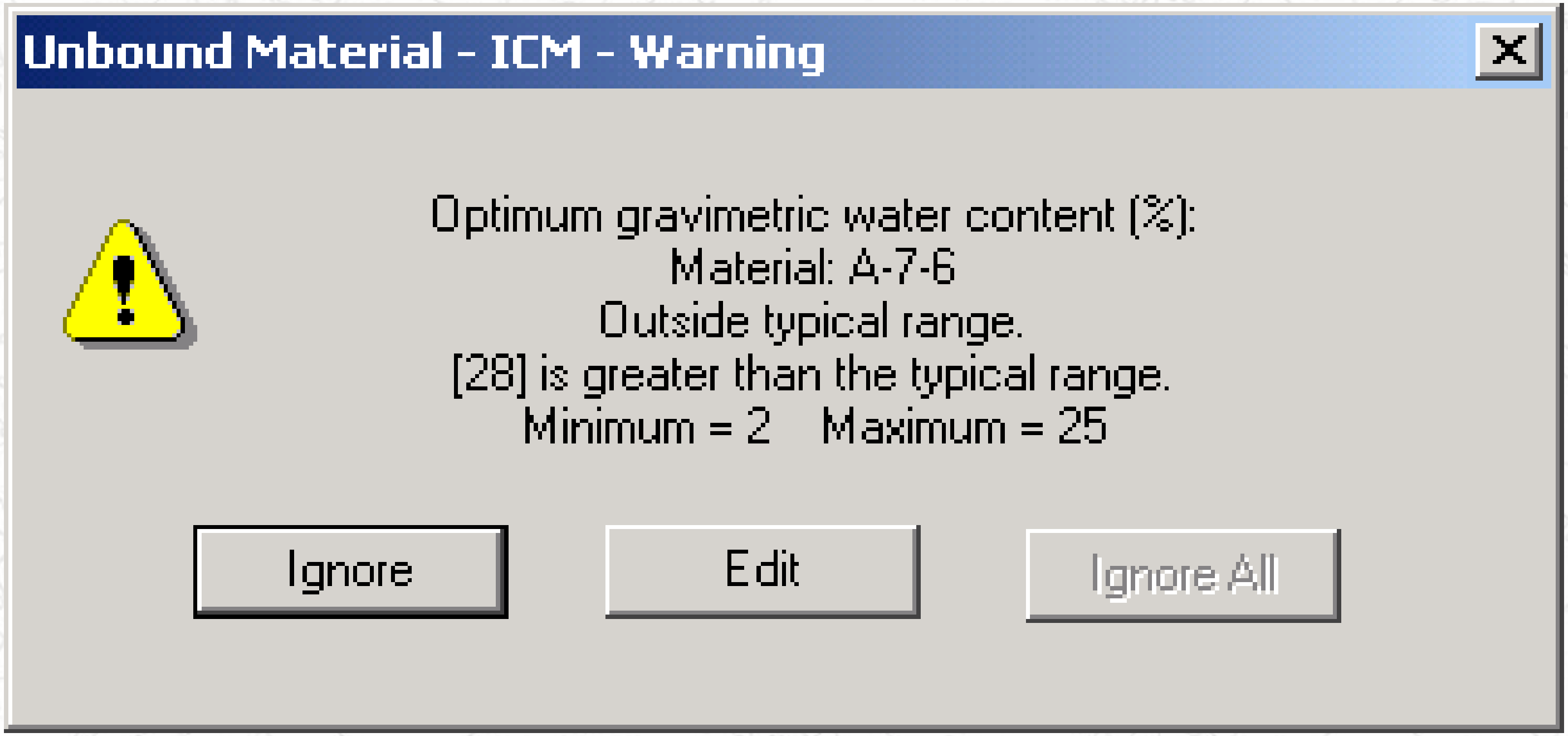
Calculated Modulus based on CBR value

Compacted Subgrade Layer - ICM Input

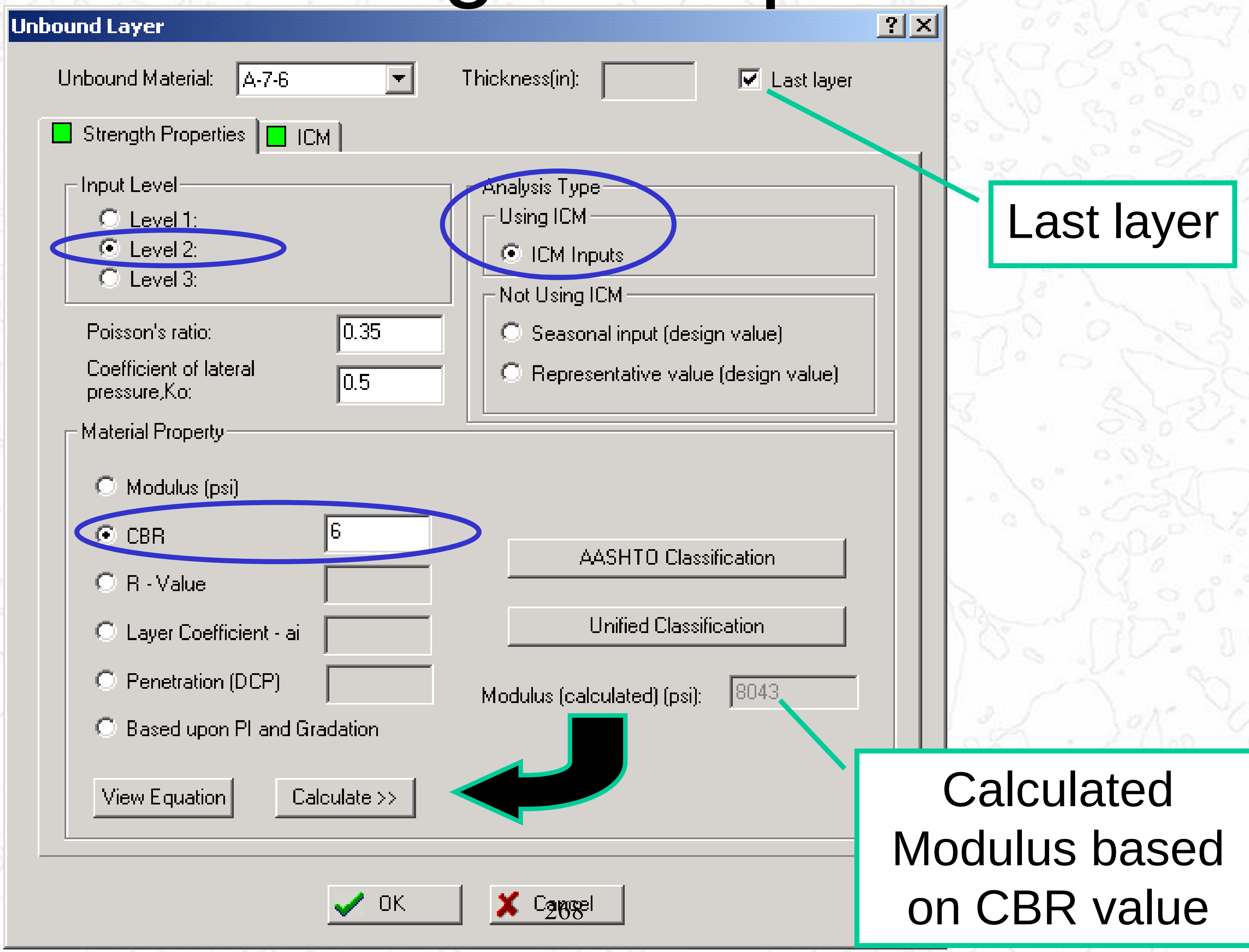
Level 2 analysis: Input measured properties



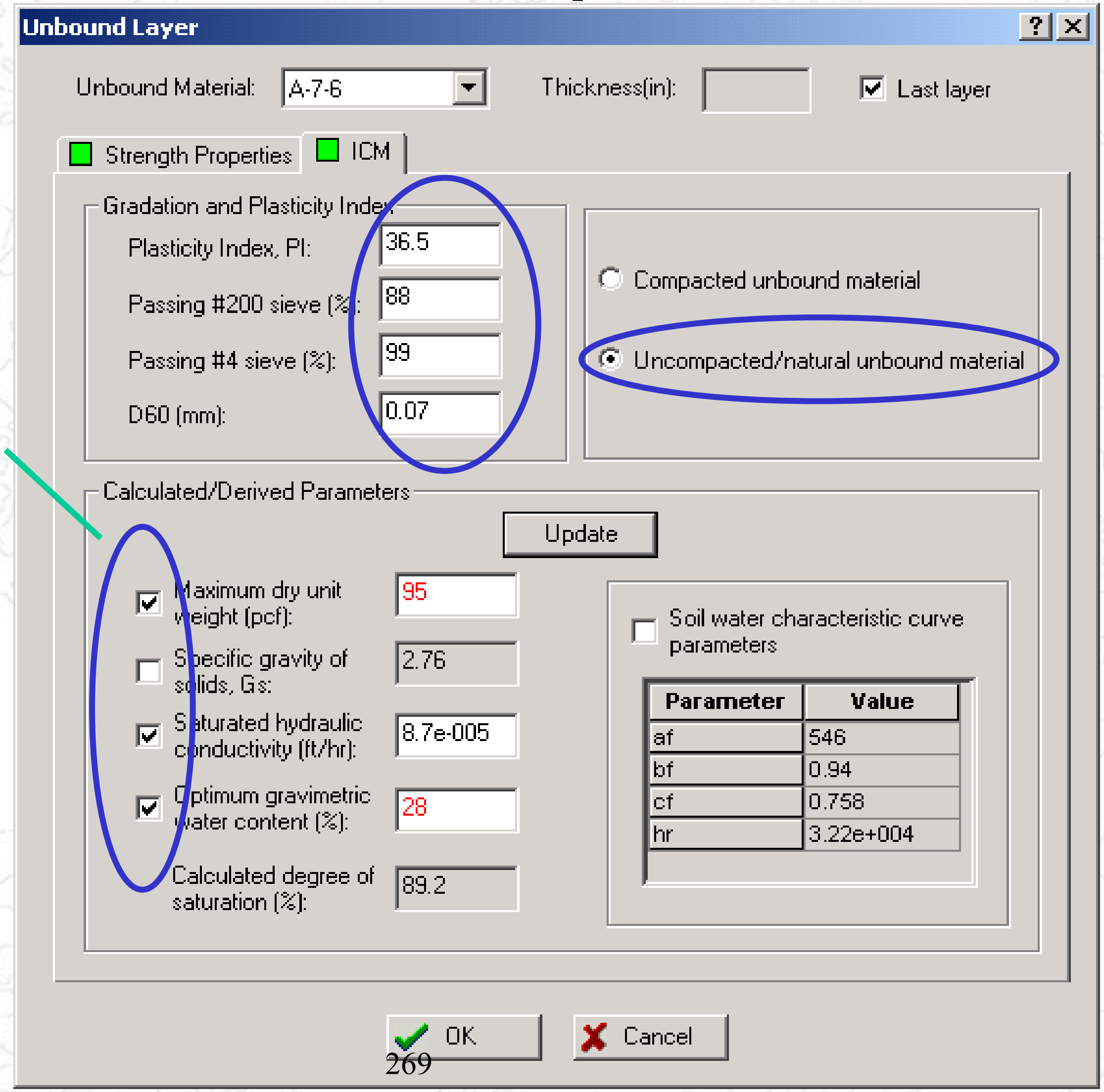
ICM Warning Capability



Natural Subgrade Layer – Strength Properties

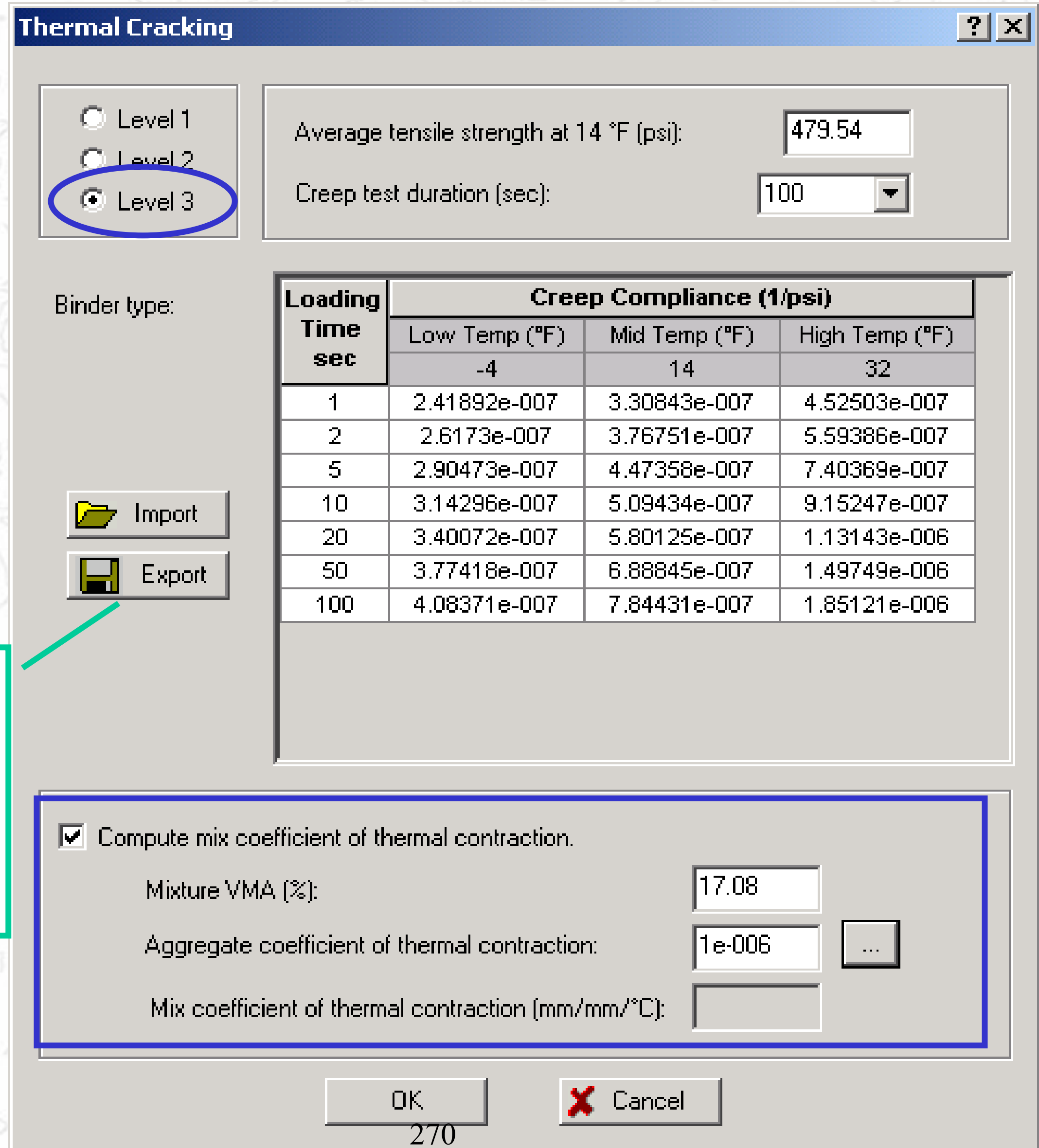


Natural Subgrade Layer - ICM Input



Level 2 analysis: Input measured properties

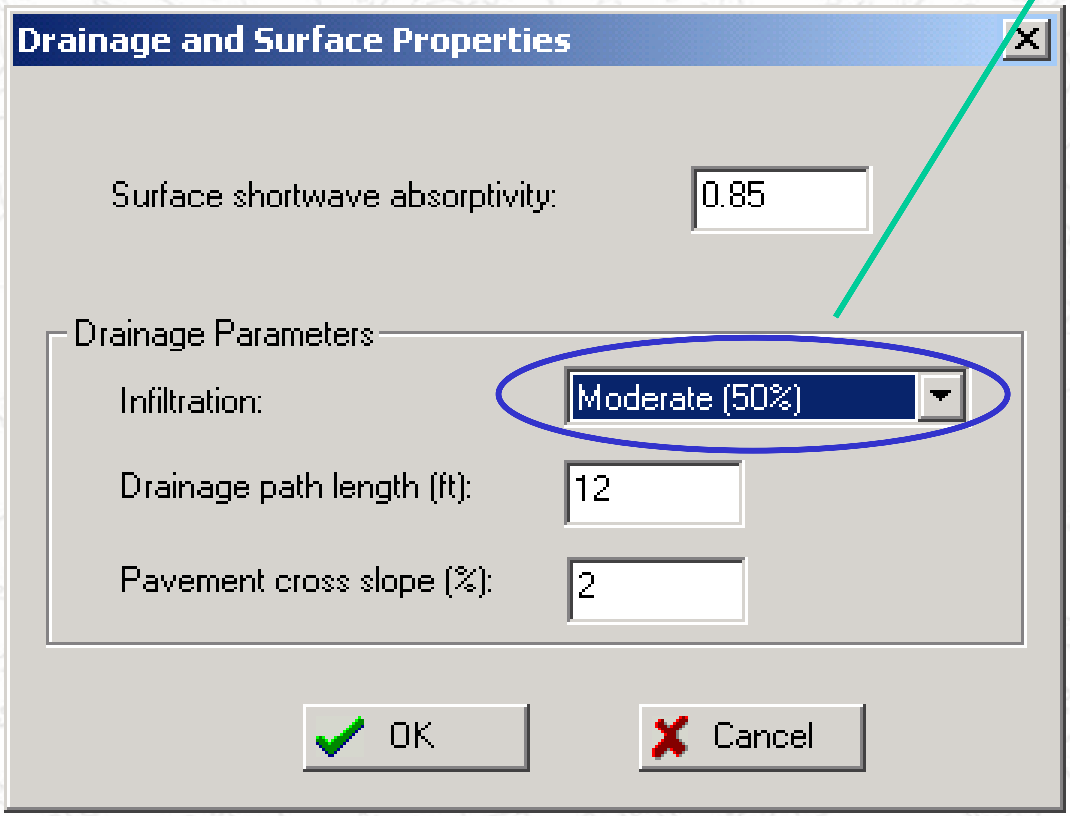
Thermal Cracking Input



Loading Time sec	Creep Compliance (1/psi)		
	Low Temp (°F)	Mid Temp (°F)	High Temp (°F)
	-4	14	32
1	2.41892e-007	3.30843e-007	4.52503e-007
2	2.6173e-007	3.76751e-007	5.59386e-007
5	2.90473e-007	4.47358e-007	7.40369e-007
10	3.14296e-007	5.09434e-007	9.15247e-007
20	3.40072e-007	5.80125e-007	1.13143e-006
50	3.77418e-007	6.88845e-007	1.49749e-006
100	4.08371e-007	7.84431e-007	1.85121e-006

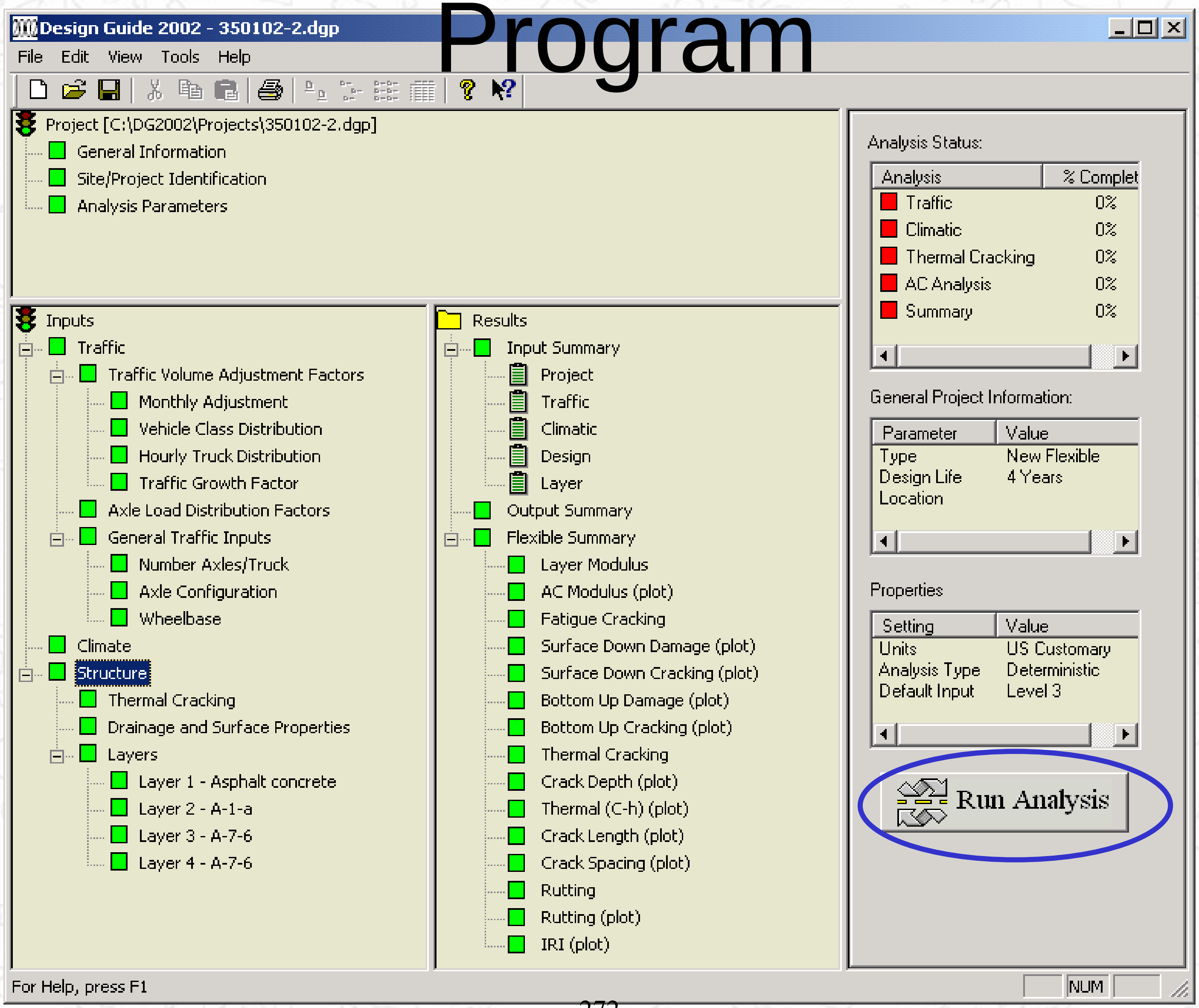
Option available to import or export a thermal cracking file

Drainage and Surface Properties



Based on shoulder type
Tied Shoulder- --Minor (10%)
Asphalt Shoulder– -Moderate (50%)
Gravel Shoulder- --Extreme (100%)

Save Project File and Run Program



2002 Design Procedure – Performance Models for Asphalt Concrete Pavements

INPUTS

2002 Design Guide Software

OUTPUTS:

Performance prediction
IRI
Surface-Down Cracking
Bottom-Up Cracking
Thermal Fracture
Permanent Deformation

Traffic, Materials and Climatic Models

Thermal cracking model

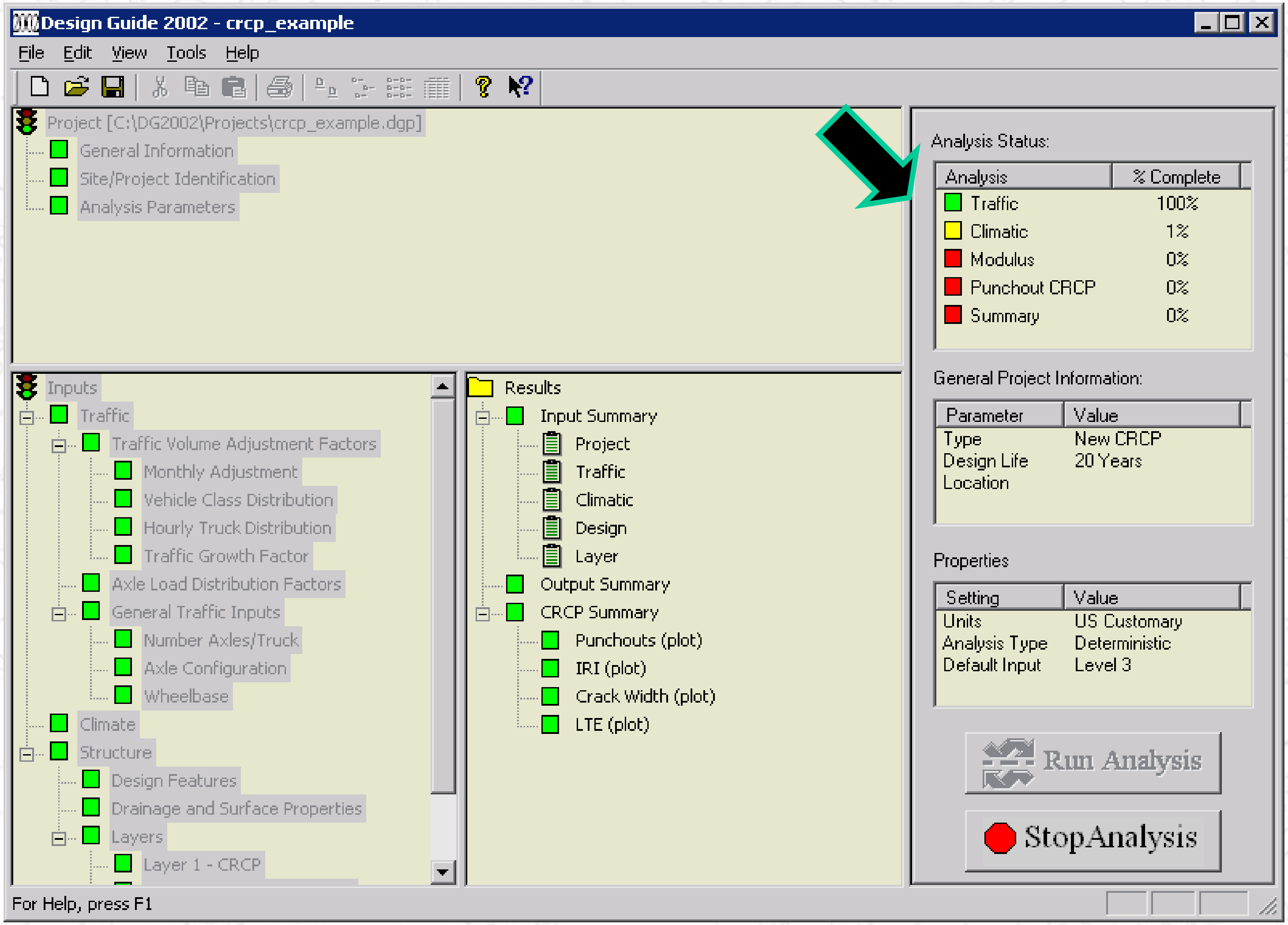
Structural response model: Linear Elastic Analysis

Damage prediction

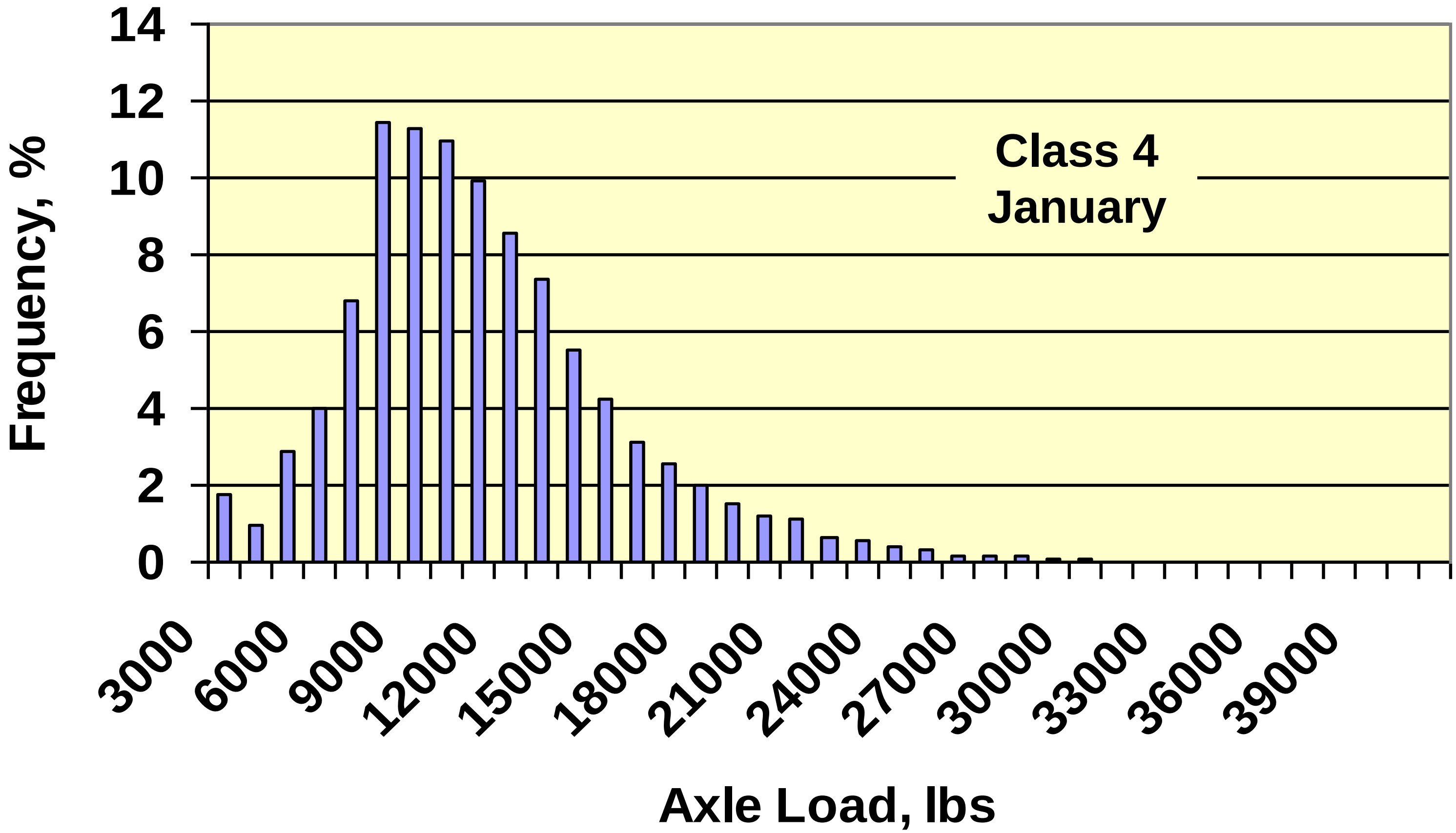
Reliability

Distress prediction models

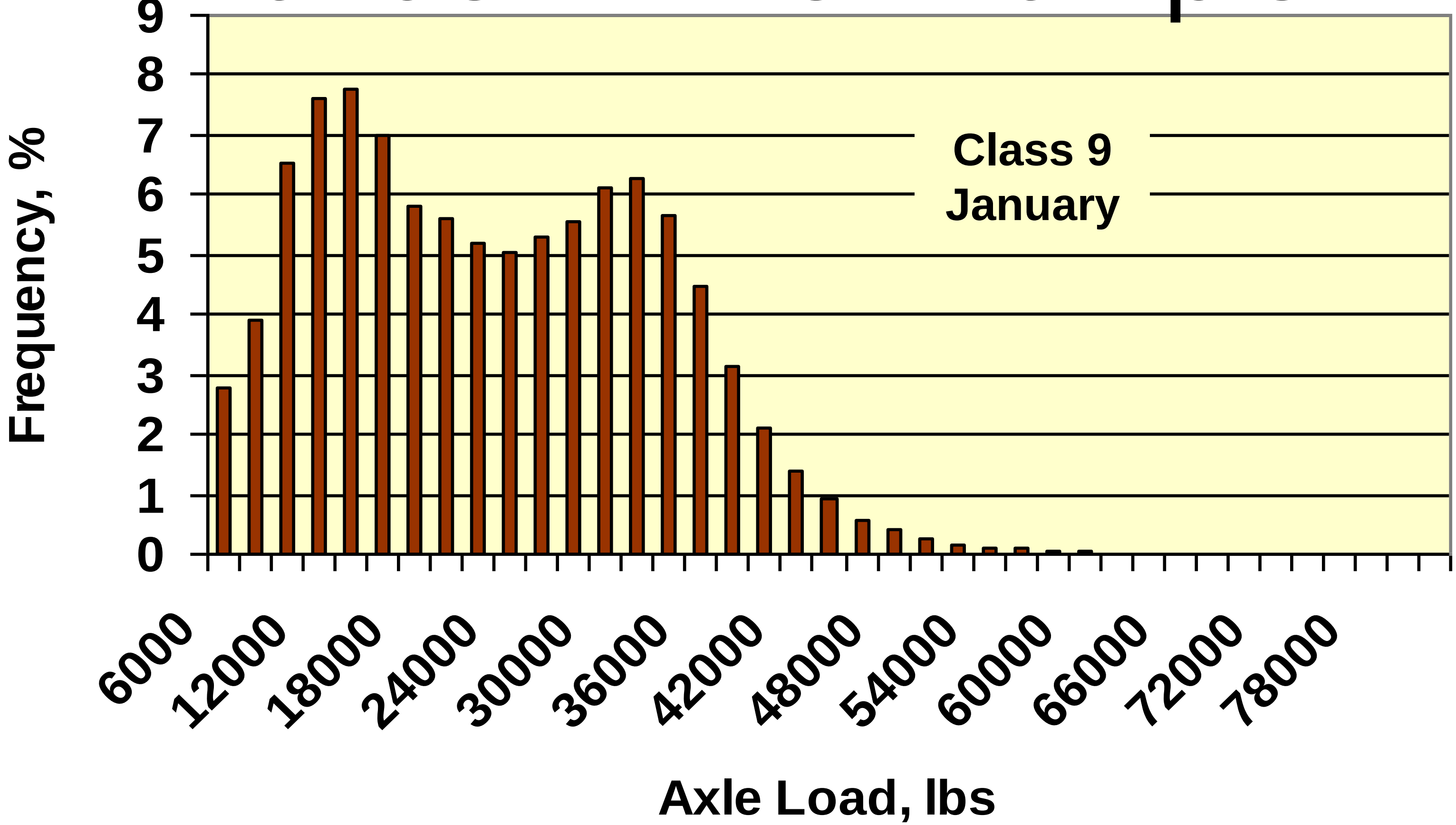
Program Runs Traffic Module



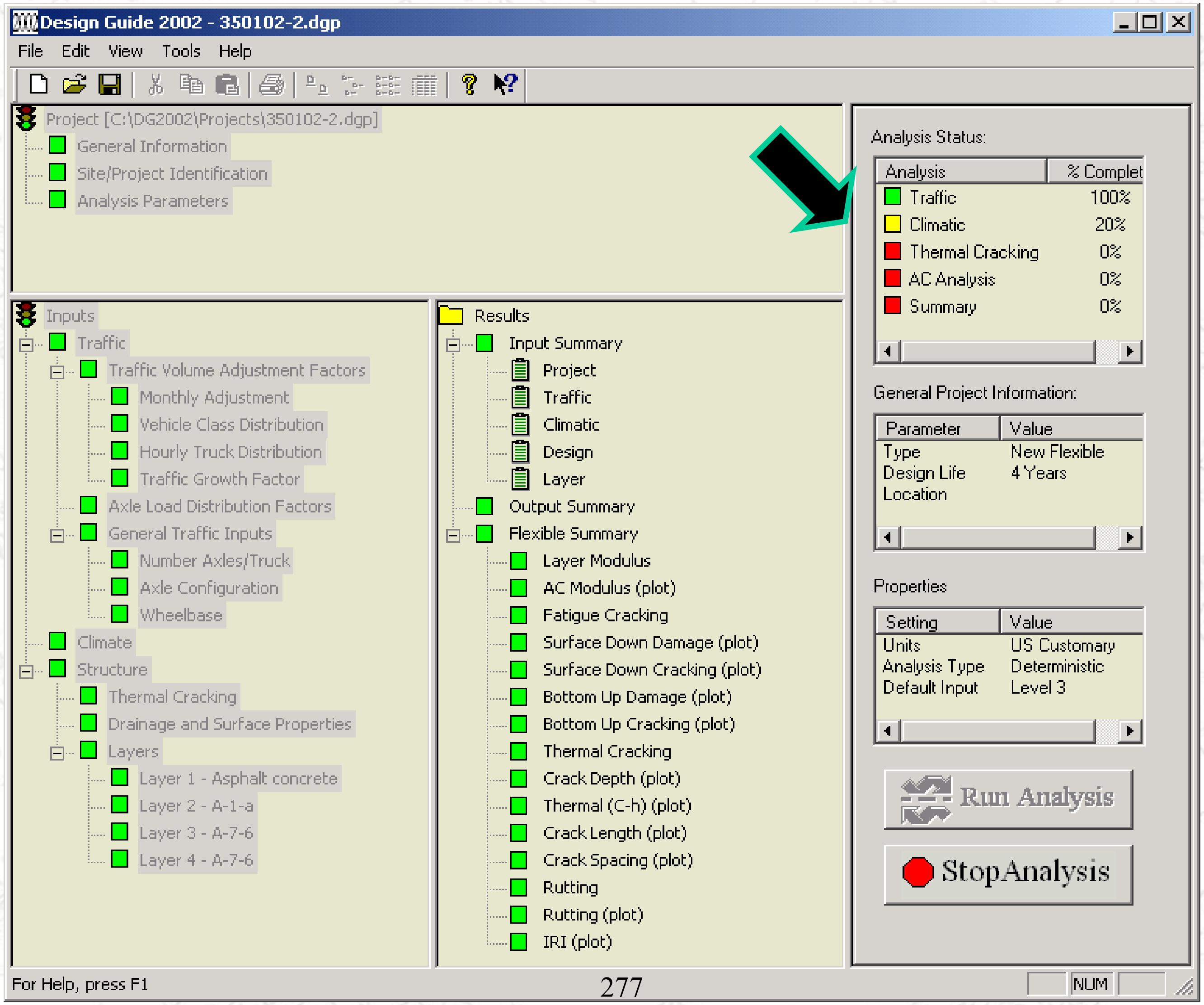
Software Creates Axle Load Distribution for Each Axle Type for Each Month – Single Axle Example



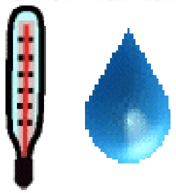
Software Creates Axle Load Distribution for Each Axle Type for Each Month – Tandem Axle Example



Run Program, cont. – Climate Module



Processing Inputs – Climate Model (EICM)



EICM Module predicts:

- ⇨ Environmental effects adjustment factors for unbou Resilient modulus

Finite Element/Linear Elastic Analysis Modules

- ⇨ Hourly temperature profile through AC layers

Thermal Cracking Module

- ⇨ Temperature Frequency Distribution at mid-depth of bound sublayers

Fatigue/Permanent Deformation Modules

- ⇨ Average moisture content for unbound materials

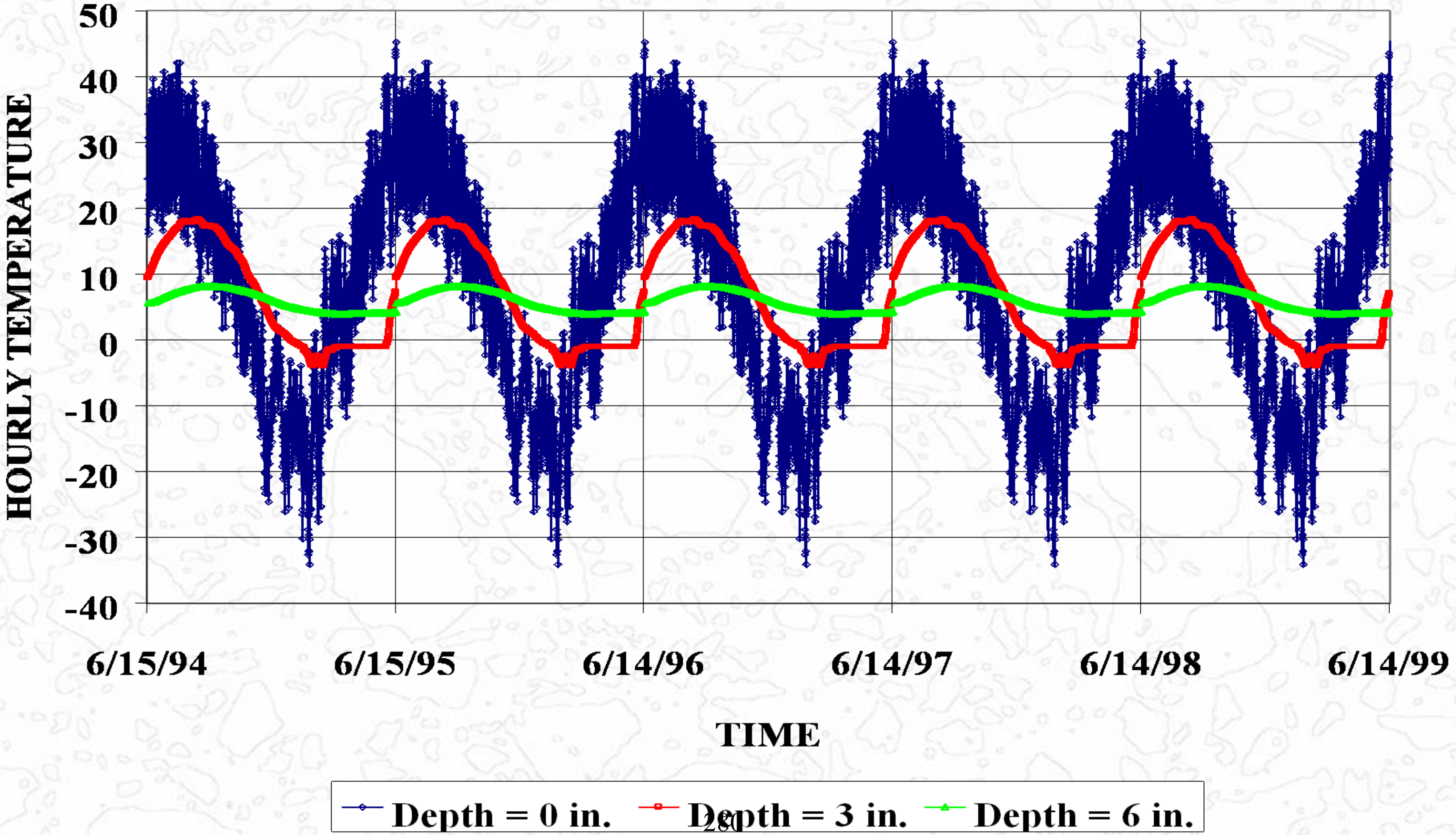
Unbound Permanent Deformation Module

Environmental Effects Adjustment Factors

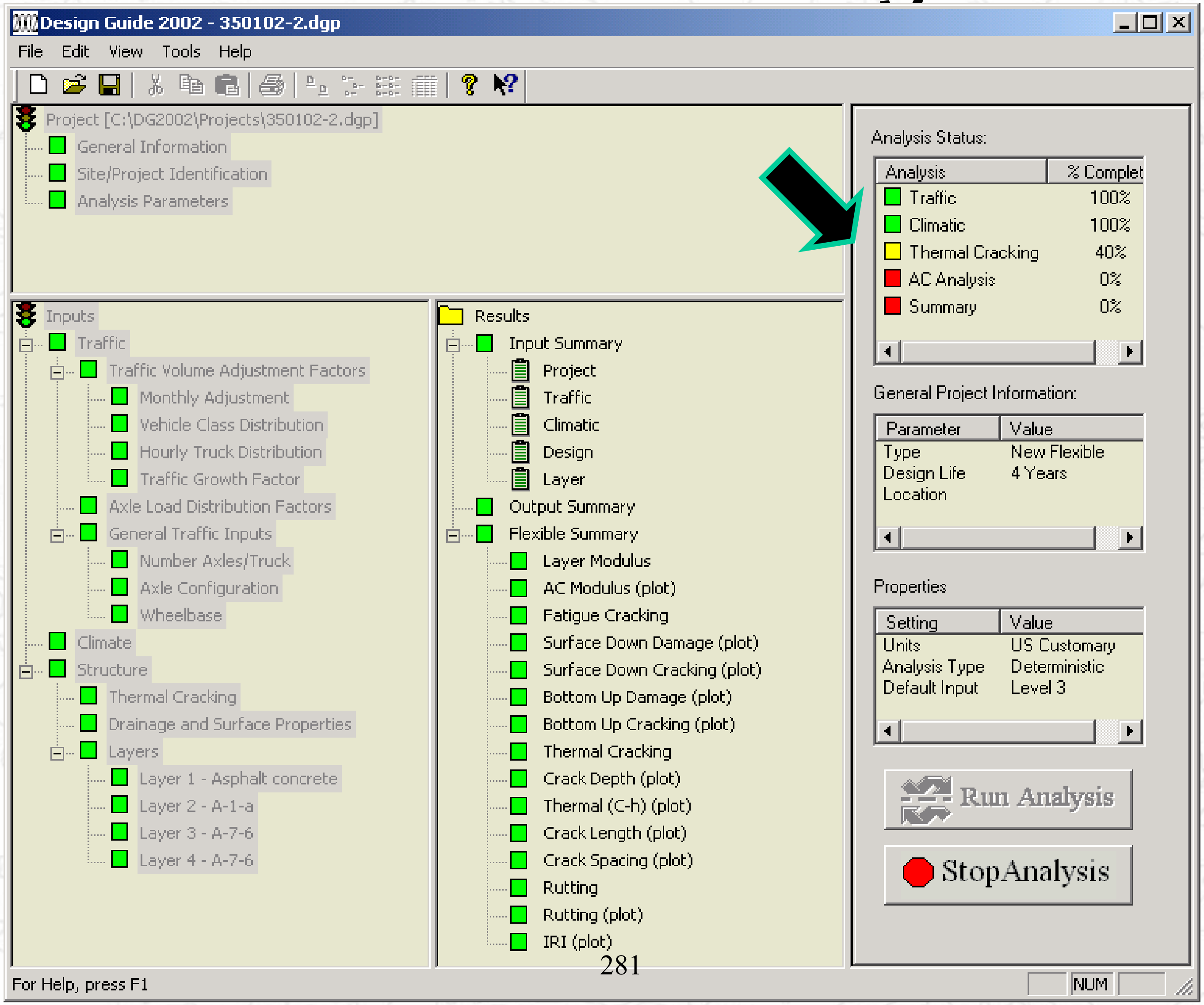
EICM computes climatic adjustment factors for the Resilient modulus for:

- ⇨ Frozen material
- ⇨ Recovering material
- ⇨ Unfrozen or fully recovered material
- ⇨ Environmental effect composite adjustment factor

Hourly Temperature Profile for AC Layers



Run Program, cont.
– Thermal Cracking Module

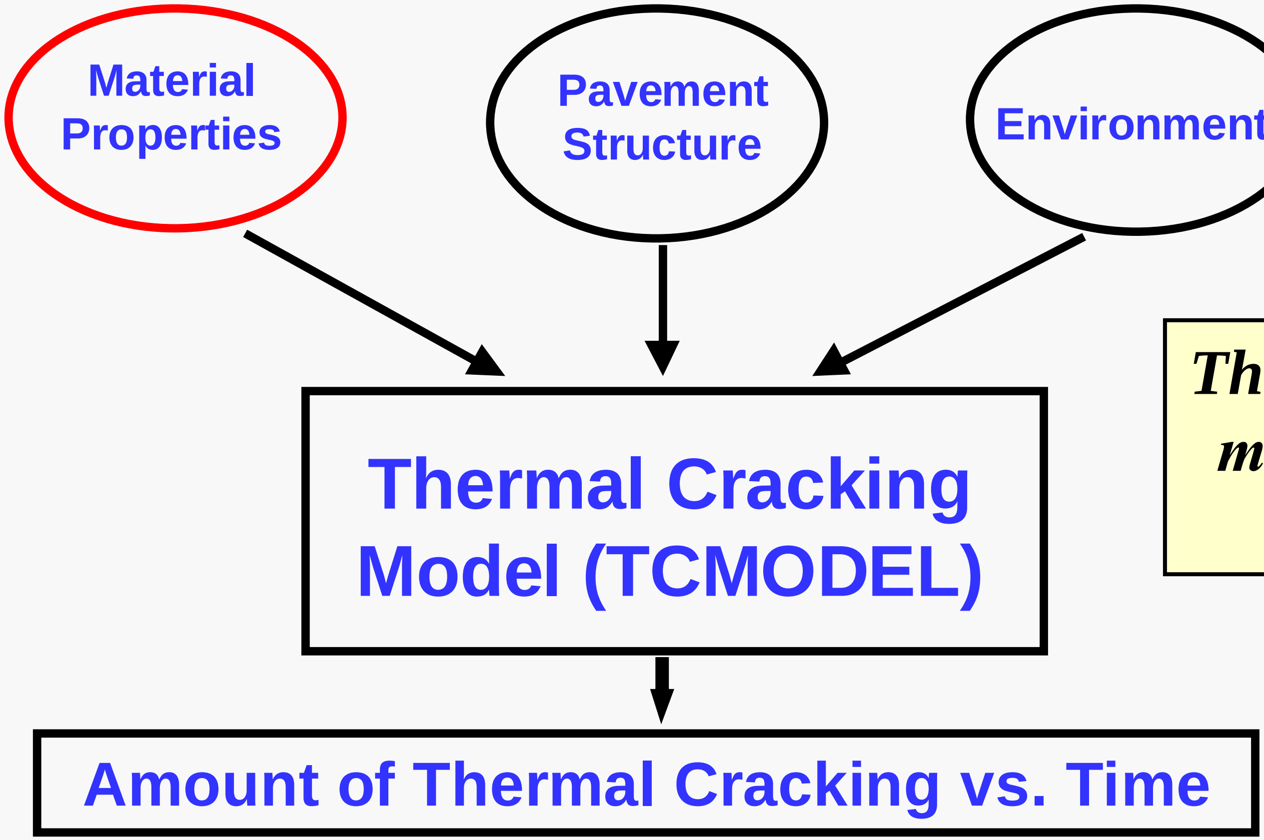


Thermal Cracking

Thermal Cracking Model

- ⇨ Uses SHRP Thermal Fracture Model
- ⇨ Use 100 sec creep data
  - ⇨ Previously required 1000 sec creep data
- ⇨ Tensile Strength Data

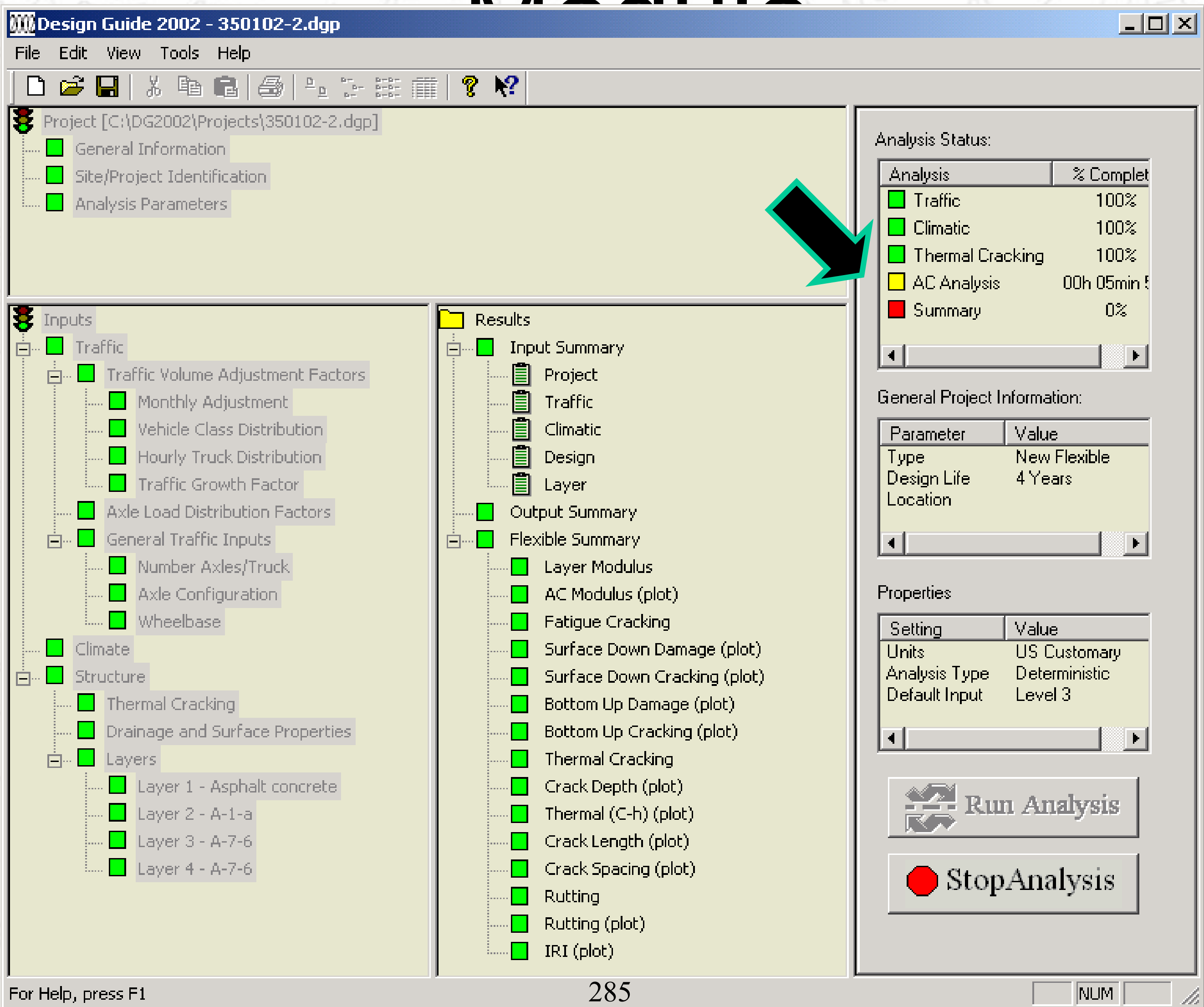
Thermal Cracking Model



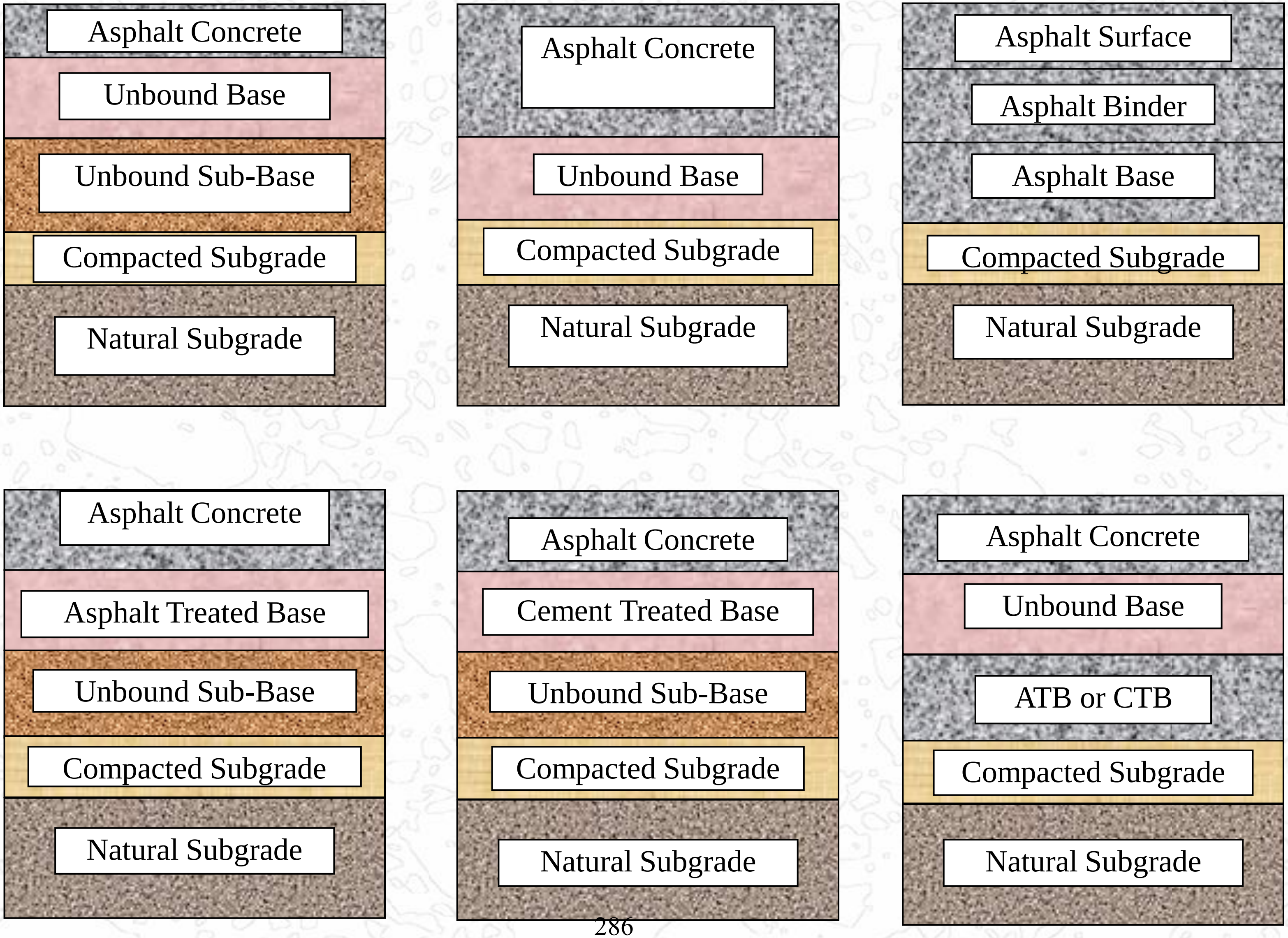
Thermal cracking model is “fully” mechanistic

Enhanced version of SHRP Thermal Cracking Model

Run Program, cont. – Asphalt Concrete Analysis Module

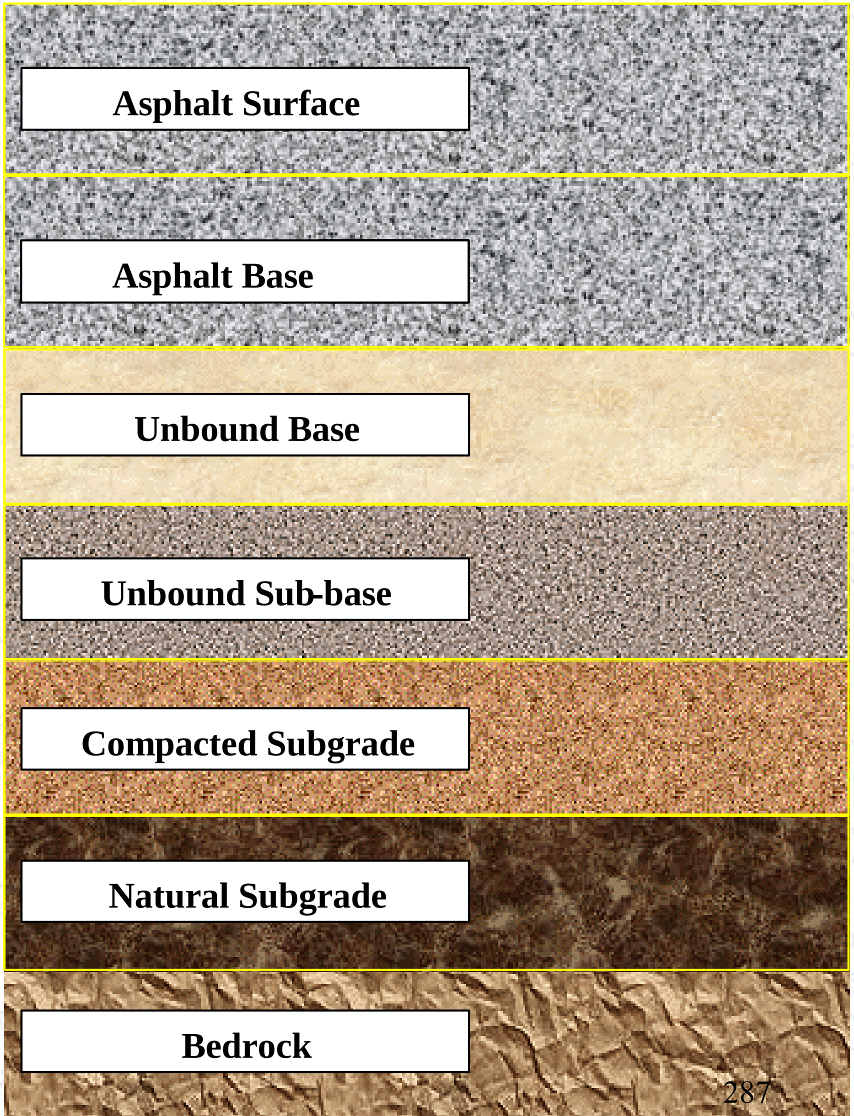


Different Strategies



Sub-Layering for Structural Analysis

Maximum Sub-Layering Depth = 8 feet



Asphalt Surface: $0.5''$; $h'_{AC} = h_{AC} - 0.5''$

Asphalt Base: *No Sub-Layering*

Unbound Base: $2''\{if\ \ h_B \geq 6\}$; $n_B = \text{int}\left(\frac{(h_B - 2)}{4} \right)\ for\ h_B > 6''$

Unbound Sub-base: $n_{SB} = \text{int}\left(\frac{h_{SB}}{4} \right)\ for\ h_{SB} \geq 8''$

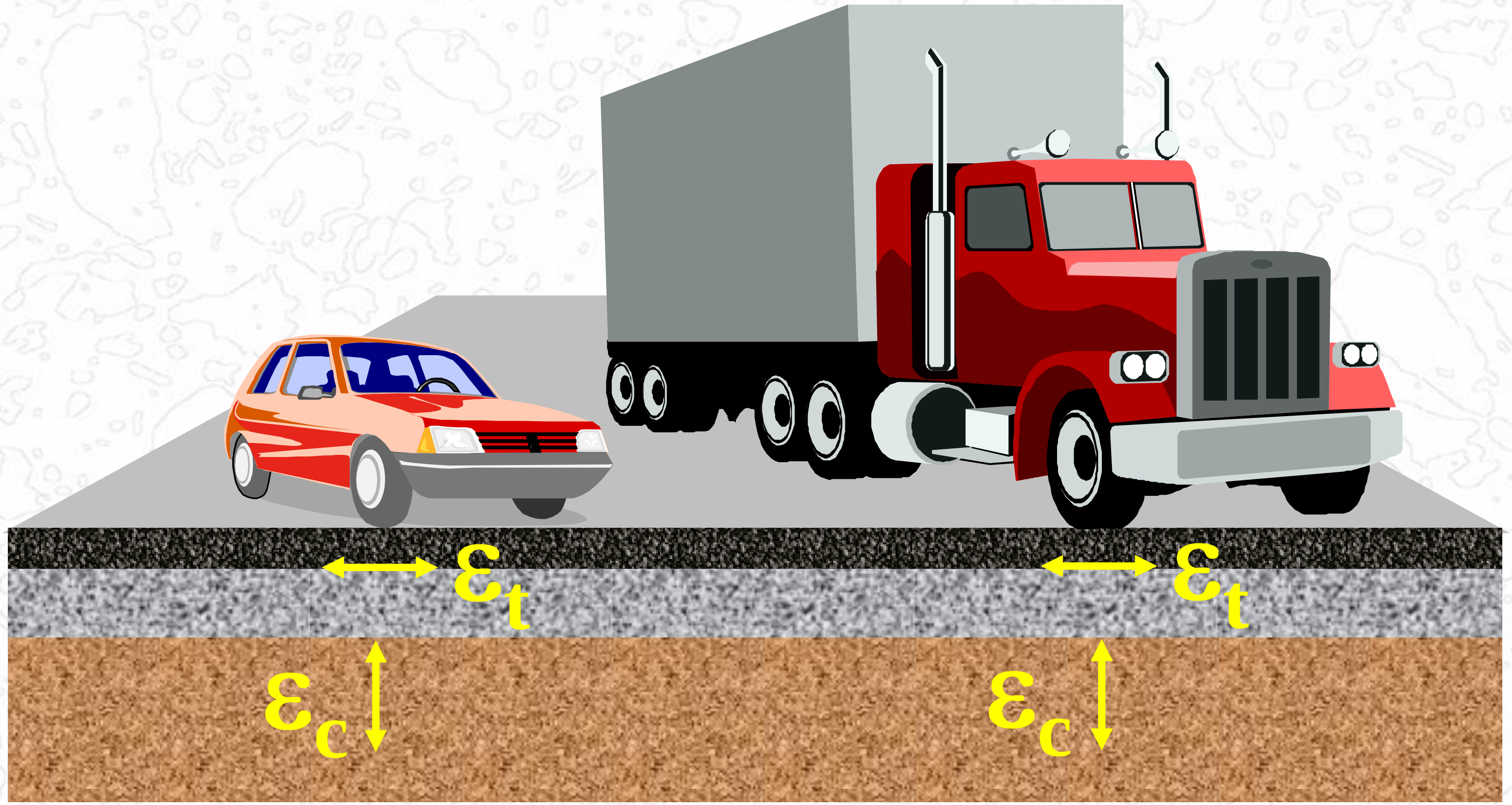
Compacted Subgrade: $n_{CSG} = \text{int}\left(\frac{h_{CSG}}{12} \right)\ for\ h_{CSG} > 12''$

Natural Subgrade: $n_{SG} = \text{int}\left(\frac{h_{SG}}{12} \right)\ for\ h_{SG} > 12''$

Computation Methodology

1. Define sub-layers
2. Adjust layer properties from EICM output.
    - Temp./Aging of HMA
    - Frost/Moisture in unbound materials
3. Simulate traffic loads.
4. Compute pavement critical response
    - FEA
    - MELT - JULEA

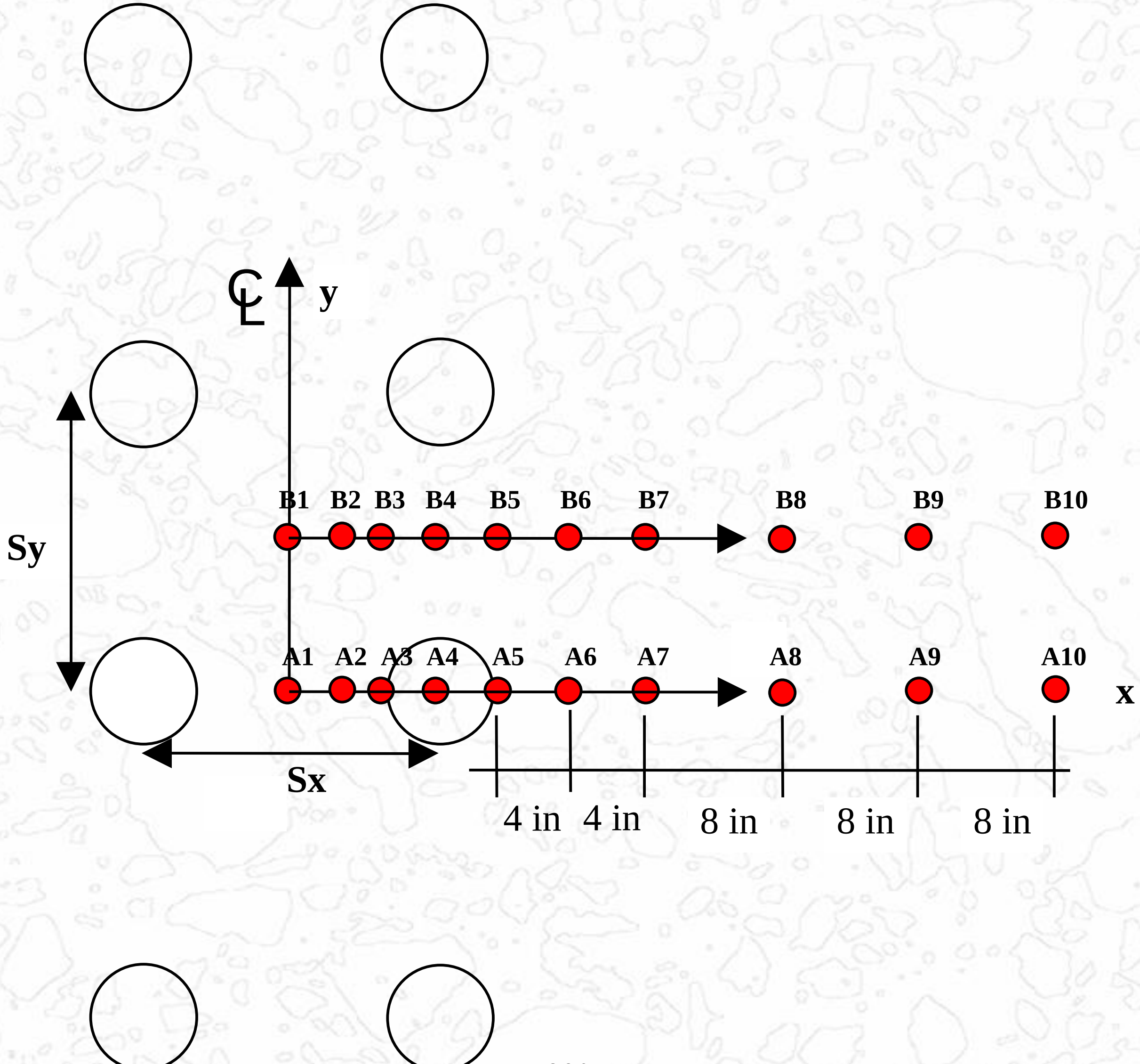
Critical Response Values



ε_t at surface + bottom of all bound layers (cracking)

ε_c at midthickness of all layers + top of subgrade (rutting)

Critical Response Locations



Computation Methodology, Cont'd

5. Calculate incremental damage for each traffic load & time period

6. Cumulate damage over time

7. Calculate distress over time

Damage Methodology

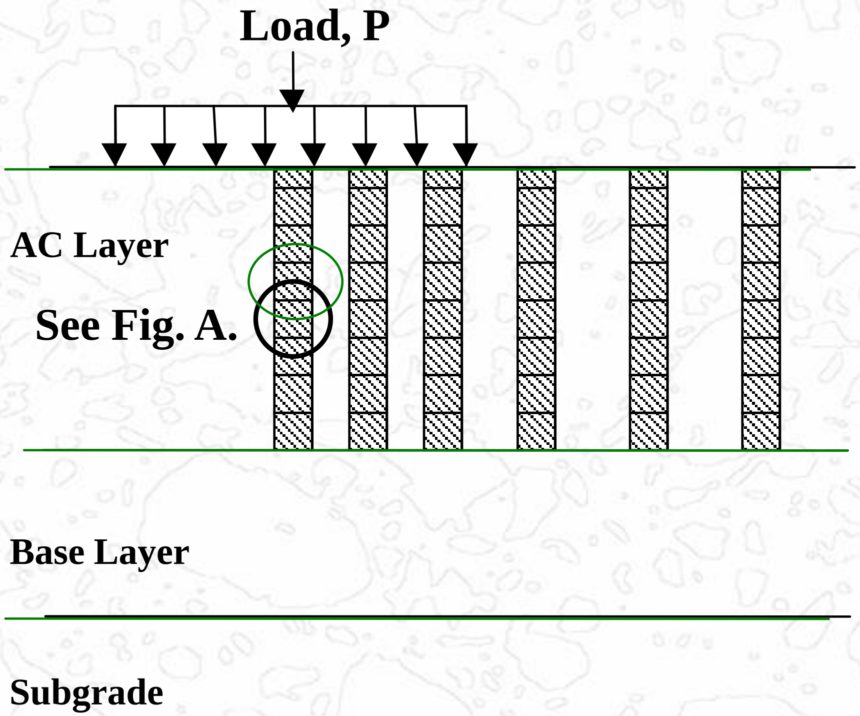
$$\Delta DI = \sum_{k=1}^{m} \sum_{i=1}^{j} \left[\frac{n_i}{N(\varepsilon_t)_i} \right]_k$$

Distortion:

$$\Delta RD = \sum_{k=1}^{m} \sum_{i=1}^{j} \sum_{d=1}^{l} \left[\varepsilon_P(d) (h_d) \right]_{k,i}$$

k = load level
i = time/season
d = sublayer

Accumulation of Rutting



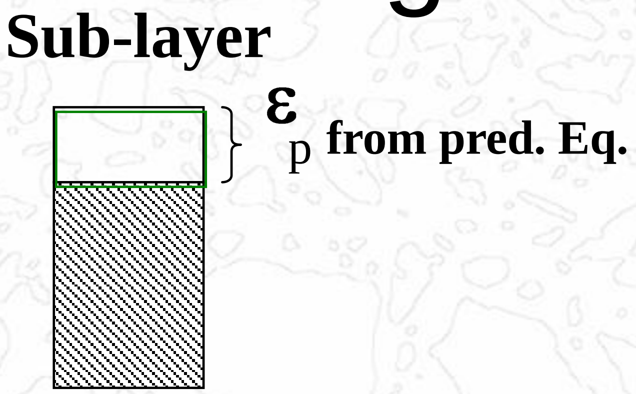
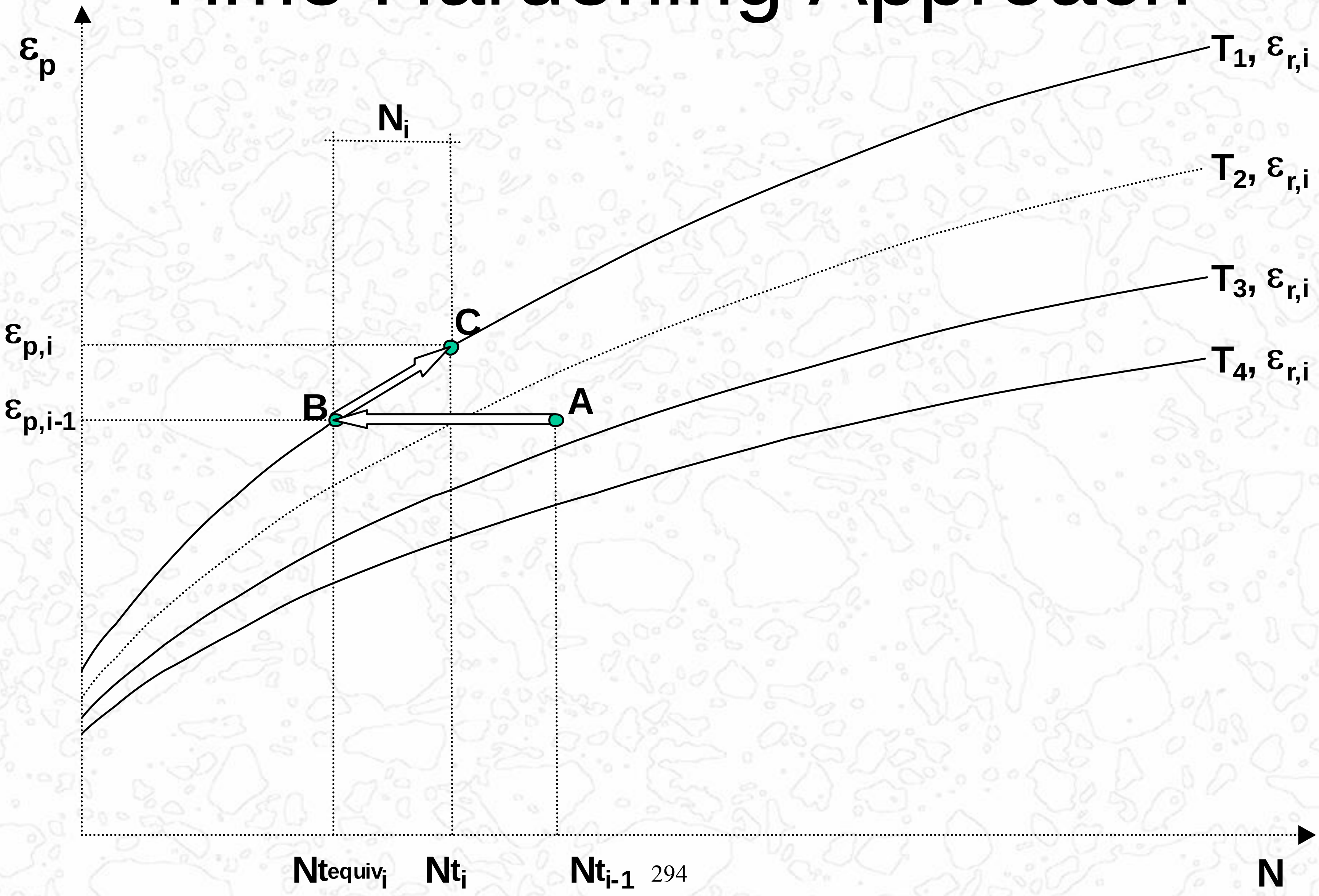


Fig. A

$$PD = \sum_{i=1}^{N\ sub\text{-}layers} \varepsilon_p^i \times h^i$$

Similar treatment for permanent deformation of unbound layers

Time Hardening Approach



Design Criteria

RUT DEPTH

Criterion

TIME

FATIGUE CRACKING

Criterion

TIME

Design Period

Predicted Distresses

Permanent Deformations

Basic Rutting Equation

Captures stress level effect

$$\log\left(\frac{\varepsilon_p}{\varepsilon_r}\right) = a_o + a_1 \log(N) + a_2 \log(T)$$

R^2 = 0.73

S_e = 0.309

S_e/S_y = 0.522

N_{tests} = 3476 (>300 mixes)

Function of material characteristics, but these less important than N and T

Similar treatment for HMA and unbound material permanent deformation

Rutting in HMA

$$\log\left(\frac{\varepsilon_p}{\varepsilon_r}\right) = -3.15552 + \log\beta_{r_1} + 1.734\beta_{r_2}\log T$$

$$+0.39937\beta_{r_3}\log N$$

ε_p = plastic strain

ε_r = resilient strain

T = layer temperature (deg F)

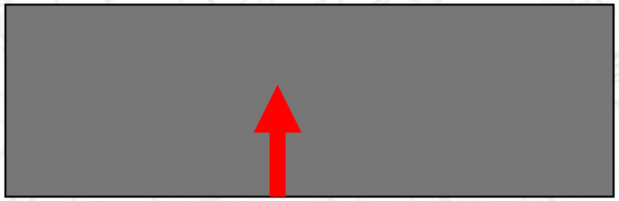
N = no of load repetition

βr_1, βr_2, βr_3 = calibration factors

Fatigue Cracking

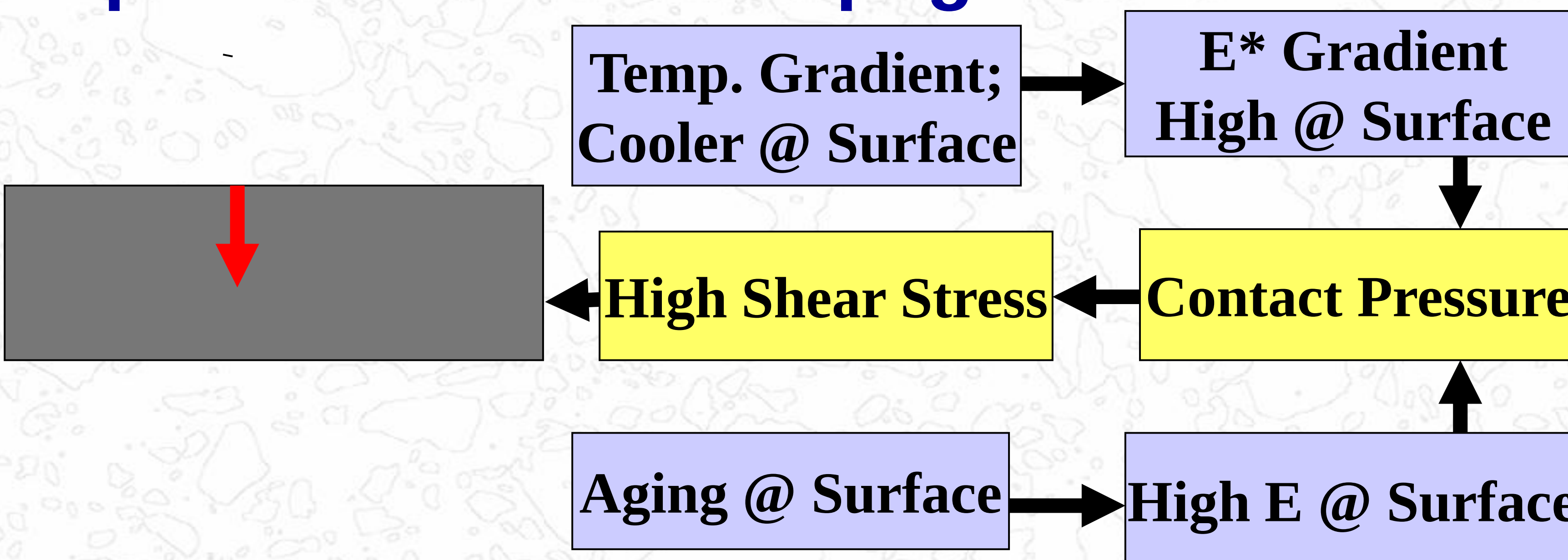
Simplified Fatigue Model

Bottom – Up Crack Propagation



Classical Fatigue Mechanism.

Top – Down Crack Propagation



Fatigue Cracking Model

$$N_f = \beta_{f1} k_1 \left(\frac{1}{\varepsilon_t}\right)^{k_2 \beta_{f2}} \left(\frac{1}{E}\right)^{k_3 \beta_{f3}}$$

β_{f1} ; β_{f2} ; β_{f3} → Calibration Factors

Pavement Smoothness

Smoothness Model

$$IRI = IRI_O + \Delta IRI_D + \Delta IRI_{SF}$$

IRI_O = **Initial IRI**

ΔIRI_D = **Change in IRI due to distress**

ΔIRI_{SF} = **Change in IRI due to site factors**

Smoothness Components

Surface Distresses D_j :

D_1 = Rut Depth Coefficient of Variation

D_2 = Fatigue Cracking

D_3 = Patching*

D_4 = Pot Holes, ..etc...D_n*

Non-Distress Variables S_f :

Rainfall

Material Gradation

Plasticity Index

Freezing Index

*Determined from separate empirical models

IRI vs. Distress Summary

Variable	Unbound Base	ATB	CTB	HMA OVERLAY	
				HMA	PCC
Site Factor	X	X			
Age	X	X		X	X
Alligator Ckg	X	X	X	X	
Rut Depth	X		X		X
Transverse Ckg.	X	X	X	X	X
Block Ckg.	X		X		
Longitudinal Ckg.	X		X	X	
Pot Holes				X	
Patching		X		X	

Connecticut Department of Transportation

M-E PDG Training Module IV

Pavement Rehabilitation/ Composite Pavement Design

Prepared by Dr. Iliya Yut

Department of Civil Engineering, UConn

May2013

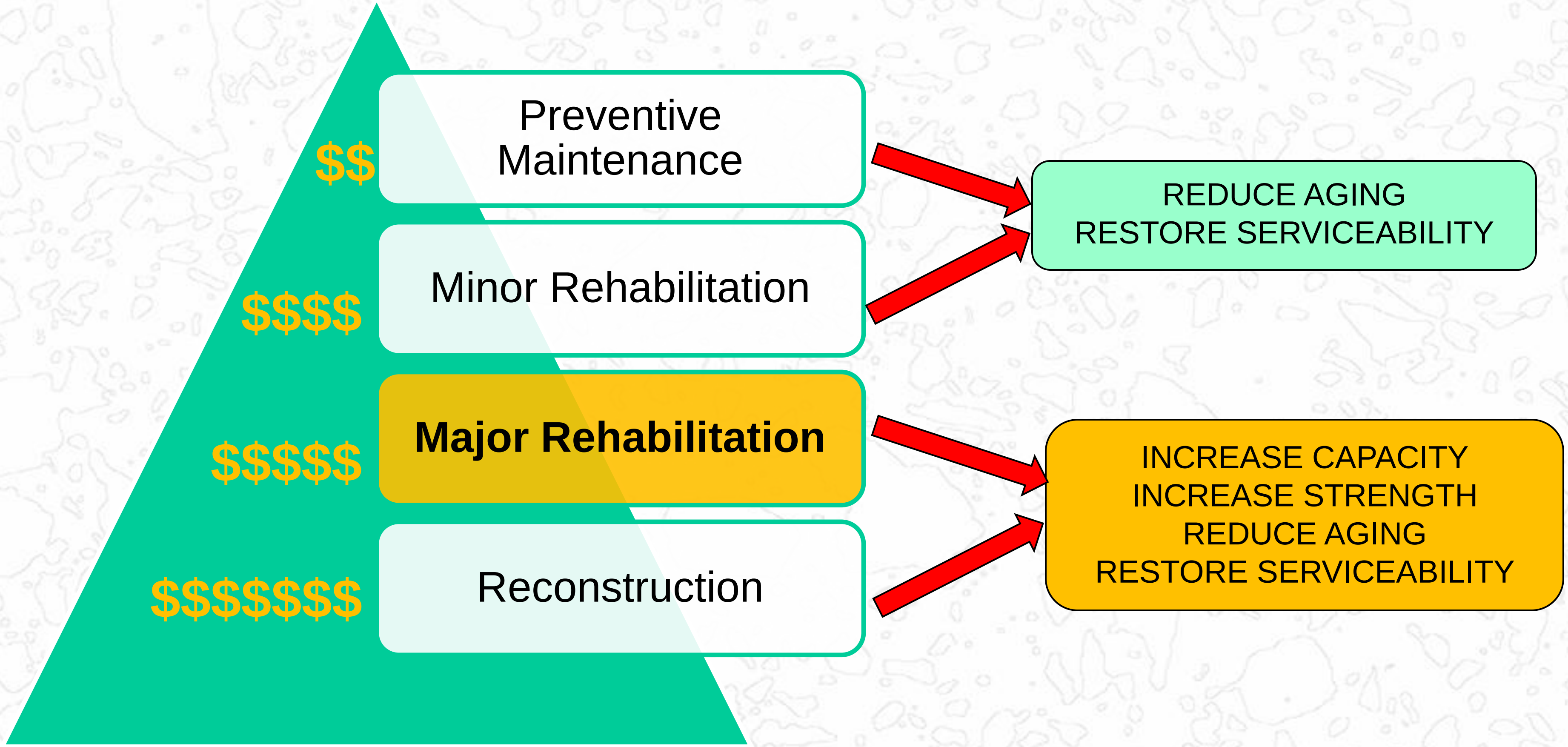
Outline

- ⇨ Overview of Rehabilitation Design Process
  - ⇨ Major Rehabilitation Strategies
  - ⇨ Recycling of Existing Pavement
  - ⇨ Identification of Feasible Strategies
- ⇨ AC Rehabilitation
- ⇨ PCC rehabilitation

Overview of Rehabilitation Design Process

Zofka, Fall 2010

Pavement Rehabilitation and Maintenance Activities



Major Rehabilitation Strategies

- Objective:
  - To repair existing deterioration and minimize future deterioration
- Parameters: ***type***, ***quantity***, and ***timing***
- Conditions addressed:
  - *Structural (distresses)*
  - *Functional (smoothness)*
  - *Material durability*
  - *Shoulder condition*

Major Rehabilitation Strategies

Reconstruction with/without Lane Additions

Pavement Type	Deficiency Addressed	Scope of Treatment
Flexible	H-severity fatigue cracking H-severity rutting Stripping Major subgrade movements Frost heave	Remove & Replace paved lane(s) Remove complete structure Add extra lane Widen existing lane
Rigid	High %% of cracked slabs High %% of deteriorated joints D-cracking Inadequate subgrade support Frost heave	

Major Rehabilitation Strategies

Structural Overlay

Overlay Type	Purpose
Thick HMA over Flexible (h > 1.5 in)	Increase structural capacity for anticipated future traffic Correct functional deficiencies
Thin HMA over Flexible (h <= 1.5 in)	Improve ride quality Increase surface friction Repair M-severity rutting, bleeding, weathering, raveling, bumps, settlement, or heaves (Does not address fatigue cracking and H-severity rutting)

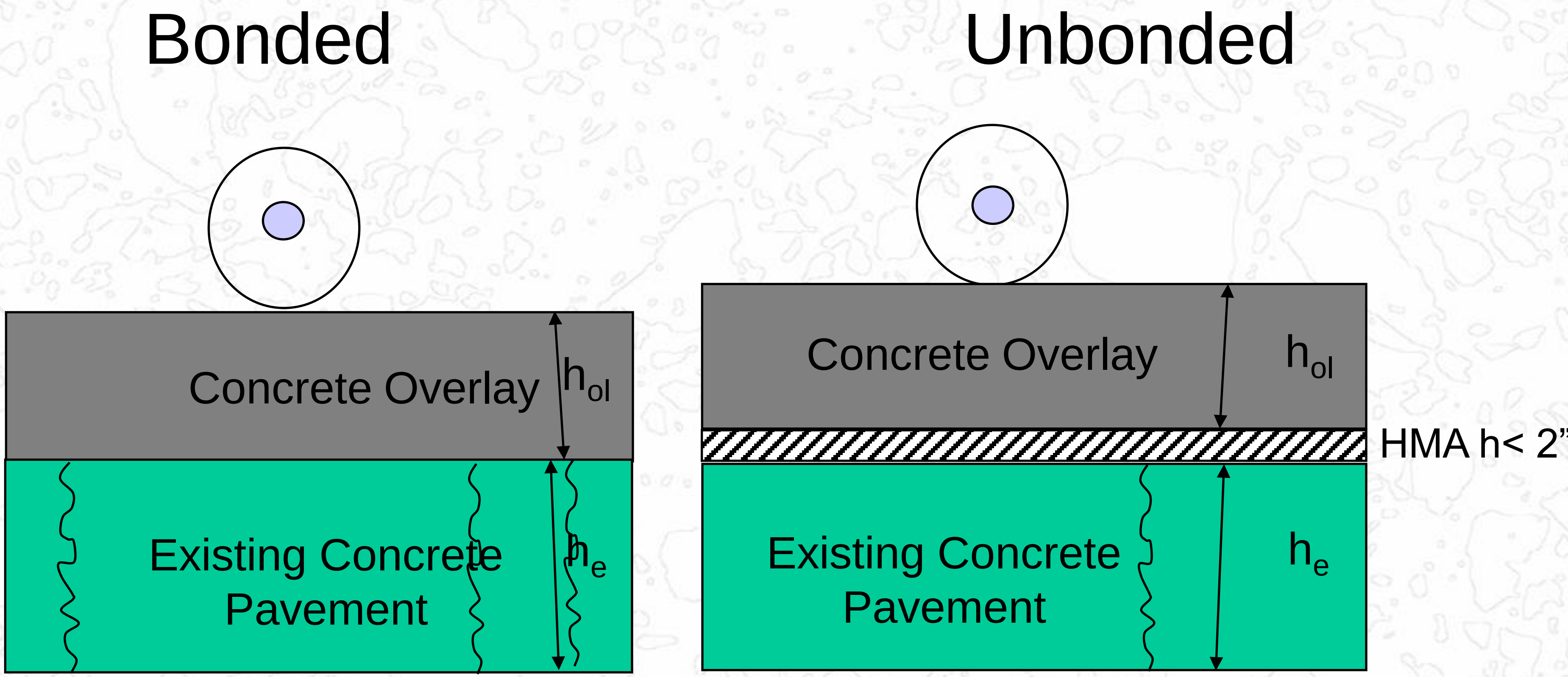
Major Rehabilitation Strategies

Structural Overlay (Cont.)

Overlay Type	Purpose
Thin HMA over Intact Rigid or Composite (1in<= h <= 3 in)	Improve ride quality Increase surface friction
Thick HMA over Intact Rigid or Composite (h > 3 in)	Increase structural capacity for anticipated future traffic Correct functional deficiencies
HMA over Intact Rigid: •Must withstand reflective cracking •Does not address excessive joint/crack deterioration	
HMA over Fractured PCC (Rubblized in12-in pieces or Crack-and-Seated in 1-3ft pieces)	Prevent reflective cracking Increase structural capacity for anticipated future traffic

Major Rehabilitation Strategies

PCC over PCC



Major Rehabilitation Strategies

Structural Overlay (Cont.)

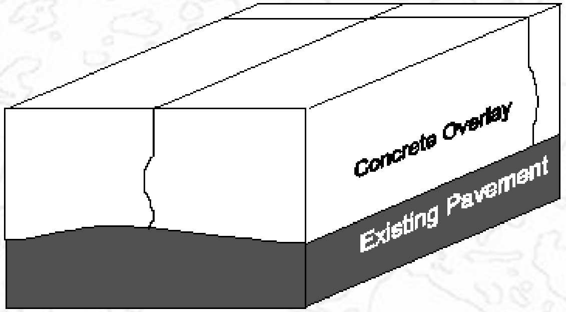
Overlay Type	Purpose
Bonded PCC over PCC (h <= 4in)	Increase structural capacity Correct L,M-severity distresses (Not recommended if H-severity deterioration or D-cracking exists)
Unbonded PCC over PCC (h > 5in)	Address H-severity distresses (separation level) Increase structural capacity

Major Rehabilitation Strategies

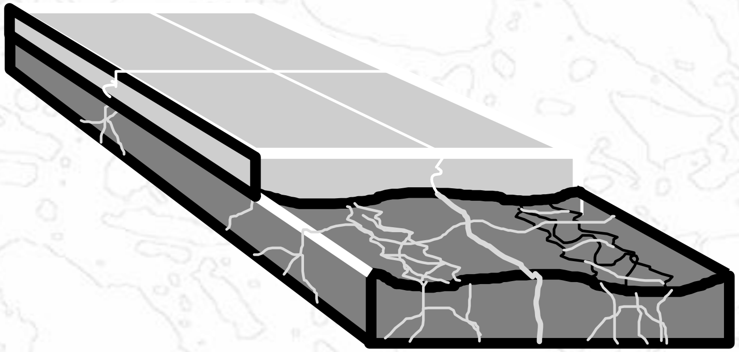
Structural Overlay (Cont.)

PCC over HMA (Whitetopping)

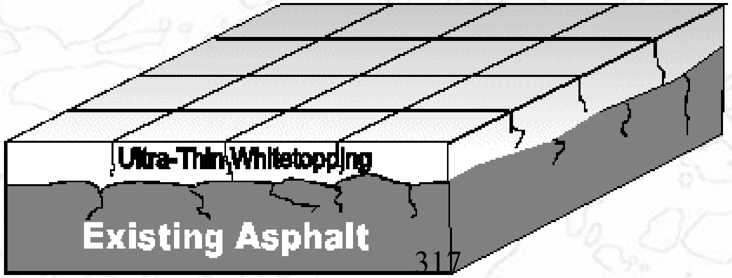
Conventional (>=8in)



Thin (4-8in)



Ultrathin (2-4in)



Major Rehabilitation Strategies

Structural Overlay (Cont.)

Overlay Type	Characteristics
Conventional Whitetopping Thin Whitetopping	Behaves as a new PCC over asphalt treated base (ATB) Increases structural capacity Repairs H-severity distresses
Ultrathin Whitetopping	Requires bonding between UTW overlay and existing HMA Requires shorter joint spacing (2-6 ft) Substantial HMA thickness is desired (e.g., full-depth HMA) Medium or low traffic volume is recommended Best addresses rutting and washboarding on parking lots and intersections

Other Repair and Preventive Treatments

Table 3.5.1. Candidate repair and preventative treatments for flexible, rigid, and composite pavements (*1*).

Pavement Type	Distress	Repair Treatments	Preventative Treatments
Flexible and composite	Alligator (fatigue) cracking	Full-depth repair	Crack sealing
	Bleeding	Apply hot sand	
	Block cracking	Seal cracks	
	Depression	Level up overlay	
	Polished aggregate	Skid resistant surface treatment Slurry seal	
	Potholes	Full-depth repair	Crack sealing and seal coats
	Raveling	Seal coats	Rejuvenating seal
	Rutting	Level up overlay and/or cold milling	
	Reflective cracking	Full or partial depth repair	Saw and seal
Rigid	Jointed concrete pavement pumping (and low joint load transfer efficiency)	Subseal (effectiveness depends on materials and procedures)	Reseal joints Restore joint load transfer Subdrainage Edge support (tied PCC shoulder edge beam)
	Jointed concrete pavement joint faulting	Grind Structural overlay	Subseal Reseal joints Restore load transfer Subdrainage Edge support (tied PCC shoulder edge beam)
	Jointed concrete pavement slab cracking	Full-depth repair Replace/recycle lane	Subseal (loss of support) Restore load transfer Structural overlay
	Jointed concrete pavement joint or crack spalling	Full-depth repair Partial-depth repair	Reseal joints
	Punchout (CRCP)	Full-depth repair	Polymer or epoxy grouting Subseal (loss of support)
	PCC disintegration	Full-depth repair	None, thick overlay

Recycling of Existing Pavements

Table 3.5.2. Highway and pavement applications and material uses (*11*).

Major Layer Category	Primary Application of Recycled Paving or Byproduct Material	Recycled Paving or Byproduct Material
Asphalt concrete or AC-treated layers	Aggregate in AC	Blast furnace slag, coal bottom ash, coal boiler slag, foundry sand mineral processing wastes, nonferrous slag, recycled asphalt pavement, scrap tires, steel slag
	Aggregate in cold mix AC	Coal bottom ash Recycled asphalt pavement
	Aggregate in seal coat or surface treatment	Blast furnace slag Coal boiler slag
	Mineral filler	Cement kiln dust, lime kiln dust, coal fly ash
PCC or cement-treated layers	Aggregate	Recycled concrete
	Supplementary cementitious materials	Coal fly ash Blast furnace slag
Pozzolan stabilized base/subbase	Aggregate	Coal bottom ash Coal boiler slag
	Cementitious material • Pozzolan • Pozzolan activator • Self-cementing material	Coal fly ash Cement kiln dust Lime kiln dust
Granular unbound base and subbase	Granular base	Blast furnace slag, coal boiler slag, mineral processing wastes Nonferrous slag, cycled asphalt pavement, Recycled concrete
Embankment or fill	Embankment or fill	Coal fly ash, mineral processing wastes, nonferrous slag Recycled asphalt pavement, Recycled concrete
Flowable fill	Aggregate	Coal fly ash Foundry sand Quarry fines
	Cementitious material • Pozzolan • Pozzolan activator • Self-cementing material	Coal fly ash Cement kiln dust Lime kiln dust

Identification of Feasible MR Strategies

Phase I: Evaluate Existing Pavement → **Phase II: Select and Design MR Alternatives** → **Phase III: Evaluate MR Alternatives** → **Phase IV: Select Most Feasible MR Alternative**

Phase I: Evaluate Existing Pavement

- Step 1: Determine Existing Condition
- Step 2: Determine Causes and Mechanism of Distress
- Step 3: Define Existing Problems and Inadequacies
- Step 4: Identify Possible Constraints

Phase II: Select and Design MR Alternatives

- Step 5: Select feasible Candidate Strategies
- Step 6: Develop Preliminary Design for Each Candidate Strategy

Phase III: Evaluate MR Alternatives

- Step 7: Perform Life-Cycle Cost Analysis
- Step 8: Determine Relevant Non-Monetary Factors

Phase IV: Select Most Feasible MR Alternative

- Step 9: Determine Most Feasible or Preferred MR Strategy

Identification of Feasible MR Strategies

⇨ Phase I Considerations and Assessments

Table 3.5.4. Areas of overall condition assessment and corresponding data sources.

Area of Assessment	Data Source						Condition Rating
	Distress Survey	Smoothness Testing	Friction Testing	Drainage Survey	Nondestructive Testing	Destructive Testing	
Structural Adequacy	√			√	√	√	Adequate Marginal Inadequate
Functional Adequacy	√	√	√				Adequate Marginal Inadequate
Drainage Adequacy	√			√	√	√	Adequate Marginal Inadequate
Materials Durability	√			√	√	√	Adequate Marginal Inadequate
Maintenance Applications	√						Adequate Marginal Inadequate
Shoulders Adequacy	√				√	√	Adequate Marginal Inadequate
Variability Along Project	√			√	√	√	Adequate Marginal Inadequate
Misc.	√			√		√	Adequate Marginal Inadequate

Identification of Feasible MR Strategies

➪ **Phase II Step 5 – Candidate MR Treatment Selection for existing HMA and HMA on PCC pavements**

Pavement Condition	Distress Types	Candidate Treatments for Developing Rehabilitation Strategy											
		Full-Depth Asphalt Repair	Partial-Depth Asphalt Repair	Cold Milling	Hot or Cold In-place Recycling	Crack Sealing	Chip Seal	AC Overlay	AC Overlay of Fractured Slab	Bonded PCC Overlay	Unbonded PCC Overlay	Subdrainage Improvement	Reconstruction (AC or PCC)
Structural	Fatigue cracking	√	√	√	√	√		√		√	√		√
	Longitudinal cracking in wheel path (low severity)	√			√	√		√		√	√		√
	Thermal cracking	√		√	√	√		√		√	√		√
	Rutting			√	√			√		√	√		√
	Reflection cracking	√	√	√				√	√	√	√		√
Functional	Excessive patching							√			√		
	Smoothness			√				√		√	√		
Drainage	Raveling		√	√	√			√					
	Stripping	√	√	√				√					
Durability	Raveling		√	√	√		√	√					
	Bleeding	√	√	√	√			√					
	Block cracking		√	√	√	√		√		√	√		√
	Shoving						√						
	Rutting			√	√			√		√	√		√
Shoulders	Same as traveled lanes	Same treatments as recommended for traveled lanes											

Identification of Feasible MR Strategies

⇨ Phase II Step 5 (Cont.) – Candidate MR Treatment Selection for existing PCC pavements

Pavement Condition	Distress Types	Candidate Treatments for Developing Rehabilitation Strategy												
		Full-Depth Repair and Slab Replacement	Partial-Depth Repair	Undersealing/Slab Jacking	Load Transfer Restoration	Joint Resealing	Diamond Grinding	Pressure Relief Joints	AC Overlay	AC Overlay of Fractured PCC Slab	Bonded PCC Overlay	Unbonded PCC Overlay	Subdrainage Improvement	Reconstruction
Structural	JPC and JRC deteriorated cracked slabs	√								√		√		√
	CRC longitudinal cracking	√								√		√		√
	JPC and JRC transverse joint/crack faulting				√		√		√	√	√	√	√	
	CRC punchouts	√								√		√		√
	JPC, JRC, and CRC patch/patch deterioration	√	√							√		√		√
Functional	Excessive patching								√			√		√
	Smoothness								√			√		√
Drainage	JPC and JRC pumping													
	JPC and JRC transverse joint/crack faulting				√		√		√	√	√	√	√	
	PCC durability (D-cracking and reactive aggregates)	√							√	√		√		√
	JPC and JRC corner breaks	√								√		√		√
Durability	PCC Durability (D-cracking and ASR)	√							√	√		√		√
	JPC, JRC, and CRC Patch/Patch Deterioration	√	√							√		√		√
	PCC Longitudinal Joint Spalling	√	√							√		√		√
	JPC and JRC Transverse Joint Spalling	√	√							√		√		√
	Treated base/subbase durability													√
Shoulders	Same as traveled lanes													
Joint condition	JPC and JRC load transfer deterioration				√									
	JPC and JRC transverse joint seal damage					√								
	JPC and JRC pumping			√	√								√	
	JPC and JRC transverse joint/crack faulting,		324		√		√		√	√	√	√	√	
	Joint surround cracking	√								√		√		√

Identification of Feasible MR Strategies

- **Phase II Step 6 – Preliminary Design of Alternatives**
  - **Information needed:**
    - Project location and right of way
    - Description of MR strategy
    - Project Layout
    - Layout of all repair work required prior to MR
    - Design data (layer geometry and features (shoulders, slopes, medians, curbs etc.)
    - Estimates of materials required for MR

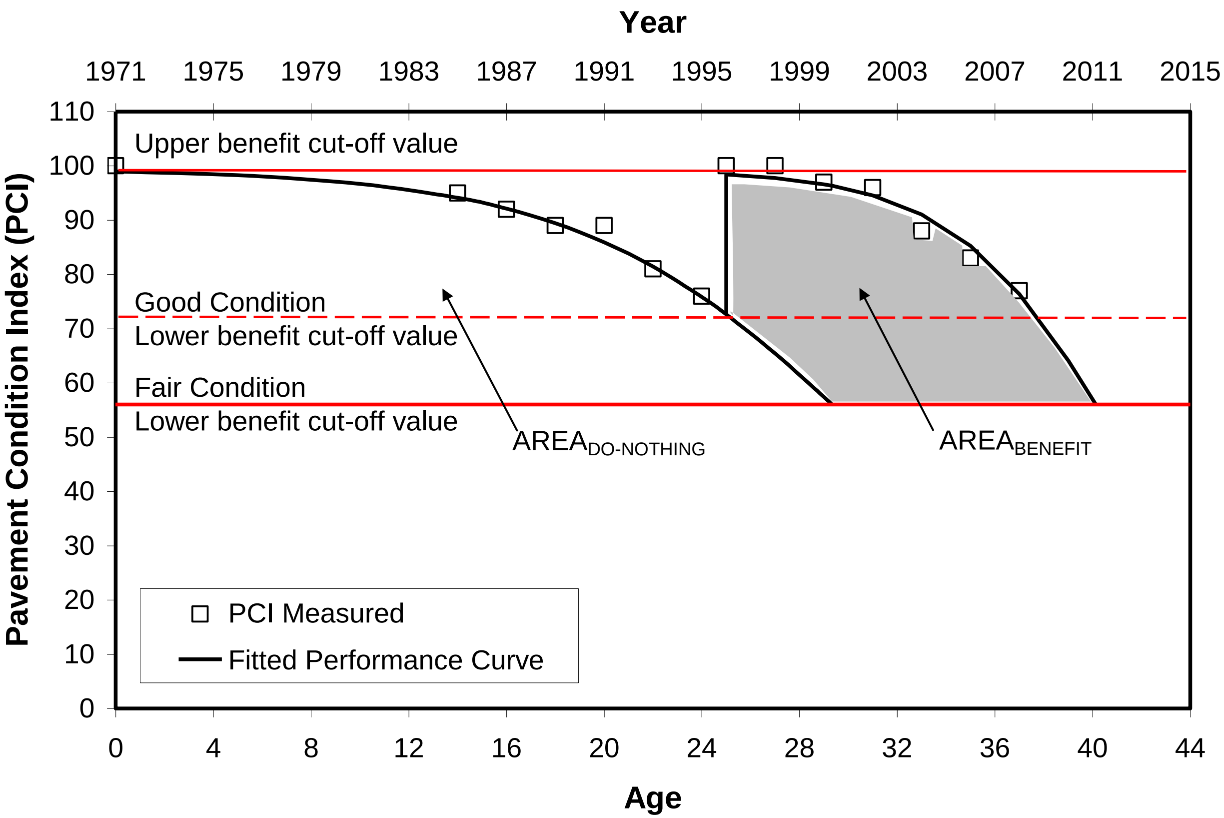
Identification of Feasible MR Strategies

- **Phase III Step 7 – Life-Cycle Cost Analysis**
  - **Objective:**
    - compare cost versus benefit (service life) of the candidate MR strategies
  - **Highway Agency Costs:**
    - Initial rehabilitation construction
    - Future Maintenance and rehabilitation
    - Future salvage value
  - **Highway User Costs:**
    - Traffic delay
    - Vehicle operation
    - Accident and discomfort

Identification of Feasible MR Strategies

⇨ Phase III Step 7 – Life-Cycle Cost Analysis

⇨ Benefit/Cost Ratio Concept



$$EUAC_i = \sum PW\$_i * \left[\frac{d(1+d)^{p_i}}{(1+d)^{p_i} - 1} \right]$$

$$\%BENEFIT = \frac{AREA_{BENEFIT}}{AREA_{DO-NOTHING}} * 100\%$$

$$B/C = \frac{\%BENEFIT}{EUAC}$$

Identification of Feasible MR Strategies

- **Phase III Step 8 – Determine Non-monetary factors that influence rehabilitation**
  - Overall policies for pavement management of a network
  - Future rehabilitation options and needs
  - Traffic volume
  - Future maintenance requirements
  - Traffic control during MR construction (safety and congestion)
  - Duration of MR construction
  - Potential foundation and climate problems
  - Performance of similar pavements in the area
  - Material availability and contractor capabilities
  - Incorporation of experimental features
  - Stimulation of competition
  - Municipal/local preference and industry recognition

Identification of Feasible MR Strategies

⇨ Phase IV Step 9 – Determine Preferred MR Strategy

⇨ Considerations:

- Cost-effectiveness
- Addressing the specific problems of the existing pavement
- Prevention of future problems
- Meeting all existing constraints of the project

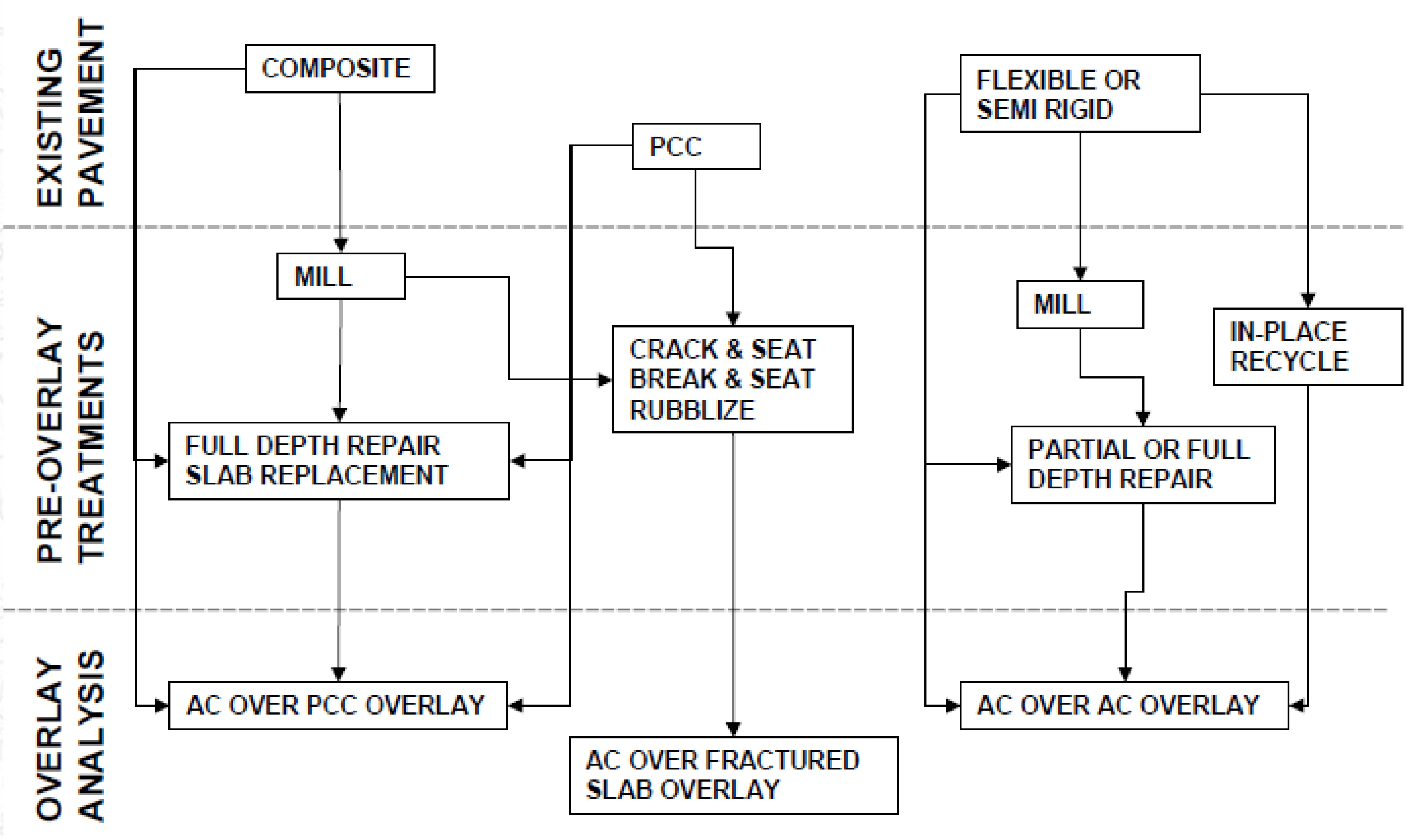
Example

	Criteria							
	Initial Cost	Duration of Construction	Service Life	Rehabilitation and Maintenance Effort	Rideability and Traffic Orientation	Proven Design in State Climate	Total Cost	Rank
Relative Importance	20%	20%	25%	15%	5 %	15%	100%	
Alternative 1	60 / 12	60 / 12	100 / 25	80 / 12	90 / 4.5	100 / 15	80.5	1
Alternative 1a	60 / 12	60 / 12	100 / 25	80 / 12	90 / 4.5	100 / 15	80.5	1
Alternative 2	60 / 12	60 / 12	70 / 17.5	50 / 7.5	60 / 3	40 / 6	58	5
Alternative 2a	60 / 12	60 / 12	70 / 17.5	50 / 7.5	60 / 3	40 / 6	58	5
Alternative 3	60 / 12	40 / 8	100 / 25	80 / 12	100 / 5	90 / 13.5	75.5	2
Alternative 4	60 / 12	80 / 6	40 / 10	20 / 3	40 / 2	20 / 3	44	8
Alternative 5	40 / 8	60 / 12	40 / 10	50 / 7.5	50 / 2.5	30 / 4.5	44.5	7
Alternative 6	70 / 14	80 / 16	60 / 12.5	50 / 7.5	80 / 4	40 / 6	60	4
Alternative 7	100 / 20	100 / 20	20 / 5	20 / 3	40 / 2	40 / 6	56	6
Alternative 8	30 / 20	60 / 12	100 / 25	100 / 15	100 / 5	30 / 4.5	67.5	3

HMA Overlay Rehabilitation Design Process

Zofka, Fall 2010

Overview of HMA Overlay Design



Inputs for HMA Rehabilitation Design

- General information
- Site/project identification
- Analysis parameters
- Traffic
- Climate
- Drainage and surface properties
- Pavement structure
  - Overlay structure
  - Existing pavement
  - Drainage and surface properties

Inputs for HMA Rehabilitation Design

General Information

Input Variable	Description/Source of Information
Project name and description	• User input
Design life	• Expected rehabilitation design life
Existing pavement construction date	• Month in which existing pavement was constructed • Year in which existing pavement was constructed
Pavement overlay construction date	• Month in which HMA overlay construction is expected • Year in which HMA overlay construction is expected
Traffic opening date	• Expected month in which rehabilitated pavement will be opened to traffic • Expected year in which rehabilitated pavement will be opened to traffic
Asphalt Concrete Overlay	• HMA overlay of existing HMA surfaced pavement o Includes conventional, deep-strength, full-depth, and semi-rigid pavements. • HMA overlay of fractured PCC slabs o Includes HMA overlays of fractured JPCP and CRCP. • HMA overlay of existing intact PCC pavement o Includes HMA overlays of intact JPCP and CRCP.

Inputs for HMA Rehabilitation Design

Analysis Parameters

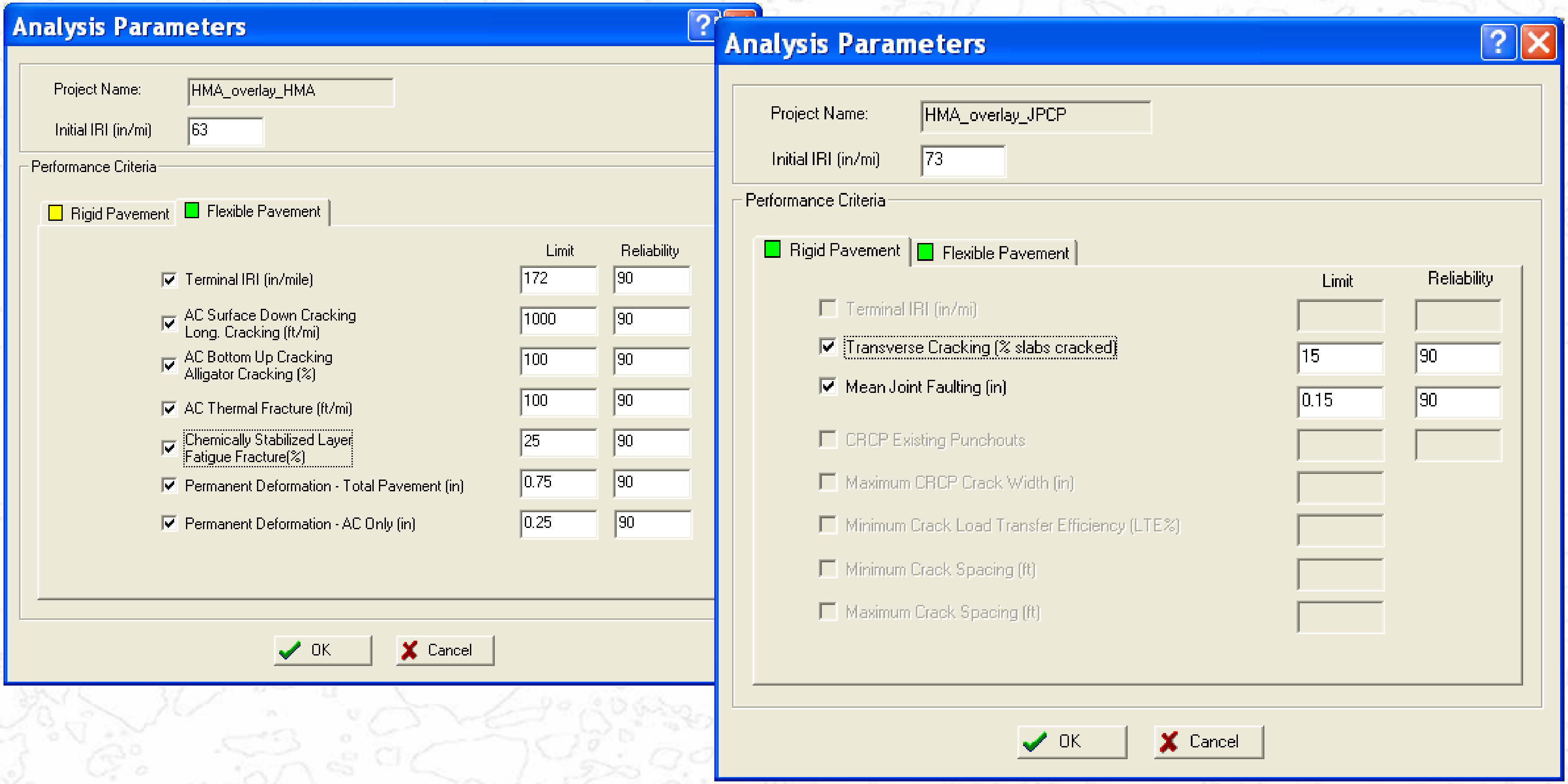
Distress	HMA over HMA	HMA over Fractured PCC	HMA over Intact PCC
Terminal Smoothness/IRI	Yes	Yes	Yes
Longitudinal Cracking	Yes	Yes	Yes
Bottom-up Fatigue (Alligator) Cracking[1]	Yes	Yes	Yes
Thermal Cracking	Yes	Yes	Yes, Unless Bonded to JPCP or CRCP
Rutting in HMA Layers	Yes	Yes	Yes
Rutting in Unbound Layers	Yes	Yes	When Used in Overlay Layers
CSM[1] Modulus Reduction	Yes	NA	NA
CSM Fatigue Cracking[2]	Yes	NA	NA
PCC: CRCP Punchouts	NA	NA	CRCP only
PCC: JPCP Transverse Cracking	NA	NA	JPCP only
Reflection Cracking	Yes	NA	Yes

[1] Alligator cracking is not expected to be a major distress type in these pavement systems unless in some special cases where the HMA overlay debonds with the PCC or when relatively thicker overlays are placed.

[2] CSM = Chemically stabilized material (e.g., cement-treated, lime flyash, soil cement bases or subbases). Note that the fatigue cracking prediction procedures for CSM layers are uncalibrated.

Inputs for HMA Rehabilitation Design

Analysis Parameters



Rehabilitation Prediction Models

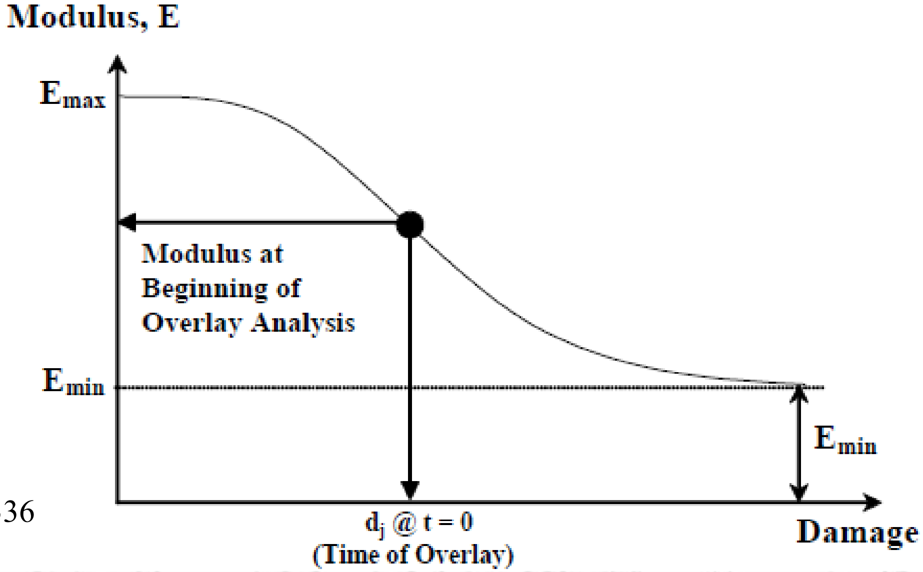
CSM Modulus Reduction

- The CSM modulus is reduced due to traffic induced damage during the overlay period (for existing HMA only).

$$E = E_{min} + \frac{(E_{max} - E_{min})}{1 + e^{a+b(d)}}$$

Where:

E = Modulus of chemically stabilized material, psi.
E_{min} = Minimum modulus, psi.
E_{max} = Maximum modulus, psi.
a and b = Fitting parameters.
d = Fatigue damage in chemically stabilized material.



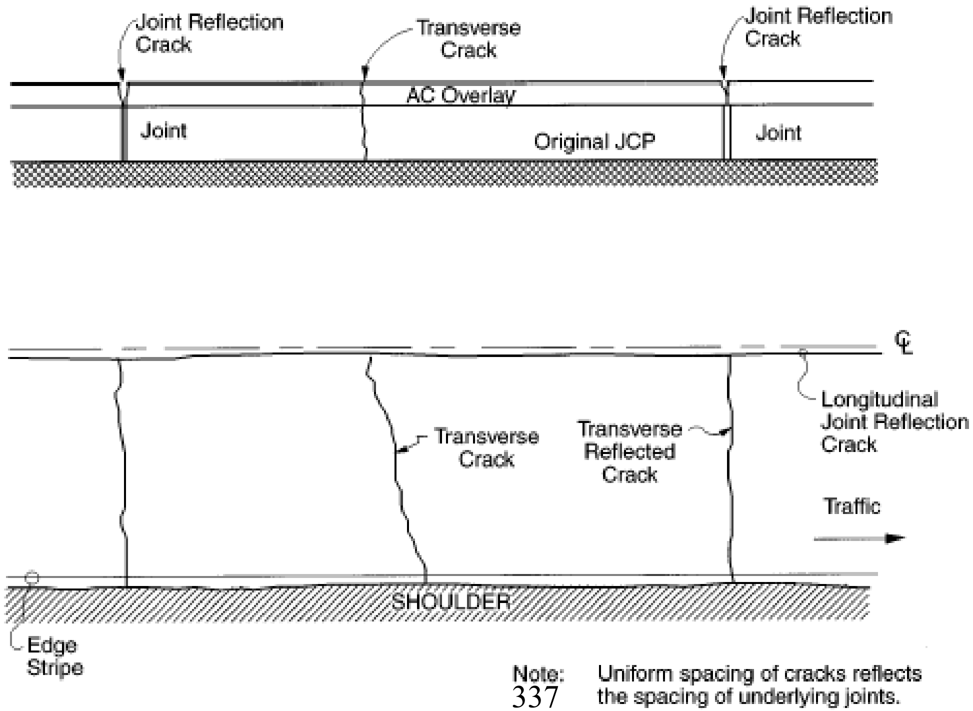
Parameter	Cement Treated Material[1]
E_{max}, psi	PART 2, Chapter 2
E_{min}, psi	50,000
A	-4
B	14

[1] These values pertain to cement treated base or subbase materials.

Rehabilitation Prediction Models

Reflection Cracking

- RC is a major distress in HMA on HMA and HMA on PCC pavements
- RC propagates from bottom up due to:
  - Load-related movements (*f(overlay h, exist. h, E, and LTE)*)
  - Temperature-induced movements (*f(dT, CTE and crack spacing)*)



Rehabilitation Prediction Models

M-EPDG Reflection Cracking Model

Distress Model Calibration Settings - Flexible Rehabilitation

Subgrade Rutting | AC Cracking | CSM Cracking | IRI

AC Fatigue | Reflective Cracking | AC Rutting | Thermal Fracture | CSM Fatigue

$$RC = \frac{100}{1 + e^{c.a + d.b.t}}$$

RC = Percent of cracks reflected, %

t = Time, years

h_{ac} = Overlay thickness(in)

a = $3.5 + 0.75(Heff)$

b = $-0.688584 - 3.37302(Heff)^{-0.915469}$

c = 1

d = Calibration parameter (user input)

	AC over AC	AC over Rigid, Good Load Transfer	AC over Rigid, Poor Load Transfer
Heff	h_{ac}	$h_{ac} - 1$	$h_{ac} - 3$

Heff	Recommended Calibration Parameter - d	
	Delay Cracking by 2 years	Accelerate Cracking by 2 years
< 4"	0.6	3
4 – 6"	0.7	1.7
> 6"	0.8	1.4

Reflective cracking c: 1.000

Reflective Cracking d: 1.000

OK Cancel

Rehabilitation Prediction Models

Analysis of Fatigue in Existing HMA Layers after Overlay

- Existing layer undergo additional fatigue damage even after overlay

$$D_m = \sum_{i=1}^{m} \Delta D_i$$

Where:

D_m = Damage for month m.

ΔD_i = Increment of damage in month i.

$$CA_m = \frac{100}{1 + e^{6 - 6 \times D_m}}$$

$$TRA_m = \sum_{i=1}^{m} RC_{m-i} \times \Delta CA_i$$

Where:

TRA = Total reflected area for month m.

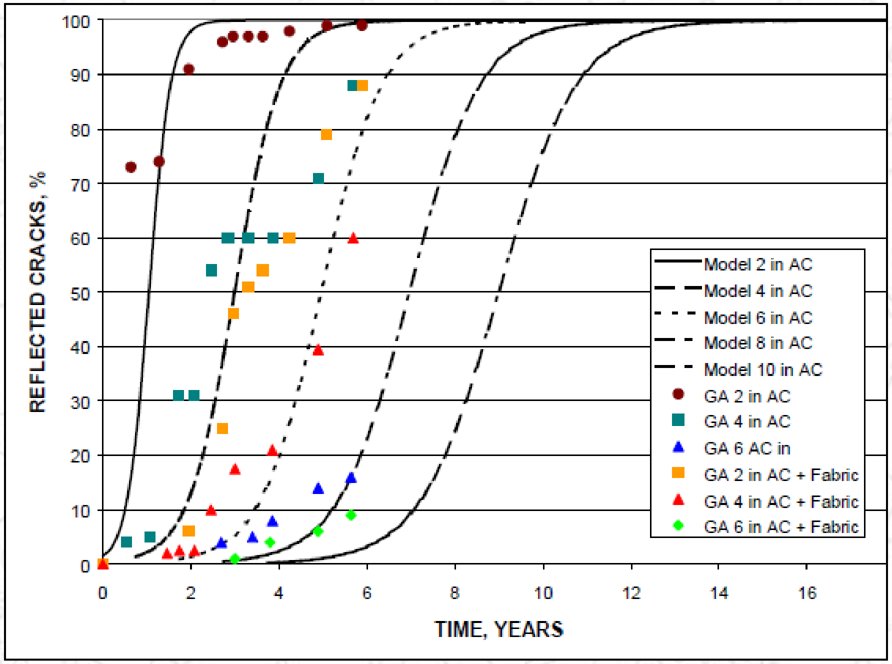
RC_{m-i} = Percent cracking reflected for Age = $m - i$; (Age in years).

ΔCA_i = Increment of fatigue cracking for month i.

Rehabilitation Prediction Models

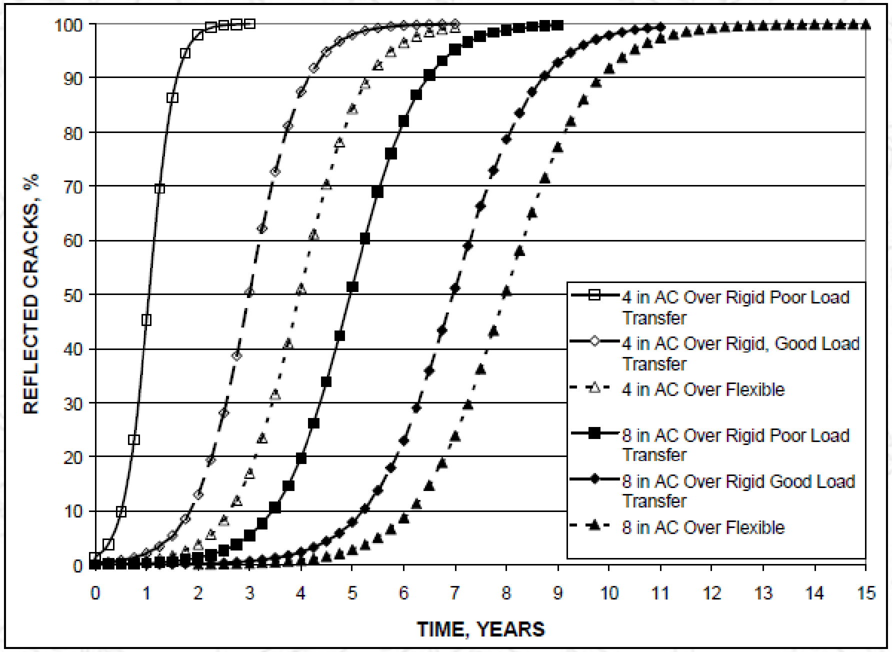
Analysis of Fatigue in Existing JPCP

⇨ Use the same cracking model as new JPCP



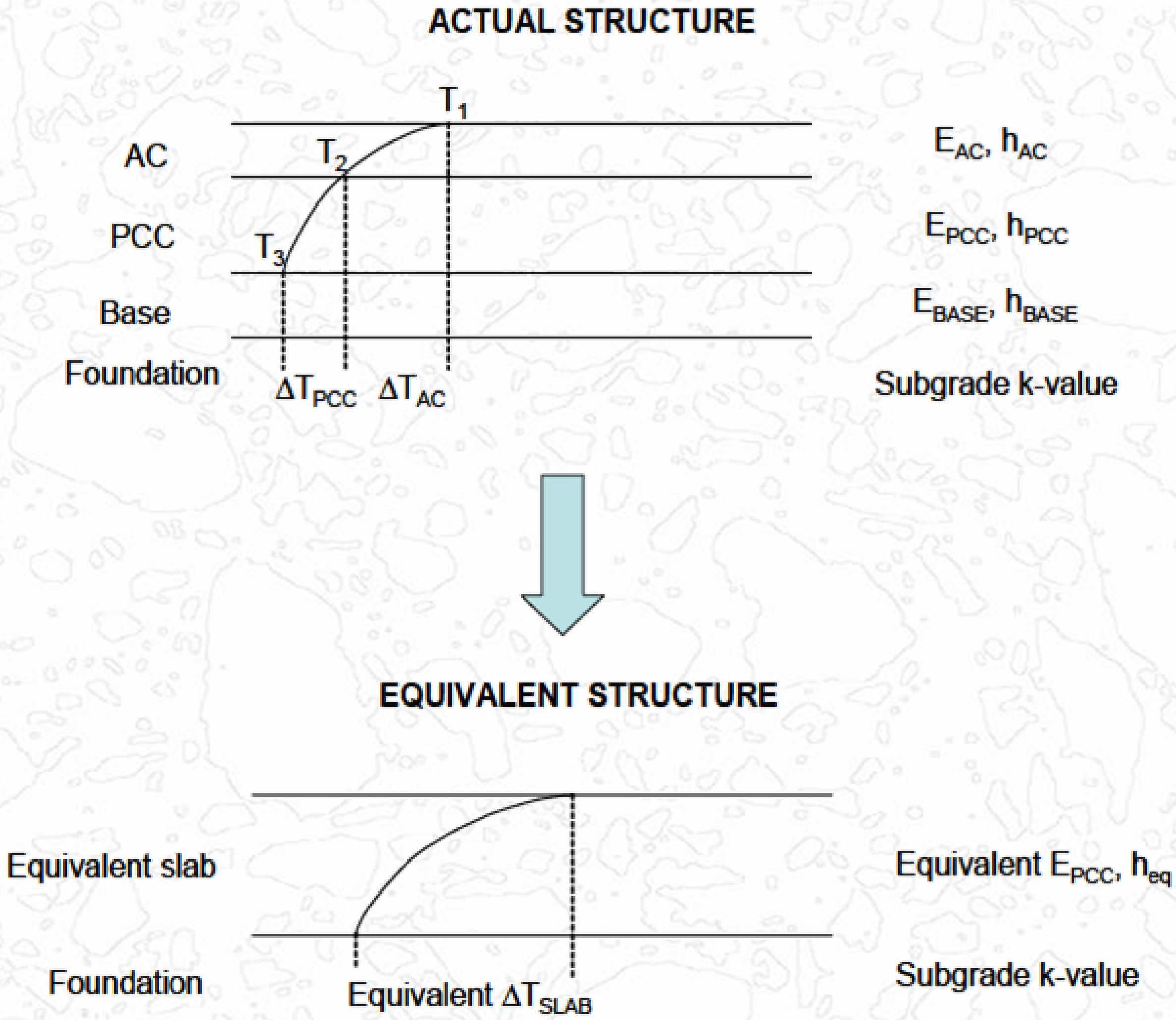
Effect of h_{HMA} on RC

Effect of h_{HMA}, pavement type and LTE on RC



Rehabilitation Prediction Models

Equivalency Principle in HMA on JPCP Analysis



Rehabilitation Prediction Models

Equivalency Principle in HMA on JPCP Analysis

- Assumptions
  - Equality of temperature graqdient moments between actual and equivalent structure
  - Equality of deflection basin at the same axle configuration and temperature loading
- Modified properties
  - Layer thickness
  - Layer modulus
  - Temperature gradients

Rehabilitation Prediction Models

Smoothness Prediction

HMA over HMA

$$IRI = IRI_0 + 0.011505(t) + 0.0035986(FC) + 3.4300573\left(\frac{1}{(TC_S)_{MH}}\right) + 0.000723(LC_S)_{MH} + 0.0112407(P)_{MH} + 9.04244(PH) \quad (3.6.6)$$

Where:

IRI_0	= Initial IRI at the time of HMA overlay placement, m/km.
t	= Time after overlay placement, years.
FC	= Total area fatigue cracking, % of wheel path area.
$(TC_S)_{MH}$	= Average spacing of medium and high severity transverse cracks, m.
LC_S	= Medium and high severity sealed longitudinal cracks in the wheel path, m/km.
$(P)_{MH}$	= Area of medium and high severity patches, % of total lane area.
(PH)	= Pot holes, % of total lane area.

Rehabilitation Prediction Models

Smoothness Prediction

HMA over PCC

$$IRI = IRI_0 + 0.0082627(t) + 0.0221832(RD) + 1.33041\left(\frac{1}{(TC_S)_{MH}}\right) \quad (3.6.7)$$

Where:

RD = Average rut depth, mm.

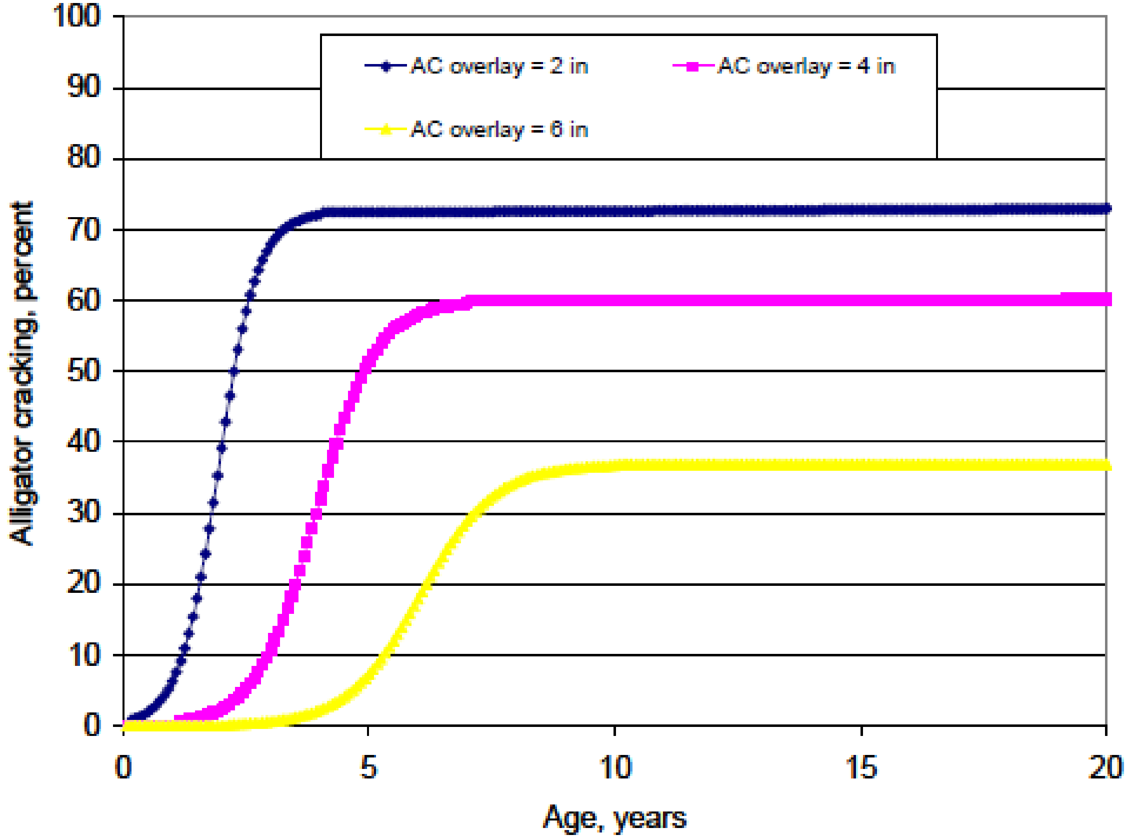
All other variables as described previously.

Pre-Overlay Treatments

HMA-on-HMA Overlay

Distress	Severity	Pre-Overlay Treatment
Alligator Cracking	Medium to High	Full-Depth Repair Cold Milling
Longitudinal Cracking	Medium to High	Cold Milling Partial-Depth Repair (for joints)
Transverse Cracking	Low to Medium High	Cold Milling Full-Depth Repair or Fabric
Rutting	Low to Medium High	Cold Milling Overlay is not recommended

Effect of Design Factors on HMA-on-HMA Performance



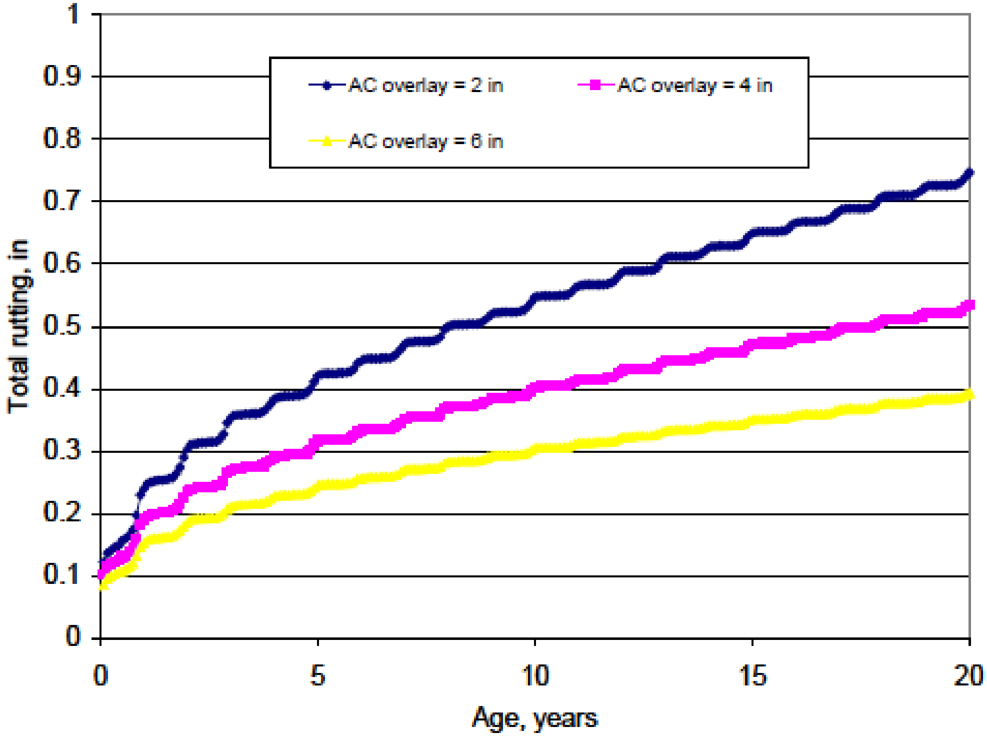
<u>Site Factors:</u>
Climate: Midwest
Traffic:18 million trucks

<u>Design Features:</u>
Existing pavement: 7.5 in HMAC, 12 in granular base, A-1-b subgrade
HMA Overlay: 2, 4, and 6 inches

<u>Design Life:</u>
20 years

Effect of HMA overlay thickness on alligator cracking

Effect of Design Factors on HMA-on-HMA Performance



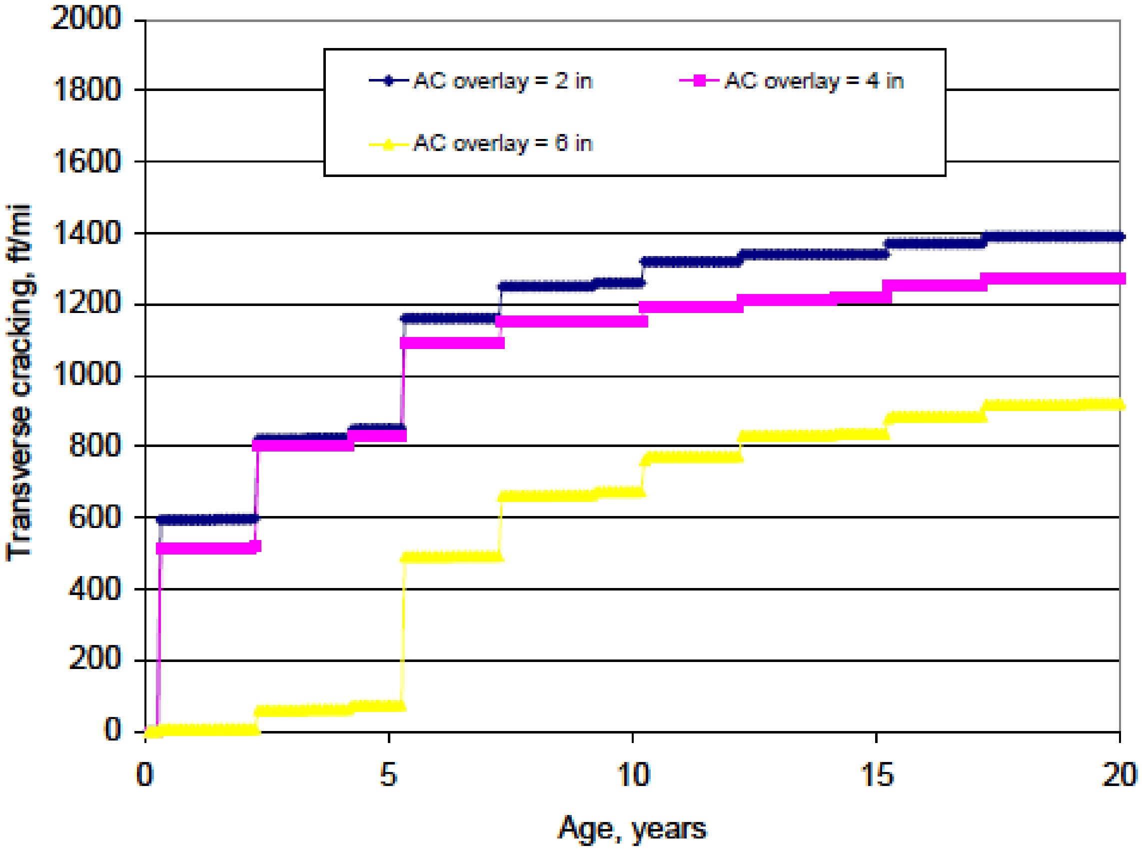
Site Factors:
Climate: Midwest
Traffic:18 million trucks

Design Features:
Existing pavement: 7.5 in HMAC, 12 in granular base, A-1-b subgrade
HMA Overlay: 2, 4, and 6 inches

Design Life:
20 years

Effect of HMA overlay thickness on total rutting

Effect of Design Factors on HMA-on-HMA Performance



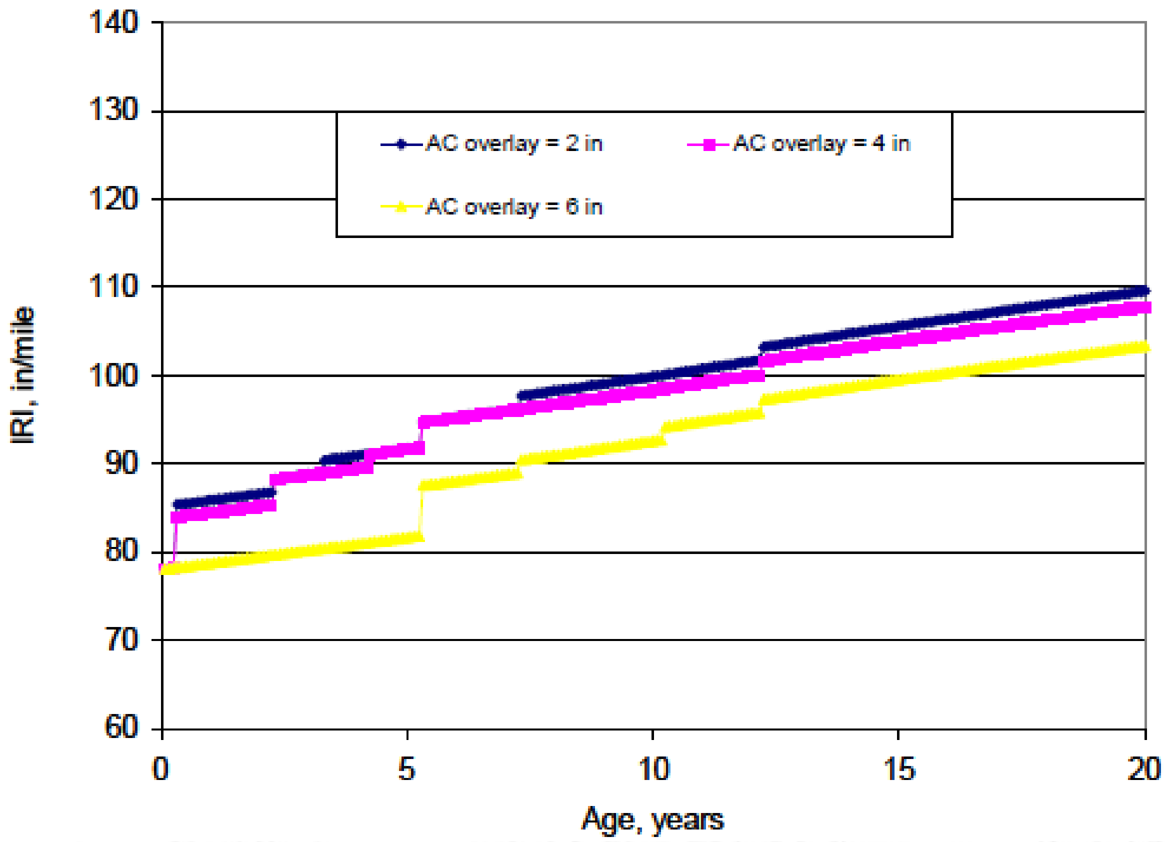
Site Factors:
Climate: Midwest
Traffic:18 million trucks

Design Features:
Existing pavement: 7.5 in HMAC, 12 in granular base, A-1-b subgrade
HMA Overlay: 2, 4, and 6 inches

Design Life:
20 years

Effect of HMA overlay thickness on transverse cracking

Effect of Design Factors on HMA-on-HMA Performance



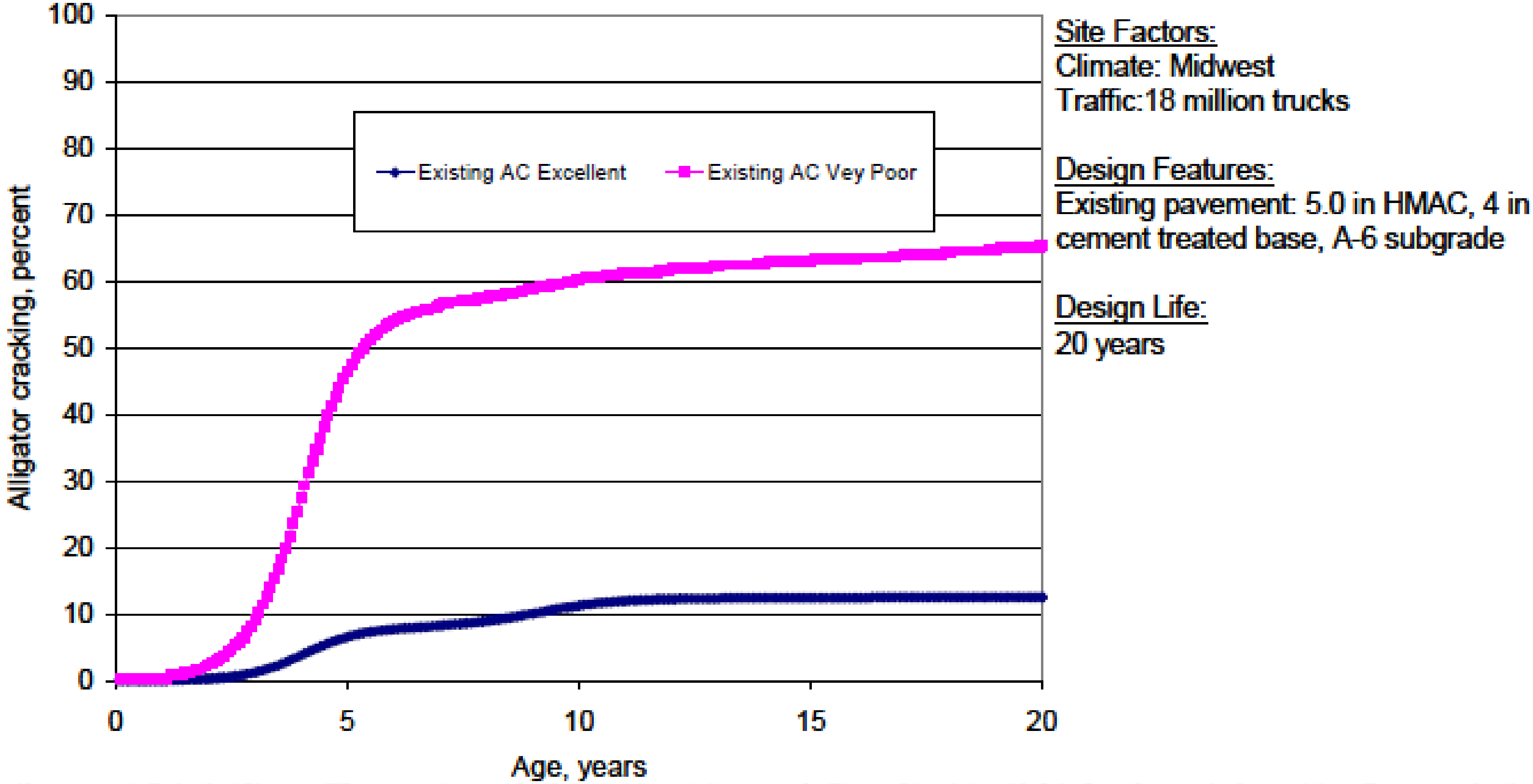
<u>Site Factors:</u>
Climate: Midwest
Traffic:18 million trucks

<u>Design Features:</u>
Existing pavement: 7.5 in HMAC, 12 in granular base, A-1-b subgrade
HMA Overlay: 2, 4, and 6 inches

<u>Design Life:</u>
20 years

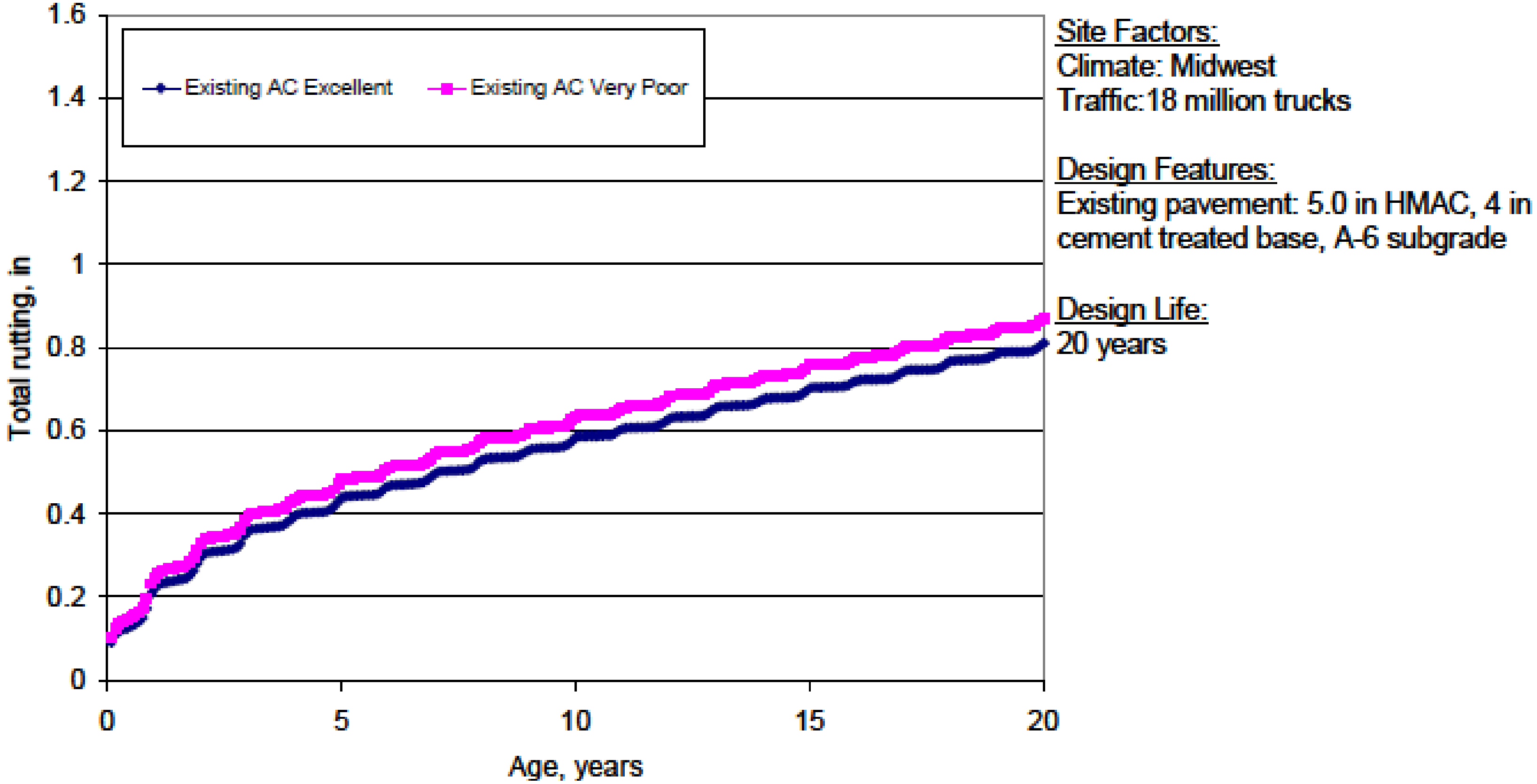
Effect of HMA overlay thickness on IRI

Effect of Design Factors on HMA-on-HMA Performance



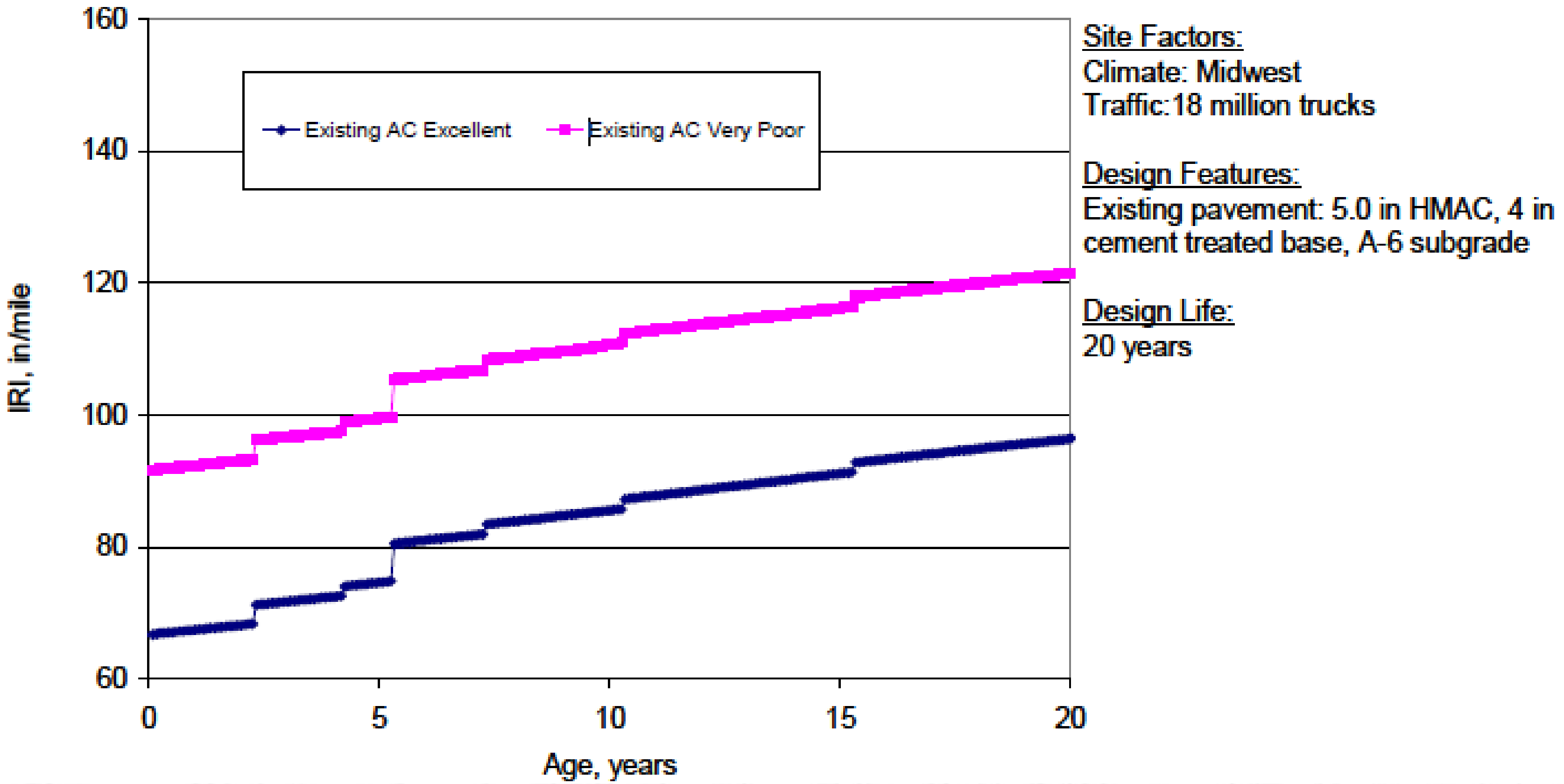
Effect of existing pavement condition on alligator cracking

Effect of Design Factors on HMA-on-HMA Performance



Effect of existing pavement condition on total rutting

Effect of Design Factors on HMA-on-HMA Performance



Effect of existing pavement condition on IRI

Pre-Overlay Treatments

HMA-on-PCC Overlay

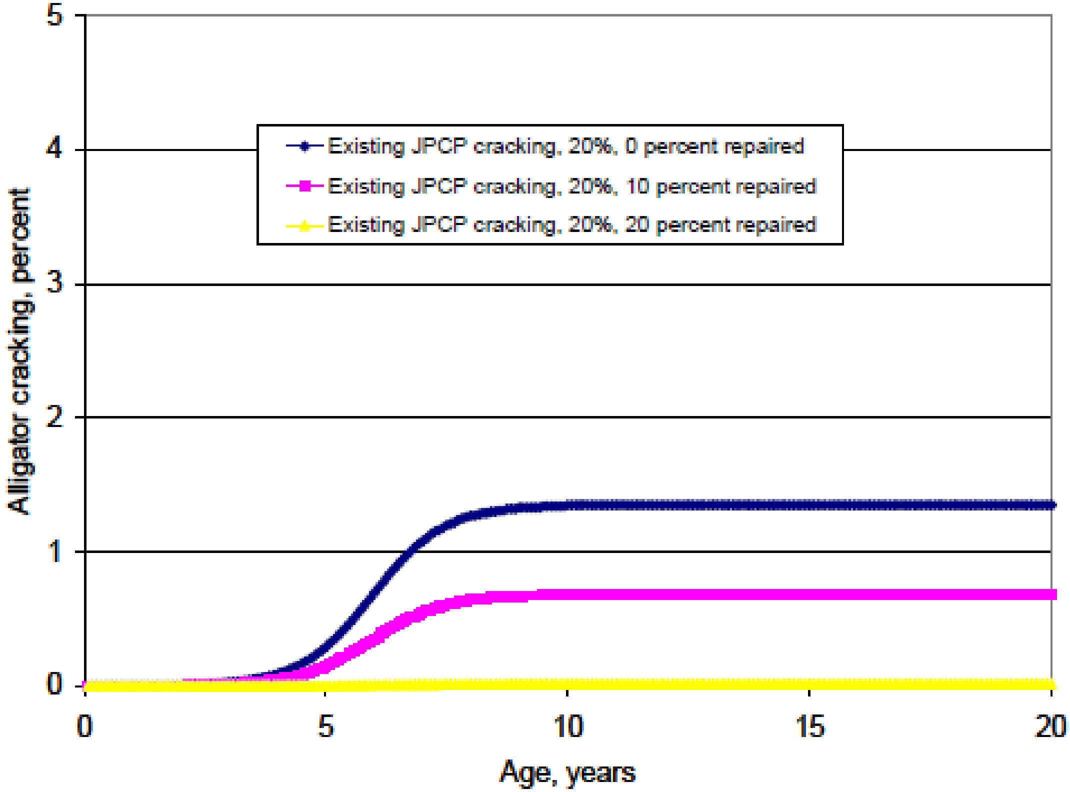
Distress	Severity	Pre-Overlay Treatment
Cracking, heaves, spalling, punchouts	Medium to High	Full-Depth PCC Repair (dowelled or tied)
Faulting and Pumping	Medium to High	Installation of edge drains, Maintenance of existing drains, Other drainage improvements Clean-up of incompressibles HMA leveling course

Pre-Overlay Treatments

HMA-on-PCC Overlay

- Reflection Crack Control:
  - Sawing and sealing joints in HMA Overlay
  - Increasing HMA Overlay thickness
  - Granular Interlayers
  - Fabric treatments and Stress Absorbing Membrane Interlayers (SAMIs)

Effect of Design Factors on HMA-on-JPCP Performance



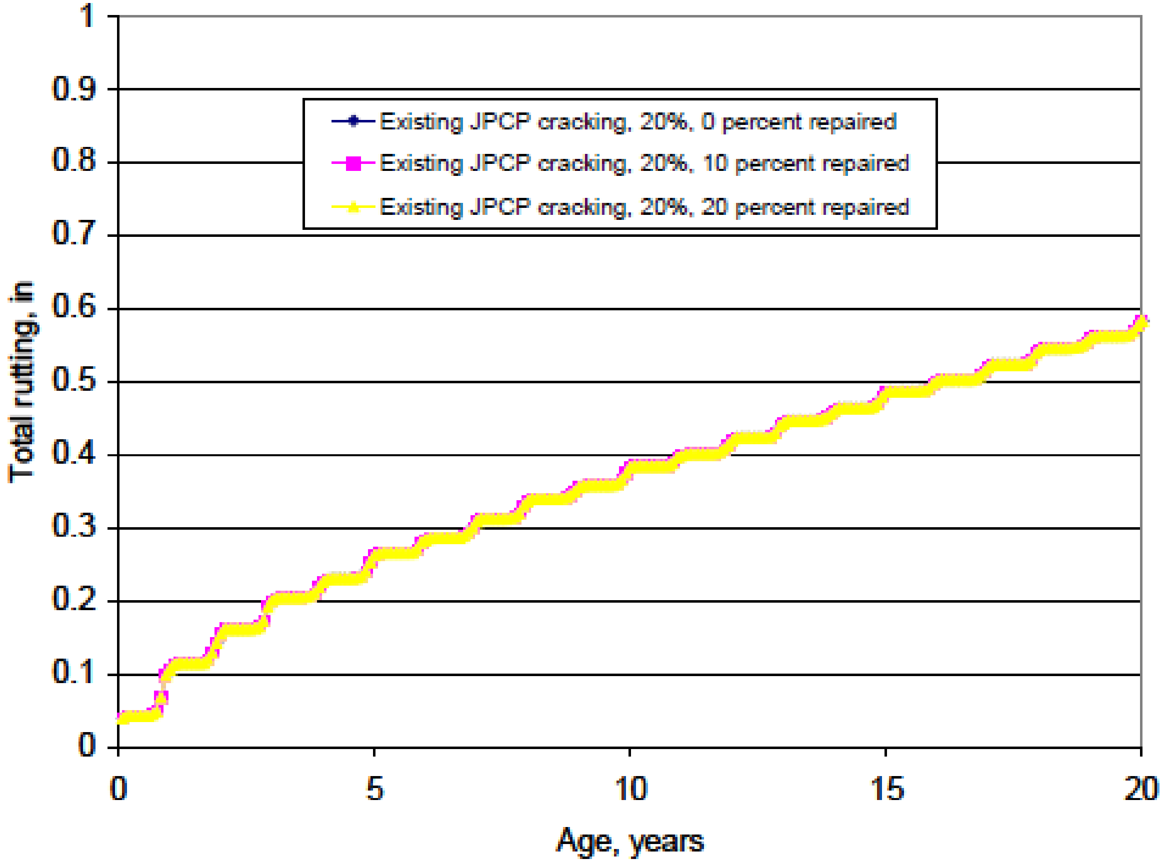
Site Factors:
Climate: Midwest
Traffic:18 million trucks

Design Features:
Existing pavement: 10 in PCC, 6 in granular base, A-6 subgrade

Design Life:
20 years

Effect of existing pavement condition on alligator cracking

Effect of Design Factors on HMA-on-JPCP Performance



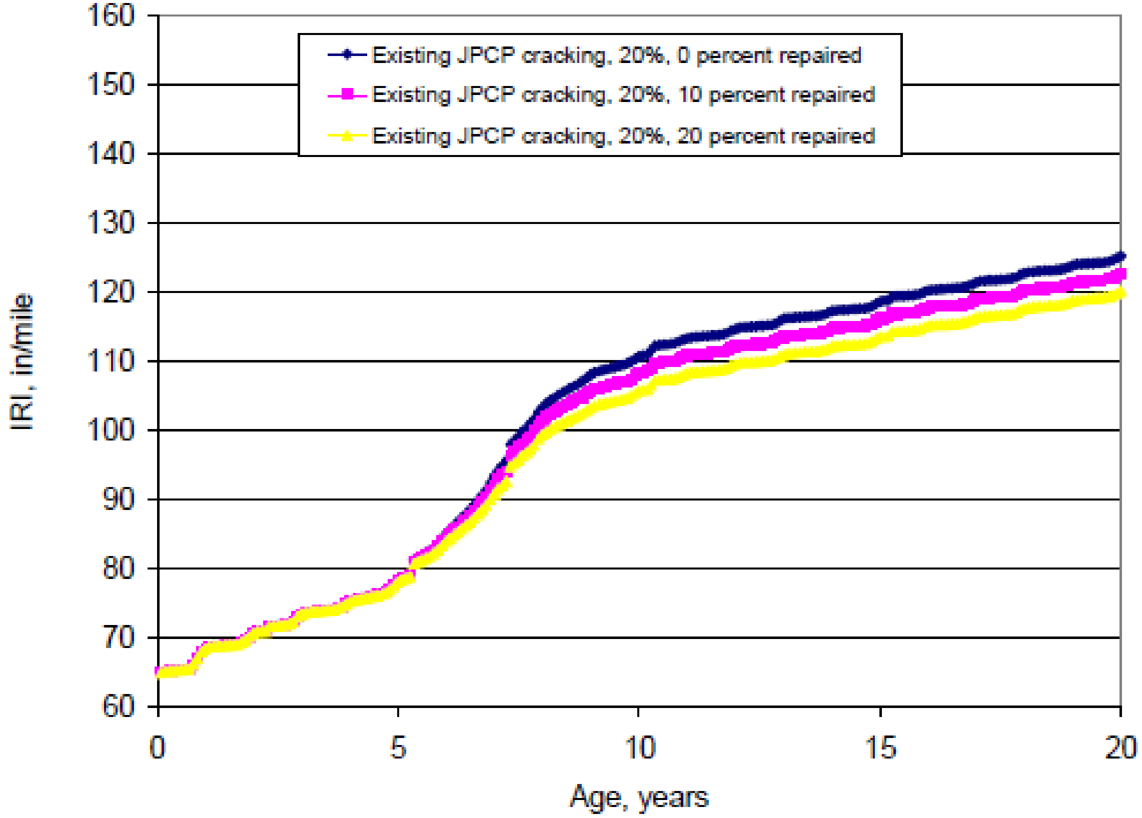
Site Factors:
Climate: Midwest
Traffic:18 million trucks

Design Features:
Existing pavement: 10 in PCC, 6 in granular base, A-6 subgrade

Design Life:
20 years

Effect of existing pavement condition on total rutting

Effect of Design Factors on HMA-on-JPCP Performance



Site Factors:
Climate: Midwest
Traffic:18 million trucks

Design Features:
Existing pavement: 10 in PCC, 6 in granular base, A-6 subgrade

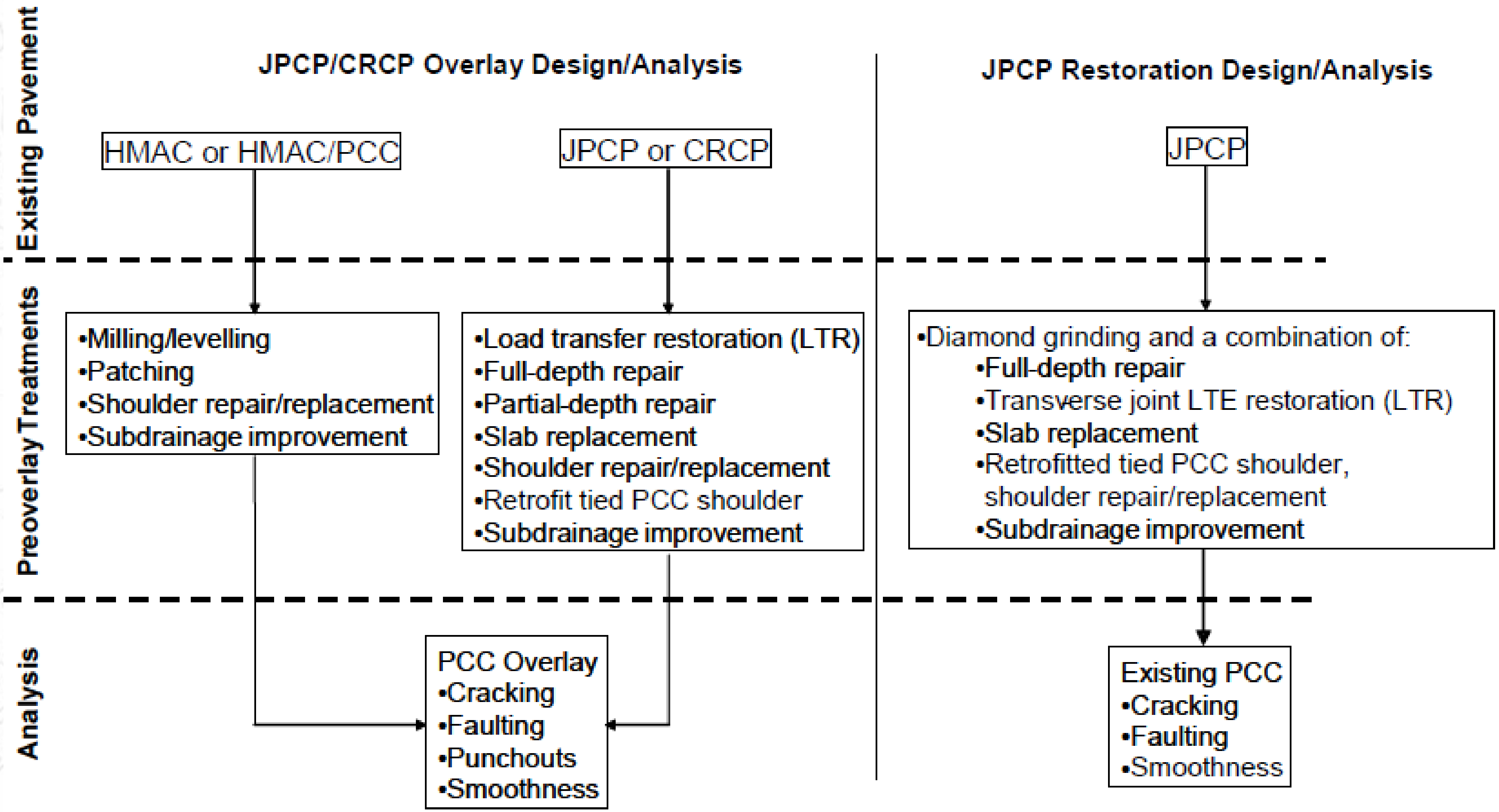
Design Life:
20 years

Effect of existing pavement condition on IRI

PCC Overlay Rehabilitation Design Process

Zofka, Fall 2010

Overview of PCC Overlay Design



JPCP Restoration Strategies

Distress	Repair Treatments	Preventive Treatments
Jointed concrete pavement pumping (and low joint load transfer efficiency)	—	• Reseal joints • Restore joint load transfer • Subdrainage • Edge support (tied PCC shoulder)
Jointed concrete pavement joint faulting	Diamond grinding Structural overlay	• Reseal joints • Restore load transfer • Subdrainage
Jointed concrete pavement slab cracking	Full-depth PCC repair Slab replacement Replace/recycle lane	• Retrofit tied PCC shoulder • Restore load transfer • Bonded and unbonded PCC overlays • Thick HMA overlays
Jointed concrete pavement joint or crack spalling	Full-depth PCC repair Partial-depth repair	• Clean and reseal joints
PCC disintegration (e.g., D-cracking and alkali-silica reaction [ASR])	Full-depth repair	• Thick hot mix AC overlay • Unbonded PCC overlay

Inputs for PCC Rehabilitation Design

- General information
- Site/project identification
- Analysis parameters
- Traffic
- Climate
- Pavement structure
- Design features
  - Drainage and surface properties
  - Layer definition and material properties
- Existing Pavement Condition

Inputs for HMA Rehabilitation Design

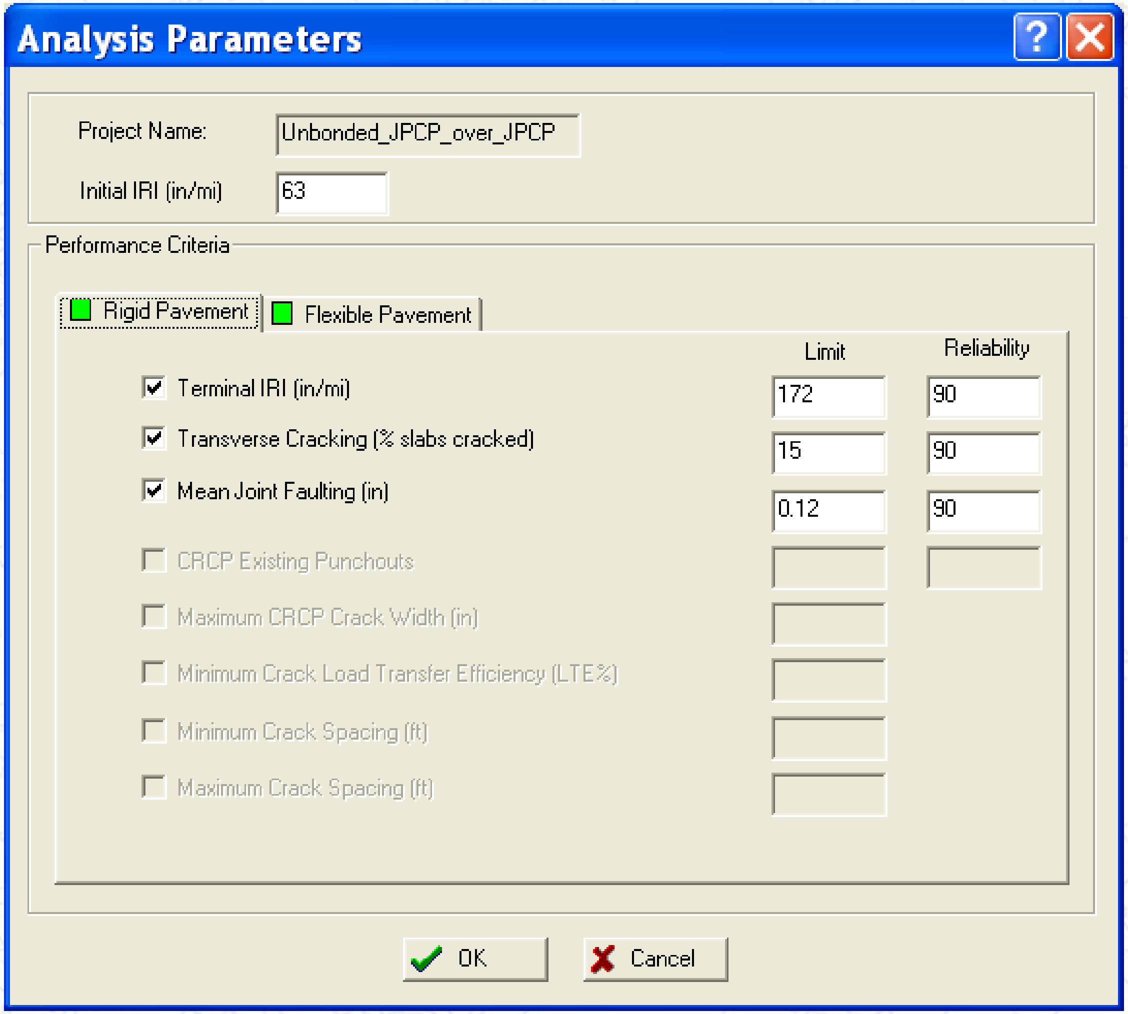
General Information

Input Variable	Description/Source of Information
Project name and description	• User input
Design life	• Expected rehabilitation design life
Existing pavement construction date	• Month in which existing pavement was constructed • Year in which existing pavement was constructed
Pavement overlay construction date[1]	• Month in which PCC overlay construction is expected • Year in which PCC overlay construction is expected
Pavement restoration date[2]	• Month in which existing PCC restoration is expected • Year in which existing PCC is restoration is expected
Traffic opening date	• Expected month in which rehabilitated pavement will be opened to traffic • Expected year in which rehabilitated pavement will be opened to traffic
Type of rehabilitation strategy	• JPCP rehabilitation without overlays 1. Existing JPCP subjected to CPR[3] • Rehabilitation with JPCP or CRCP overlays 1. Existing JPCP, JRCP, CRCP, or composite overlaid with unbonded JPCP overlay 2. Existing JPCP, JRCP, CRCP, or composite overlaid with unbonded CRCP overlay 3. Existing JPCP and CRCP overlaid with bonded PCC overlay 4. Existing flexible pavement overlaid with JPCP overlay 5. Existing flexible pavement overlaid with CRCP overlay

1. Applicable to PCC overlays only.
2. Applicable to existing JPCP subjected to CPR only.
3. CPR is defined as diamond grinding with a combination of CPR treatments such as full-depth patching, load transfer restoration, shoulder replacement, and lane widening.

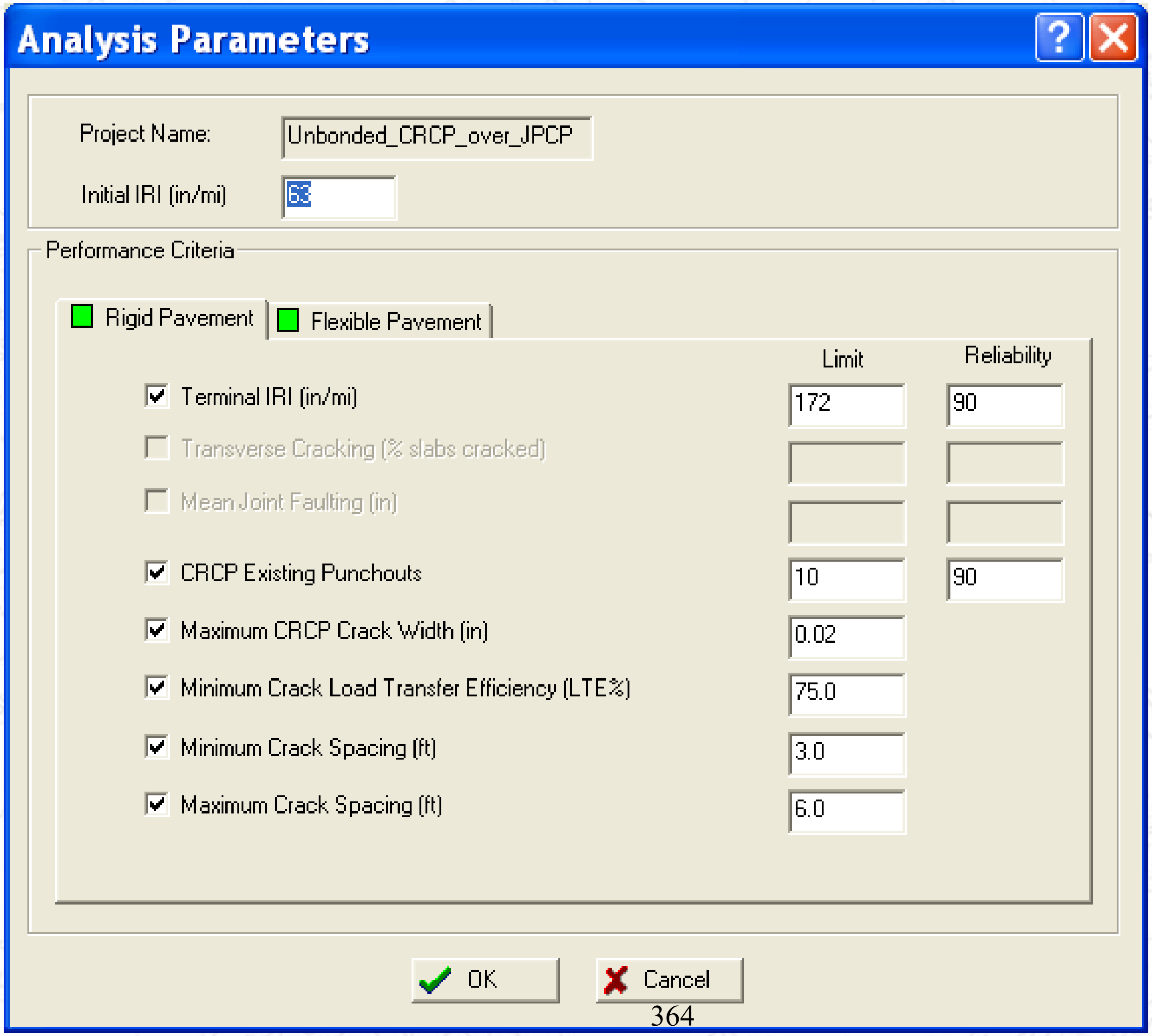
Inputs for HMA Rehabilitation Design

Analysis Parameters – JPCP Overlay



Inputs for HMA Rehabilitation Design

Analysis Parameters – CRCP Overlay



Inputs for HMA Rehabilitation Design

Analysis Parameters – Pavement Condition

Existing Pavement Type	Structural Condition			
	Good	Moderate	Severe	Rubblized
JPCP (percent slabs cracked)[1]	<10	10 to 50	> 50 or crack and seat	Rubblized
JRCP (percent area deteriorated)[2]	< 5	5 to 25	> 25 percent or break and seat	Rubblized
CRCP (percent area deteriorated)[3]	< 3	3 to 10	> 10	Rubblized

[1]Percent slabs cracked with all severities and types of cracks plus any repairs.

[2]Percent area including repairs or patches, deteriorated joints, and deteriorated cracks (deteriorated joints and cracks converted to repair areas).

[3]Percent area includes repairs, patches, and localized failures and punchouts converted to repair areas.

Pre-Overlay Treatments

Unbonded JPCP/CRCP on JPCP

Distress	Pre-Overlay Treatment
Spalling (H-severity)	Remove any loose material If HMA separator layer >=1in, no repair is necessary
Faulting	If HMA separator layer >=1in, no repair is necessary If LTE<50, HMA sep. layer >=1.5 in is needed Fracturing of existing pavement Increase CRCP reinforcement
D-cracking	HMA separator layer >=1in Remove loose pieces Improve drainage Fracture existing slabs
Loss of support	Slab replacement Level settlements with HMA layer Fracture existing slabs

Pre-Overlay Treatments

Unbonded JPCP/CRCP on CRCP

Distress	Pre-Overlay Treatment
Punchouts	Full-depth CRCP repair Repair foundation beyond the distress boundary
Deteriorated Transverse Cracks	Full-depth CRCP patch
Joint Spalling	Full-depth patch

Pre-Overlay Treatments

Bonded PCC on PCC

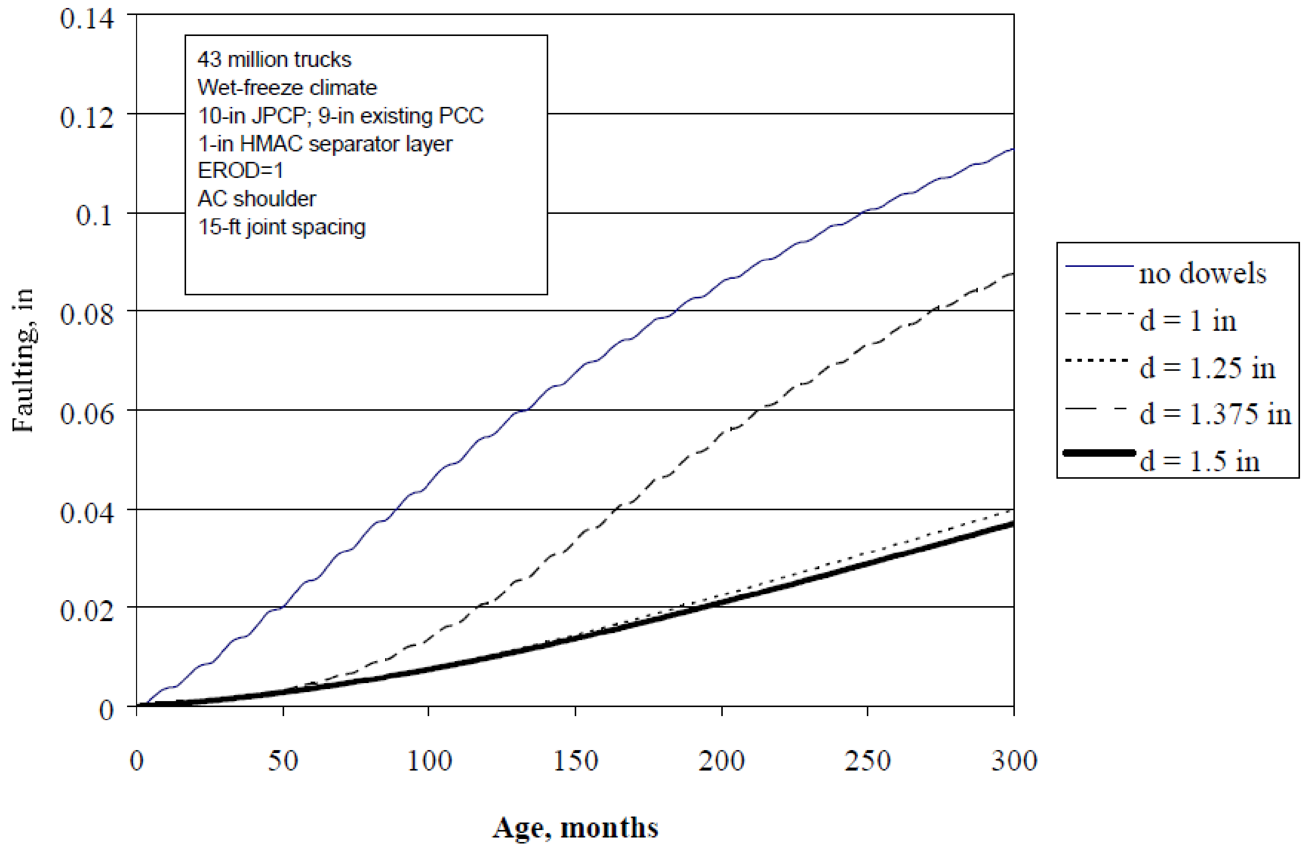
Distress	Critical Severity	Pre-Overlay Treatment
Corner Breaks	Low	Slab stabilization LTE restoration with full-depth repair
Punchouts (CRCP only)	Low	Full-depth reinforced repair
Joint Spalling	Medium	Partial-depth repair Full depth repair (where deterioration extends beyond mid depth)
D-Cracking	Medium	Partial-depth repair Full depth repair (where deterioration extends beyond mid depth)
Transverse cracking	Medium	LTE restoration with full-depth repair Saw joint above repair oint
Longitudinal Cracking	Medium	Cross-stitch crack Place reinforcement bars across crack

Pre-Overlay Treatments

PCC on HMA

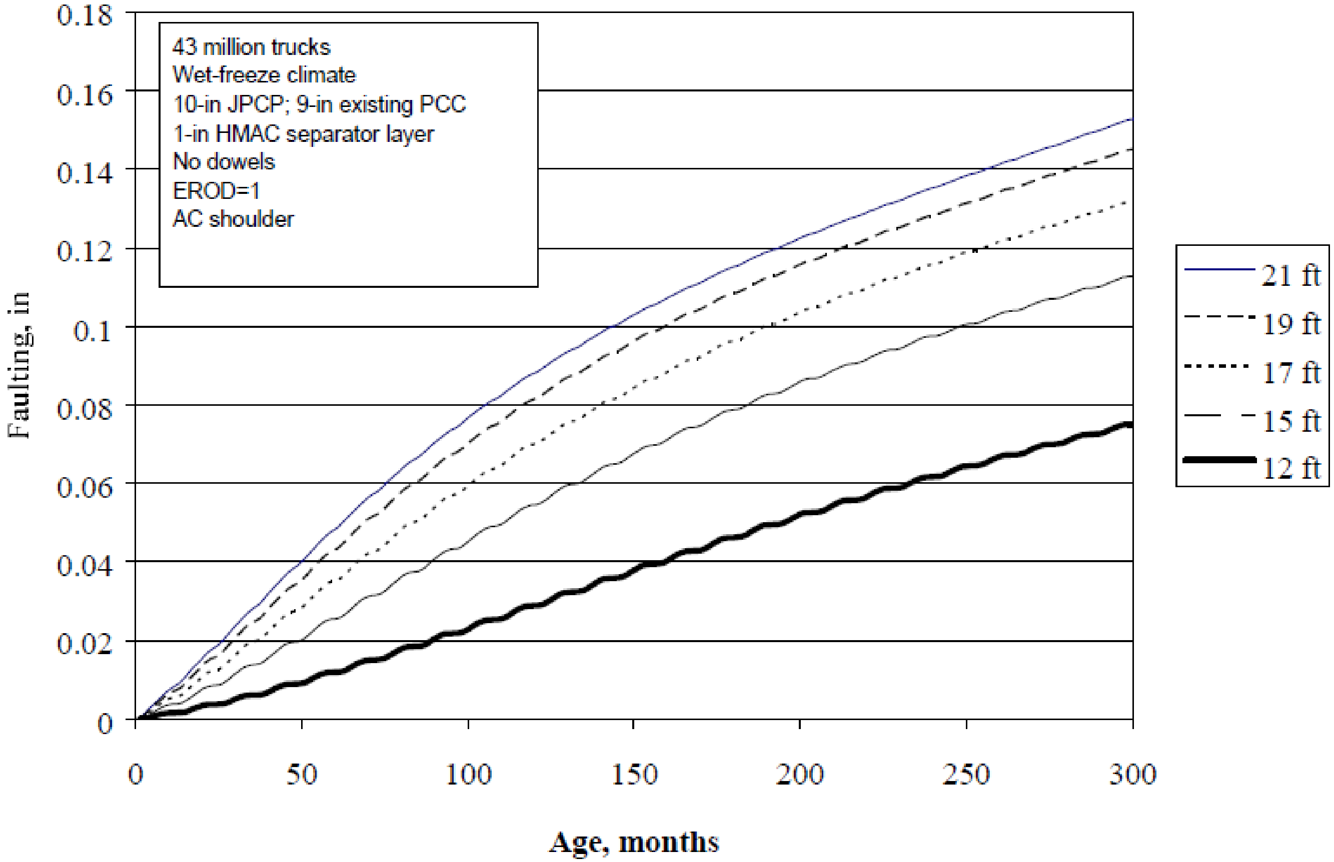
Distress	Critical Severity	Pre-Overlay Treatment
Rutting	Medium(<=1 in)	No milling (direct placement)
Rutting	High (>1in)	Milling Leveling course

Effect of Design Factors on Unbonded JPCP Overlay Performance



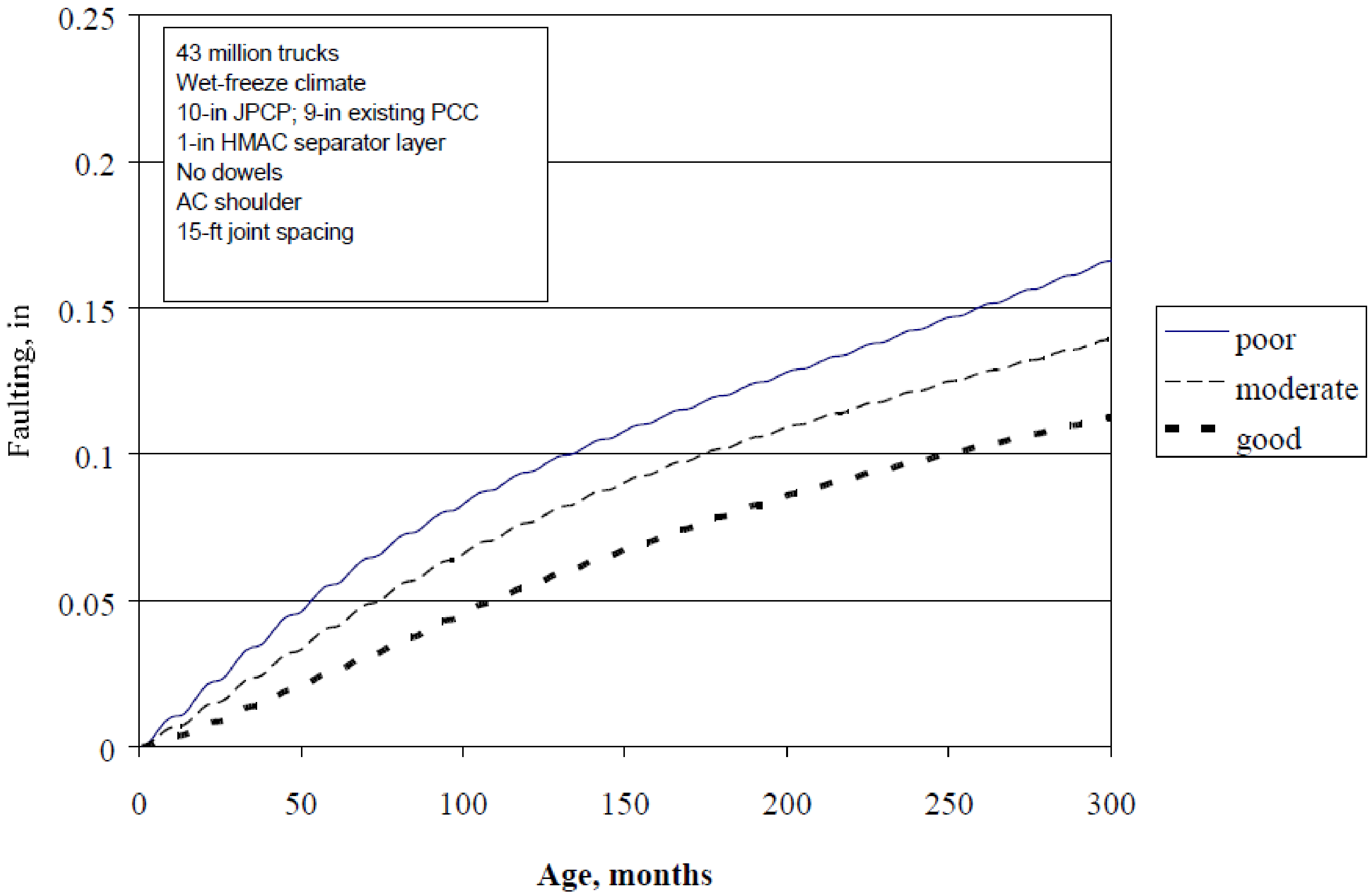
Effect of dowel diameter on faulting

Effect of Design Factors on Unbonded JPCP Overlay Performance



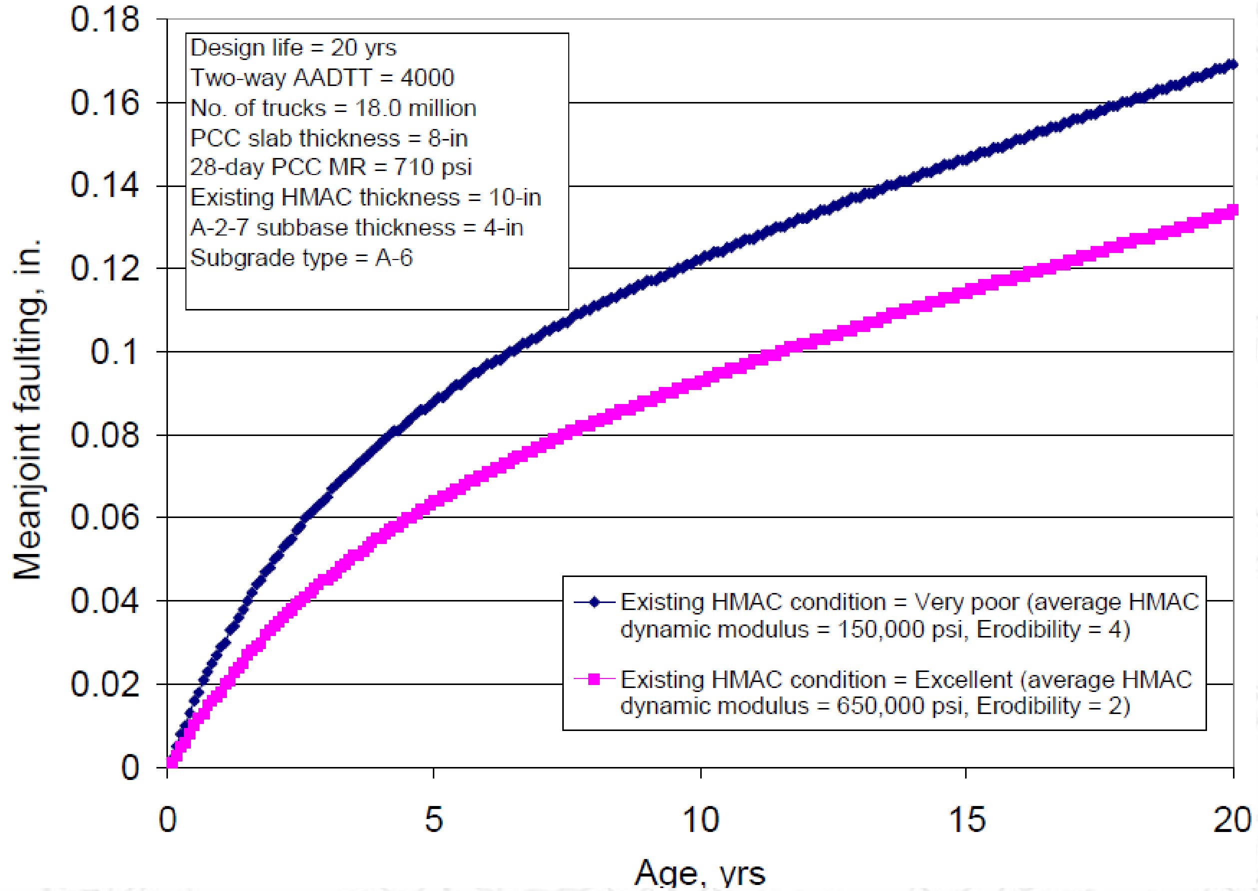
Effect of joint spacing on faulting

Effect of Design Factors on Unbonded JPCP Overlay Performance



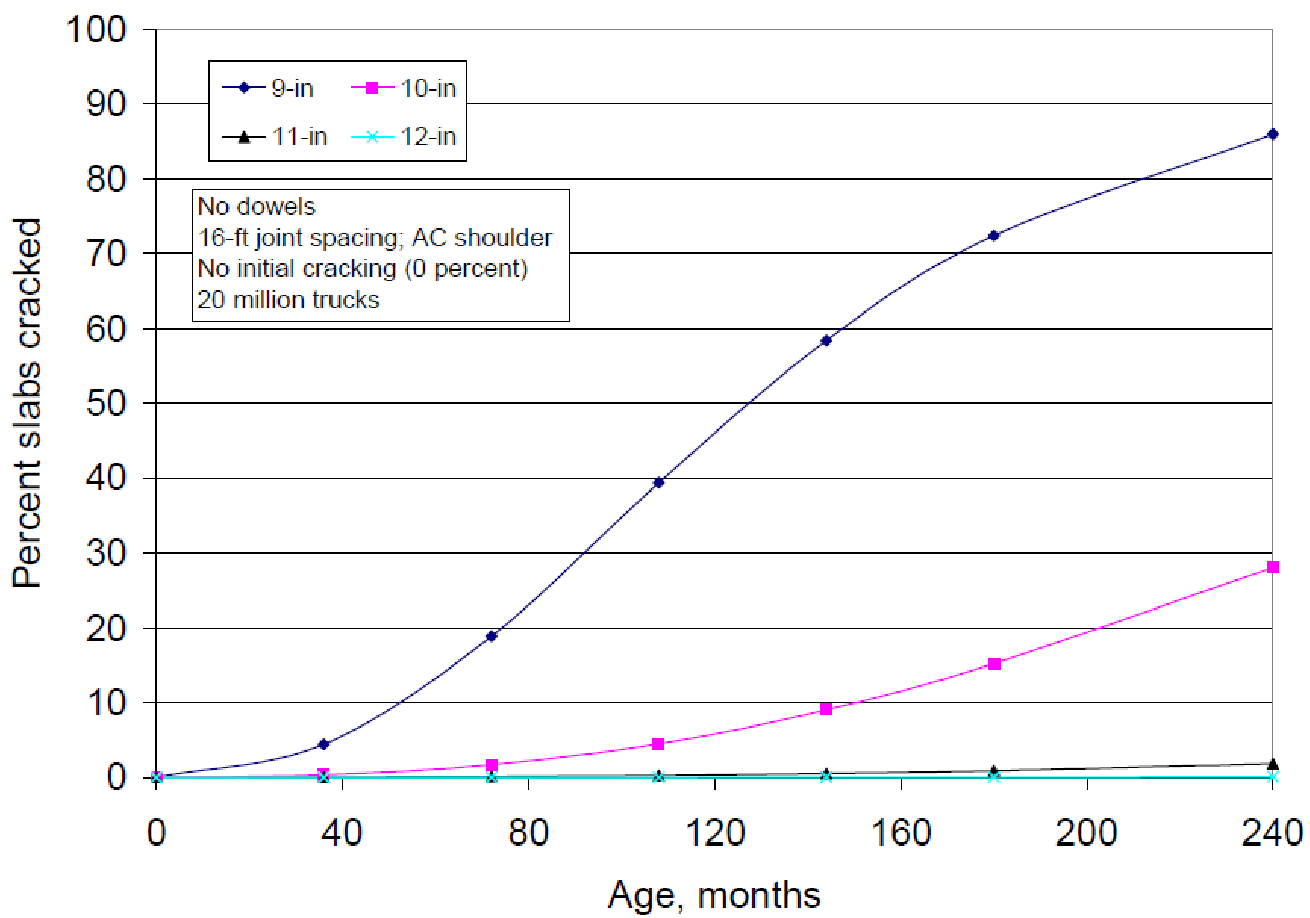
Effect of existing PCC condition on faulting

Effect of Design Factors on Unbonded JPCP Overlay Performance



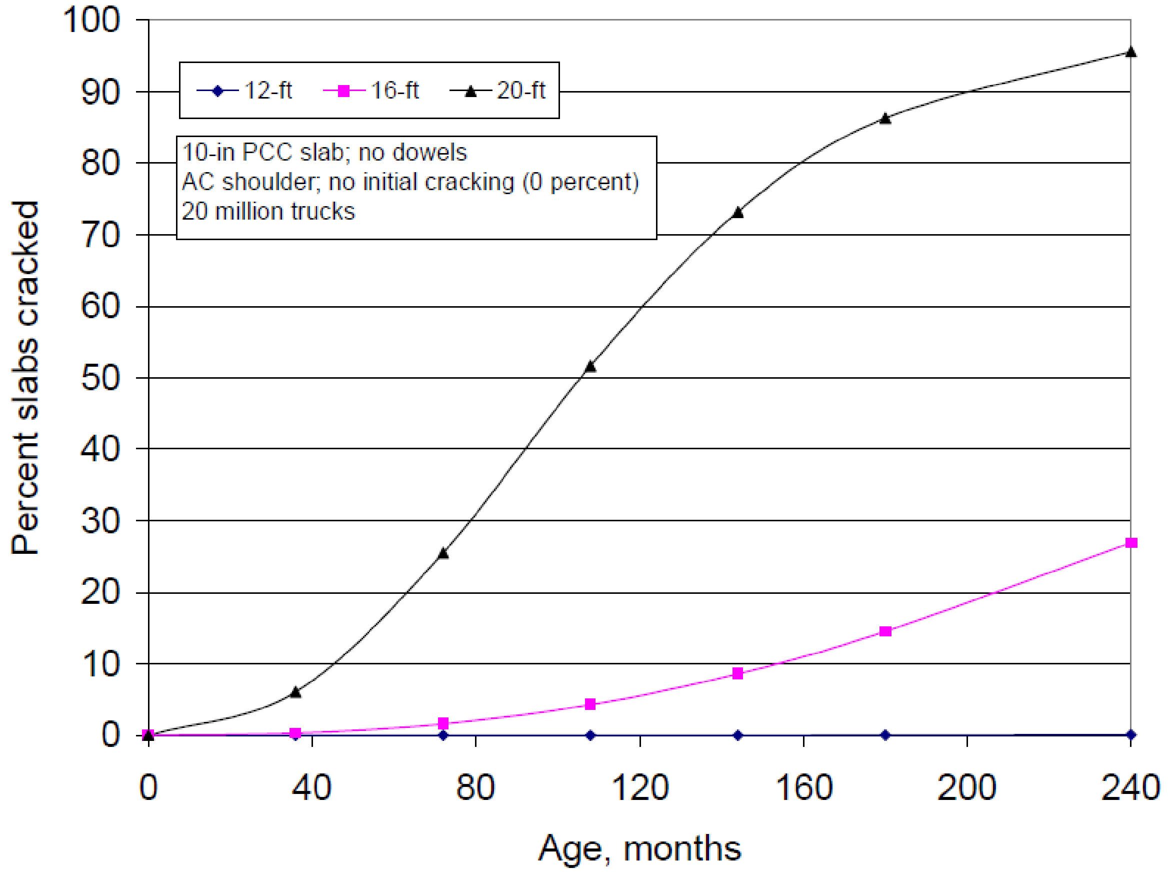
Effect of existing HMA condition on faulting

Effect of Design Factors on Unbonded JPCP Overlay Performance



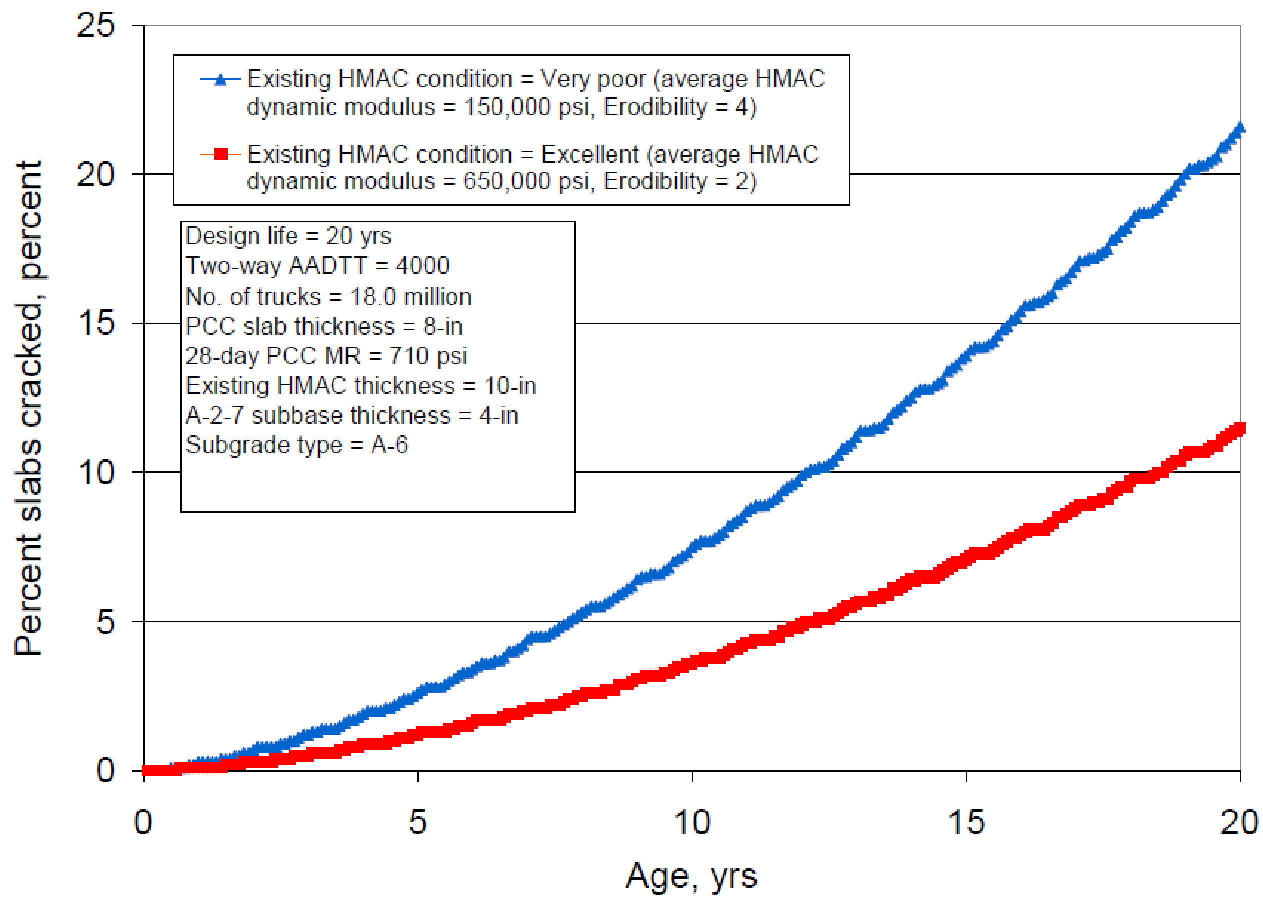
Effect of slab thickness on transverse cracking

Effect of Design Factors on Unbonded JPCP Overlay Performance



Effect of joint spacing on transverse cracking

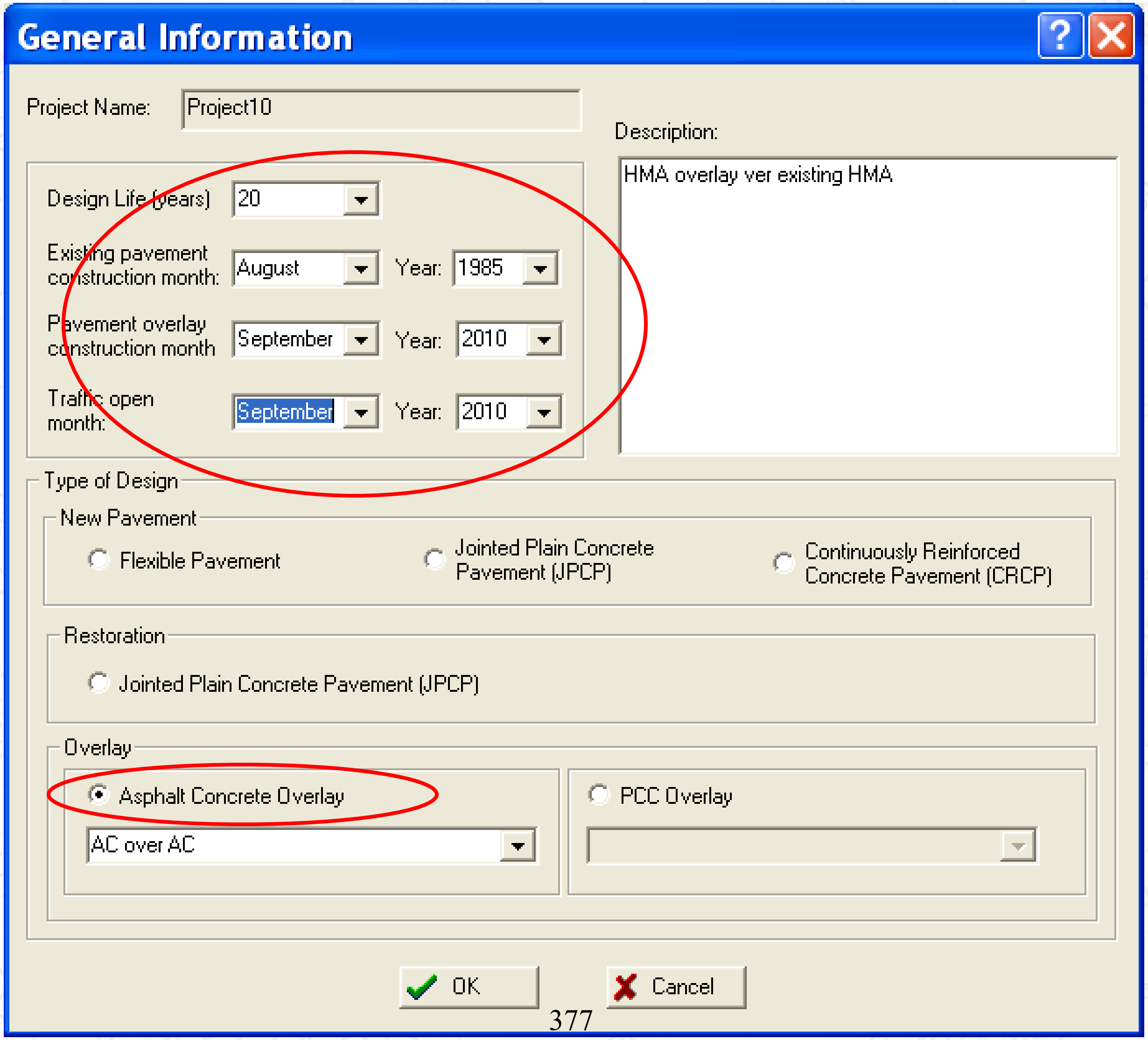
Effect of Design Factors on JPCP over HMA Performance



Effect of existing HMA condition on transverse cracking

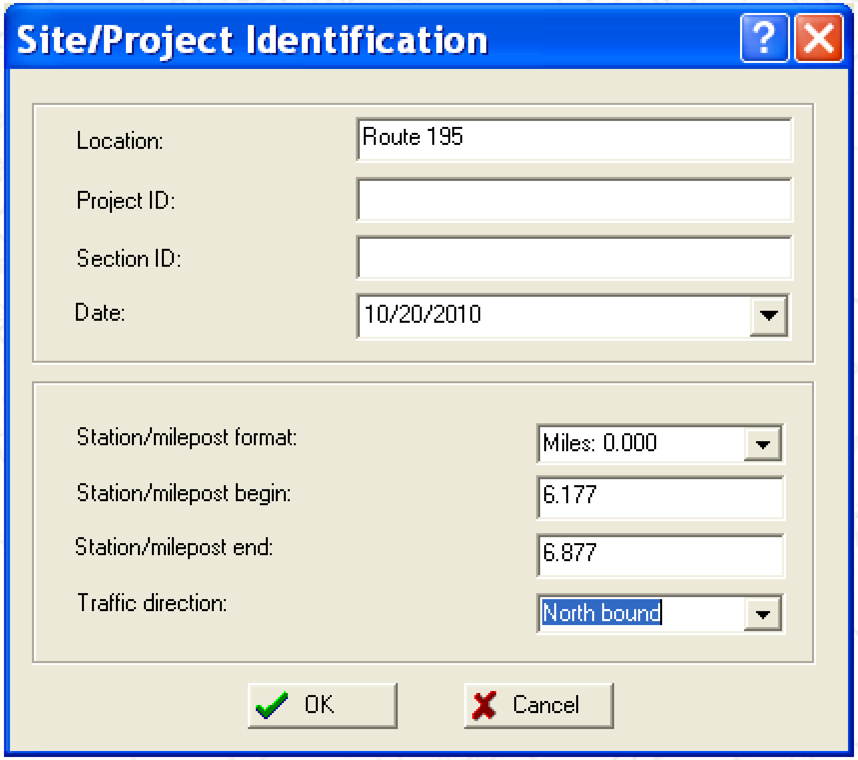
M-EPDG Example -HMA on HMA

General Information Screen - Inputs



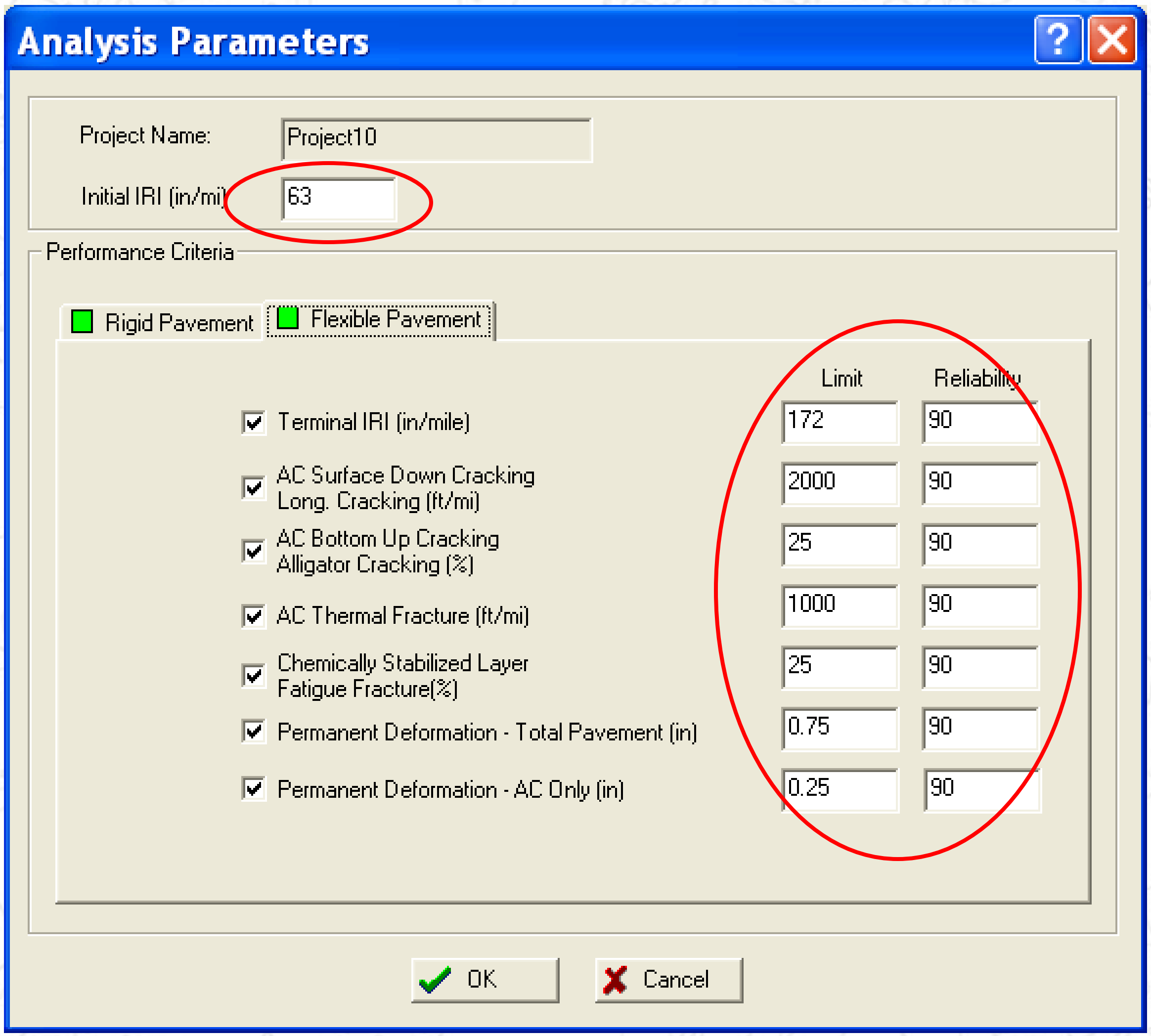
M-EPDG Example -HMA on HMA

Site/Project Identification



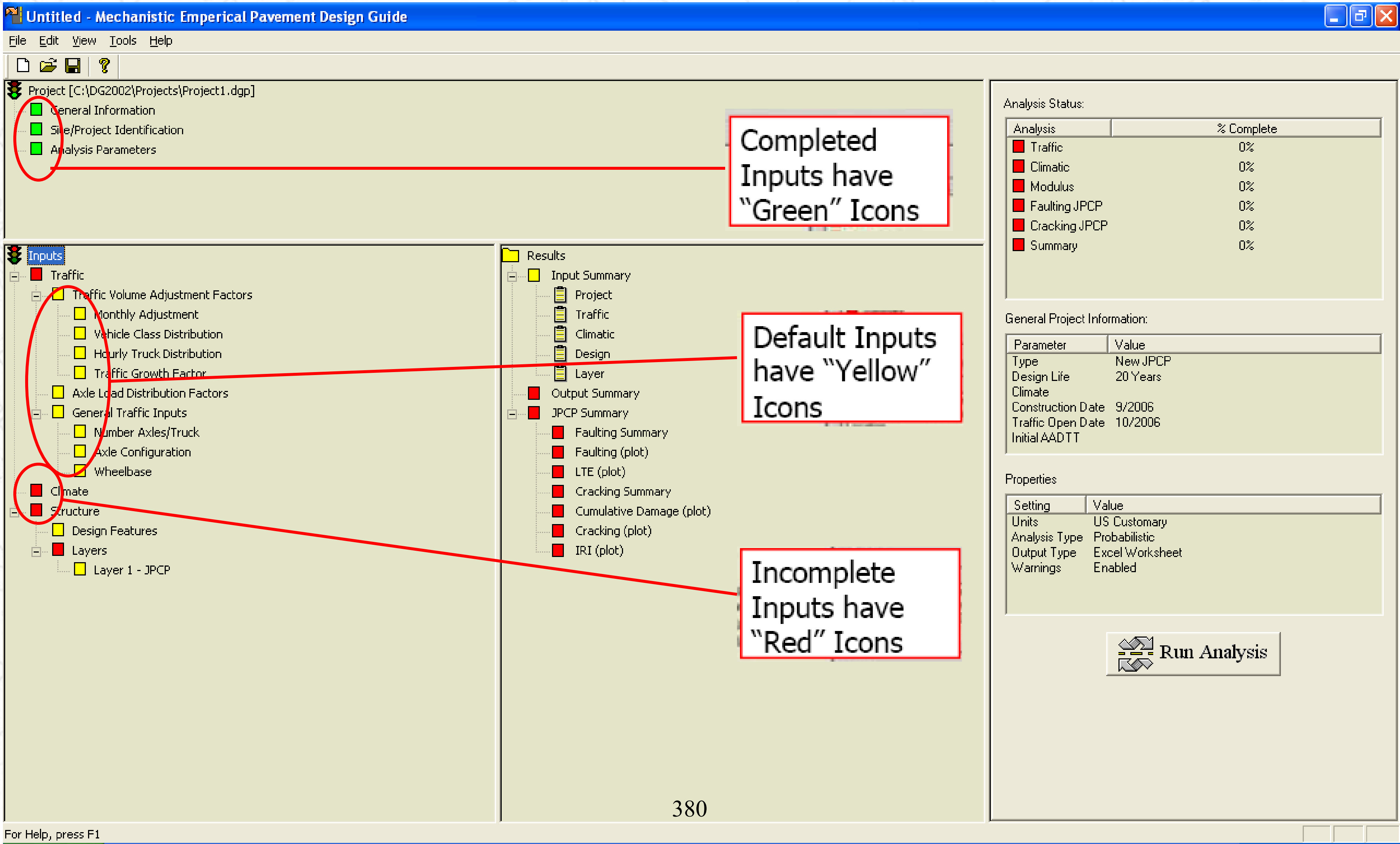
M-EPDG Example -HMA on HMA

Analysis Parameters

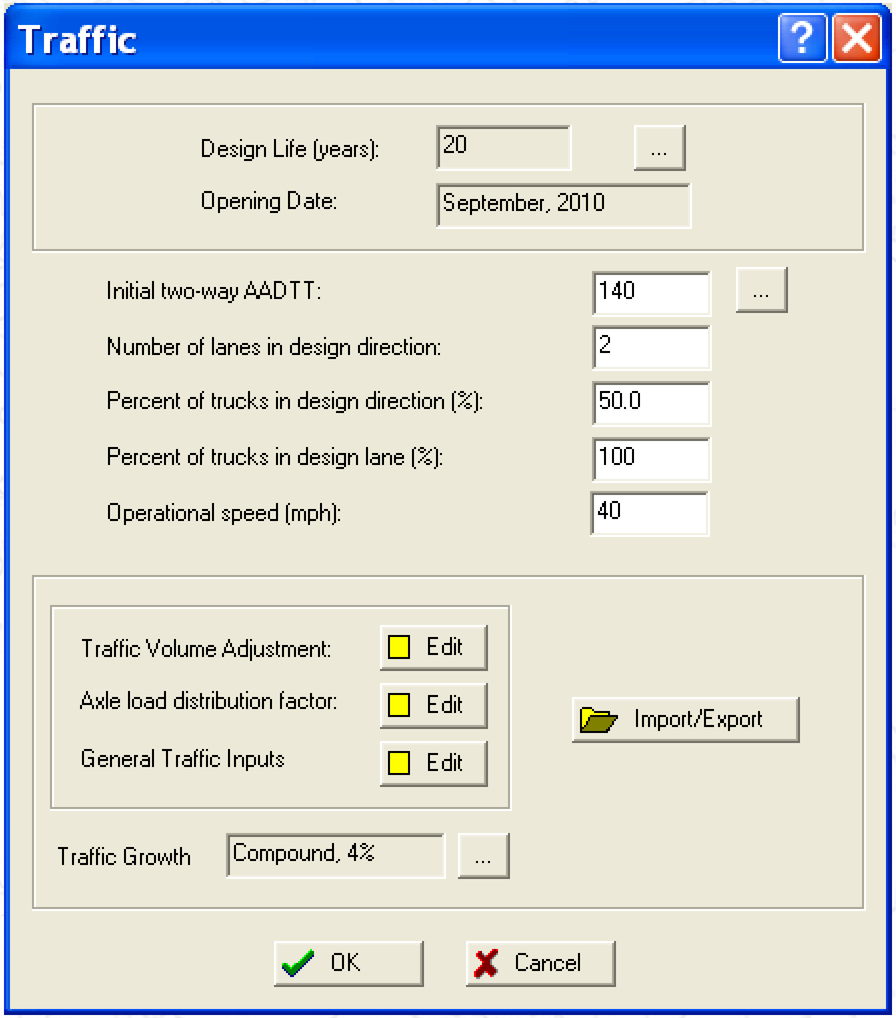


Example – JPCP Design

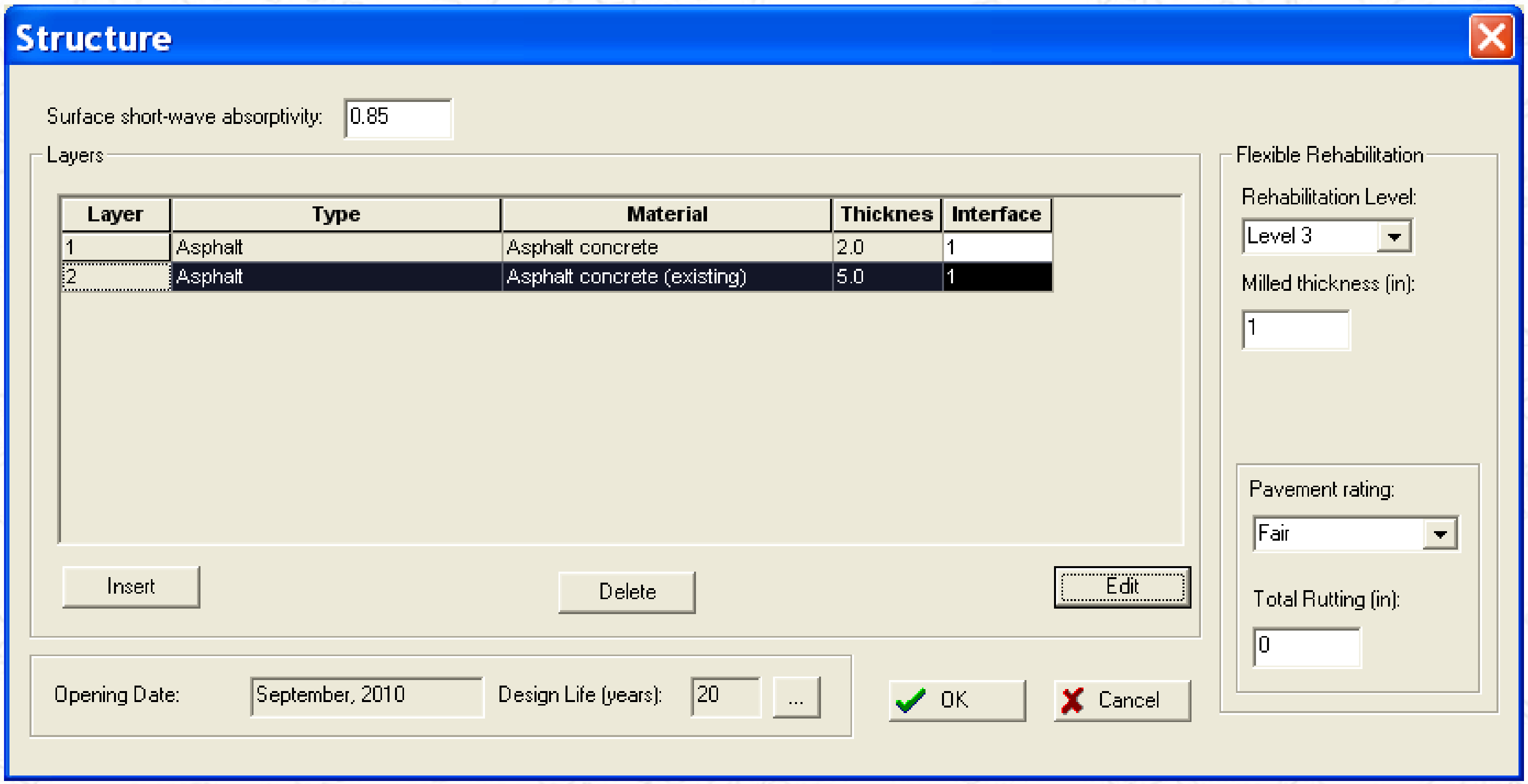
Program Indicates Status of Inputs



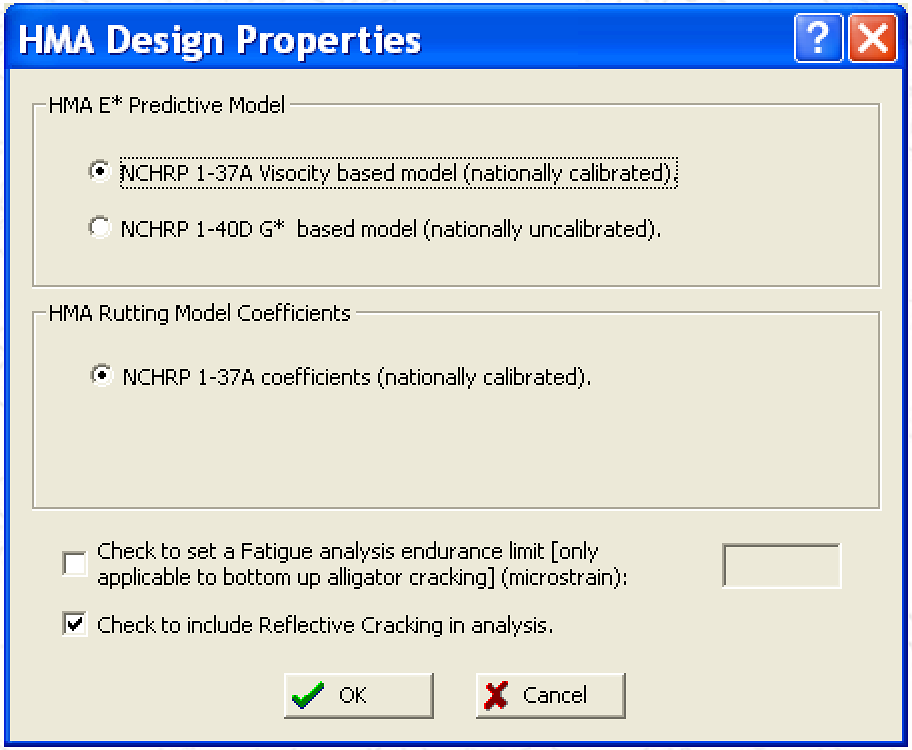
M-EPDG Example -HMA on HMA



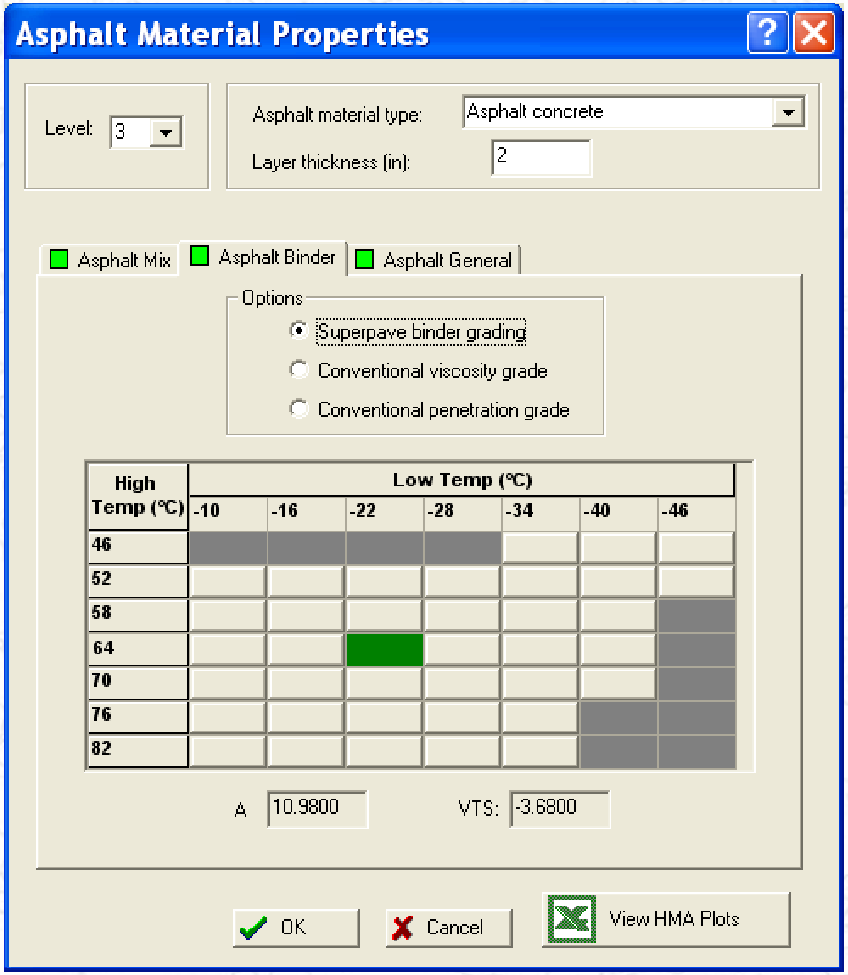
M-EPDG Example -HMA on HMA

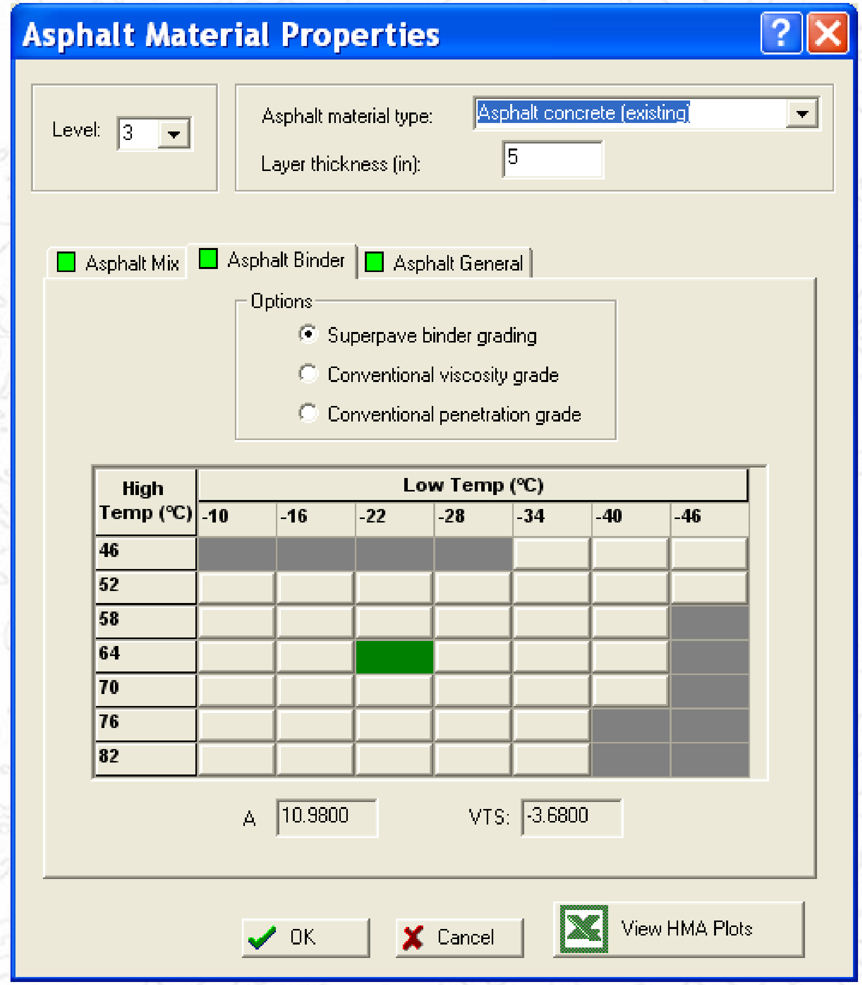


M-EPDG Example -HMA on HMA

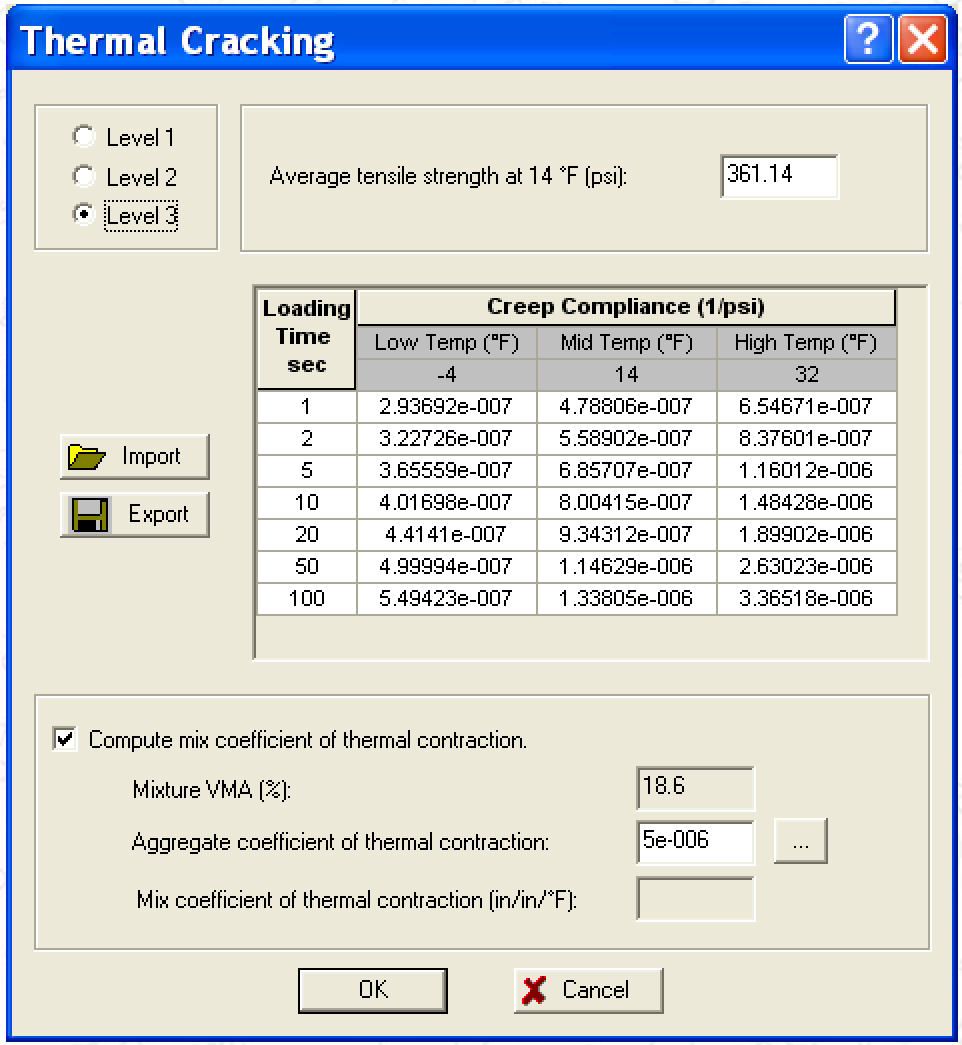


M-EPDG Example -HMA on HMA





M-EPDG Example -HMA on HMA



Thermal Cracking

Level 1
Level 2
Level 3

Average tensile strength at 14 °F (psi): 361.14

Loading Time sec	Creep Compliance (1/psi)		
	Low Temp (°F)	Mid Temp (°F)	High Temp (°F)
	-4	14	32
1	2.93692e-007	4.78806e-007	6.54671e-007
2	3.22726e-007	5.58902e-007	8.37601e-007
5	3.65559e-007	6.85707e-007	1.16012e-006
10	4.01698e-007	8.00415e-007	1.48428e-006
20	4.4141e-007	9.34312e-007	1.89902e-006
50	4.99994e-007	1.14629e-006	2.63023e-006
100	5.49423e-007	1.33805e-006	3.36518e-006

Import
Export

Compute mix coefficient of thermal contraction.

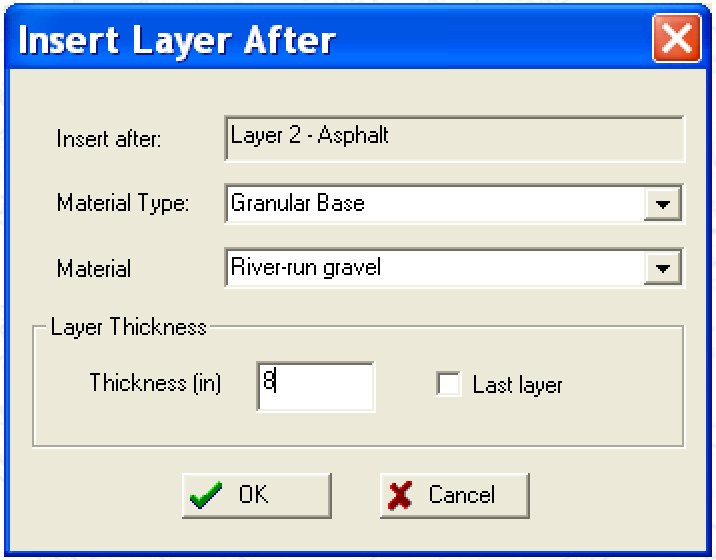
Mixture VMA (%): 18.6

Aggregate coefficient of thermal contraction: 5e-006 ...

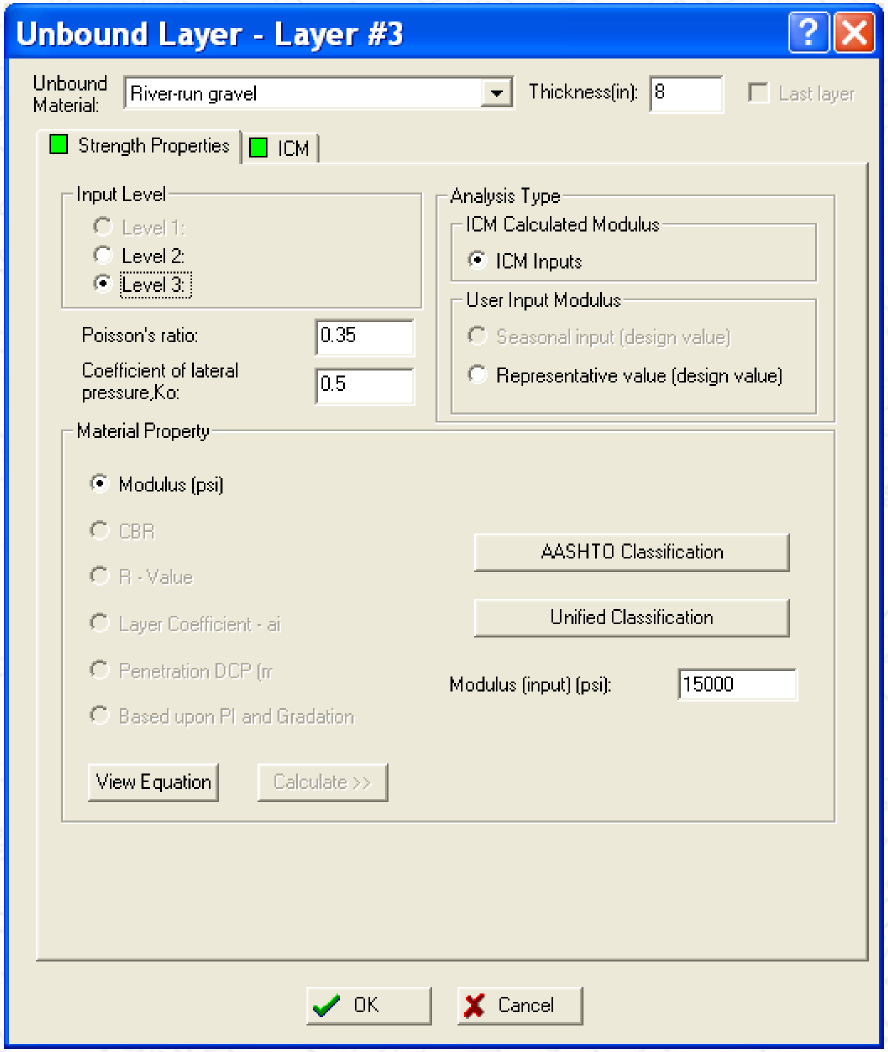
Mix coefficient of thermal contraction (in/in/°F):

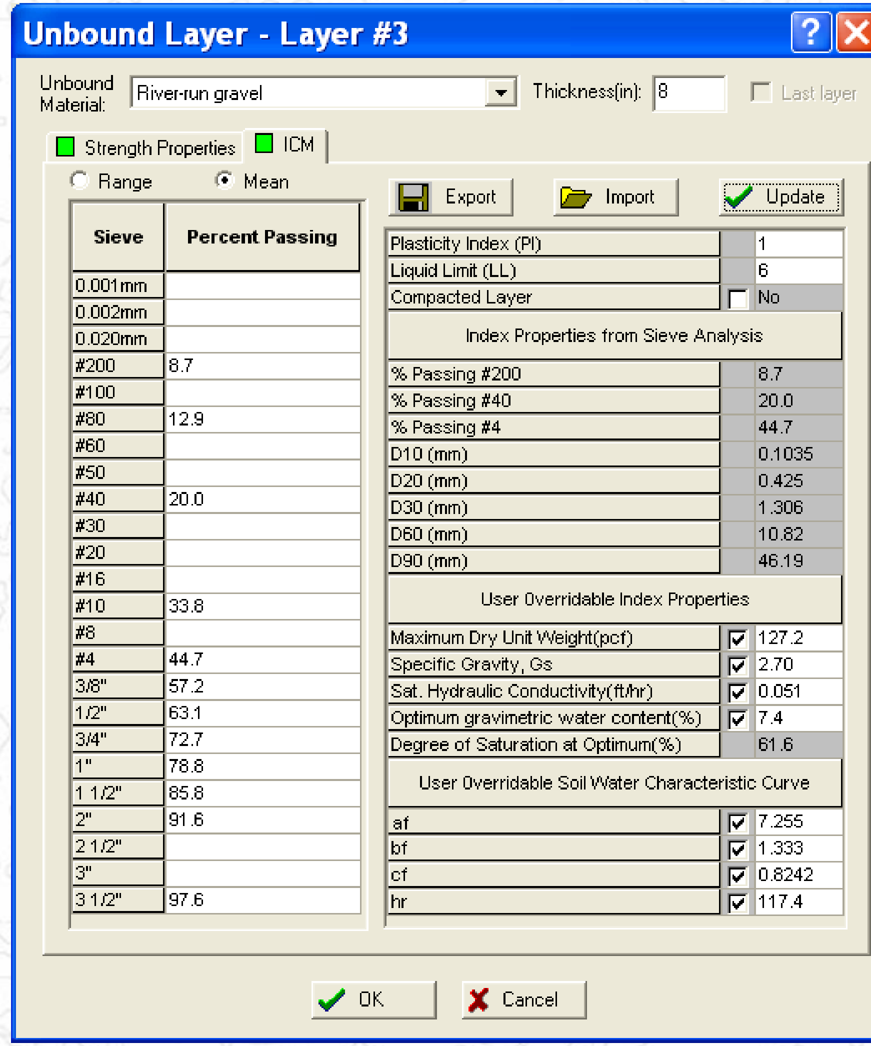
OK
Cancel

M-EPDG Example -HMA on HMA

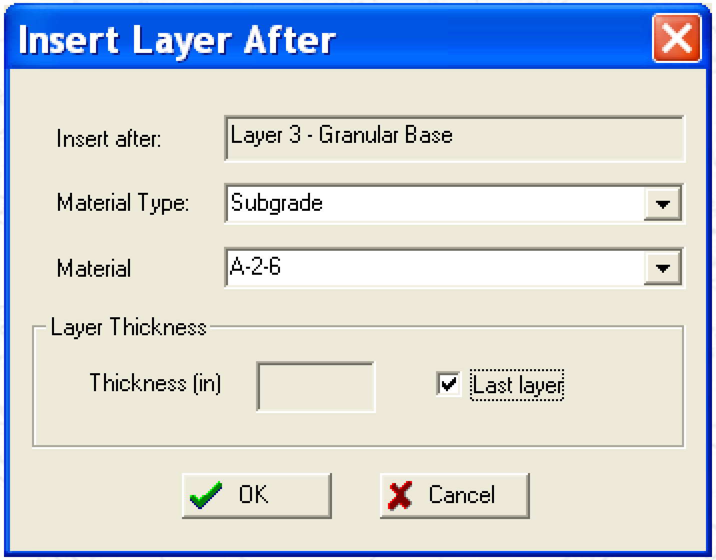


M-EPDG Example -HMA on HMA

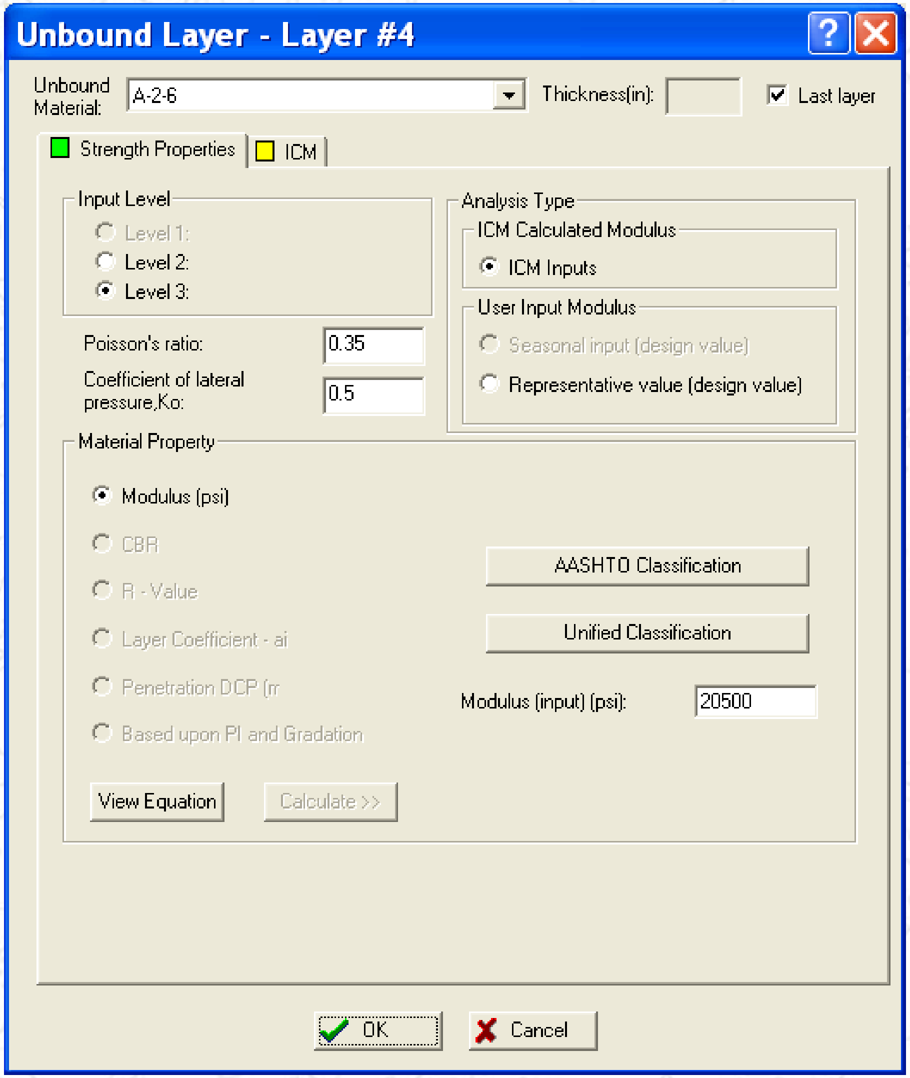


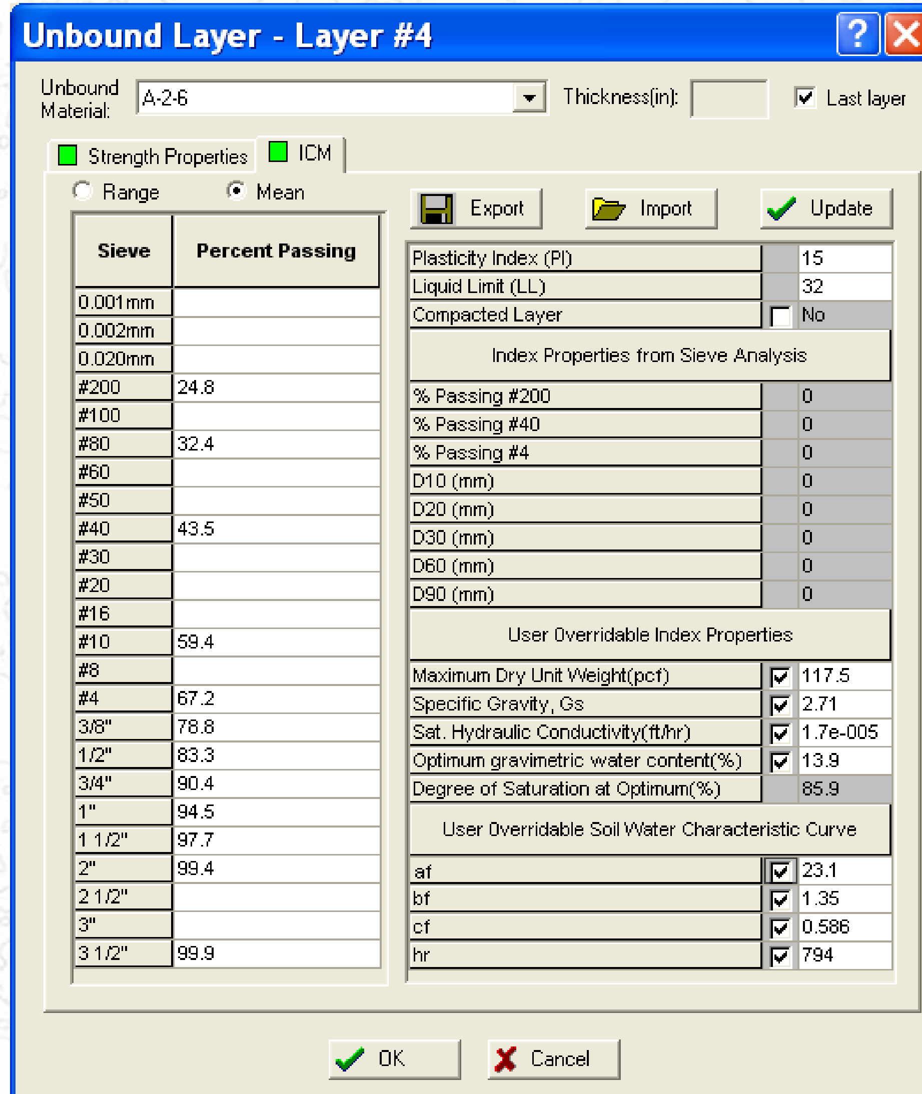


M-EPDG Example -HMA on HMA

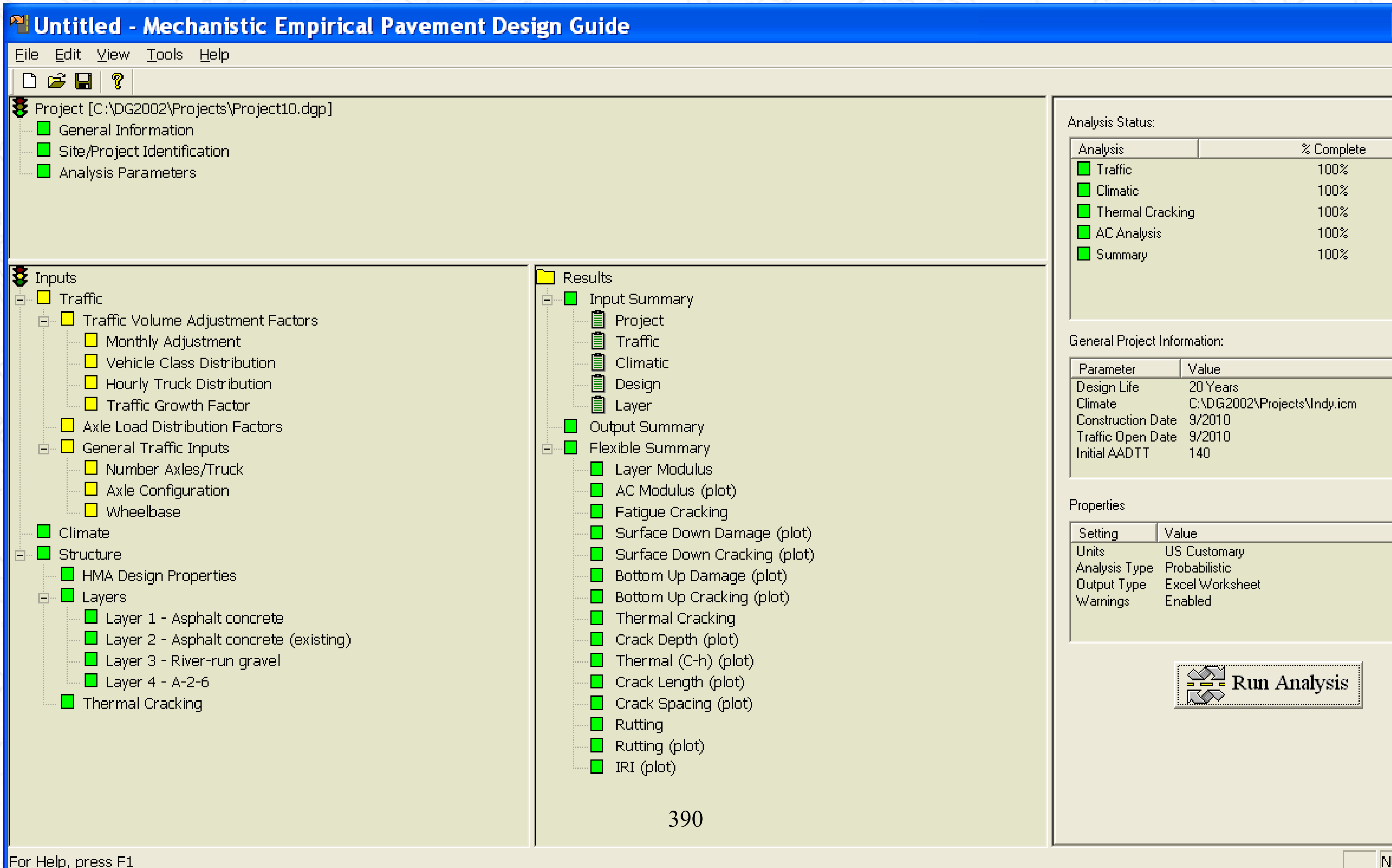


M-EPDG Example -HMA on HMA





M-EPDG Example -HMA on HMA



Appendix C. Inputs and Outputs

New AC

FNAME	AADTT	DWT	HAC1	ACOLGRAD	ACOLBIND	HAC2	ACGRAD	ACBIND	STRUCT	CLIMATE	HBASE	EB	SG	ES	BASE	SUBGRAD	LONGCRA	ALLIGCRA	TRANSCRA	ACRUT	TOTRUT	IRI	TRUCKS
NAC11122	400	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	10000	Grading B	Soil A	0.02	0.0638	0.2	0.048	0.26	99.5	1912450
NAC11222	400	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.19	0.0516	0.2	0.049	0.211	97.6	1912450
NAC11322	400	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	20000	Grading B	Soil C	0.59	0.0451	0.2	0.05	0.203	97	1912450
NAC12122	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	10000	Grading B	Soil A	0.05	0.152	0.2	0.068	0.303	101.3	4781120
NAC12212	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	10	25000	A-1-b	15000	Grading B	Soil B	0.41	0.128	0.2	0.069	0.246	99	4781120
NAC12221	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	20000	A-1-b	15000	Grading A	Soil B	0.52	0.137	0.2	0.068	0.252	99.3	4781120
NAC12222	1000	10	3	S0.375	70-22	5	S0.5	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	45.7	0.5	0.3	0.081	0.293	101.1	4781120
NAC12222	1000	10	4	S0.375	64-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.47	0.12	0.3	0.079	0.258	99.5	4781120
NAC12222	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.45	0.122	0.2	0.07	0.25	99.1	4781120
NAC12222	1000	10	4	S0.375	76-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.27	0.114	0.1	0.057	0.235	98.6	4781120
NAC12222	1000	10	3	S0.375	70-22	3	S0.5	64-22	3+3+6	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0	0.321	0.1	0.062	0.209	97.6	4781120
NAC12223	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	30000	A-1-b	15000	Grading C	Soil B	0.34	0.109	0.2	0.071	0.246	99	4781120
NAC12232	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	18	25000	A-1-b	15000	Grading B	Soil B	0.52	0.118	0.2	0.07	0.253	99.3	4781120
NAC12322	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	20000	Grading B	Soil C	1.47	0.106	0.2	0.071	0.24	98.5	4781120
NAC13122	2500	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	10000	Grading B	Soil A	0.12	0.37	0.2	0.099	0.358	103.6	11952800
NAC13222	2500	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	1.17	0.297	0.2	0.102	0.3	101.3	11952800
NAC13322	2500	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	SHORE	14	25000	A-1-b	20000	Grading B	Soil C	4.01	0.259	0.2	0.104	0.288	100.5	11952800
NAC21122	400	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	10000	Grading B	Soil A	0.02	0.0638	0.2	0.048	0.26	99.5	1912450
NAC21222	400	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.19	0.0516	0.2	0.049	0.211	97.6	1912450
NAC21322	400	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	20000	Grading B	Soil C	0.59	0.0451	0.2	0.05	0.203	97	1912450
NAC22122	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	10000	Grading B	Soil A	0.05	0.152	0.2	0.068	0.303	101.3	4781120
NAC22212	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	10	25000	A-1-b	15000	Grading B	Soil B	0.41	0.128	0.2	0.069	0.246	99	4781120
NAC22221	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	20000	A-1-b	15000	Grading A	Soil B	0.52	0.137	0.2	0.068	0.252	99.3	4781120
NAC22222	1000	10	3	S0.375	70-22	5	S0.5	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	45.7	0.5	0.3	0.081	0.293	101.1	4781120
NAC22222	1000	10	4	S0.375	64-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.47	0.12	0.3	0.079	0.258	99.5	4781120
NAC22222	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.45	0.122	0.2	0.07	0.25	99.1	4781120
NAC22222	1000	10	4	S0.375	76-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.27	0.114	0.1	0.057	0.235	98.6	4781120
NAC22222	1000	10	3	S0.375	70-22	3	S0.5	64-22	3+3+6	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0	0.321	0.1	0.062	0.209	97.6	4781120
NAC22223	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	30000	A-1-b	15000	Grading C	Soil B	0.34	0.109	0.2	0.071	0.246	99	4781120
NAC22232	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	18	25000	A-1-b	15000	Grading B	Soil B	0.52	0.118	0.2	0.07	0.253	99.3	4781120
NAC22322	1000	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	20000	Grading B	Soil C	1.47	0.106	0.2	0.071	0.24	98.5	4781120
NAC23122	2500	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	10000	Grading B	Soil A	0.12	0.37	0.2	0.099	0.358	103.6	11952800
NAC23222	2500	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	1.17	0.297	0.2	0.102	0.3	101.3	11952800
NAC23322	2500	10	4	S0.375	70-22	6	S0.5	64-22	4+6+0	INLAND	14	25000	A-1-b	20000	Grading B	Soil C	4.01	0.259	0.2	0.104	0.288	100.5	11952800
NAC31122	400	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	14	25000	A-1-b	10000	Grading B	Soil A	0.02	0.0587	312	0.044	0.261	102.9	1912450
NAC31222	400	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.13	0.0474	313	0.045	0.234	101.8	1912450
NAC31322	400	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	14	25000	A-1-b	20000	Grading B	Soil C	0.41	0.0414	316	0.045	0.217	100.9	1912450
NAC32122	1000	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	14	25000	A-1-b	10000	Grading B	Soil A	0.03	0.14	312	0.062	0.302	104.5	4781120
NAC32212	1000	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	10	25000	A-1-b	15000	Grading B	Soil B	0.27	0.118	314	0.063	0.268	103.2	4781120
NAC32221	1000	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	14	20000	A-1-b	15000	Grading A	Soil B	0.33	0.125	309	0.062	0.276	103.5	4781120
NAC32222	1000	100	3	S0.375	70-22	5	S0.5	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	26.5	0.427	170	0.066	0.31	103.9	4781120
NAC32222	1000	100	4	S0.375	64-22	6	S0.5	64-22	4+6+0	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.35	0.111	549	0.074	0.283	105.7	4781120
NAC32222	1000	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.29	0.112	313	0.063	0.272	103.3	4781120
NAC32222	1000	100	4	S0.375	76-22	6	S0.5	64-22	4+6+0	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.2	0.106	159	0.055	0.261	101.7	4781120
NAC32223	1000	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	14	30000	A-1-b	15000	Grading C	Soil B	0.23	0.101	309	0.064	0.267	103.1	4781120
NAC32232	1000	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	18	25000	A-1-b	15000	Grading B	Soil B	0.34	0.108	313	0.064	0.275	103.5	4781120
NAC32322	1000	100	4	S0.375	70-22	6	S0.5	64-22	4+6+0	MOUNT	14	25000	A-1-b	20000	Grading B	Soil C	0.99	0.0977	316	0.064	0.252	102.3	4781120

AC over AC

FNAME	AADTT	DWT	HMILL	PR	TotRutExi	HAC1	ACOLGRA	ACOLBINI	HAC2	ACGRAD	ACBIND	STRUCT	CLIMATE	HBASE	EB	SG	ES	VCD	THD	TGR	TPRESS	BASE	SUBGRAD	LONGCRA	ALLIGCRA	REFCRACI	SURFCRA(	TRANSCRA	ACRUT
OAC_1112	400	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	10000	High	Low	0.02	120	Grading B	Soil A	0.96	0.0079	34.23	34.2379	3.4	0.042
OAC_1122	400	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	4.76	0.0075	34.23	34.2375	3.4	0.044
OAC_1132	400	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	20000	High	Low	0.02	120	Grading B	Soil C	10.8	0.0072	34.22	34.2272	3.6	0.044
OAC_1212	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	10000	High	Low	0.02	120	Grading B	Soil A	2.33	0.0172	34.28	34.2972	3.4	0.06
OAC_1221	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	10	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	12.6	0.0163	34.27	34.2863	3.3	0.062
OAC_1222	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	20000	A-1-b	15000	High	Low	0.02	120	Grading A	Soil B	17.7	0.0168	34.28	34.2968	3.5	0.061
OAC_1222	1000	10	2	Fair	0.5	3.5	S0.5	64-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	17.2	0.0127	34.27	34.2827	4.1	0.075
OAC_1222	1000	10	2	Fair	0.5	2	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	199	0.0015	28.88	28.8815	2.5	0.076
OAC_1222	1000	10	2	Fair	0.5	3.5	S0.375	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	8.32	0.0105	34.27	34.2805	0.4	0.064
OAC_1222	1000	10	1	Fair	0.5	3.5	S0.5	70-22	2	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	1.39	0.0119	34.24	34.2519	2.7	0.058
OAC_1222	1000	10	2	Poor	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	131	0.265	50.03	50.295	3.4	0.07
OAC_1222	1000	10	2	Fair	0	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	12.7	0.0162	34.27	34.2862	3.4	0.079
OAC_1222	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	12.7	0.0162	34.27	34.2862	3.4	0.062
OAC_1222	1000	10	2	Fair	1	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	12.7	0.0162	34.27	34.2862	3.4	0.062
OAC_1222	1000	10	2	Good	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	3.58	0.0025	17.74	17.7425	3.4	0.061
OAC_1222	1000	10	3	Fair	0.5	3.5	S0.5	70-22	5	S0.5	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	23.2	0	0	0	3.8	0.073
OAC_1222	1000	10	2	Fair	0.5	3.5	S1	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	14.4	0.0285	34.26	34.2885	10.5	0.055
OAC_1222	1000	10	2	Fair	0.5	5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	0.44	0.013	35.75	35.763	1.4	0.059
OAC_1222	1000	10	2	Fair	0.5	3.5	S0.5	76-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	9.02	0.0203	34.26	34.2803	1.1	0.05
OAC_1222	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	30000	A-1-b	15000	High	Low	0.02	120	Grading C	Soil B	9.14	0.0159	34.26	34.2759	3.6	0.064
OAC_1223	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	18	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	13.8	0.0162	34.27	34.2862	3.3	0.063
OAC_1232	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	20000	High	Low	0.02	120	Grading B	Soil C	30	0.0156	34.26	34.2756	3.6	0.063
OAC_1312	2500	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	10000	High	Low	0.02	120	Grading B	Soil A	6.33	0.0393	34.4	34.4393	3.4	0.088
OAC_1322	2500	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	37	0.0371	34.36	34.3971	3.4	0.091
OAC_1332	2500	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	SHORE	14	25000	A-1-b	20000	High	Low	0.02	120	Grading B	Soil C	90.1	0.0357	34.34	34.3757	3.6	0.093
OAC_2112	400	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	10000	High	Low	0.02	120	Grading B	Soil A	1.6	0.0088	34.24	34.2488	29	0.05
OAC_2122	400	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	7.15	0.0083	34.23	34.2383	29.1	0.052
OAC_2132	400	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	20000	High	Low	0.02	120	Grading B	Soil C	15.5	0.008	34.23	34.238	30.7	0.053
OAC_2212	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	10000	High	Low	0.02	120	Grading B	Soil A	4	0.0191	34.29	34.3091	29	0.071
OAC_2221	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	10	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	19.5	0.0181	34.28	34.2981	27.4	0.073
OAC_2222	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	20000	A-1-b	15000	High	Low	0.02	120	Grading A	Soil B	28	0.0187	34.29	34.3087	30.5	0.072
OAC_2222	1000	10	2	Fair	0.5	3.5	S0.5	64-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	26.7	0.0138	34.28	34.2938	33.3	0.09
OAC_2222	1000	10	2	Fair	0.5	2	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	271	0.0018	28.91	28.9118	20.5	0.092
OAC_2222	1000	10	2	Fair	0.5	3.5	S0.375	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	12.7	0.0116	34.28	34.2916	2.5	0.076
OAC_2222	1000	10	1	Fair	0.5	3.5	S0.5	70-22	2	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	2.32	0.0134	34.24	34.2534	28	0.068
OAC_2222	1000	10	2	Poor	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	168	0.297	50.03	50.327	29.1	0.082
OAC_2222	1000	10	2	Fair	0	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	19.4	0.018	34.27	34.288	29.1	0.092
OAC_2222	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	19.4	0.018	34.27	34.288	29.1	0.074
OAC_2222	1000	10	2	Fair	1	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	19.4	0.018	34.27	34.288	29.1	0.073
OAC_2222	1000	10	2	Good	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	6.07	0.0027	17.75	17.7527	29.1	0.073
OAC_2222	1000	10	3	Fair	0.5	3.5	S0.5	70-22	5	S0.5	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	37.7	0	0	0	38	0.085
OAC_2222	1000	10	2	Fair	0.5	3.5	S1	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	22.3	0.0315	34.27	34.3015	104	0.066
OAC_2222	1000	10	2	Fair	0.5	5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	0.81	0.0144	35.75	35.7644	13.9	0.07
OAC_2222	1000	10	2	Fair	0.5	3.5	S0.5	76-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	13.9	0.023	34.27	34.293	8.6	0.059
OAC_2222	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	30000	A-1-b	15000	High	Low	0.02	120	Grading C	Soil B	13.7	0.0177	34.27	34.2877	31.8	0.075
OAC_2223	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	18	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	20.8	0.018	34.27	34.288	30.7	0.074
OAC_2232	1000	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	20000	High	Low	0.02	120	Grading B	Soil C	43.8	0.0173	34.27	34.2873	30.7	0.075
OAC_2312	2500	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	10000	High	Low	0.02	120	Grading B	Soil A	11.1	0.0435	34.42	34.4635	29	0.104
OAC_2322	2500	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	57.2	0.0411	34.38	34.4211	29.1	0.108
OAC_2332	2500	10	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	INLAND	14	25000	A-1-b	20000	High	Low	0.02	120	Grading B	Soil C	133	0.0396	34.36	34.3996	30.7	0.11
OAC_3112	400	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	10000	High	Low	0.02	120	Grading B	Soil A	0.71	0.0063	34.23	34.2363	1180	0.039
OAC_3122	400	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	3.56	0.006	34.23	34.236	1190	0.04
OAC_3132	400	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	20000	High	Low	0.02	120	Grading B	Soil C	8.21	0.0058	34.22	34.2258	1240	0.041
OAC_3212	1000	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	10000	High	Low	0.02	120	Grading B	Soil A	1.72	0.0138	34.28	34.2938	1180	0.056
OAC_3221	1000	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	10	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	9.3	0.0131	34.26	34.2731	1170	0.057
OAC_3222	1000	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	20000	A-1-b	15000	High	Low	0.02	120	Grading A	Soil B	13	0.0135	34.27	34.2835	1230	0.056
OAC_3222	1000	100	2	Fair	0.5	3.5	S0.5	64-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	13.7	0.0101	34.27	34.2801	894	0.071
OAC_3222	1000	100	2	Fair	0.5	2	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	162	0.0011	28.85	28.8511	1270	0.068
OAC_3222	1000	100	2	Fair	0.5	3.5	S0.375	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	6.19	0.0084	34.26	34.2684	651	0.059
OAC_3222	1000	100	1	Fair	0.5	3.5	S0.5	70-22	2	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	0.98	0.0095	34.23	34.2395	879	0.053
OAC_3222	1000	100	2	Poor	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	103	0.223	50.03	50.253	1190	0.064
OAC_3222	1000	100	2	Fair	0	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	9.44	0.013	34.26	34.273	1190	0.073
OAC_3222	1000	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	9.44	0.013	34.26	34.273	1190	0.058
OAC_3222	1000	100	2	Fair	1	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	9.44	0.013	34.26	34.273	1190	0.057
OAC_3222	1000	100	2	Good	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	2.6	0.0019	17.73	17.7319	1190	0.057
OAC_3222	1000	100	3	Fair	0.5	3.5	S0.5	70-22	5	S0.5	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	18.2	0	0	0	1330	0.067
OAC_3222	1000	100	2	Fair	0.5	3.5	S1	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	10.8	0.0231	34.26	34.2831	1410	0.051
OAC_3222	1000	100	2	Fair	0.5	5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	0.3	0.0107	35.75	35.7607	834	0.054
OAC_3222	1000	100	2	Fair	0.5	3.5	S0.5	76-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	7.15	0.0163	34.26	34.2763	884	0.048
OAC_3222	1000	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	30000	A-1-b	15000	High	Low	0.02	120	Grading C	Soil B	6.87	0.0128	34.26	34.2728	1230	0.059
OAC_3223	1000	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	18	25000	A-1-b	15000	High	Low	0.02	120	Grading B	Soil B	10.3	0.013	34.26	34.273	1230	0.058
OAC_3232	1000	100	2	Fair	0.5	3.5	S0.5	70-22	1	S0.375	64-22	3+5+0	MOUNT	14	25000	A-1-b	20000	High	Low	0.02	120	Grading B	Soil C	22.5	0.0126	34.25	34.2626	1240	0.058

AC over Rubblized PCC

FNAME	AADTT	DWT	HACOL	ACOLGRA	ACOLBINI	HPCC	EPCC	CLIMATE	HBASE	EB	SG	ES	BASE	SUBGRAD	LONGCRA	ALLIGCRA	TRANSCR	ACRUT	TOTRUT	IRI	TRUCKS
OPC11122	400	10	3.5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	10000	Grading B	Soil A	0	0.0038	1610	0.047	0.236	113.2	1912450
OPC11222	400	10	3.5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.02	0.004	1610	0.049	0.22	112.5	1912450
OPC11322	400	10	3.5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	20000	Grading B	Soil C	0.05	0.004	1610	0.05	0.206	112	1912450
OPC12122	1000	10	3.5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	10000	Grading B	Soil A	0.02	0.0127	1610	0.068	0.283	115	4781120
OPC12212	1000	10	3.5	S0.5	70-22	9	500000	SHORE	10	25000	A-1-b	15000	Grading B	Soil B	0.07	0.0134	1610	0.07	0.259	114.1	4781120
OPC12221	1000	10	3.5	S0.5	70-22	9	500000	SHORE	14	30000	A-1-b	15000	Grading C	Soil B	0.03	0.0112	1620	0.074	0.253	113.9	4781120
OPC12222	1000	10	3.5	S0.5	64-22	9	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.1	0.0128	1610	0.086	0.28	114.9	4781120
OPC12222	1000	10	2	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0	0	1450	0.037	0.251	112.4	4781120
OPC12222	1000	10	3.5	S0.375	70-22	9	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.05	0.0086	733	0.072	0.265	107.3	4781120
OPC12222	1000	10	3.5	S0.5	70-22	9	200000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	7.06	0.26	1610	0.077	0.297	116	4781120
OPC12222	1000	10	3.5	S0.5	70-22	8	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.3	0.0175	1610	0.069	0.273	114.6	4781120
OPC12222	1000	10	3.5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.08	0.0132	1610	0.07	0.262	114.2	4781120
OPC12222	1000	10	3.5	S0.5	70-22	10	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.02	0.0105	1620	0.071	0.254	113.9	4781120
OPC12222	1000	10	3.5	S0.5	70-22	9	1000000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0	0.0006	1610	0.071	0.244	113.4	4781120
OPC12222	1000	10	3.5	S1	70-22	9	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.08	0.0193	1630	0.061	0.251	113.9	4781120
OPC12222	1000	10	5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.06	0.0169	13.2	0.092	0.267	100.8	4781120
OPC12222	1000	10	3.5	S0.5	76-22	9	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.07	0.0136	1200	0.062	0.254	110.6	4781120
OPC12223	1000	10	3.5	S0.5	70-22	9	500000	SHORE	14	20000	A-1-b	15000	Grading A	Soil C	0.06	0.0124	1620	0.072	0.266	114.4	4781120
OPC12232	1000	10	3.5	S0.5	70-22	9	500000	SHORE	18	25000	A-1-b	15000	Grading B	Soil B	0.08	0.013	1610	0.07	0.265	114.3	4781120
OPC12322	1000	10	3.5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	20000	Grading B	Soil C	0.19	0.0135	1610	0.072	0.247	113.6	4781120
OPC13122	2500	10	3.5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	10000	Grading B	Soil A	0.06	0.0419	1610	0.099	0.342	117.5	11952800
OPC13222	2500	10	3.5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	15000	Grading B	Soil B	0.28	0.0435	1610	0.102	0.318	116.5	11952800
OPC13322	2500	10	3.5	S0.5	70-22	9	500000	SHORE	14	25000	A-1-b	20000	Grading B	Soil C	0.71	0.0444	1610	0.105	0.3	115.8	11952800
OPC21122	400	10	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	10000	Grading B	Soil A	0.01	0.0036	137	0.066	0.257	102	1912450
OPC21222	400	10	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.03	0.0037	137	0.067	0.24	101.3	1912450
OPC21322	400	10	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	20000	Grading B	Soil C	0.07	0.0038	137	0.068	0.225	100.7	1912450
OPC22122	1000	10	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	10000	Grading B	Soil A	0.02	0.0119	137	0.094	0.311	104.2	4781120
OPC22212	1000	10	3.5	S0.5	70-22	9	500000	INLAND	10	25000	A-1-b	15000	Grading B	Soil B	0.11	0.0125	131	0.095	0.286	103.1	4781120
OPC22221	1000	10	3.5	S0.5	70-22	9	500000	INLAND	14	30000	A-1-b	15000	Grading C	Soil C	0.04	0.0105	138	0.101	0.28	102.9	4781120
OPC22222	1000	10	3.5	S0.5	64-22	9	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.14	0.0119	139	0.117	0.312	104.2	4781120
OPC22222	1000	10	2	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0	0	75.6	0.047	0.263	101.7	4781120
OPC22222	1000	10	3.5	S0.375	70-22	9	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.07	0.008	18.8	0.098	0.292	102.4	4781120
OPC22222	1000	10	3.5	S0.5	70-22	9	200000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	7.28	0.261	137	0.102	0.324	105	4781120
OPC22222	1000	10	3.5	S0.5	70-22	8	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.41	0.0164	128	0.094	0.299	103.6	4781120
OPC22222	1000	10	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.11	0.0124	137	0.096	0.29	103.3	4781120
OPC22222	1000	10	3.5	S0.5	70-22	10	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.03	0.0099	148	0.098	0.281	103	4781120
OPC22222	1000	10	3.5	S0.5	70-22	9	1000000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.01	0.0005	137	0.099	0.274	102.6	4781120
OPC22222	1000	10	3.5	S1	70-22	9	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.12	0.0182	347	0.085	0.277	104.5	4781120
OPC22222	1000	10	5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.07	0.0166	84.9	0.108	0.284	102.7	4781120
OPC22222	1000	10	3.5	S0.5	76-22	9	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.09	0.0128	54.2	0.08	0.273	102	4781120
OPC22223	1000	10	3.5	S0.5	70-22	9	500000	INLAND	14	20000	A-1-b	15000	Grading A	Soil C	0.08	0.0117	142	0.098	0.293	103.5	4781120
OPC22232	1000	10	3.5	S0.5	70-22	9	500000	INLAND	18	25000	A-1-b	15000	Grading B	Soil B	0.11	0.0122	137	0.096	0.293	103.4	4781120
OPC22322	1000	10	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	20000	Grading B	Soil C	0.25	0.0126	137	0.097	0.273	102.7	4781120
OPC23122	2500	10	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	10000	Grading B	Soil A	0.09	0.0393	137	0.137	0.382	107	11952800
OPC23222	2500	10	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	15000	Grading B	Soil B	0.39	0.0408	137	0.139	0.356	106	11952800
OPC23322	2500	10	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	20000	Grading B	Soil C	0.92	0.0418	137	0.141	0.337	105.3	11952800
OPC31122	400	100	3.5	S0.5	70-22	9	500000	INLAND	14	25000	A-1-b	10000	Grading B	Soil A	0.01	0.0037	1630	0.052	0.278	115	1912450
OPC31222	400	100	3.5	S0.5	70-22	9	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.02	0.0039	1630	0.053	0.257	114.2	1912450
OPC31322	400	100	3.5	S0.5	70-22	9	500000	MOUNT	14	25000	A-1-b	20000	Grading B	Soil C	0.05	0.004	1610	0.05	0.234	113.1	1912450
OPC32122	1000	100	3.5	S0.5	70-22	9	500000	MOUNT	14	25000	A-1-b	10000	Grading B	Soil A	0.02	0.0125	1630	0.074	0.329	117.1	4781120
OPC32212	1000	100	3.5	S0.5	70-22	9	500000	MOUNT	10	25000	A-1-b	15000	Grading B	Soil B	0.08	0.0132	1630	0.075	0.299	115.9	4781120
OPC32221	1000	100	3.5	S0.5	70-22	9	500000	MOUNT	14	30000	A-1-b	15000	Grading C	Soil C	0.03	0.011	1630	0.079	0.284	115.3	4781120
OPC32222	1000	100	3.5	S0.5	64-22	9	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.11	0.0126	1620	0.094	0.322	116.8	4781120
OPC32222	1000	100	2	S0.5	70-22	9	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0	0	2110	0.039	0.291	119.4	4781120
OPC32222	1000	100	3.5	S0.375	70-22	9	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.05	0.0085	1180	0.077	0.305	112.5	4781120
OPC32222	1000	100	3.5	S0.5	70-22	9	200000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	7.1	0.261	1630	0.082	0.342	117.9	4781120
OPC32222	1000	100	3.5	S0.5	70-22	8	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.33	0.0173	1630	0.074	0.314	116.5	4781120
OPC32222	1000	100	3.5	S0.5	70-22	9	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.08	0.013	1630	0.075	0.303	116	4781120
OPC32222	1000	100	3.5	S0.5	70-22	10	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.02	0.0104	1630	0.077	0.292	115.6	4781120
OPC32222	1000	100	3.5	S0.5	70-22	9	1000000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.01	0.0006	1630	0.077	0.282	115.2	4781120
OPC32222	1000	100	3.5	S1	70-22	9	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.09	0.019	1990	0.066	0.291	118.4	4781120
OPC32222	1000	100	5	S0.5	70-22	9	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.06	0.0171	1510	0.084	0.291	114.6	4781120
OPC32222	1000	100	3.5	S0.5	76-22	9	500000	MOUNT	14	25000	A-1-b	15000	Grading B	Soil B	0.07	0.0134	1620	0.067	0.293	115.6	4781120
OPC32223	1000	100	3.5	S0.5	70-22	9	500000	MOUNT	14	20000	A-1-b	15000	Grading A	Soil C	0.06	0.0122	1640	0.077	0.299	115.9	4781120
OPC32232	1000	100	3.5	S0.5	70-22	9	500000	MOUNT	18	25000	A-1-b	15000	Grading B	Soil B	0.09	0.0129	1630	0.076	0.306	116.1	4781120
OPC32322	1000	100	3.5	S0.5	70-22	9	500000	MOUNT	14	25000	A-1-b	20000	Grading B	Soil C	0.2	0.0133	1630	0.077	0.282	115.2	4781120