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RESEARCH REPORT

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Study of the Effectiveness of ITD Pavement Design Method

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16. Abstract
The current ITD design method for flexible pavement is an empirical procedure based on R-value of the subgrade. There are claims that this design method is conservative and that ITD's calculated Equivalent Single Axle Loads (ESALs) are also conservative compared to the surrounding states. Thus, the main objective of this research was to compare the pavement design using the ITD's current design method with the pavement design using the AASHTO 1993 design guide and the Mechanistic-Empirical Pavement Design Guide (MEPDG). Furthermore, the research aimed to evaluate the methods of ESAL calculations as well as the traffic volume projection methods that are adopted in Idaho. A total of 8 in-service pavement sections located in different regions in the state were selected in coordination with ITD engineers. The key factor was the availability of design information for these sections. For each pavement section, the ITD design was checked, and the section was redesigned using the AASHTO 1993 design guide and MEPDG. All designs using AASHTO 1993 and MEPDG were developed at 50 and 85 percent reliability levels. The nationally calibrated MEPDG software (Version 1.1) was used to predict the performance of the 3 design methods. Design inputs for MEPDG were established as Level 2 for Hot Mix Asphalt and subgrade materials. All other MEPDG inputs were selected as Level 3. To evaluate the designs by various methods, MEPDG was used to predict performance for the pavement sections designed by the three methods, and then compared to each other. Also, the performance predicted by MEPDG was compared to actual performance to the extent of performance data available in ITD pavement management system. For ESAL and traffic evaluation, ITD's ESAL calculation method and traffic volume projection methods were studied, analyzed, and compared with other states. New truck factors were developed for ITD based on the analysis of weight-in-motion (WIM) data located in Idaho. ITD climatic factors were also analyzed and compared with MEPDG. Results showed that, relative to AASHTO 1993 and MEPDG procedures, ITD design method leads to thicker unbound layer(s) than the other methods. On the other hand, the AASHTO 1993 and MEPDG guides show reasonable agreement on the resulting pavement structure. ITD's current truck classification system and truck factors yield highly conservative ESALs compared to other state factors as well as regional and statewide factors developed. Finally, current ITD climatic zones are not consistent and yield different stresses based on MEPDG analysis.

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4		mm
ft	feet	0.3048		m
yd	yards	0.914		m
mi	Miles (statute)	1.61		km
		AREA		
in^2	square inches	645.2	millimeters squared	cm^2
ft^2	square feet	0.0929	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
mi^2	square miles	2.59	kilometers squared	km^2
ac	acres	0.4046	Hectares	ha
		MASS (weight)		
oz	Ounces (avdp)	28.35	Grams	g
lb	Pounds (avdp)	0.454	Kilograms	kg
T	Short tons (2000 lb)	0.907	Megagrams	mg
		VOLUME		
fl oz	fluid ounces (US)	29.57	Milliliters	mL
gal	Gallons (liq)	3.785	Liters	liters
ft^3	cubic feet	0.0283	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3

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

Symbol	When You Know	Multiply By	To Find	Symbol
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (°F-32)	Celsius temperature	°C
		ILLUMINATION		
fc	Foot-candles	10.76	Lux	lx
fl	foot-lamberts	3.426	candela/m^2	cd/cm^2
		FORCE and PRESSURE or STRESS		
lbf	pound-force	4.45	newtons	N
psi	pound-force 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 (statute)	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
km^2	kilometers squared	0.39	square miles	mi^2
ha	hectares (10,000 m^2)	2.471	acres	ac
		MASS (weight)		
g	grams	0.0353	Ounces (avdp)	oz
kg	kilograms	2.205	Pounds (avdp)	lb
mg	megagrams (1000 kg)	1.103	short tons	T
		VOLUME		
mL	milliliters	0.034	fluid ounces (US)	fl oz
liters	liters	0.264	Gallons (liq)	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 °C+32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/cm^2	candela/m^2	0.2919	foot-lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	pound-force	lbf
kPa	kilopascals	0.145	pound-force per square inch	psi

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List of Acronyms

AADT	Annual Average Daily Traffic
AADTT	Annual Average Daily Truck Traffic
ADTT	Average Daily Truck Traffic
AASHO	American Association of State Highway Officials (predecessor to AASHTO)
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ALS	Axle Load Spectra
CAADT	Commercial Annual Average Daily Traffic
Caltrans	California Department of Transportation
CBR	California Bearing Ratio
CF	Climatic Factor
CI	Cracking Index
DHV	Design Hourly Volume
DOT	Department of Transportation
ESALs	Equivalent Single Axle Loads
EQL	Equivalent Wheel Load
FHWA	Federal Highway Agency
GE	Gravel Equivalence
G_f	Gravel Factor (Substitution Ratio)
HMA	Hot Mix Asphalt
ITD	Idaho Transportation Department
LCCA	Life Cycle Cost Analysis
LEF	Load Equivalency Factor
LTPP	Long Term Pavement Performance
MEPDG	Mechanistic-Empirical Pavement Design Guide
PPMIS	Pavement Performance Management Information System
RD	Rut Depth
RI	Roughness Index
SHAs	State Highway Agencies
SN	Structure Number
TTC	Truck Traffic Classification
TWRG	Truck Weight Road Group
UDOT	Utah Department of Transportation
UI	University of Idaho
WASHO	Western Association of State Highway Officials
WIM	Weigh-In-Motion
WSDOT	Washington State Department of Transportation

Executive Summary

Introduction

The current flexible pavement design method adopted by the Idaho Transportation Department (ITD) is an empirical procedure based on the California Department of Transportation (Caltrans) R-Value method. In this method, traffic is characterized in terms of Traffic Index, which is a function of the design 18-kip Equivalent Single Axle Load (ESAL). A Climatic Factor (CF) is used to reflect the various geographical regions in the State of Idaho. ITD's procedure determines the pavement thickness as a Gravel Equivalence (GE) based on an empirical equation. The GE is then transferred to various layer thicknesses through a Gravel factor (G_f) for each type of material. Observations in Idaho showed that many existing roadways that have been designed with ITD's design method have performed well beyond their design lives, and still perform adequately. Furthermore, based on information from adjacent states, ESALs calculated by ITD are extremely conservative.

This research investigated the current ITD design method and compared it with both American Association of State Highway and Transportation Officials (AASHTO) 1993 and the Mechanistic-Empirical Pavement Design Guide (MEPDG) procedures. In addition, the current ITD truck factors and traffic volume projection methods as well as Idaho climatic factors were also investigated.

Several in-service pavement sections located in different districts and designed according to ITD design method were identified. These pavement sections were redesigned using AASHTO 1993 and MEPDG procedures. All designs using AASHTO 1993 and MEPDG were conducted at 50 and 85 percent reliability levels. The nationally calibrated MEPDG software was used to predict the performance of the three design alternatives. Level 2 Hot Mix Asphalt (HMA) and subgrade material characterization inputs were used in the MEPDG analysis. All other MEPDG inputs were Level 3. Pavement distresses and smoothness predicted using MEPDG related to the three design methods were compared to each other. In addition, MEPDG predicted performance was compared to measured field performance. The measured field performance was obtained from ITD's Pavement Performance Management Information System (PPMIS). Furthermore, a comparison was made between ITD's and other states' ESAL calculations. Additionally, current ITD truck factors were compared with truck factors developed for Idaho from the analysis of traffic Weigh-In-Motion (WIM) data. Moreover, the accuracy of the ITD traffic volume projection method was investigated. Finally, ITD climatic factors were analyzed and compared with MEPDG climatic data.

Research Methodology

This project was conducted in eight major tasks. The following tasks were conducted:

Task 1: Reviewed other state agencies design procedures, focusing on the western states.

Task 2: Obtained and reviewed selected states' design methodologies and the latest version of MEPDG.

Task 3: Identified and selected one or two projects in each ITD district for analysis.

Task 4: Analyzed ITD traffic data to determine accuracy.

Task 5: Analyzed material properties to determine basic design parameters.

Task 6: Analyzed climatic factors for the State of Idaho.

Task 7: Analyzed and re-designed recruited pavement sections using AASHTO 1993, and MEPDG.

Task 8: Evaluated pavement performance using MEPDG.

This report documents all research work conducted under these tasks for ITD.

Key Findings

The key findings of this research work are summarized below:

- The unbound granular layer(s) thickness(s) resulting from the ITD design method were much thicker (2 to 4.5 times as thick) compared to the AASHTO 1993 and MEPDG designs.

- The 3 design methods yielded reasonably similar thickness for the Asphalt Concrete (AC) layer at 50 percent reliability. However, at higher reliability levels, MEPDG yielded thicker AC thickness compared to both methods, especially in the case of very weak subgrade strength.

- ITD truck factors used in ESAL calculations are more conservative compared to other state DOTs and AASHTO factors.

- Truck traffic data obtained from WIM sites in Idaho were analyzed to develop regional and statewide truck factors for Idaho. Researchers found that the current ITD truck factors are highly conservative compared to the developed regional and statewide factors. Furthermore, ITD's current truck classification is based on the classical Equivalent Wheel Load (EWL) and does not accurately represent the current truck traffic classifications.

- Comparison between total rutting predicted using MEPDG and the actual measured rutting for the investigated projects revealed that the nationally calibrated rutting models in MEPDG are significantly over predicting the total rutting.

- ITD's current cracking rating method measures and reports cracking differently compared to MEPDG required distress survey method.

- MEPDG climatic inputs are much more comprehensive compared to empirical ITD climatic factors. In addition, ITD climatic zones were found to be inconsistent with MEPDG. When MEPDG was run on sections located in the same climatic zones it yielded different distresses.

Conclusions

The main conclusion of this research is that the current ITD design method for flexible pavement structures yields highly conservative pavement structures compared to the widely used AASHTO 1993 design method and the newly developed MEPDG procedure. In addition, current ITD truck factors and truck traffic classification yield highly conservative values compared to other states' factors as well as factors developed from the analysis of Idaho WIM data.

Recommendations

Based on the findings of this research, the following recommendations are offered:

- ITD should continue with the implementation and calibration of MEPDG in Idaho to replace its current design method as soon as practical.
- To ensure consistency with MEPDG distress prediction, ITD should consider performing pavement condition surveys in accordance with the Long-Term Pavement Performance (LTPP) method of data collection.
- ITD should adopt the truck factors that were developed in this study, and regularly update them using Idaho WIM site data.
- ITD should consider changing its current truck classification system, which was based on the EWL principals. The Federal Highway Administration (FHWA) truck classification system, or a simplified system based on it, should be used.
- ITD should consider replacing its current method for projecting future traffic volume needs, as it consistently over predicts traffic volume. There are several traffic forecasting methods that ITD may investigate. These methods include: time series forecasting, regression, clustering, and neural networks.

Chapter 1
Introduction

Background

The majority of the State Department of Transportation (DOTs) are currently using different versions of the AASHTO method for pavement structure design. The AASHTO methods are empirical methods based on relationships between traffic loading, materials, and pavement serviceability developed from the AASHO Road Test in the late 1950s.(1) ITD uses an empirical design procedure adapted from Caltrans.(2) These empirical procedures for pavement structure design have many limitations and concerns regarding climate, traffic, materials, and pavement performance. In fact, many existing pavement sections that have been designed with the ITD design procedure were found to perform beyond their design lives, and are still performing adequately. This raises the question of the cost effectiveness of the ITD design procedure, especially with the development of MEPDG.

Problem Statement

With limited funding, there is increased emphasis on building structurally adequate, yet cost effective, flexible pavements. Current flexible design methods range from empirical designs based on data from the 1950s AASHO Road Test, to methods developed by FHWA, ITD, University of Idaho (UI), and from other states.(1) In addition, the newly developed MEPDG is now available. Based on information from surrounding states, there may be evidence that ITD's calculated ESALs for design are extremely conservative. Many existing roadways have performed beyond their calculated design lives, and still perform adequately. ITD needs to evaluate existing design methodologies to determine if they are still applicable to current needs or if modifications can improve performance and reduce costs.

Research Objectives

The key objectives of this research project were to:

1) Evaluate the current ITD flexible pavement top-down design method and design methods from selected other states against the MEPDG analysis tool.
2) Review selected district projects designed with ITD's top-down design procedure, for performance and longevity. Evaluate performance using MEPDG and compare the predicted performance to the actual performance in the ITD pavement management database, where available.
3) Review the current ITD ESAL calculation and traffic volume projection methods and methods from other states. Provide recommendations for any proposed changes.
4) Evaluate the current ITD climatic factors.

Scope of Work

To investigate the current ITD design method, eight in-service flexible pavements designed using ITD's design method and located in different districts in Idaho, were identified. These pavement projects were redesigned using ITD's method and with both the AASHTO 1993 and MEPDG procedures at 2 different reliability levels.[3, 4] MEPDG predicted distresses and smoothness for the three design alternatives for each investigated project were compared to each other. Furthermore, pavement performance predicted using the nationally calibrated version of the MEPDG for the ITD in-service pavement sections was compared to actual measured field performance.

The current ITD ESAL calculation method was studied and compared with methods from the neighboring states. Moreover, statewide and regional truck factors were developed for Idaho based on analysis of traffic (WIM) data. These truck factors were compared with the current factors. The current ITD simple method for traffic projection was also evaluated. Finally, current ITD climatic factors and climatic zones were investigated and compared to MEPDG climatic weather stations in Idaho.

Report Organization

This report is organized in 7 chapters as described below:

Chapter 1 covers the problem statement, research objectives, and scope of work.

Chapter 2 presents a review of the current flexible pavement design practices in the U.S. It also provides an overview of the current ITD, AASHTO 1993, and MEPDG design procedures and major inputs required by each of these methods.

Chapter 3 presents the selected projects for analysis. The major inputs required by the investigated design methods for each project are presented. This chapter covers the redesign of each project using ITD's, AASHTO 1993 and MEPDG at different levels of reliability showing the results and analysis.

Chapter 4 investigates the actual field performance of the selected projects against MEPDG predicted performance. It also presents the variations of the current ITD distress survey compared to the requirements of MEPDG.

Chapter 5 studied and investigated current ITD ESAL calculation method and compares it with different state methods. It also presents the development of truck factors for Idaho using WIM data for a more precise traffic characterization.

Chapter 6 presents an evaluation of the current ITD climatic zones and factors in comparison with MEPDG climatic factors.

Finally, Chapter 7 summarizes the key findings of this research as well as recommendations for ITD.

References and several supporting appendices are included at the end of the report.

Chapter 2
Flexible Pavement Design Practice in the U.S.

Introduction

Pavement design is the process of determining the pavement layer thicknesses and the appropriate material properties. The designed pavement structure should safely and economically sustain the expected traffic loads and environmental conditions for the intended service life of the pavement. Several design methods for flexible pavements are currently practiced in the U.S. and around the world. These methods range from very simple empirical methods to more advanced and sophisticated mechanistic based methods. The empirical pavement design methods are generally based on empirical correlations that relate the design traffic to the pavement section and its material properties such as (R-value, California Bearing Ratio: CBR, layer coefficient, etc.). It is often based on local experience of observed performance and some engineering judgment. Most of the current pavement design procedures practiced in the U.S. and around the world are empirical procedures. Both ITD's and AASHTO 1993 methods belong to the empirical pavement design procedure category. These empirical design procedures have many limitations in terms of the characterization of materials, traffic, climate, and pavement performance. However, they worked well when computing capabilities were limited. On the other hand, the advanced pavement design methods are based on Mechanistic-Empirical (M-E) principals. The Asphalt Institute's method and MEPDG are examples of these design methods. M-E design methods rely on calculating stresses, strains, and deformations based on fundamental engineering mechanics. Then, the calculated stresses, strains, and deformations are transformed into field distresses such as fatigue cracking, thermal cracking, and rutting using empirical transfer functions. This method would not be possible without the computing power of today's computers.

This chapter presents an overview of the ITD design method for flexible pavement structures. It also covers the current flexible pavement design practice in the U.S. with the focus on the design methods used in the western states especially Idaho's neighboring states.

Current Flexible Pavement Design Practices in the U.S.

Literature searches showed that the current design practices for flexible pavements in the U.S. include the following methods:

- AASHTO 1972.
- AASHTO 1993.
- State Procedures (ITD's and Caltrans' design methods, for example).
- Combination of AASHTO and State procedures.
- MEPDG for forensic analysis and comparison studies.

A survey was conducted in 2007 regarding MEPDG.[5] This survey included 65 questions sent to the Department of Transportation (DOTs) addressing their current design procedures, MEPDG knowledge, implementation activities, partnering activities, and training needs.[5] The 50 state DOT responders showed that, 63 percent use the 1993 AASHTO Pavement Design Guide, 12 percent use the 1972 AASHTO Design Guide, 13 percent use individual state design procedures, 8 percent use a combination of AASHTO and state procedures, and the remaining use other design procedures.[5] This distribution is shown in Figure 1.

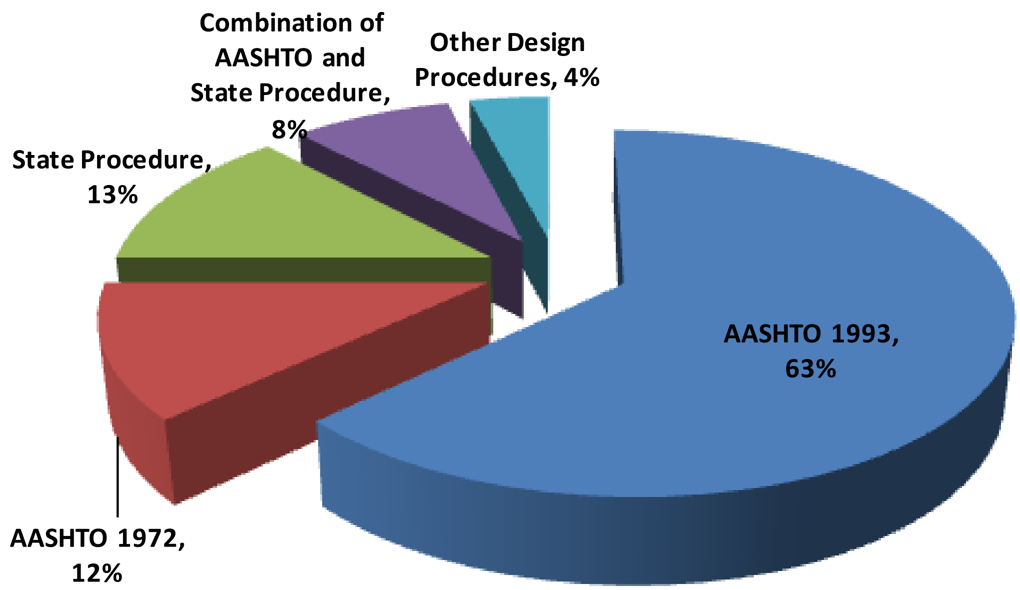

Figure 1. 2007 Survey Results of the States Current Design Practices.[5]

The same survey results showed that, about 80 percent of the DOTs stated that they have plans to implement MEPDG.[5, 6] An older FHWA survey that was completed in 2003 showed at that time, only 42 percent of the DOTs had implementation plans for MEPDG.[7] This means that MEPDG is gaining more attention with time. Figure 2 illustrates the DOTs, in 2007 that had implementation plans for MEPDG.

It should be noted that Idaho is one of the states that has an implementation plan for MEPDG. In fact, ITD contracted with the University of Idaho to do a research project to evaluate the implementation of MEPDG in Idaho.[8] Another research project between ITD and UI has been proposed to calibrate the MEPDG distress models for Idaho conditions.

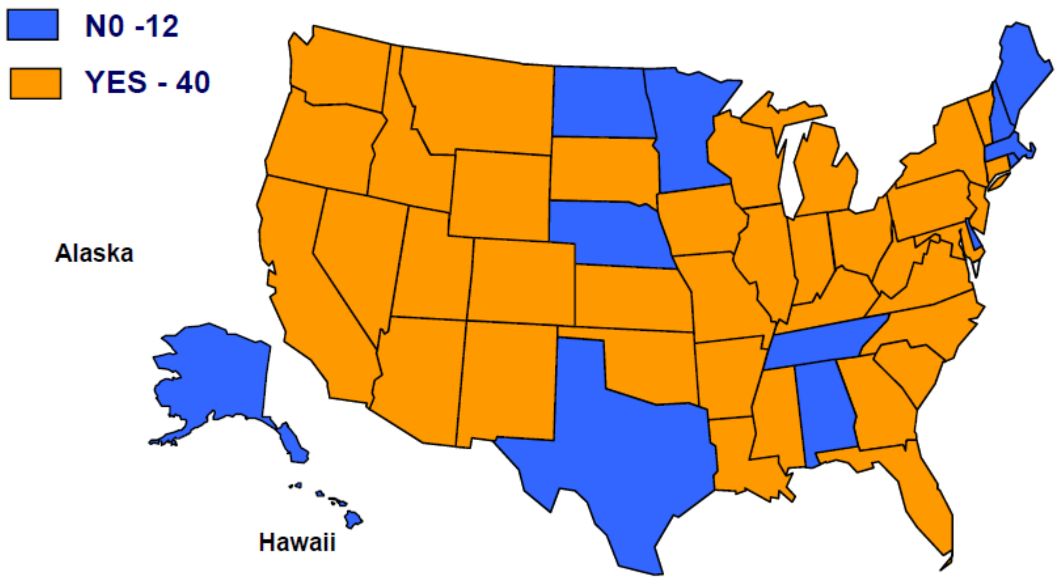

Figure 2. States with MEPDG Implementation Plans[5]

Current Flexible Pavement Design Practice in the Western States

Table 1 shows the flexible pavement design methods currently practiced in the western United States.[7] This table clearly shows that the AASHTO 1993 design procedure is the most practiced within the western states and especially states neighboring Idaho.

Based on the presented survey results, it was concluded that the AASHTO 1993 is the most widely used design method in the U.S. It is also the most practiced method in the western states and those surrounding Idaho. The same survey results showed that 80 percent of the DOTs, including Idaho, have MEPDG implementation plans in place.[6]

Based on the results from the literature searches, it was decided to evaluate ITD's flexible pavement design method against AASHTO 1993 and MEPDG design methods.

Table 1. Western States Current Design Practice[7]

State	Current Pavement Design Method
Arizona	AASHTO 1993
California	State Procedure
Colorado	AASHTO 1993
Montana	AASHTO 1972 & 1993
Nevada	AASHTO 1993
New Mexico	Combination of AASHTO 1972 and State Procedures
Oregon	Combination of AASHTO 1993 and State Procedures
Utah	AASHTO 1993
Washington	AASHTO 1993
Wyoming	AASHTO 1993

Overview of Idaho Flexible Pavement Design Method

The current flexible pavement design method adopted by ITD is an empirical procedure based on the R-value of the subgrade.[2] It incorporates traffic in terms of Traffic Index (TI) which is a function of the design 18-kip Equivalent-Single Axle Load (ESAL).[16, 17] It also incorporates Climatic Factors (CF) to reflect the various geographical regions within Idaho. ITD's procedure determines the pavement thickness as a Gravel Equivalence (GE) based on an empirical equation adopted from the California Department of Transportation (Caltrans). The GE is then transferred to various layer thicknesses through a gravel factor (G_f) for each type of material. The minimum design standards for this design method are based on recommendations of Caltrans, AASHTO, Asphalt Institute, and local experience.[2] It should be noted that ITD design method is a deterministic design method (i.e., it does not incorporate reliability into design). However, it incorporates factors of safety on the GE factors.

The benefit of ITD's design method is that it can be considered a perpetual pavement design concept. The ballast section is thick enough to allow major rehabilitation in the surface layer only without the need for full depth reconstruction.

ITD Design Method Required Inputs

The major inputs required by ITD design method are as follows:

- Design life (of at least 20 years).
- Traffic in terms of Traffic Index which is a function of the 18-kip ESAL.
- Resistance values (R-values) for the supporting layers (base, subbase, and subgrade).

- Climatic factor (CF) which is based on the geographical location of the project.
- Gravel factors (G_f) for various layers. Gravel factor is an empirical factor that relates the thickness of a layer to its equivalent thickness of gravel.

More detail on the ITD's design method can be found in the ITD's *Materials Manual*.[2]

Overview of AASHTO 1993 Pavement Design Method

This empirical design procedure is based on the results of the AASHO road test conducted in Ottawa, Illinois, in the late 1950s and early 1960s.[3] The first design guide was published in 1961 and was revised in 1972, 1981, and 1986, and 1993. The empirical performance equations obtained from the road test under certain traffic, climatic and subgrade conditions are used to compute the pavement layer thickness. The various versions of the AASHTO design guides have served well for several decades.[18] However, deficiencies and limitations associated with the AASHTO 1993 design guide motivated the development of MEPDG.[4]

AASHTO 1993 Design Method Required Inputs

The major inputs required by the AASHTO 1993 design method are as follows:

- Design life.
- Serviceability.
- Reliability and overall standard deviation.
- Traffic in terms of 18-kip ESAL.
- Stiffness (resilient moduli) of the supporting layers.
- Structural layers coefficients (a_i).
- Drainage coefficients (m_i).

The AASHTO 1993 recommended design reliability levels are shown in Table 2.

Table 2. AASHTO 1993 Recommended Reliability Levels[3]

Functional Classification	Recommended Level of Reliability	
	Urban	Rural
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

Major assumptions for AASHTO design method regarding the design reliability levels, unbound granular base/subbase materials and subgrade soils characterization methods, conversion equation for the resilient modulus (M_r) utilized in some of the western states are summarized in Table 3.

Table 3. Major Assumptions for AASHTO Design Method Utilized in Selected Western States

State	Flexible Pavement Design Method	Design Reliability Level	Primary Unbound Materials and Subgrade Soils Property	Conversion Equation if M_r is not Measured	Reference Number
Arizona	AASHTO 1993	Interstates : 99% ≥ 10,000 ADT : 95% 2,001-10,000 ADT: 90% 501-2,000 ADT : 85% ≤ 500 ADT : 75%	R-value	$M_r = \frac{1815 + 225(\text{Rmean}) + 2.4(\text{Rmean})^2}{0.6(\text{SVF})^{0.6}}$	9
Colorado	AASHTO 1993	Reliability selected Based on the Functional Class	R-Value	$M_r = 10^{[(SSV+18.72)/6.24]}$ $SSV=[(R-5)/11.29]+3$	10
Montana	AASHTO 1993	Reliability selected Based on the Functional Class	R-Value, M_r	$M_r = -0.51 (R)^2+297 (R) + 3292$	11
New Mexico	AASHTO 1972 and State Procedure	No Reliability (50%). However, it uses @-risk software to calculate design R-value, ESAL, and AC thickness.	R-value	Convert R-value to SSV Not M_r	12
Oregon	AASHTO 1993 and State Procedure	AASHTO Recommendations based on Functional Class	M_r form lab or correlation with DCP	$M_r =49023(C_f)(DCP)^{-0.39}$	13
Utah	AASHTO 1993	Interstates: 95% Others : 90%	CBR	$M_r = 1500(CBR)$	14
Washington	AASHTO 1993	ESAL < 10,000,000: 85% ESAL ≥10,000,000: 95%	Currently: M_r Historically :CBR until 1951, then R-value	-	15

ADT = average daily traffic; M_r = resilient modulus, psi; CBR = California bearing ratio
R = R-value; DCP = dynamic cone penetrometer, mm/blow
C_f = correction factor; SSV = soil support value; R_{mean} = weighted average R-value
SVF = seasonal variation factor

Overview of MEPDG Pavement Design Method

MEPDG is a state-of-the-art tool for the analysis/design of new and rehabilitated pavement structures based on mechanistic-empirical principles. The flexible pavement portion of the software mechanistically calculates the structural response (stresses, strains, and deflections), within a pavement system, using the multi-layer elastic theory or finite element analysis.[4] Moisture and temperature variations within the pavement structure are calculated internally using the Enhanced Integrated Climatic Model (EICM). EICM utilizes a comprehensive database from 851 weather stations throughout the United States. Pavement distresses (rutting, bottom-up and top-down fatigue cracking, thermal cracking) and roughness are predicted using empirical transfer functions. In the current software version of the MEPDG (version 1.1), these transfer functions are nationally calibrated based on field data from

94 LTPP sections distributed all over the U.S. The software also allows users to input their calibration coefficients to reflect certain conditions.

MEPDG Required Inputs

Unlike ITD and AASHTO 1993, MEPDG requires an extensive amount of input data. It requires 100+ inputs in order to design/analyze a pavement section. MEPDG also utilizes hierarchical levels of the design inputs. This feature provides the user with flexibility in obtaining the design inputs of the project based on its importance and anticipated funding cost. The inputs for the MEPDG may also be obtained using a mix of the three hierarchical levels. The MEPDG three levels of inputs regarding traffic and material properties are as follows.

- Level 1: represents the highest level of accuracy and lowest level of errors for the inputs. Input parameters for this level must be measured directly either in the laboratory or in the field. This level of input has the highest cost in testing and data collection.
- Level 2: represents an intermediate level of accuracy. Parameters are estimated from built-in-correlations based on limited routine laboratory test results or selected from an agency database.
- Level 3: represents the lowest level of accuracy. Usually, typical default values (best estimates) of input parameters are used in this level.

The main inputs of the MEPDG are divided into four main categories. These categories are project, traffic, climate, and structure. More details regarding these inputs can be found in the NCHRP 1-37A Report and the Manual of Practice.[4, 19] The MEPDG recommend levels of reliability for different functional classification of roadway are presented in Table 4.

Table 4. MEPDG Recommended Reliability Levels[19]

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

Table 5 presents the performance criteria (threshold values) recommended for use with the MEPDG for flexible pavement design based on the roadway functional class.

Table 4. MEPDG Recommended Design Criteria[19]

Distress	Threshold Value at Design Reliability
Terminal IRI (in./mile)	Interstate: 160 Primary: 200 Secondary: 200
Alligator Cracking (percent lane area)	Interstate: 10 Primary: 20 Secondary: 35
Thermal Fracture (Transverse Cracking) (ft/mile)	Interstate: 500 Primary: 700 Secondary: 700
Total Rutting (in.)	Interstate: 0.40 Primary: 0.50 Others < 45 mph: 0.65

Chapter 3
Comparison of Idaho Flexible Pavement Design Procedure with AASHTO 1993 and MEPDG Methods

Introduction

The previous chapter presented a literature review of the flexible pavement design methods practiced in the U.S. as well as an overview of ITD's, AASHTO 1993, and MEPDG design methods. It was found in the literature that AASHTO 1993 design method is the most practiced design method by DOTs across the U.S., especially in the western states.[5, 6, 7] This chapter presents a comparison of ITD's design method to AASHTO 1993, and MEPDG methods using real in-service pavement sections designed according to ITD's design method.

Selected Projects

The original work plan called for identifying 1 or 2 projects from each of the 6 districts in Idaho. However, ITD was only able to provide UI's research team with projects from Districts 1, 2, 3, and 6. The total number of the recruited projects was 8. There was 1 project from District 4 presented, however the data was incomplete and did not allow for analysis. There were no projects from District 5. The 8 recruited projects are shown in Table 5. The US-2 project (from Wrenco Loop to Dover) has 3 different sections with the same design inputs with the exception of subgrade strength. This allows for direct evaluation of the influence of the subgrade strength on the designed pavement structure using the three pavement design methods considered in this study.

Table 5. Selected Projects

Project	US-2 (a) Wrenco Loop to Dover (MP 22.01-MP 22.32)	US-2(b) Wrenco Loop to Dover (MP 22.32-MP 23.91)	US-2(c) Wrenco Loop to Dover (MP 23.91-MP 24.99)	SH-62 Oak Street	SH-3 Arrow to Turkey Farm	SH-19 Greenleaf to Simplot	US-95 Devil's Elbow	US-93 Tom Cat Hill, East
Project No.	F-5121(019)	F-5121(019)	F-5121(019)	ST-4749(612)	STP-4170(101)	STP-RS 3712(008)	F-3112(42)	ST-6350(652)
Key No.	0717	0717	0717	9338	5956	0135	2224	7768
County	Bonner	Bonner	Bonner	Lewis	Nez Perce	Canyon	Washington	Butte
District	1	1	1	2	2	3	3	6
Functional Class*	Principal Arterial-Other (Rural)	Principal Arterial-Other (Rural)	Principal Arterial-Other (Rural)	Major Collector (Rural)	Minor Arterial (Rural)	Minor Arterial (Rural)	Principal Arterial-Other (Rural)	Principal Arterial-Other (Rural)
AC Layer Thickness (in.)**	6.0	6.0	6.0	4.2	5.4	4.2	3.6	6.0
Granular Base Layer Thickness (in.)**	6.0	6.0	6.0	22.2	19.2	5.4	6.0	9.0
Granular Subbase Layer Thickness (in.)**	26.4	9.0	30.0	-	-	12.0	30.0	6.0

* From Idaho State Highway Functional Classification 2015 Map.[20]

** These values represent the constructed typical section.

ITD Pavement Performance Management Information System

ITD is using Arizona's method to evaluate pavement surface distresses.[21] This distress survey is conducted visually on the most travelled lane. ITD measures its pavement performance based on roughness index (RI), cracking index (CI), skid number, and rut depth (RD).[21] In order to determine the RI, the International Roughness Index (IRI) is first measured in units of (in./mile) using a laser mounted on the Profiler Van. The measured IRI values are then compressed into 0.0 to 5.0 scale, where 0.0 is a very rough and 5.0 is a very smooth pavement surface. The skid data is collected by towing a small trailer that measures force on a wheel that is locked and not rotating. The skid number ranges from 20 to 200 with a threshold value less than 35.

A crack index value between 0.0 and 5.0 is given to the pavement, based on the size and location of cracks, percentage of the roadway surveyed that shows distresses, and type of road surface. A rating of

5.0 is good pavement with no visible cracking and 0.0 is for pavement with the maximum distress classification. It should be noted that the CI represents all types of cracks occurring in the pavement together (alligator, longitudinal, and transverse). In collecting this distress survey data, ITD started using Pathway © Profiler Van technology to gather the majority of their roadway data since 1995.(21) Profiler Vans drive over the pavement and produce digital images of the pavement surface across the width and length of the roadway segment being evaluated. The rutting is measured using rutting detection scanning lasers attached to the profiler vans. ITD considers the pavement to be deficient based on the RI or CI and the functional class of the roadway according to the criteria shown in Table 6.

Table 6. ITD Pavement Deficiency Criteria(21)

Pavement Condition	Interstates and Arterials	Collectors
Good	(CI or RI) > 3.0	(CI or RI) > 3.0
Fair	2.5 ≤ (CI or RI) ≤ 3.0	2.0 ≤ (CI or RI) ≤ 3.0
Poor	2.0 ≤ (CI or RI) < 2.5	1.5 ≤ (CI or RI) < 2.0
Very Poor	(CI or RI) < 2.0	(CI or RI) < 1.5

Performance Indicators for the Selected Pavement Sections

ITD's online Pavement Performance Management Information System (PPMIS) was used to find the performance of these pavement sections.(22) Table 7 shows the performance indicators of the investigated projects as of 2008. The data in this table shows good performance for almost all investigated projects. Even though the US-95 pavement section is near the end of its service life, it rates at a fair condition. In addition, the 2009 statewide pavement condition for Idaho shows that 60 percent of the roadway network is in good condition, 22 percent in fair condition, and only 18 percent of the roadway network fall in the poor to very poor category.(21) This is shown in Figure 3.

Table 7. Performance Indicators of the Investigated Projects as of 2008

Design Project	US-2(a)	US-2(b)	US-2(c)	SH-62	SH-3	SH-19	US-95	US-93
Years in Service	6.0	6.0	6.0	New	10.0	11.0	18.0	2.0
RI	4.7	3.4	4.1	-	3.1	3.9	3.5	3.7
CI	5.0	4.7	5.0	-	5.0	5.0	3.0	5.0
Skid Number	44.0	44.0	44.0	-	51.0	4.0	47.1	57.0
Rut Depth (in.)	0.23	0.16	0.24	-	0.17	0.14	0.18	0.10

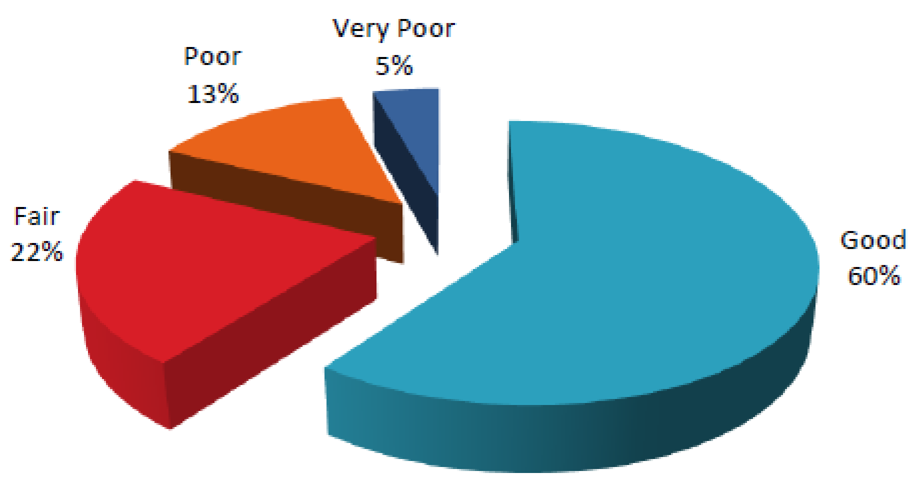

Figure 3. 2009 Statewide Pavement Condition[21]

Analysis Procedure

The projects presented previously in Table 5 represent in-service pavement sections that were designed according to the ITD's design method. These projects were redesigned using the AASHTO 1993 and MEPDG (version 1.10) procedures in addition to redesigning ITD to compare with the documented structure. Since ITD's flexible pavement design method is a deterministic method, (does not incorporate reliability into design), all designs were conducted at 50 percent reliability level using both AASHTO 1993 and MEPDG procedures. The investigated projects were also redesigned using AASHTO 1993 and MEPDG methods at 85 percent reliability level. The 85 percent reliability level analyses were conducted in order to compare ITD's method with AASHTO 1993 and MEPDG guides at a higher reliability level. This higher reliability level is the recommended design reliability for the analyzed projects if AASHTO or MEPDG design methods are used.

Both ITD and AASHTO 1993 design methods use one representative value for the strength characterization for each of the unbound granular layers and subgrade. Additionally, for traffic characterization ITD uses TI which is a function of ESAL. AASHTO 1993 uses ESAL while MEPDG uses Axle Load Spectra (ALS). Thus, in order to simulate ITD and AASHTO 1993, MEPDG's Enhanced Integrated Climatic Model (EICM), which adjusts the resilient modulus of the unbound materials and subgrade soils, was deactivated. The MEPDG representative modulus option was then used in all MEPDG analyses. Moreover, MEPDG traffic data was characterized in terms of ESALs instead of axle load spectra. This was made because it compares all design methods on the same assumptions.

The nationally calibrated MEPDG (version 1.1) was also used to predict the performance of the investigated projects designed using the 3 design methods. The predicted performance includes alligator cracking, rutting, and IRI.

Project Input Data for Each Design Method

Input data as well as the design criteria vary significantly among the three design methods. To redesign each of the investigated projects using the selected design methods, data was collected from several sources. ITD design phase reports as well as Google Earth, ITD's PPMIS, ITD highway information through the ITD's official website, and the United States Geological Survey (USGS) website were used.[23, 24, 25] Reasonable assumptions were made for some of the input parameters where information was not available. The following subsections summarize the major design inputs for each studied project for each of the three design methods.

ITD's Inputs for the Investigated Projects

ITD's input data required for the investigated projects are shown in Table 8. This data was the easiest and most straight forward as they are available in the ITD phase reports and design sheets recruited from ITD. Design details of these projects are shown in Appendix A. These designs were done by ITD.

Table 8. Design Input Data for ITD Design Method

Design Input	Design Project							
	US-2(a)	US-2(b)	US-2(c)	SH-62	SH-3	SH-19	US-95	US-93
Design Life, years	20	20	20	20	20	20	20	20
Traffic Index	11.51	11.51	11.51	8.78	10.5	9.57	9.9	10.27
Climatic Factor	1.1	1.1	1.1	1.1	1.0	1.0	1.0	1.05
Base R-Value*	80	80	80	80	80	80	80	80
Subbase R-Value*	65	65	65	-	-	65	65	60
Subgrade R-Value**	20	50	15	8	25	44	5	50

*Estimated values
**Laboratory measured values

AASHTO 1993 Inputs for the Investigated Projects

Table 10 summarizes the AASHTO 1993 Input data required for the investigated projects. For all of the investigated projects, initial and final serviceability values of 4.5, and 2.5, respectively, were assumed. The structural layer coefficients (a_i) were estimated based on the modulus of each layer as recommended by the design method. The primary unbound granular base/subbase materials and subgrade soils characterization input required by AASHTO 1993 and MEPDG methods is the resilient modulus. The resilient modulus values of the base and subbase layers were estimated based on the assumed R-values for these layers. For the resilient moduli of the subgrade layers, the laboratory measured design R-values were used to estimate them using the Asphalt Institute (AI) equation. The AASHTO 1993 and MEPDG guides recommend the use of this equation to convert R-values to M_r.[3, 4] This equation is given in Figure 4 as follows:[26]

$$M_r = 1155 + 555\,(R)$$

where:

M_r = resilient modulus, psi
R = R-value

Figure 4. Asphalt Institute Equation to Estimate Resilient Modulus from R-Value

ITD uses an approximate relationship between M_r and R-value for the subgrade soils. This relationship is shown in Figure 5.(2) ITD's M_r-R-value relationship was not used in this analysis. The AI equation was used instead as it is recommended by AASHTO and MEPDG.(4) In addition, ITD's M_r-R-value relationship yields conservative results (lower R-values).

$$\text{Log } M_r = (222 + R) / 67$$

where:

M_r = resilient modulus, psi

R = R-value

Figure 5. ITD Equation to Estimate Resilient Modulus from R-Value

As previously mentioned, all performed designs using the AASHTO 1993 method were performed at 2 different reliability levels of 50 percent and 85 percent.

Table 10. Design Input Data for AASHTO 1993 Method

Design Input	Design Project							
	US-2(a)	**US-2(b)**	**US-2(c)**	**SH-62**	**SH-3**	**SH-19**	**US-95**	**US-93**
Design Reliability, %	50% & 85%							
Design ESALs	7,920,000	7,920,000	7,920,000	816,000	3,696,000	1,677,000	2,240,000	3,034,000
ΔPSI (Loss of Serviceability)	2	2	2	2	2	2	2	2
Structural Layer Coefficients (a_i)								
a_1	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44
a_2	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17
a_3	-	-	-	-	-	-	0.19	-
Drainage Coefficients (m_i)								
m_2	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
m_3	-	-	-	-	-	-	1.2	-
M_r (psi), Base*	38,000	38,000	38,000	38,000	38,000	38,000	38,000	38,000
M_r (psi), Subbase*	-	-	-	-	-	-	32,000	-
M_r (psi), Subgrade**	12,255	28,905	9,480	5,595	15,030	25,575	3,930	28,905
M_r (psi), Subgrade***	4092	11473	3446	2710	4860	9336	2444	11474

* Estimated Values

** Estimated from measured R-values using the Asphalt Institute equation given in Figure 4 and used in the analysis.

* ** Estimated from measured R-values using the Asphalt Institute equation given in Figure 5 and was not used in the analysis.

MEPDG Inputs for the Investigated Projects

Most of the inputs used in MEPDG for the investigated project are considered Level 3 inputs. For the Hot Mix Asphalt (HMA) material characterization, the dynamic modulus (E*) is the primary input. Level 2 input data which includes some gradation parameters as well as volumetric mix parameters was used for the HMA characterization for all investigated projects as shown in Table 11. The NCHRP 1-37A viscosity-based model incorporated in to the software was selected to estimate E* for the analyses.[4] HMA volumetric properties required by MEPDG are the in-situ air voids at the time of construction and the effective binder content (V_{beff}). Because the actual in-situ air voids at the time of construction for the investigated projects were not available, this parameter was assumed to be 7 percent. The V_{beff} for HMA used for each project was calculated from the gravimetric asphalt content with the help of the following formula shown in Figure 6:[28]

$$V_{beff} = \frac{(100 - V_a)}{100} \times \frac{(100 \times G_{sb} - 100 \times G_{mm} + AC\% \times G_{mm})}{G_{sb}}$$

where:

V_{beff} = effective binder content by volume (%)
V_a = in-situ target air voids (%)
G_{sb} = bulk specific gravity of aggregates
G_{mm} = theoretical maximum specific gravity
AC % = gravimetric asphalt content percentage (by % of total mix weight)

Figure 6. Equation to Determine Effective Binder Content of HMA

For the subgrade material characterization inputs, MEPDG Level 2 was used as the R-value of the subgrade is measured in the laboratory. MEPDG then uses the Asphalt Institute equation presented in Figure 4 to compute the resilient modulus from the R-value. The appropriate weather station data for each location was chosen based on the latitude, longitude and elevation of the project.

For the traffic data inputs, MEPDG uses the axle load spectra while ITD uses the TI (function of the classical 18-kip ESAL) and AASHTO 1993 design method uses the classical ESALs as the only traffic input. In order to maintain consistent traffic inputs for the three methods, the traffic load spectra analysis in the MEPDG was modified to reflect the design ESALs. The use of 18-kips-ESALs in MEPDG was conducted by:

1. using 100 percent FHWA Truck Class 5 and 0 percent for all other truck classes.
2. using 100 percent 18,000 lb single axle load and 0 percent for all other single axle loads.
3. using 100 percent of trucks in design direction.
4. using 100 percent trucks in design lane. In addition, the design ESALs expected to use the pavement was assumed to be uniformly distributed over the design life.

Furthermore, the representative modulus module for the base/subbase and subgrade layers was used in the analyses. MEPDG recommended a criterion shown previously in Table 5 was followed. The major inputs for the investigated projects using MEPDG design method are illustrated in Table 11.

Table 11. Design Input Data for MEPDG

Design Input	Design Project							
	US-2(a)	US-2(b)	US-2(c)	SH-62	SH-3	SH-19	US-95	US-93
Location:								
County	Bonner	Bonner	Bonner	Lewis	Nez Perce	Canyon	Washington	Butte
Latitude, Deg. Min.	48.15	48.15	48.15	46.14	46.28	43.4	44.19	43.26
Longitude, Deg. Min.	-116.39	-116.39	-116.39	-116.14	-116.45	-116.46	-116.55	-113.37
Elevation, ft	2,085	2,085	2,085	3,215	850	2,330	2,490	5,641
Main Traffic Inputs:								
Design Life, years	20	20	20	20	20	20	20	20
Speed, mph	60	60	60	35	55	60	60	65
AADTT (design lane)	542	542	542	56	253	115	153	208
Hot Mix Asphalt (HMA) Material Properties:								
Binder Type	PG58-28	PG58-28	PG58-28	PG58-28	PG70-28	AC-10	AC-10	PG64-34
Cumulative, % Retained ¾" Sieve	1	1	1	0	3	0	0	0
Cumulative, % Retained $^{3}/_{8}$" Sieve	27	27	27	15	23	27	26	26.4
Cumulative, % Retained No.4 Sieve	53	53	53	45	54	49	49	53
% Passing No.200 Sieve	6	6	6	8.2	4	5.6	4.9	5.3
% V_{beff}	9.95	9.95	9.95	11.01	11.6	9.66	9.38	10.09
% Air Voids	7	7	7	7	7	7	7	7
Unbound Granular Base Course Properties:								
Material Type	A-1-a	A-1-a	A-1-a	A-1-a	A-1-a	A-1-a	A-1-a	A-1-a
M_r, psi	38,000	38,000	38,000	38,000	38,000	38,000	38,000	38,000
Unbound Granular Subbase Course Properties:								
Material Type	Permeable Aggregate	Permeable Aggregate	Permeable Aggregate	-	-	Permeable Aggregate	Permeable Aggregate	Permeable Aggregate
M_r, psi	32,000*	32,000*	32,000*	-	-	32,000*	32,000	32,000*
Subgrade Properties:								
Material Type	SP-SM	ML	CL	CL	SM	ML	MH	SW
M_r, psi	12,255	28,905	9,480	5,595	15,030	25,575	3,930	28,905
GWT Depth, ft	10	10	10	6	45	7	7	30

* For ITD's Design Method sections only

Pavement Structure Design

Three designs were considered for each of the 8 selected projects. ITD's design method was used to design each of these projects. The projects were also structurally designed using both the AASHTO 1993 guide and MEPDG method. It should be noted that, the AASHTO 1993 design method recommends minimum layer thicknesses as a function of the design traffic level in terms of ESALs. The AASHTO 1993 minimum recommended layer thicknesses are shown in Table 12. Thus, for AASHTO 1993 design, if the method yielded thicknesses lower than the ones shown in Table 12, the design thicknesses were then taken from this table.

Table 12. AASHTO 1993 Minimum Layer Thicknesses[3]

Traffic (ESALs)	Asphalt Concrete	Aggregate Base
Less than 50,000	1.0	4.0
50,001 - 150,000	2.0	4.0
150,001 - 500,000	2.5	4.0
500,001 - 2,000,000	3.0	6.0
2,000,001 - 7,000,000	3.5	6.0
Greater than 7,000,000	4.0	6.0

Currently, MEPDG software is an analysis tool rather than a design tool. In order to find the design structure for each project using MEPDG, MEPDG software version 1.10 was ran several times based on the criteria for primary roadways presented in Table 5.

Results and Analysis

The following subsections present a comparison between the resulting pavement structure using ITD, AASHTO 1993, and MEPDG procedures at 50 percent and 85 percent reliability levels. It also present a comparison of the predicted performance using MEPDG for the three pavement designs for each of the investigated projects. AASHTO 1993 designs were conducted using the flexible pavement design utility which is available online.[27] This flexible pavement design utility solves the AASHTO 1993 basic design equation for flexible pavements.

Structure Design at a 50 Percent Reliability Level

Table 13 summarizes the resulted structures of the investigated projects using ITD's design method. The designed pavement structures using the AASHTO 1993 guide are shown in Table 14. The data in this table shows that for most of studied projects, the base layer thickness was chosen according to the minimum thickness criteria as a function of traffic recommended by AASHTO (See Table 12). Thus the actual structure number (SN) of these projects is much larger than the required SN as shown in Table 14. The resulted design structures based on the MEPDG procedure are shown in Table 15.

Table 13. ITD Design Structures for the Investigated Projects

Parameter	Design Project							
	US-2(a)	US-2(b)	US-2(c)	SH-62	SH-3	SH-19	US-95	US-93
HMA Thickness (in.)	6.0	6.0	6.0	4.2	4.2	4.2	4.2	5.4
Base Thickness (in.)	7.2	7.2	7.2	22.2*	22.2*	5.4	7.8	7.8
Subbase Thickness (in.)	19.2*	7.2*	21.0*	-	-	10.2	18.0*	5.4

*Rock cap

Table 14. AASHTO 1993 Design Structures for the Investigated Projects at 50 Percent Reliability

Parameter	Design Project							
	US-2(a)	US-2(b)	US-2(c)	SH-62	SH-3	SH-19	US-95	US-93
HMA Thickness (in.)	5.5	5.0	5.5	3.5	4.5	4.0	4.0	4.5
Base Thickness (in.)	6.0*	6.0*	7.5	9.5	6.0*	6.0*	4.0	6.0*
Subbase Thickness (in.)	-	-	-	-	-	-	7.5	-
Required AASHTO SN	3.845	2.395	3.595	3.045	2.695	1.945	3.995	2.045
Actual AASHTO SN	3.920	3.160	3.620	3.060	2.940	2.720	4.020	2.940

* Minimum recommended thickness by the 1993 design method

Table 15. MEPDG Design Structures for the Investigated Projects at 50 Percent Reliability

Parameter	Design Project							
	US-2(a)	US-2(b)	US-2(c)	SH-62	SH-3	SH-19	US-95	US-93
HMA Thickness (in.)	5.5	4.5	5.0	3.5	5.0	3.0	5.0	3.5
Base Thickness (in.)	6.0	6.0	7.0	6.0	6.0	4.0	5.0	5.0
Subbase Thickness (in.)	-	-	-	-	-	-	8.0	-

A comparison between the computed pavement structures using the 3 design methods at the 50 percent reliability level is shown in Figure 7. Figure 8 through Figure 10 compare the resulting thicknesses of the AC, base, and subbase layers from the 3 design methods at the 50 percent reliability level, respectively. These figures clearly show that ITD's design method always yields pavement structures that are significantly thicker compared to the other two methods. Figure 8 shows that that AASHTO 1993 and MEPDG methods yield very similar AC thicknesses for most of the investigated projects. Furthermore, a reasonable agreement was found between the three design methods regarding the design thickness of the asphalt layer except in case of projects with either very high strength or very weak subgrade foundation. However, the unbound granular layer(s) thickness(s) were found to be much thicker in the ITD's design method (2 to 4.5 times). For all practical purposes, the AASHTO 1993 and MEPDG methods produce reasonably similar pavement structures.

It should be noted that the assumptions used in this study are different from that ITD used when implementing AASHTO 1993 design guide. ITD used M_r-R-value equation that led to low M_r values. Hence, when using AASHTO 1993 guide it produced thicker pavement than ITD's design method. In this study the Asphalt Institute equation of M_r-R-value was used as per AASHTO recommendation. This, in turn, led to much higher M_r values for the same R-values, and consequently produced thinner pavements than ITD method. Hence the key difference in the results between AASHTO 1993 design guide and ITD method is in the transfer equation of M_r from R-value.

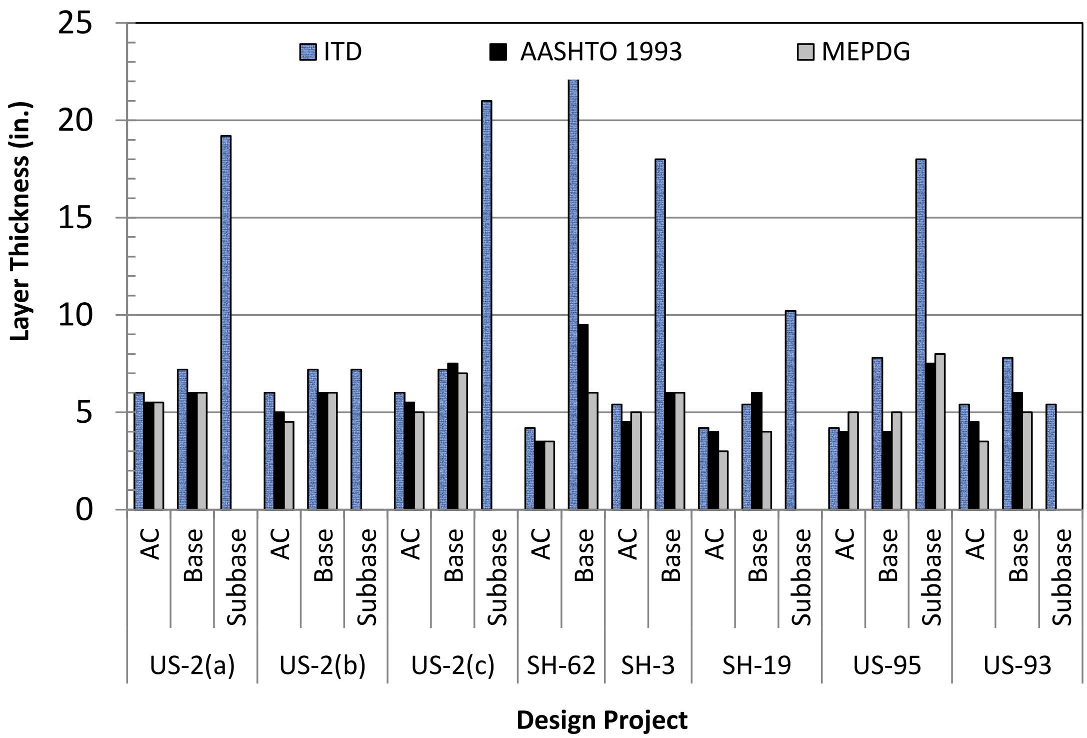

Figure 7. Comparison of the Recommended Pavement Structure by the Investigated Design Methods at 50 Percent Reliability

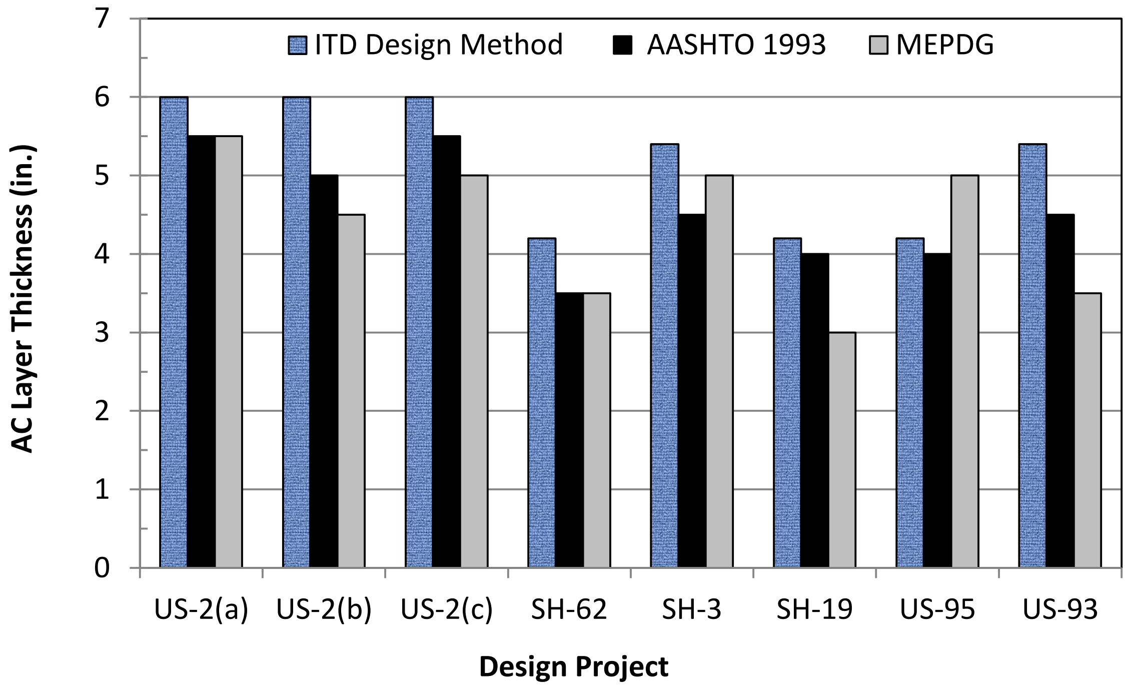

Figure 8. Comparison of the Recommended AC Layer Thickness by the Investigated Design Methods at 50 Percent Reliability

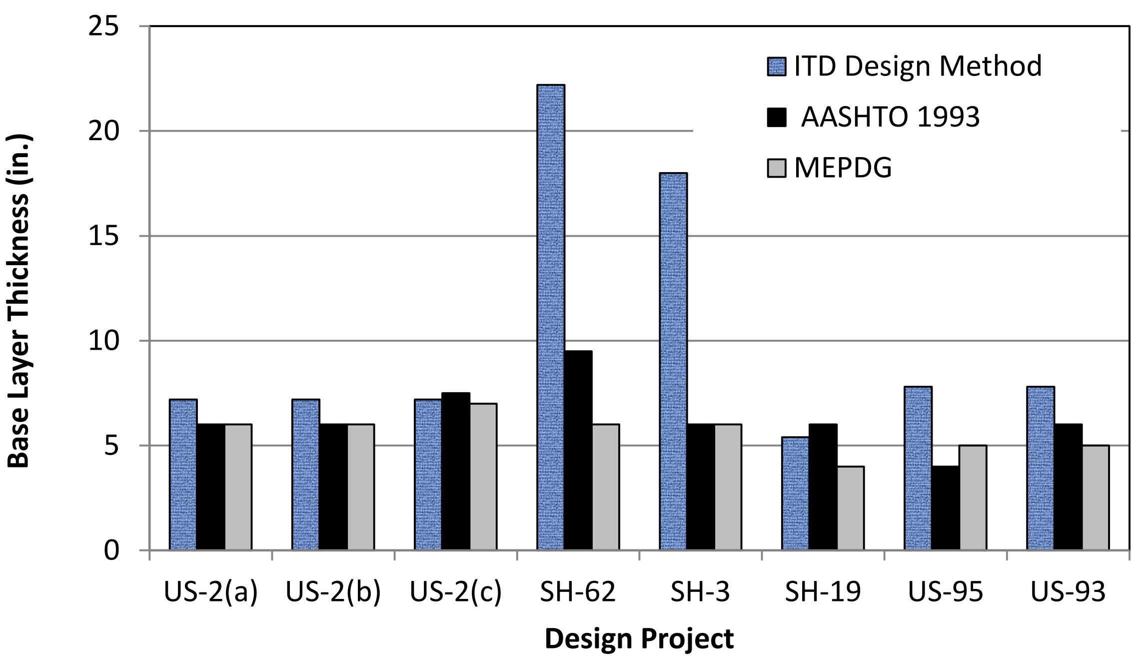

Figure 9. Comparison of the Recommended Base Layer Thickness by the Investigated Design Methods at 50 Percent Reliability

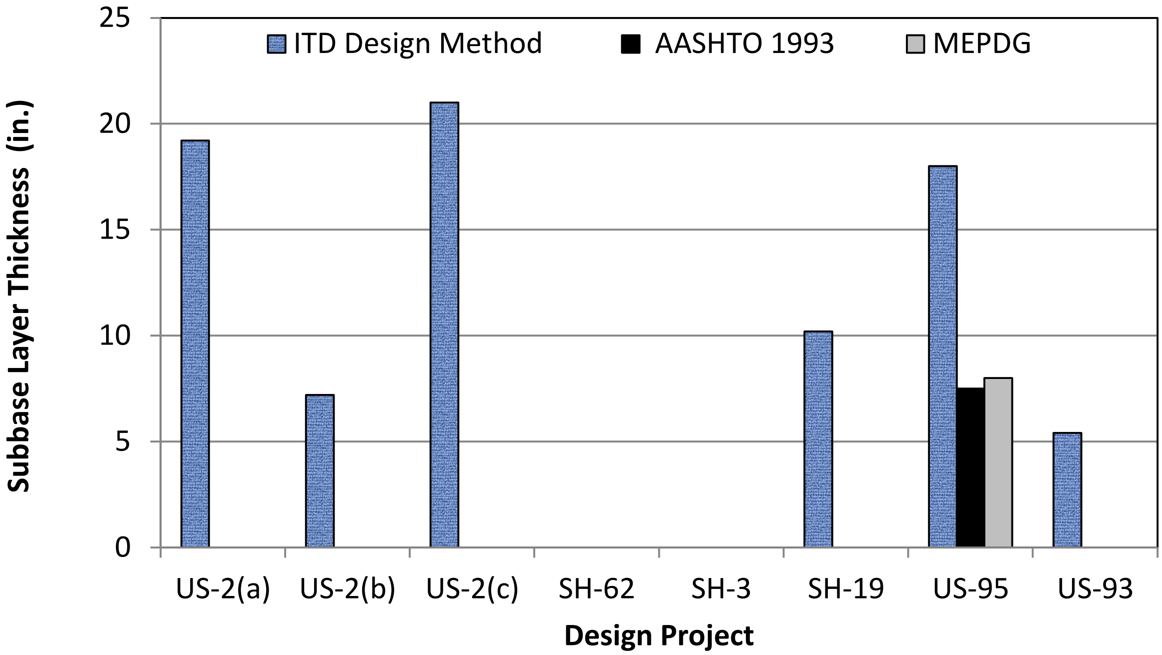

Figure 10. Comparison of the Recommended Subbase Layer Thickness by the Investigated Design Methods at 50 Percent Reliability

The US-2 project has 3 sections with the same inputs except for the subgrade strength. The designed pavement structures using the 3 different methods for the 3 US-2 sections show that the subgrade strength is overemphasized by the ITD's design method compared to the other 2 design methods.[29] This is shown in Figure and Figure by the dramatic increase in the subbase thickness with the decrease in the subgrade strength based on ITD's design.

MEPDG Predicted Distresses at 50 Percent Reliability

Rutting Analysis

MEPDG predicted distresses and roughness, at the end of design life, using the pavement structures resulting from the 3 design methods for the investigated projects are shown in Figure 11 through Figure 16. No thermal cracking was predicted for any of the pavement structures resulted by any of the investigated design methodologies. A comparison of MEPDG predicted distresses and roughness, during the service life, using the pavement structures resulted from the three design methods for the investigated projects are shown in Appendix B.

For all practical purposes, Figure 11 shows no significant difference in the predicted total rutting for the structures designed with AASHTO 1993 and MEPDG as these 2 methods generally yielded similar pavement structures. This figure also shows no significant difference in the predicted total rutting between the structures resulting from ITD's design method and the other 2 design methods, despite the fact that ITD's method always yielded a much thicker pavement structures.

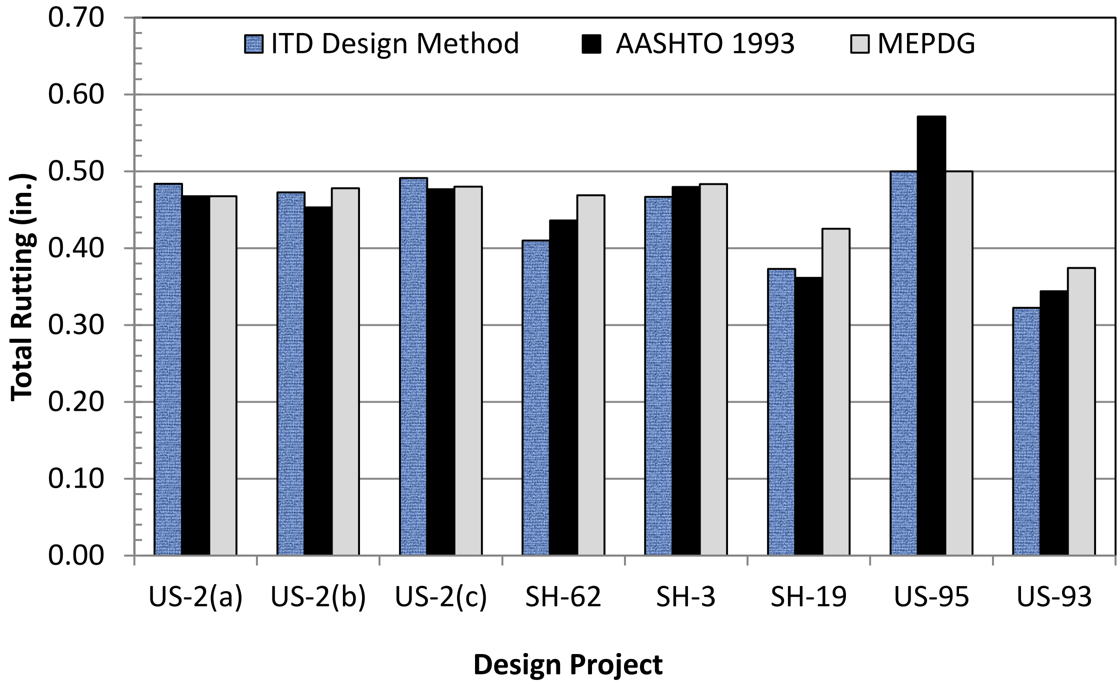

Figure 11. Comparison of MEPDG Total Predicted Rutting From the 3 Design Methods at 50 Percent Reliability

Furthermore, one can surmise from Figure 12 that, there is no significant difference in the predicted AC rutting for all pavement structures resulted from the 3 design methods. The primary reason for this is that, generally, there was no significant difference in the design AC thickness resulting from the three different design methods. In addition, the AC rutting is only a function of the AC layer properties, traffic, and environment and it is not affected by the foundation properties. This observation agrees with other studies.[30, 31]

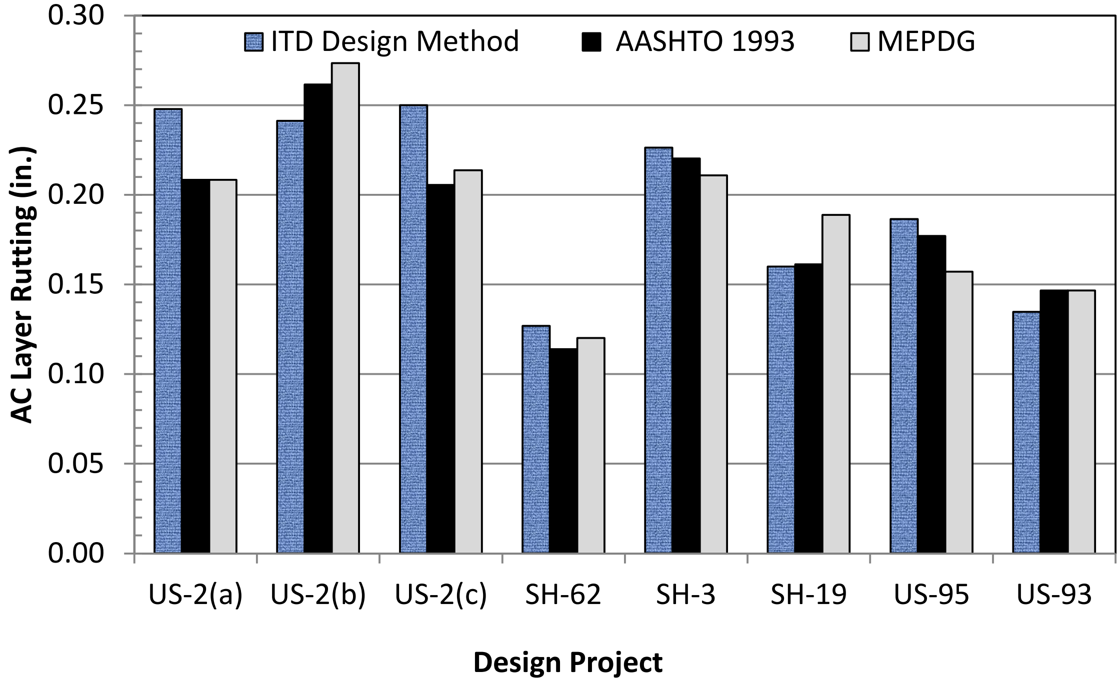

Figure 12. Comparison of MEPDG Predicted AC Rutting from the 3 Design Methods at 50 Percent Reliability

It can be also deduced from Figure 7 and Figure 13 that as the thickness of the unbound base/subbase layers increases, the rutting within this layer(s) also increases. This is obvious in Figure 13 for the structures designed with ITD's method. This fact is the main reason that, although ITD's design method always yielded larger total pavement thickness compared to AASHTO 1993 and MEPDG, the total rutting predicted from the 3 design methods is not significantly different.

Figure 14 shows that, the predicted subgrade rutting for the pavement structures designed by ITD's method is the lowest. This observation is basically because ITD's design always yielded thicker pavement structures. It is obvious that if the pavement structure above the subgrade is thicker, the compressive strain at the top as well as any point within the subgrade is lower, hence subgrade rutting decreases.

Only the US-95 section at the 50 percent reliability level, AASHTO 1993 design yielded a structure with a total rutting slightly in excess of MEPDG recommended threshold value. All other structures conform to MEPDG recommended design criteria.

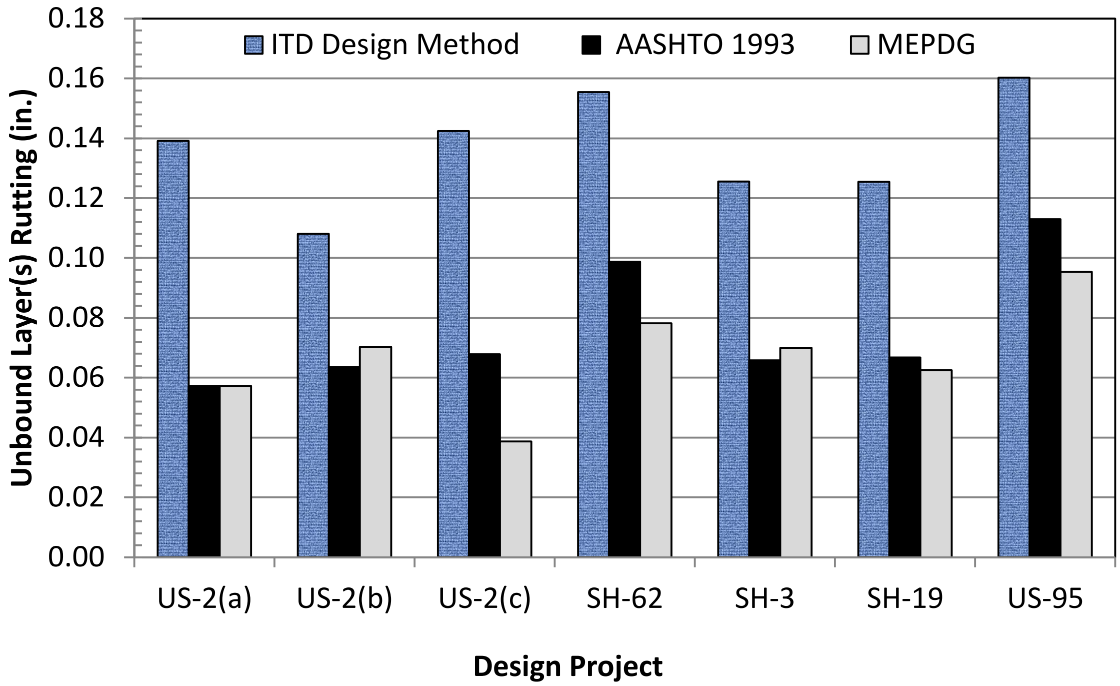

Figure 13. Comparison of MEPDG Predicted Unbound Layers Rutting from the 3 Design Methods at 50 Percent Reliability

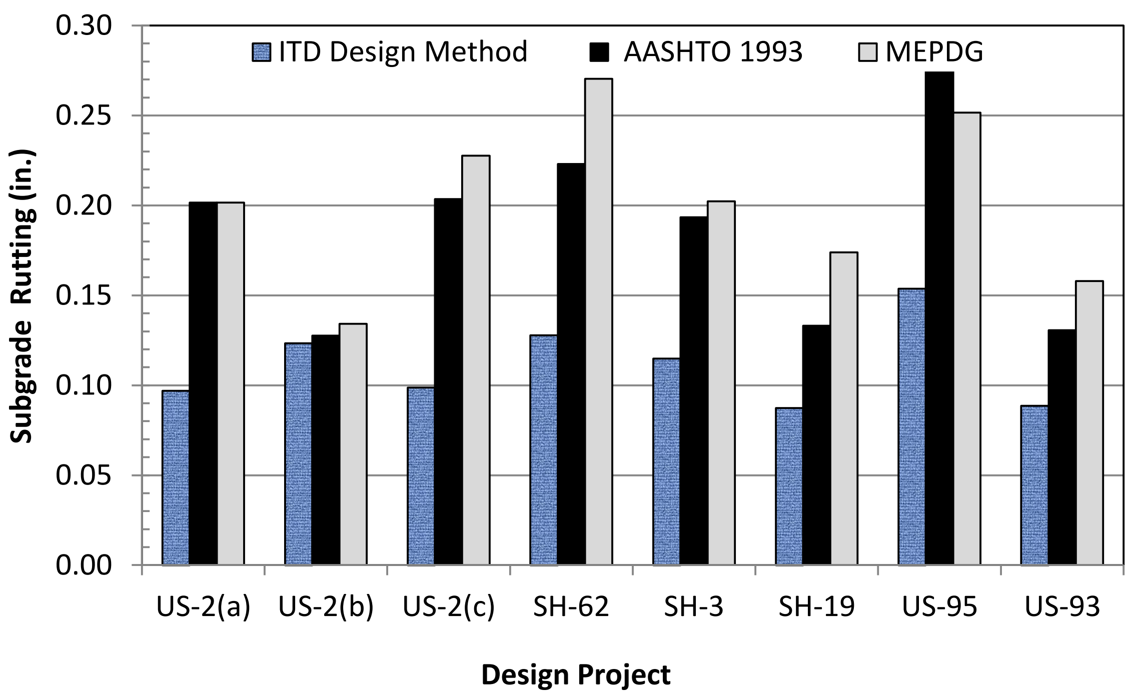

Figure 14. Comparison of MEPDG Predicted Subgrade Rutting from the 3 Design Methods at 50 Percent Reliability

Fatigue Cracking Analysis

Regarding alligator fatigue cracking, Figure 15 clearly shows that among the 3 design methods, ITD's design method always yielded pavement sections with the least amount of fatigue cracking. This is obviously because ITD's method always yields thicker (at least 2.0 times as thick) unbound granular layers which results in a stronger support for the AC layer. Thus less tensile strain at the bottom of the AC layer occurs. This figure also shows that MEPDG predicted alligator fatigue cracking values for all structures resulted for the 3 design methods are way below the MEPDG recommended threshold value of 20 percent.[(19)]

Roughness (IRI) Analysis

A comparison between the IRI predicted using MEPDG for the pavement structures resulted from the 3 design methods, at the 50 percent reliability level is shown in Figure 16. No significant difference in the predicted IRI was observed as shown in this figure. Additionally, MEPDG predicted IRI values for all structures resulted from the 3 design methods, are way below MEPDG recommended threshold value of 200 in./mile.

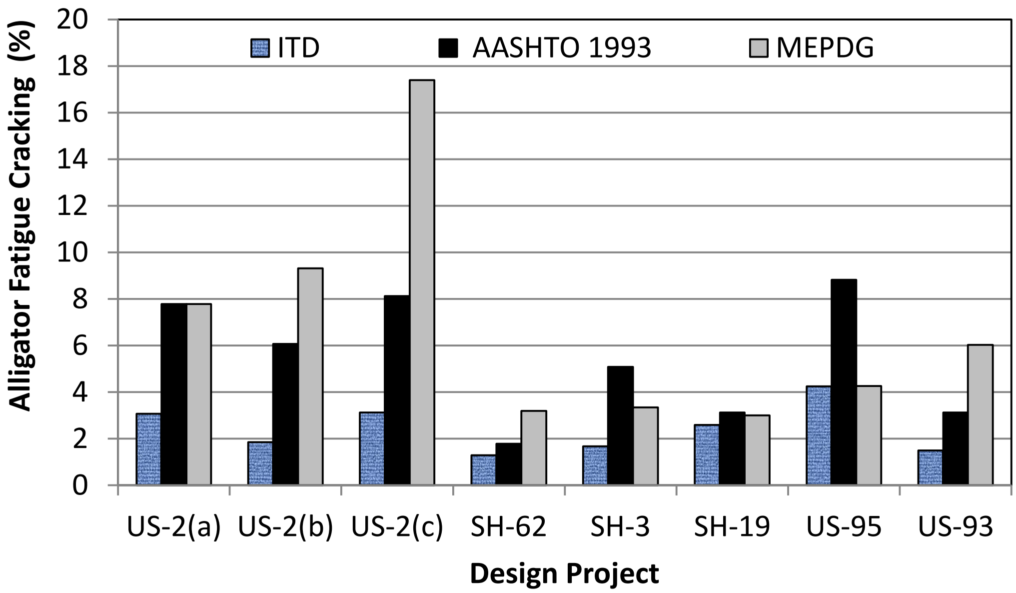

Figure 15. Comparison of MEPDG Predicted Alligator Fatigue Cracking from the 3 Design Methods at 50 Percent Reliability

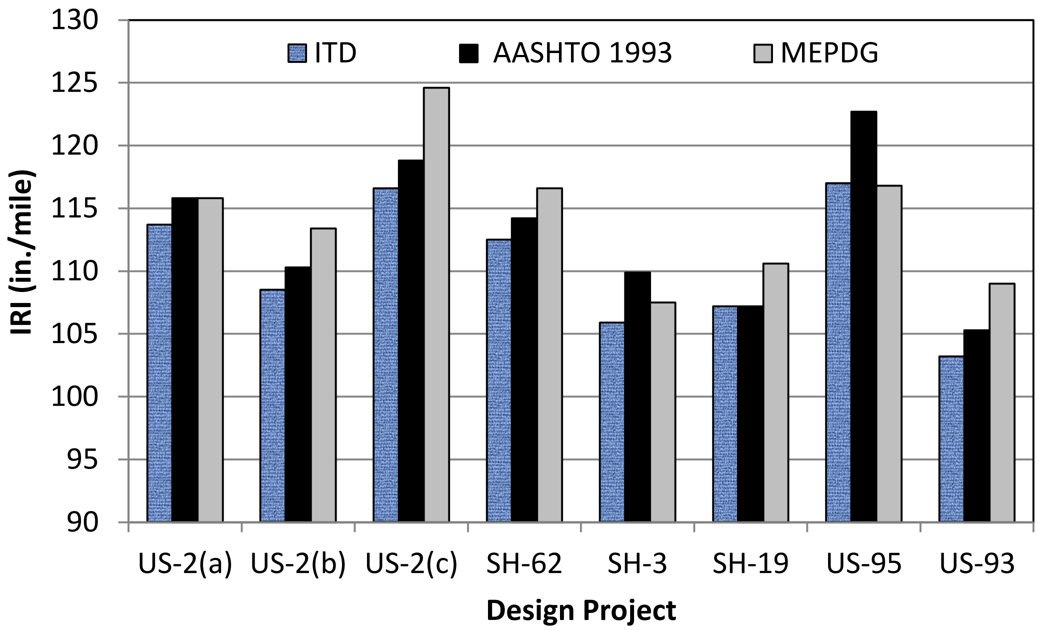

Figure 16. Comparison of MEPDG Predicted IRI from the 3 Design Methods at 50 Percent Reliability

Finally, a comparison between Figure 11, Figure 15, and Figure 16 along with MEPDG recommended design criteria reveals that MEPDG designs were governed by the total pavement rutting not by cracking or IRI. One direct reason for this is the use of ESALs instead of axle load spectra in all MEPDG computer simulation runs. In general, load associated cracking are more sensitive to axle load spectra compared to rutting.[8]

Structure Design at 85 Percent Reliability Level

ITD's design method for flexible pavements is a deterministic method that does not incorporate reliability. On the other hand, both AASHTO 1993 and MEPDG methods allow design at different reliability levels. Results at the 50 percent reliability level show that ITD's design method always yielded significantly thicker unbound layer thicknesses compared to both AASHTO 1993 and MEPDG guides. It was decided to redesign the investigated projects at 85 percent reliability levels using both AASHTO 1993 and MEPDG methods and compare the resulting structures and predicted distresses with ITD's design.

The designed pavement structures at 85 percent reliability level and combined standard deviation of 0.45 using the AASHTO 1993 guide are shown in Table 16. As expected comparing the thicknesses in Table 14 and Table 16 show that, the higher the reliability, the thicker the pavement thickness. This is true especially on the AC layer thickness. Table 17 summarizes the designed pavement structures at 85 percent reliability levels using MEPDG.

Table 16. AASHTO 1993 Design Structure for the Investigated Projects at 85 Percent Reliability

Parameter	Design Project							
	US-2(a)	US-2(b)	US-2(c)	SH-62	SH-3	SH-19	US-95	US-93
HMA Thickness (in.)	6.0	6.0	6.0	4.0	5.5	5.0	5.0	5.5
Base Thickness (in.)	8.0	6.0*	9.5	11.5	6.0*	6.0*	4.0	6.0*
Subbase Thickness (in.)	-	-	-	-	-	-	8.0	-
Required SN	3.845	2.795	4.100	3.595	3.195	2.295	4.595	2.395
Actual SN	3.920	3.600	4.160	3.600	3.380	3.160	4.676	3.160

* Minimum Recommended Thickness by the 1993 Design Method

Table 17. MEPDG Design Structure for the Investigated Projects at 85 Percent Reliability

Parameter	Design Project							
	US-2(a)	US-2(b)	US-2(c)	SH-62	SH-3	SH-19	US-95	US-93
HMA Thickness (in.)	7.0	6.5	7.5	4.0	7.0	4.0	7.0	4.5
Base Thickness (in.)	8.0	6.0	9.0	8.0	6.0	4.0	4.0	6.0
Subbase Thickness (in.)	-	-	-	-	-	-	8.5	-

Figure 17 shows a comparison between the computed pavement structures using the 3 design methods at the 85 percent reliability level. A comparison of the resulting thicknesses of the AC, base, and subbase layers from the 3 design methods at the 85 percent reliability level, is shown in Figure 18 through Figure 20, respectively. Similar to the results at the 50 percent reliability level, AASHTO 1993, and MEPDG methods at 85 percent reliability still result in significantly thinner unbound layer thicknesses compared to ITD design method. However, at this reliability level, generally for most of the investigated projects, both AASHTO 1993 and MEPDG methods yielded thicker AC layer thicknesses compared to ITD's method as shown in Figure 18. Figure 18 also reveals that MEPDG desired AC thickness is usually slightly larger than the one required by ITD's and AASHTO 1993 methods. The significant increase in the AC layer thickness for projects designed with MEPDG occurred with the US-95 project which has a very weak subgrade (R- value = 5).

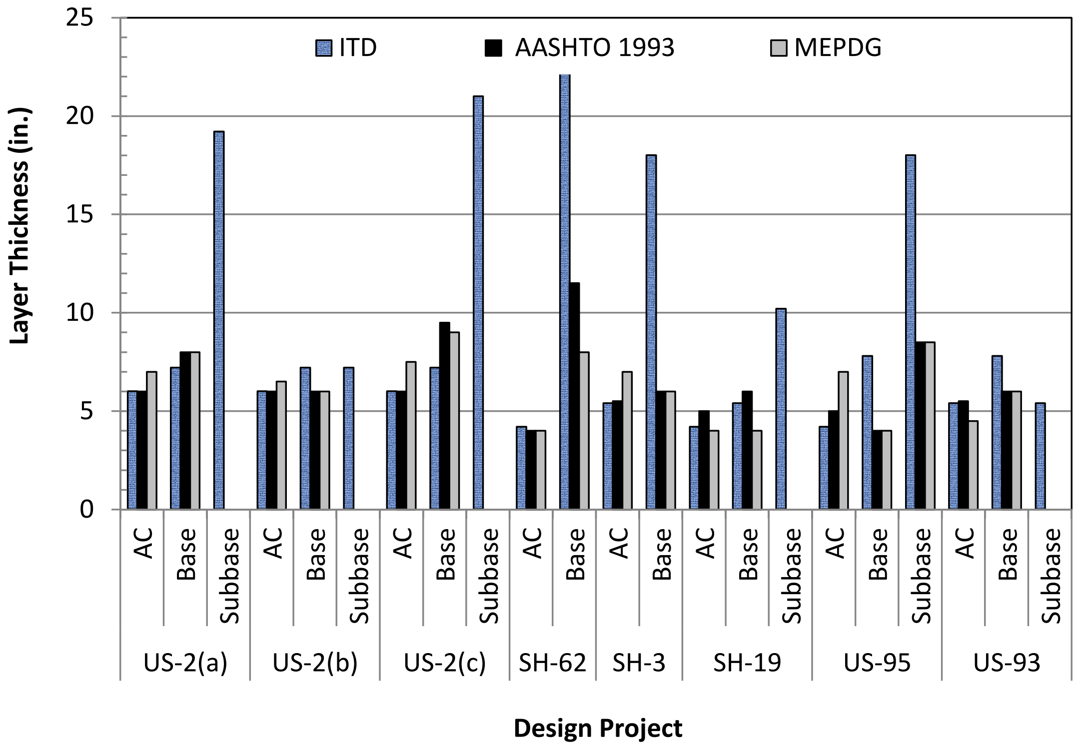

Figure 17. Comparison of the Recommended Pavement Structure by the Investigated Design Methods at 85 Percent Reliability

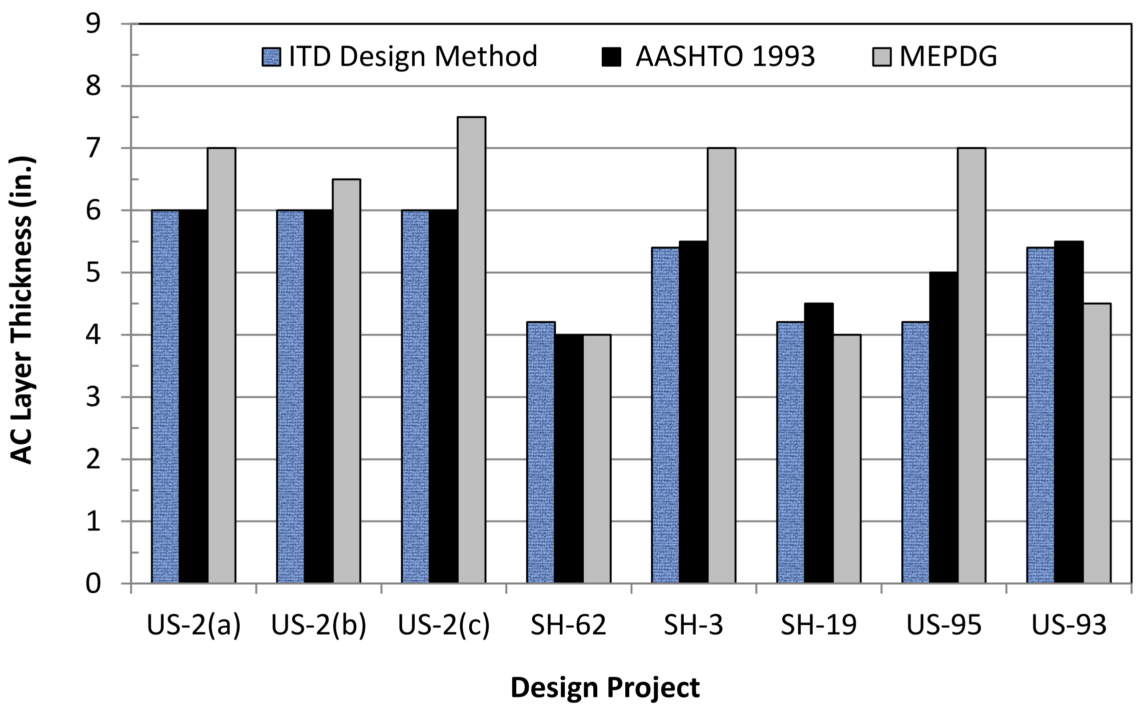

Figure 18. Comparison of the Recommended AC Layer Thickness by the Investigated Design Methods at 85 Percent Reliability

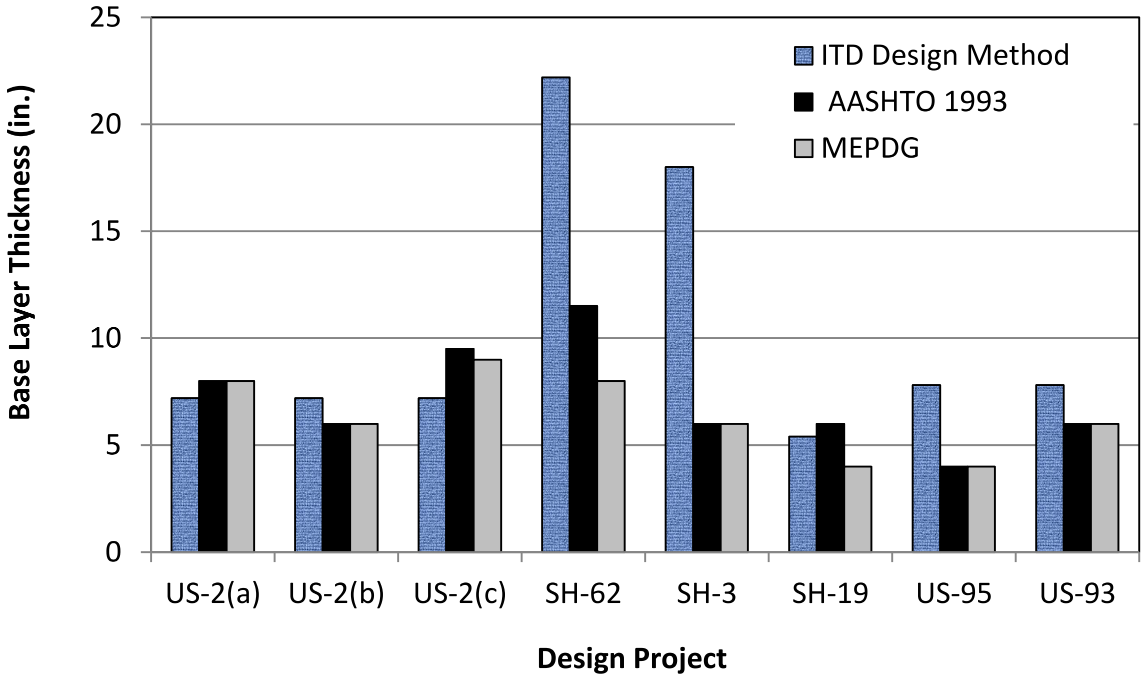

Figure 19. Comparison of the Recommended Base Layer Thickness by the Investigated Design Methods at 85 Percent Reliability

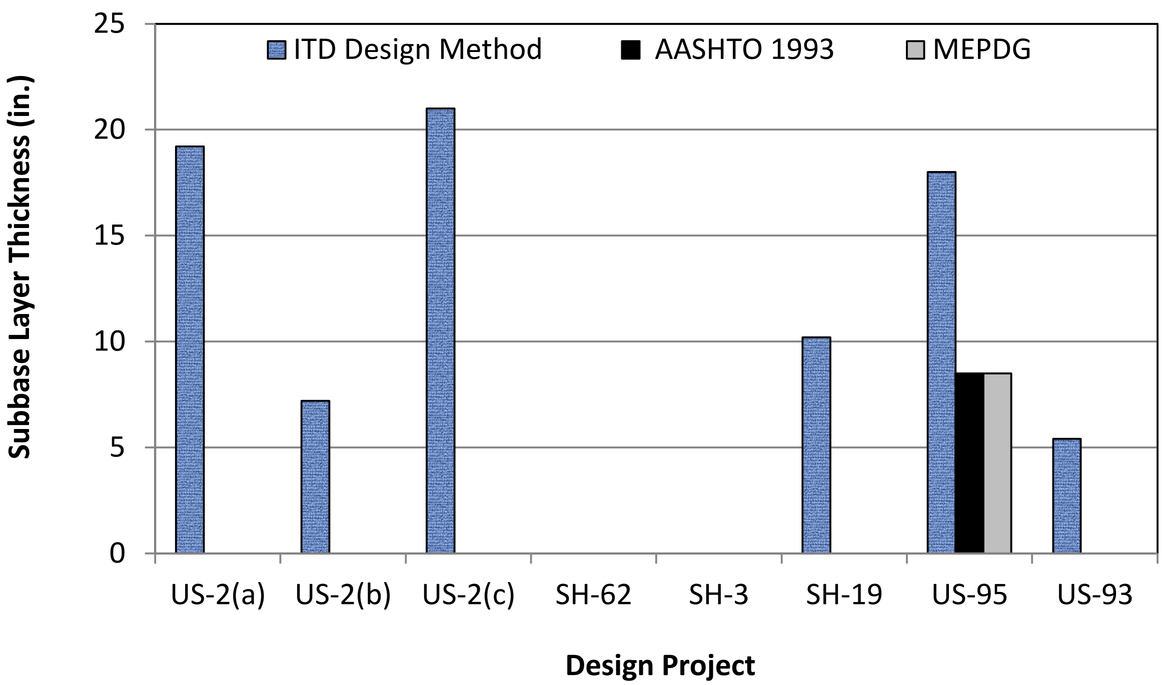

Figure 20. Comparison of the Recommended Subbase Layer Thickness by the Investigated Design Methods at 85 Percent Reliability

MEPDG Predicted Distresses for Structures Designed at 85 Percent Reliability

MEPDG predicted distresses and roughness, at the end of design life, using the pavement structures resulting from the 3 design methods for the investigated projects are shown in Figure 21 through Figure 26. Again, no thermal cracking was predicted for any of the pavement structures resulting by any of the investigated design methodologies at this level of reliability. A comparison of MEPDG predicted distresses and roughness for structures designed at 85 percent reliability using MEPDG and AASHTO 1993 compared to structures designed using ITD's design method which does not incorporate reliability are shown in Appendix C.

Rutting Analysis

Similar to the results at 50 percent reliability, Figure 21 shows no significant difference in MEPDG predicted total rutting for the structures designed with the 3 investigated methods. However, the AC layer rutting is generally higher for the structures designed using ITD's method as the thickness of the AC was smaller in these structures. This is shown in Figure 22. Again, the rutting in the unbound layers is significantly higher for the structures designed with ITD's method as shown in Figure 23.

Similar to results at 50 percent reliability, Figure 24 shows that, the predicted subgrade rutting for the pavement structures designed by ITD's method is the lowest. Furthermore, this figure shows that all investigated structures conform to MEPDG recommended design criteria.

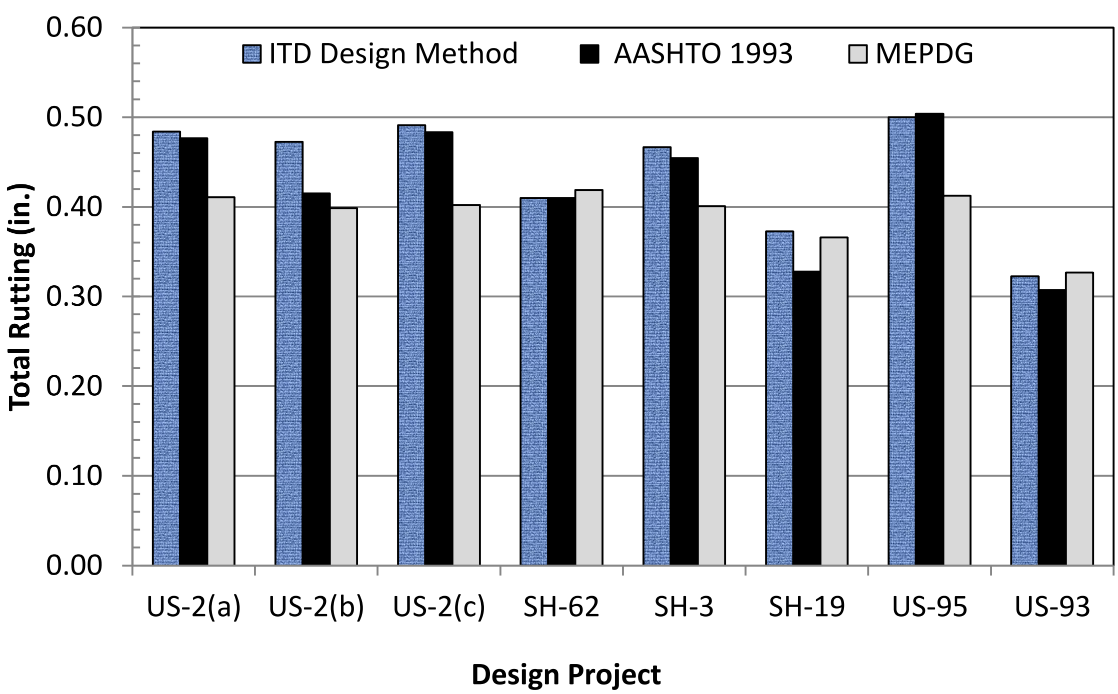

Figure 21. Comparison of MEPDG Total Predicted Rutting from the 3 Design Methods at 85 Percent Reliability

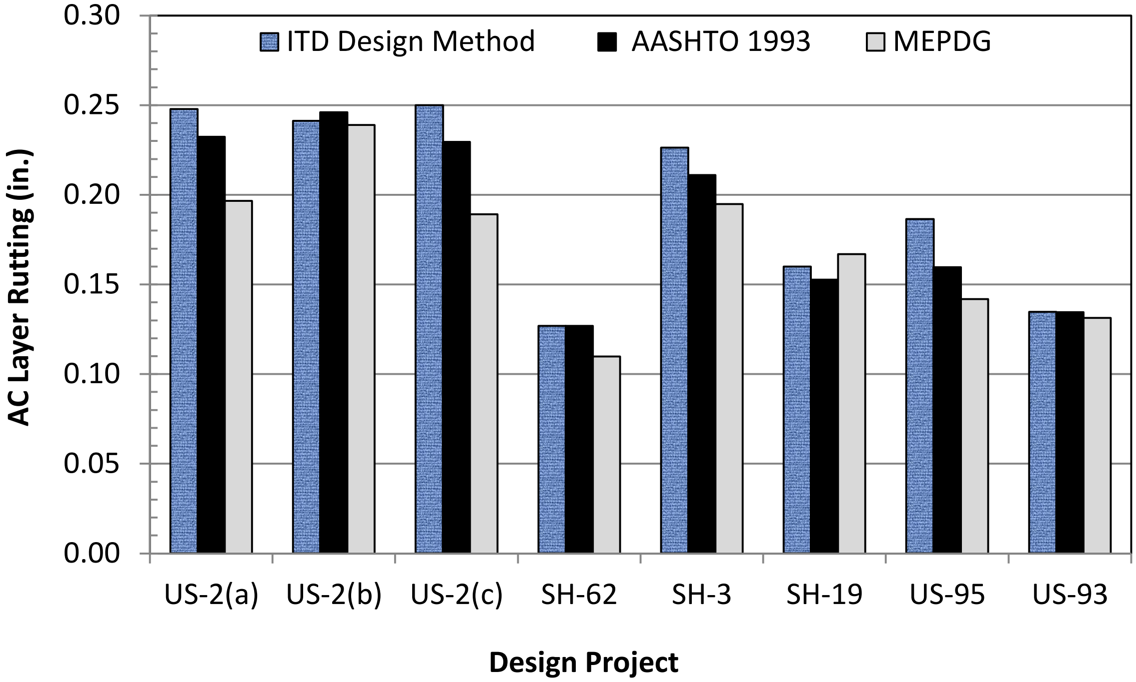

Figure 22. Comparison of MEPDG Predicted AC Rutting from the 3 Design Methods at 85 Percent Reliability

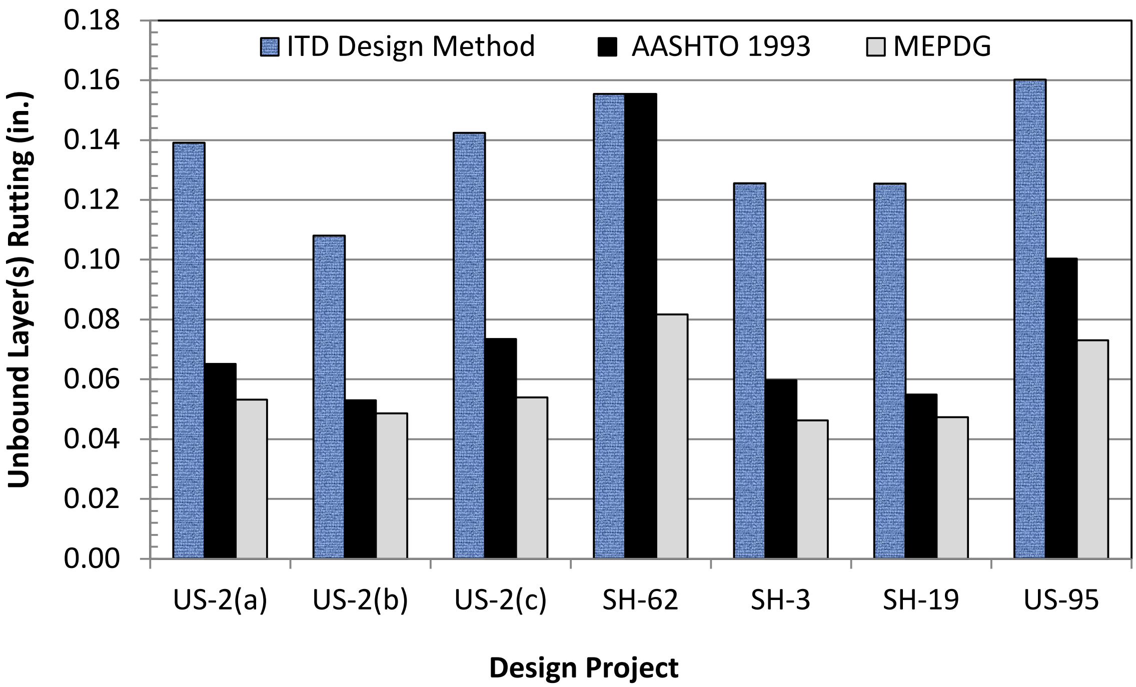

Figure 23. Comparison of MEPDG Predicted Unbound Layers Rutting from the 3 Design Methods at 85 Percent Reliability

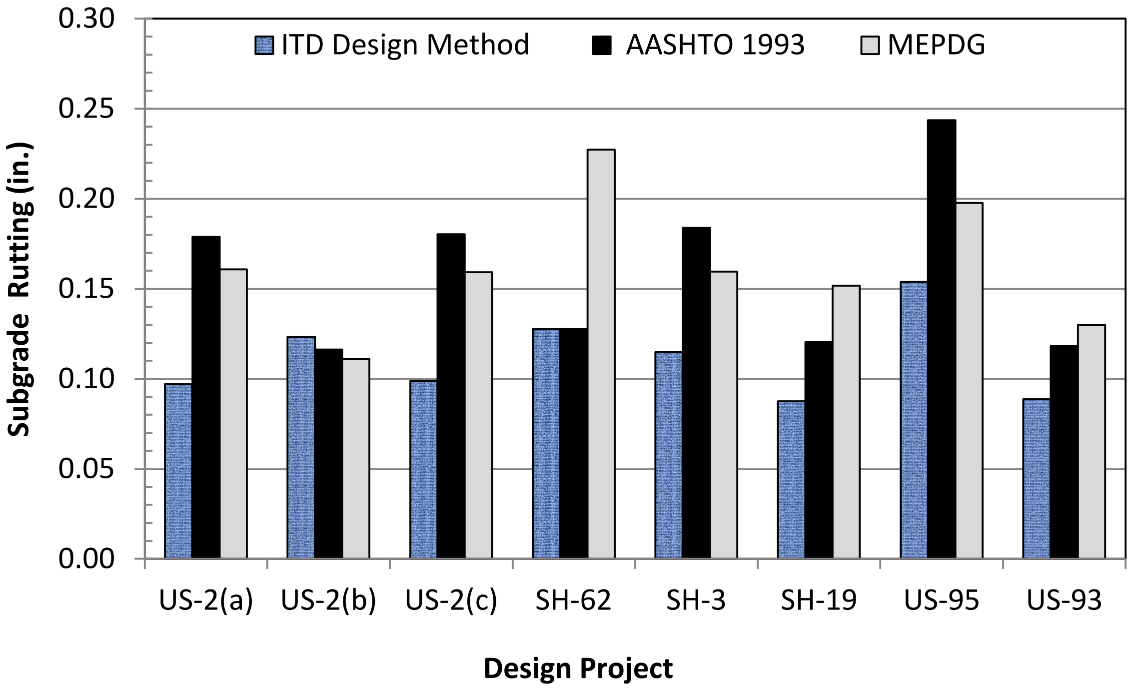

Figure 24. Comparison of MEPDG Predicted Subgrade Rutting from the 3 Design Methods at 85 Percent Reliability

Fatigue Cracking Analysis

A comparison of MEPDG predicted alligator fatigue cracking resulting from pavement structures using the 3 investigated design alternatives, is shown in Figure 25. This figure shows that among the three design methods, MEPDG design method yielded pavement sections with the least amount of fatigue cracking. This is obviously because MEPDG resulting AC layer thicknesses were thicker compared to the other two methods. One can conclude from Figure 25 and Figure 15 that increasing the AC layer thickness is more effective in resisting fatigue cracking compared to increasing the unbound layer thickness. As the AC layer thickness increased, the tensile strain at the bottom of the AC layer becomes lower, hence less fatigue occurs. Figure 25 also shows that MEPDG predicted fatigue cracking values for all structures resulted from the 3 design methods, are way below MEPDG recommended threshold value of 20 percent.

Roughness (IRI) Analysis

Finally, a comparison between the IRI predicted using MEPDG for the pavement structures resulted from the 3 design methods, at the 85 percent reliability level is shown in Figure 26. No significant difference in the predicted IRI was observed as shown in this figure. Additionally, MEPDG predicted IRI values for all structures resulted from the three design methods, are way below MEPDG recommended threshold value of 200 in./mile.

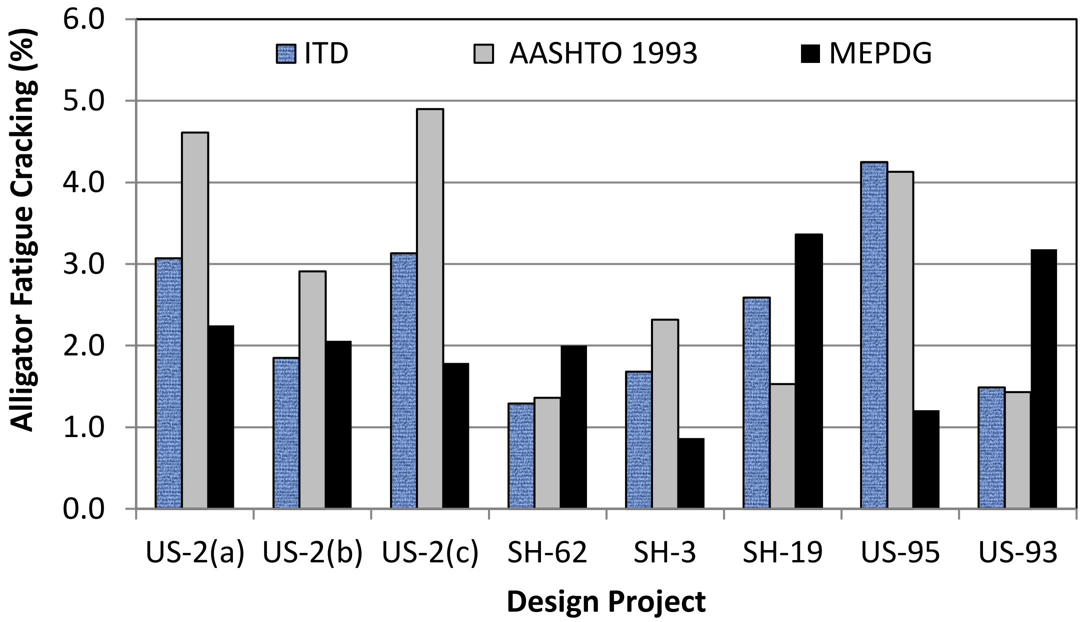

Figure 25. Comparison of MEPDG Predicted Alligator Fatigue Cracking from the 3 Design Methods at 85 Percent Reliability

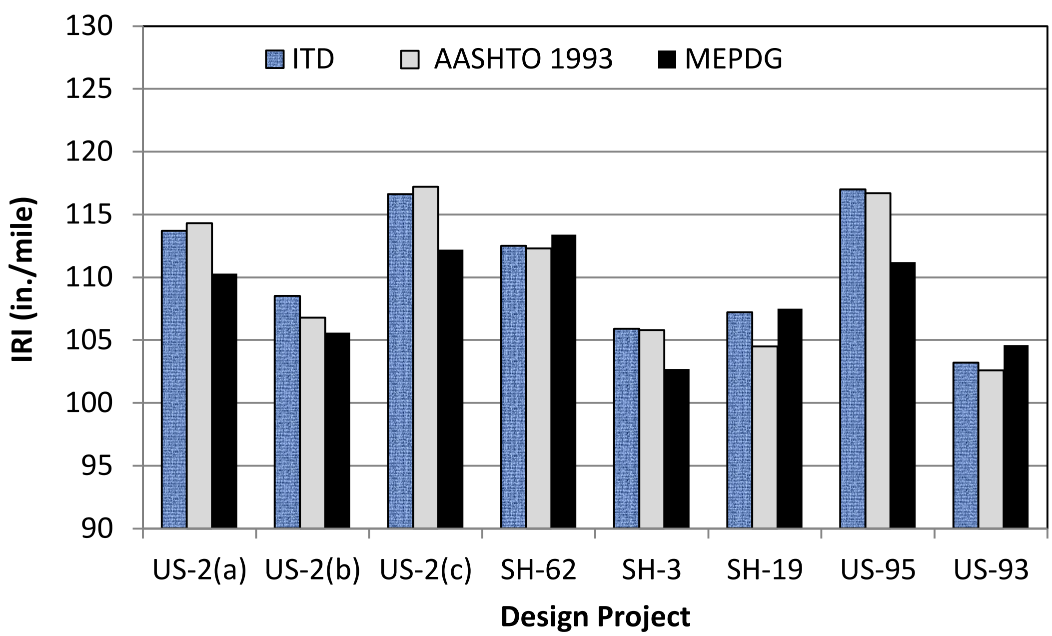

Figure 26. Comparison of MEPDG Predicted IRI from the 3 Design Methods at 85 Percent Reliability

Construction Cost Comparison

In order to quantify the savings that may be achieved when using AASHTO 1993 or MEPDG design methods instead of the current ITD's design method, construction cost comparison was performed on the resulted pavement structures from the 3 design methods. This comparison was conducted following ITD's recommendations and with the help of the ITD's excel sheet for Life Cycle Cost Analysis (LCCA).(32) It should be noted that the performed analysis is intended only for comparing the design alternatives rather than estimating their actual construction cost.

A comparison of the initial construction cost of structures using ITD, AASHTO 1993, and MEPDG at 50 percent reliability level is shown in Figure 27. Figure 28 compares the initial construction cost at 85 percent reliability level. The construction cost analysis conducted on the 8 investigated projects shows that structures designed using the AASHTO 1993 design method, at 50 percent reliability, yielded an average saving of 39 percent compared to ITD's designs. The average saving was 22 percent when the designs were conducted at 85 percent reliability using AASHTO 1993.

For MEPDG designs at 50 percent reliability, the conducted cost analysis shows an average saving of 48 percent compared to ITD's designs. The average saving was 17 percent when the designs were conducted at 85 percent reliability using MEPDG. This analysis shows that, even at the 85 percent reliability level, a considerable saving could be achieved if AASHTO 1993 or MEPDG design methods were used instead of ITD's design method.

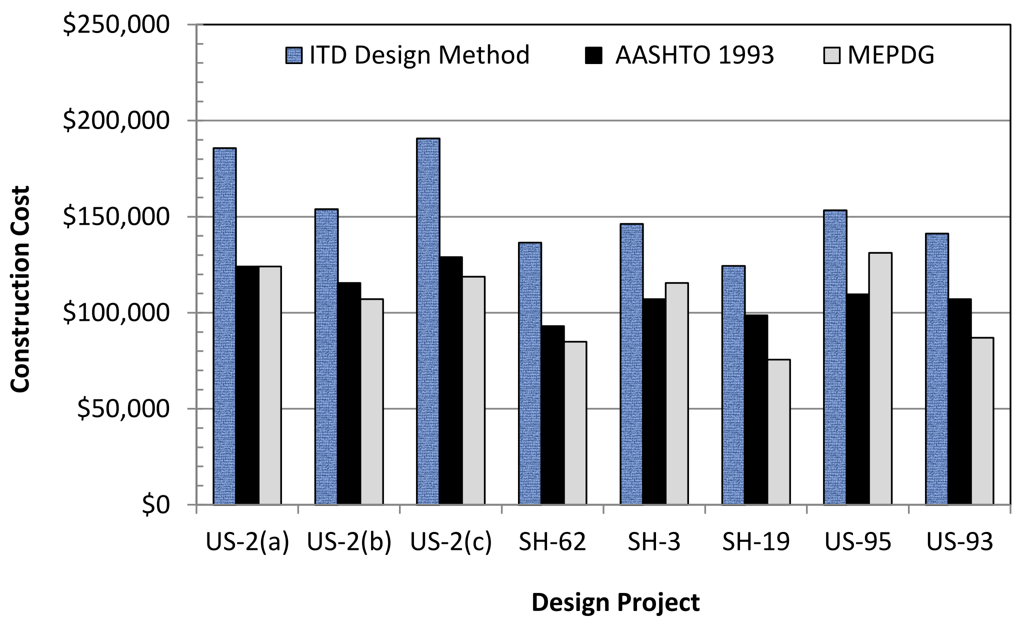

Figure 27. Comparison of the Initial Construction Cost at 50 Percent Reliability Level

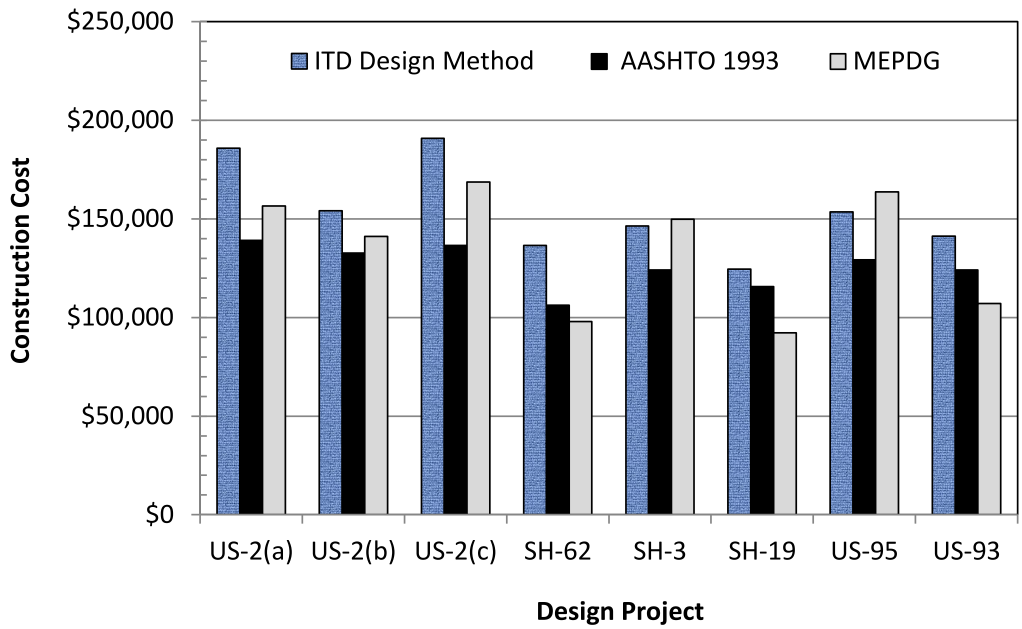

Figure 28. Comparison of the Initial Construction Cost at 85 Percent Reliability Level

Chapter 4
Pavement Performance Evaluation Using MEPDG

Introduction

Unlike both ITD's and AASHTO 1993 design methods, MEPDG is able to simulate and predict pavement performance over the service life of the pavement. Four different distress types as well as pavement smoothness are predicted using MEPDG for flexible pavements. The distresses predicted are permanent deformation (rutting), bottom-up alligator fatigue cracking, top-down longitudinal fatigue cracking, and thermal (transverse) cracking. Pavement smoothness (roughness) is expressed as IRI in MEPDG.

In the current version of the MEPDG software (Version 1.10), pavement distresses are predicted, from mechanistically calculated strains and deformations, using statistical transfer functions. These transfer functions are nationally calibrated based on field data from 94 LTPP sections distributed all over the U.S. The software also allows users to input user defined calibration coefficients to reflect certain conditions.

In MEPDG, IRI is predicted empirically as a function of pavement distresses, site factors that represent the foundation's shrink/swell and frost heave capabilities, and an estimate of the initial IRI (at time of construction).(19)

This chapter presents a comparison between field measured and MEPDG predicted distresses based on the evaluated ITD projects.

Investigated Projects and Input Data

No performance data was available for the SH-62 project as it is a new project. Consequently, a total of 7 out of the 8 projects presented previously in Table 5 and designed by ITD's flexible pavement design method were investigated for performance. These projects represent a wide range of subgrade strength, climatic conditions, and traffic levels.

The actual cross section for each of the investigated projects was taken from ITD's design reports. MEPDG Level 2 input data was used for the characterization of the HMA and subgrade soils. A statewide Axle Load Spectra (ALS) and number of axles per truck class data developed for Idaho as part of the MEPDG implementation in Idaho which is close to completion were used for this task.(8) Idaho statewide ALS data is presented in Table 18 through Table 21 for single, tandem, tridem, and quad axles, respectively. Table 22 summarizes the number of axles per truck per axle type. The Annual Average Daily Truck Traffic (AADTT) distribution is shown in Table 23. This data belongs to MEPDG default values for the Truck Traffic Classification (TTC) Group 3. The traffic axle configuration and general traffic data are shown in Table 24. All other required inputs were MEPDG Level 3 default values.

Table 18. Statewide Single Axle Load Spectra

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
3,000	4.07	9.14	1.82	5.81	15.18	2.13	1.16	9.74	8.25	5.21
4,000	1.91	10.92	2.83	3.02	10.52	2.15	0.78	6.44	5.84	5.81
5,000	3.18	10.80	3.51	2.44	9.48	2.64	1.72	9.26	4.66	5.87
6,000	6.18	12.22	5.14	5.03	9.05	3.02	2.74	9.79	6.56	6.65
7,000	6.30	7.69	6.82	6.59	7.04	4.89	3.53	7.82	7.12	7.75
8,000	10.77	8.31	9.85	8.93	10.41	7.45	7.30	9.01	10.57	7.20
9,000	8.39	6.94	9.12	9.03	6.37	9.20	10.35	6.72	9.77	8.34
10,000	9.01	5.70	10.59	9.35	7.18	13.36	15.49	7.70	11.94	11.01
11,000	7.49	4.60	9.13	9.15	4.45	14.00	13.92	5.83	9.51	8.15
12,000	7.39	4.47	10.23	9.18	4.00	14.58	15.04	4.73	7.04	8.59
13,000	6.94	3.31	8.47	7.99	3.11	9.22	10.78	3.34	4.67	5.86
14,000	6.22	2.50	5.75	5.07	2.09	4.02	3.94	2.74	2.80	3.48
15,000	6.21	2.40	5.67	3.51	2.15	3.42	3.28	2.82	2.55	3.78
16,000	3.46	1.80	2.97	3.84	1.19	2.05	1.22	2.23	1.78	2.50
17,000	2.68	1.81	2.48	3.13	1.18	1.77	0.96	2.03	1.39	2.63
18,000	1.83	1.48	1.41	2.21	1.01	1.34	0.60	1.72	1.04	1.87
19,000	1.58	1.42	1.18	1.49	1.26	1.18	1.21	1.53	0.71	1.54
20,000	1.02	0.94	0.70	0.87	0.82	0.79	2.29	1.06	0.49	0.96
21,000	0.88	0.74	0.75	0.75	1.01	0.67	1.61	0.83	0.59	0.69
22,000	0.83	0.45	0.80	0.40	0.60	0.52	0.66	0.74	0.31	0.41
23,000	0.74	0.43	0.38	0.66	0.41	0.47	0.24	0.84	0.27	0.27
24,000	0.55	0.29	0.10	0.51	0.23	0.27	0.32	0.56	0.37	0.30
25,000	0.58	0.15	0.12	0.25	0.14	0.14	0.29	0.31	0.31	0.31
26,000	0.43	0.17	0.03	0.13	0.14	0.15	0.11	0.17	0.27	0.12
27,000	0.32	0.19	0.02	0.21	0.11	0.10	0.04	0.22	0.14	0.09
28,000	0.24	0.29	0.02	0.09	0.10	0.06	0.05	0.12	0.11	0.06
29,000	0.15	0.19	0.01	0.16	0.07	0.03	0.02	0.14	0.06	0.06
30,000	0.09	0.11	0.00	0.01	0.07	0.07	0.06	0.25	0.06	0.06
31,000	0.09	0.08	0.00	0.01	0.06	0.04	0.02	0.17	0.06	0.04
32,000	0.11	0.07	0.00	0.04	0.06	0.03	0.01	0.16	0.06	0.04
33,000	0.10	0.04	0.02	0.03	0.06	0.02	0.03	0.15	0.04	0.02
34,000	0.07	0.04	0.00	0.00	0.05	0.05	0.05	0.13	0.05	0.04
35,000	0.04	0.03	0.00	0.00	0.05	0.03	0.02	0.15	0.05	0.03
36,000	0.04	0.04	0.00	0.01	0.04	0.01	0.01	0.09	0.08	0.02
37,000	0.01	0.04	0.01	0.01	0.04	0.03	0.01	0.08	0.09	0.04
38,000	0.01	0.02	0.00	0.01	0.03	0.02	0.01	0.06	0.08	0.03
39,000	0.03	0.01	0.00	0.00	0.04	0.01	0.02	0.06	0.06	0.03
40,000	0.01	0.04	0.02	0.05	0.05	0.01	0.03	0.08	0.09	0.04
41,000	0.05	0.13	0.05	0.03	0.15	0.06	0.08	0.18	0.16	0.10

Table 19. Statewide Tandem Axle Load Spectra

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
6,000	4.34	0.00	5.52	11.08	30.69	1.69	3.74	21.91	7.33	6.03
8,000	2.25	0.00	6.01	6.54	11.45	2.92	5.89	9.97	4.42	6.60
10,000	2.60	0.00	6.93	9.47	9.39	5.61	6.01	15.71	8.03	7.20
12,000	3.52	0.00	7.25	9.73	11.11	8.14	7.41	20.39	8.45	9.54
14,000	2.64	0.00	7.09	7.18	7.52	6.94	7.82	13.50	8.20	5.77
16,000	4.20	0.00	6.27	5.76	6.04	6.23	8.24	4.49	10.64	6.20
18,000	4.40	0.00	6.45	5.82	4.66	5.35	5.73	2.91	13.47	6.00
20,000	5.91	0.00	5.45	4.39	3.58	5.22	5.06	1.91	7.83	5.97
22,000	9.56	0.00	5.47	4.15	2.42	4.87	5.70	1.04	8.38	4.79
24,000	10.61	0.00	5.74	4.68	3.64	5.67	6.39	0.57	6.51	5.46
26,000	7.87	0.00	6.18	4.54	3.15	5.93	4.06	0.43	3.84	6.28
28,000	6.64	0.00	5.36	3.97	1.51	6.03	5.21	0.57	3.13	6.13
30,000	6.89	0.00	4.73	3.93	0.90	6.35	5.75	0.86	2.59	5.67
32,000	6.93	0.00	3.75	2.64	0.66	5.48	5.30	0.84	1.88	3.80
34,000	4.51	0.00	3.39	3.24	0.59	5.31	4.04	0.85	1.28	3.37
36,000	3.71	0.00	2.63	3.07	0.55	4.76	2.85	0.89	0.79	2.95
38,000	2.90	0.00	2.43	2.07	0.40	3.81	2.13	0.30	0.68	1.84
40,000	1.72	0.00	1.83	1.68	0.24	2.74	1.83	0.27	0.35	1.79
42,000	1.30	0.00	1.56	1.42	0.18	2.25	1.59	0.20	0.42	1.14
44,000	0.79	0.00	1.88	0.59	0.18	1.47	0.66	0.21	0.36	0.91
46,000	0.76	0.00	1.26	0.45	0.15	1.18	0.54	0.23	0.42	0.53
48,000	0.51	0.00	0.96	0.40	0.12	0.62	0.42	0.17	0.15	0.33
50,000	1.07	0.00	0.46	0.42	0.10	0.38	0.57	0.14	0.10	0.28
52,000	1.41	0.00	0.24	0.35	0.12	0.17	0.24	0.08	0.15	0.44
54,000	0.91	0.00	0.19	0.26	0.08	0.31	0.15	0.10	0.09	0.36
56,000	0.60	0.00	0.55	0.29	0.08	0.19	0.09	0.18	0.04	0.12
58,000	0.16	0.00	0.12	0.15	0.05	0.12	0.08	0.12	0.04	0.06
60,000	0.03	0.00	0.07	0.18	0.04	0.05	0.08	0.13	0.03	0.09
62,000	0.09	0.00	0.07	0.40	0.05	0.05	0.04	0.06	0.06	0.03
64,000	0.22	0.00	0.03	0.32	0.05	0.04	0.02	0.11	0.04	0.06
66,000	0.24	0.00	0.02	0.21	0.08	0.04	0.13	0.12	0.03	0.05
68,000	0.38	0.00	0.01	0.11	0.06	0.03	0.51	0.13	0.05	0.03
70,000	0.16	0.00	0.02	0.08	0.02	0.01	0.71	0.16	0.06	0.01
72,000	0.00	0.00	0.02	0.10	0.01	0.00	0.68	0.09	0.06	0.06
74,000	0.01	0.00	0.03	0.07	0.01	0.00	0.24	0.08	0.04	0.03
76,000	0.01	0.00	0.01	0.05	0.01	0.00	0.02	0.03	0.02	0.01
78,000	0.00	0.00	0.01	0.09	0.02	0.00	0.01	0.02	0.00	0.01
80,000	0.00	0.00	0.00	0.04	0.04	0.03	0.01	0.05	0.01	0.01
82,000	0.15	0.00	0.01	0.08	0.05	0.01	0.05	0.18	0.03	0.05

Table 20. Statewide Tridem Axle Load Spectra

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	42.61	13.22	14.86	40.49	12.16	3.66	30.50	19.41
15,000	0.00	0.00	7.04	3.73	9.56	12.48	7.10	3.84	6.29	7.94
18,000	0.00	0.00	7.37	4.61	25.09	9.37	5.68	16.10	14.17	5.64
21,000	0.00	0.00	9.01	6.32	22.10	7.78	5.51	22.67	3.32	3.85
24,000	0.00	0.00	8.84	5.22	13.32	3.49	4.62	9.36	1.36	3.05
27,000	0.00	0.00	7.59	6.66	2.38	4.49	4.11	8.81	4.76	4.87
30,000	0.00	0.00	7.06	7.04	1.71	6.07	7.31	1.71	8.20	7.18
33,000	0.00	0.00	1.46	6.45	1.08	2.40	6.40	4.17	7.21	10.89
36,000	0.00	0.00	4.40	8.94	0.51	3.14	8.83	2.37	4.84	9.89
39,000	0.00	0.00	1.25	8.90	0.64	1.93	8.71	0.71	3.61	6.94
42,000	0.00	0.00	1.28	6.76	0.68	1.79	7.36	0.68	2.13	5.11
45,000	0.00	0.00	1.20	5.90	0.55	1.63	6.54	1.19	1.91	5.20
48,000	0.00	0.00	0.47	5.37	0.64	1.69	5.39	0.23	1.84	2.64
51,000	0.00	0.00	0.22	3.33	0.28	1.46	3.16	0.74	1.62	1.22
54,000	0.00	0.00	0.18	2.43	0.57	0.29	2.42	5.72	1.76	1.41
57,000	0.00	0.00	0.01	1.82	0.42	0.27	1.48	2.87	1.06	1.22
60,000	0.00	0.00	0.01	1.14	0.46	0.17	1.24	3.80	0.74	0.57
63,000	0.00	0.00	0.00	0.60	0.37	0.09	0.51	4.92	1.03	0.68
66,000	0.00	0.00	0.00	0.27	0.75	0.07	0.48	1.44	0.56	0.51
69,000	0.00	0.00	0.00	0.25	0.71	0.18	0.27	1.95	0.13	0.35
72,000	0.00	0.00	0.00	0.09	0.27	0.09	0.24	1.53	0.33	0.29
75,000	0.00	0.00	0.00	0.09	0.43	0.02	0.08	0.34	0.34	0.10
78,000	0.00	0.00	0.00	0.12	0.67	0.02	0.05	0.00	0.17	0.11
81,000	0.00	0.00	0.00	0.02	0.46	0.05	0.10	0.00	0.59	0.08
84,000	0.00	0.00	0.00	0.02	0.06	0.08	0.01	0.00	0.86	0.13
87,000	0.00	0.00	0.00	0.02	0.11	0.02	0.01	0.40	0.14	0.04
90,000	0.00	0.00	0.00	0.04	0.41	0.05	0.03	0.00	0.16	0.12
93,000	0.00	0.00	0.00	0.21	0.16	0.00	0.02	0.00	0.13	0.11
96,000	0.00	0.00	0.00	0.02	0.14	0.00	0.03	0.08	0.22	0.03
99,000	0.00	0.00	0.00	0.02	0.05	0.08	0.02	0.71	0.02	0.05
102,000	0.00	0.00	0.00	0.39	0.56	0.31	0.13	0.00	0.00	0.37

Table 21. Statewide Quad Axle Load Spectra

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	0.00	10.85	27.34	18.21	4.77	0.00	14.78	8.29
15,000	0.00	0.00	0.00	3.91	8.72	6.68	3.52	0.00	4.66	2.56
18,000	0.00	0.00	0.00	3.22	6.30	13.83	2.94	2.72	3.31	3.06
21,000	0.00	0.00	0.00	4.57	6.60	10.70	2.27	16.20	5.90	2.04
24,000	0.00	0.00	0.00	6.90	2.62	8.81	1.91	17.69	7.13	1.86
27,000	0.00	0.00	0.00	7.74	5.86	6.19	2.55	10.22	6.20	2.22
30,000	0.00	0.00	0.00	6.54	5.18	3.71	2.34	6.51	7.84	3.20
33,000	0.00	0.00	0.00	4.61	3.54	1.08	3.47	9.77	2.08	6.76
36,000	0.00	0.00	0.00	2.94	1.35	2.05	5.47	13.31	3.97	3.74
39,000	0.00	0.00	0.00	3.88	4.80	4.52	9.09	10.48	9.08	4.61
42,000	0.00	0.00	0.00	3.56	4.73	3.38	6.89	9.99	4.38	4.79
45,000	0.00	0.00	0.00	2.82	5.68	2.40	10.90	2.53	2.93	5.77
48,000	0.00	0.00	0.00	4.11	1.24	2.12	10.80	0.58	1.91	4.29
51,000	0.00	0.00	0.00	4.83	2.22	0.72	9.04	0.00	0.37	5.44
54,000	0.00	0.00	0.00	3.69	2.53	1.13	6.06	0.00	1.22	3.99
57,000	0.00	0.00	0.00	2.84	1.25	2.85	4.23	0.00	0.13	4.85
60,000	0.00	0.00	0.00	1.48	1.64	0.95	2.69	0.00	1.06	4.74
63,000	0.00	0.00	0.00	1.36	2.01	1.80	2.46	0.00	0.13	4.72
66,000	0.00	0.00	0.00	1.27	2.05	1.50	2.16	0.00	0.93	4.02
69,000	0.00	0.00	0.00	1.33	0.51	1.60	1.78	0.00	2.45	4.60
72,000	0.00	0.00	0.00	1.64	0.47	0.74	1.50	0.00	2.40	4.17
75,000	0.00	0.00	0.00	1.28	1.03	0.81	1.23	0.00	3.14	1.83
78,000	0.00	0.00	0.00	1.16	0.00	1.64	0.58	0.00	3.84	1.41
81,000	0.00	0.00	0.00	1.65	0.00	0.70	0.20	0.00	4.12	1.00
84,000	0.00	0.00	0.00	0.75	0.04	1.71	0.11	0.00	1.94	1.13
87,000	0.00	0.00	0.00	1.89	0.21	0.17	0.08	0.00	1.31	1.01
90,000	0.00	0.00	0.00	1.90	0.25	0.00	0.07	0.00	1.00	0.60
93,000	0.00	0.00	0.00	2.42	0.20	0.00	0.14	0.00	0.17	0.58
96,000	0.00	0.00	0.00	1.65	0.20	0.00	0.14	0.00	0.09	0.57
99,000	0.00	0.00	0.00	1.20	0.64	0.00	0.09	0.00	0.26	0.27
102,000	0.00	0.00	0.00	2.01	0.79	0.00	0.52	0.00	1.27	1.88

Table 22. Number of Axles per Truck

FHWA Truck Class	Number of Axles			
	Single	Tandem	Tridem	Quad
Class 4	1.59	0.34	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.00	1.00	0.00	0.00
Class 7	1.00	0.22	0.83	0.10
Class 8	2.52	0.60	0.00	0.00
Class 9	1.25	1.87	0.00	0.00
Class 10	1.03	0.85	0.95	0.26
Class 11	4.21	0.29	0.01	0.00
Class 12	3.24	1.16	0.07	0.01
Class 13	3.32	1.79	0.14	0.02

Table 23. AADTT Traffic Distribution by Vehicle Class

FHWA Truck Class	Percentage of Truck
Class 4	0.9
Class 5	11.6
Class 6	3.6
Class 7	0.2
Class 8	6.7
Class 9	62.0
Class 10	4.8
Class 11	2.6
Class 12	1.4
Class 13	6.2

Table 24. Axle Configurations and General Traffic Data

Input	Value
Average Axle Width (edge to edge) Outside Dimensions (ft)	8.5
Dual Tire Spacing (in.)	12.0
Tire Pressure (psi)	120.0
Tandem Axle Spacing (in.)	51.6
Tridem Axle Spacing (in.)	49.2
Quad Axle Spacing (in.)	49.2
Design Lane Width (ft)	12.0
Standard Deviation of Traffic Wander (in.)	10.0

For MEPDG simulation runs to predict performance, the EICM module in the software was allowed to adjust the modulus of the unbound base/subbase materials and subgrade soil. It was performed this way to better simulate the actual field conditions.

Field Measured and MEPDG Predicted Performance

As explained in the previous chapter, ITD is using the Arizona method to evaluate the surface distresses. This method combines thermal cracking; bottom-up alligator fatigue cracking, and top-down longitudinal cracking into one Cracking Index (CI) on a scale from 0 to 5.0. On the other hand, MEPDG predicts each one of these distresses separately. The longitudinal and thermal cracking, in MEPDG, are predicted in units of ft/mile while the alligator fatigue cracking is computed as the area of cracking in (ft^2/500 ft/lane width) and expressed as a percentage. For rutting, ITD measures the total rutting at the AC surface. This total rutting includes, HMA as well as the unbound and subgrade layers. MEPDG, on the other hand, predicts the rutting within each layer. The total pavement rutting is then computed as the sum of the rutting occurring within each individual layer. Moreover, ITD's PPMIS reports the pavement roughness in terms of Roughness Index (RI) on a scale from 0 to 5, while the MEPDG reports it in terms of IRI in inches/mile. Unfortunately, this discrepancy in measuring and reporting the distresses did not allow for direct comparison of MEPDG predicted cracking and roughness with the measured field data for the investigated projects. The only distress that can be evaluated is the total pavement rutting.

Rutting Performance of the Investigated Projects

The field measured rutting for each of the investigated projects was recruited from ITD's PPMIS.[22] This data is only available until 2008. Simulation runs using MEPDG were performed on the investigated projects. MEPDG total predicted rutting, for each project, was compared with the available field measured rutting. This is shown graphically in Figure 29 through Figure 35**.** These figures show that, MEPDG significantly over predicts rutting for Idaho sections. This finding may explain why MEPDG designed pavement structures for Idaho conditions were governed by rutting.

Furthermore, these figures also show that there are small errors in the rutting measurements in the field. This is shown by the occasional decrease in the measured rutting for almost all of the investigated projects. Obviously, measured rutting should either increase with time or at least stays constant. These errors may be attributed to the variability of the relative location of rutting measurement from year to year and delineating the measured rutting over the pavement section. However, for all practical purposes these errors are not significant.

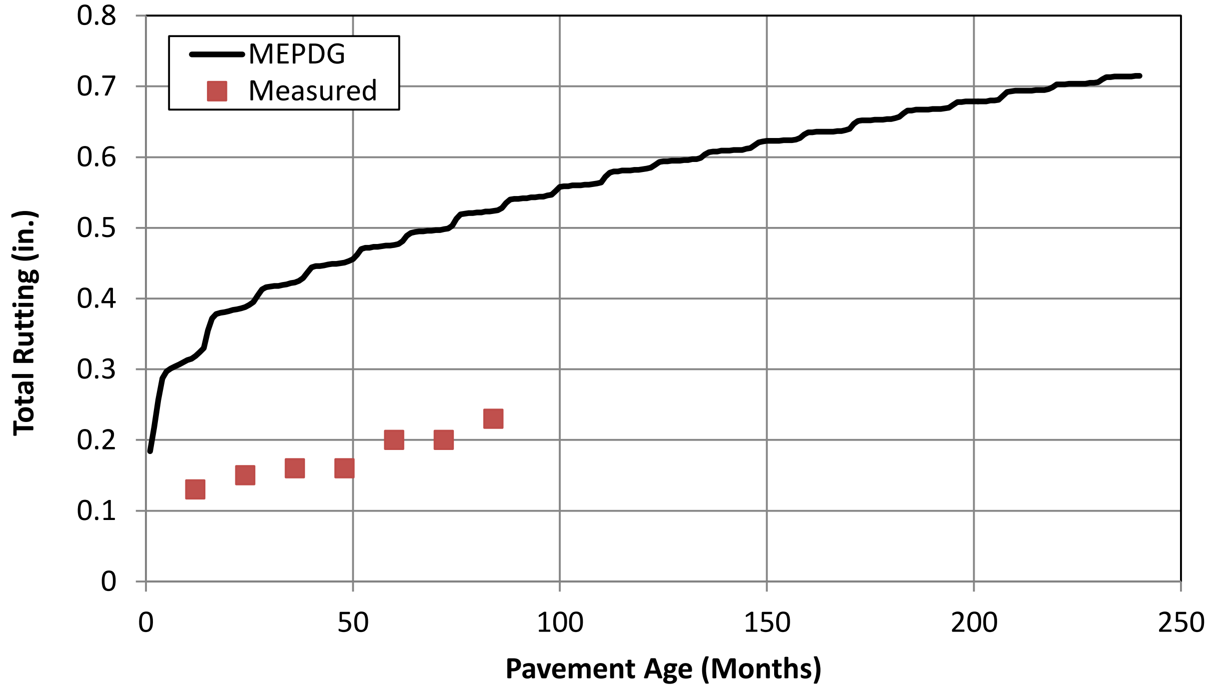

Figure 29. Comparison of MEPDG Predicted Rutting and Field Measured Rutting for US-2(a) Project

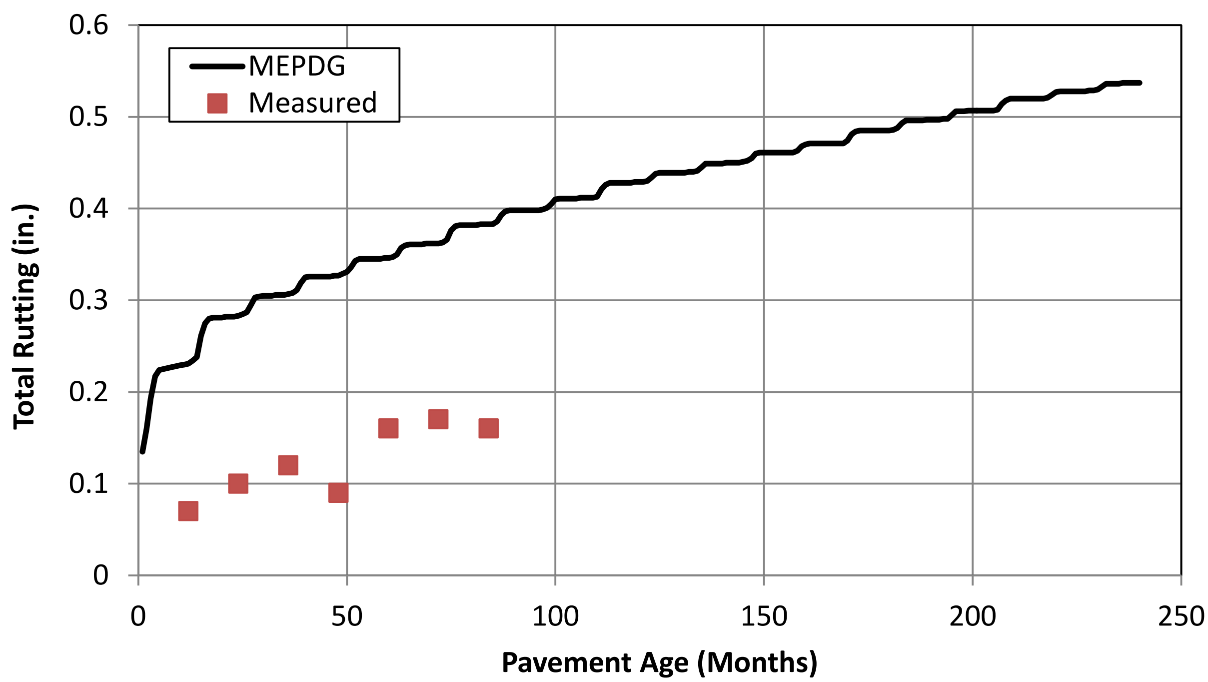

Figure 30. Comparison of MEPDG Predicted Rutting and Field Measured Rutting for US-2(b) Project

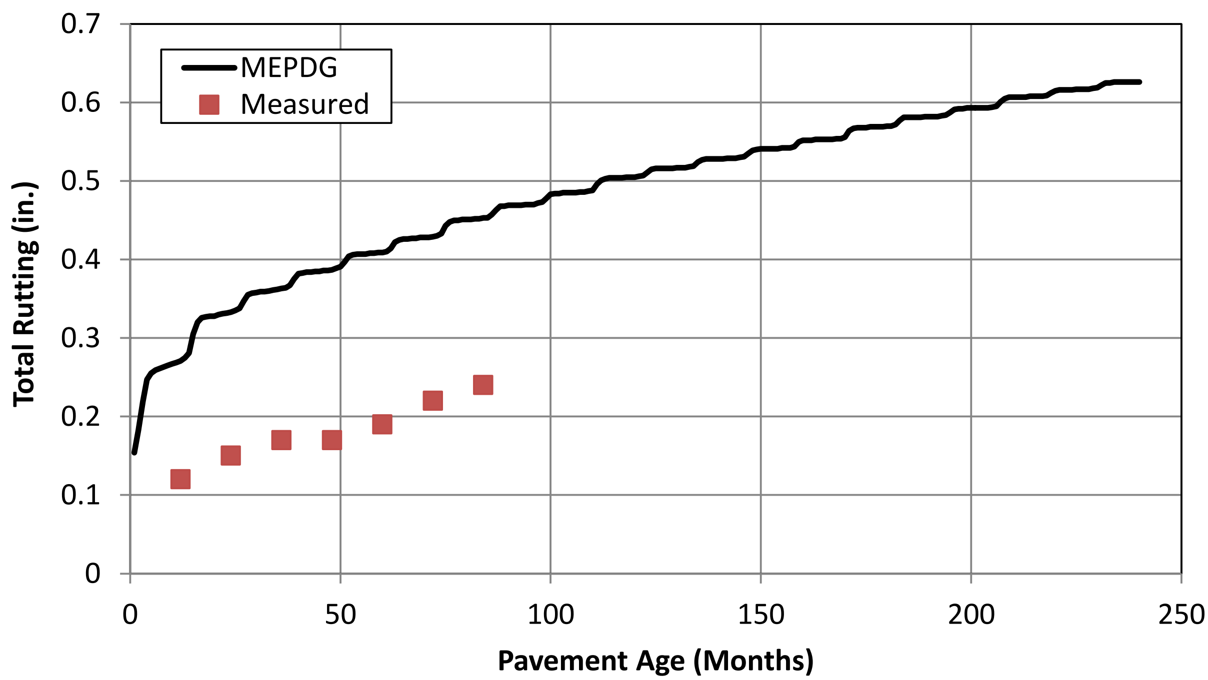

Figure 31. Comparison of MEPDG Predicted Rutting and Field Measured Rutting for US-2(c) Project

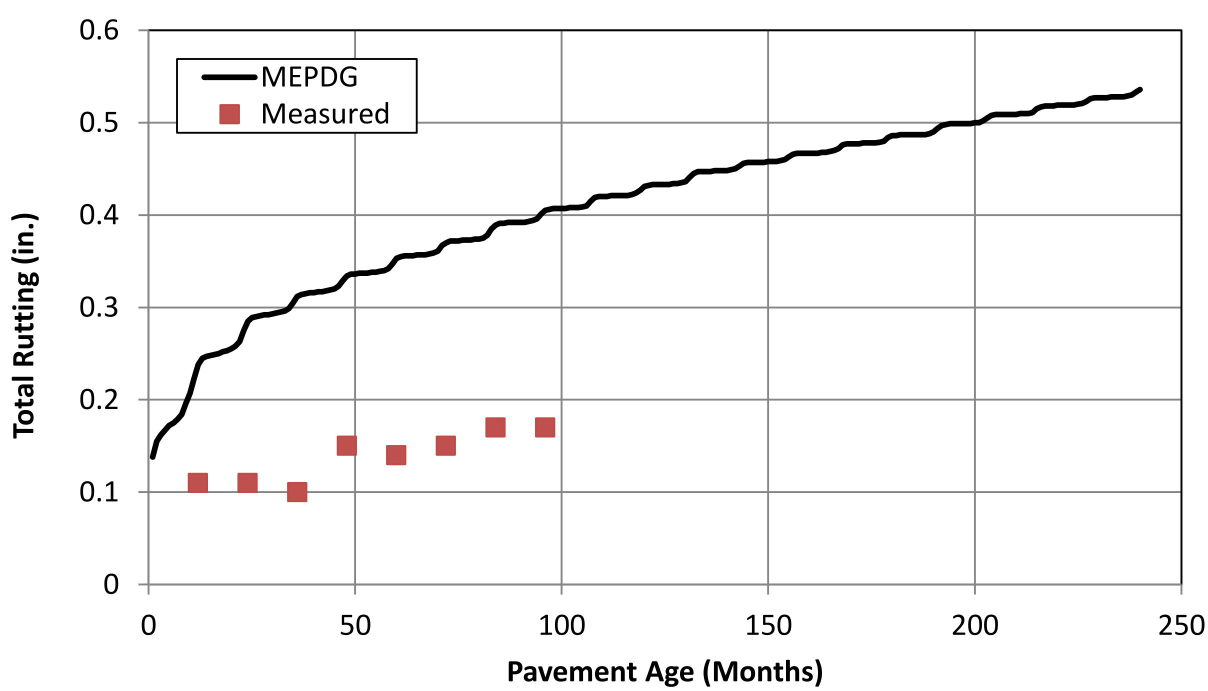

Figure 32. Comparison of MEPDG Predicted Rutting and Field Measured Rutting for SH-3 Project

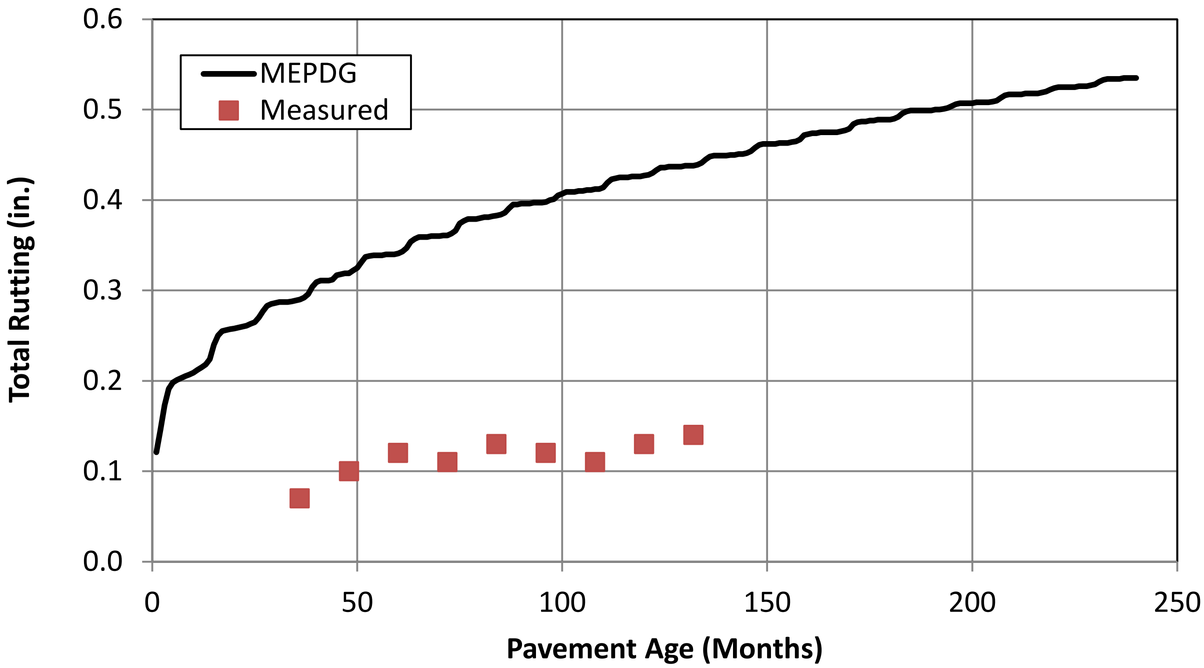

Figure 33. Comparison of MEPDG Predicted Rutting and Field Measured Rutting for SH-19 Project

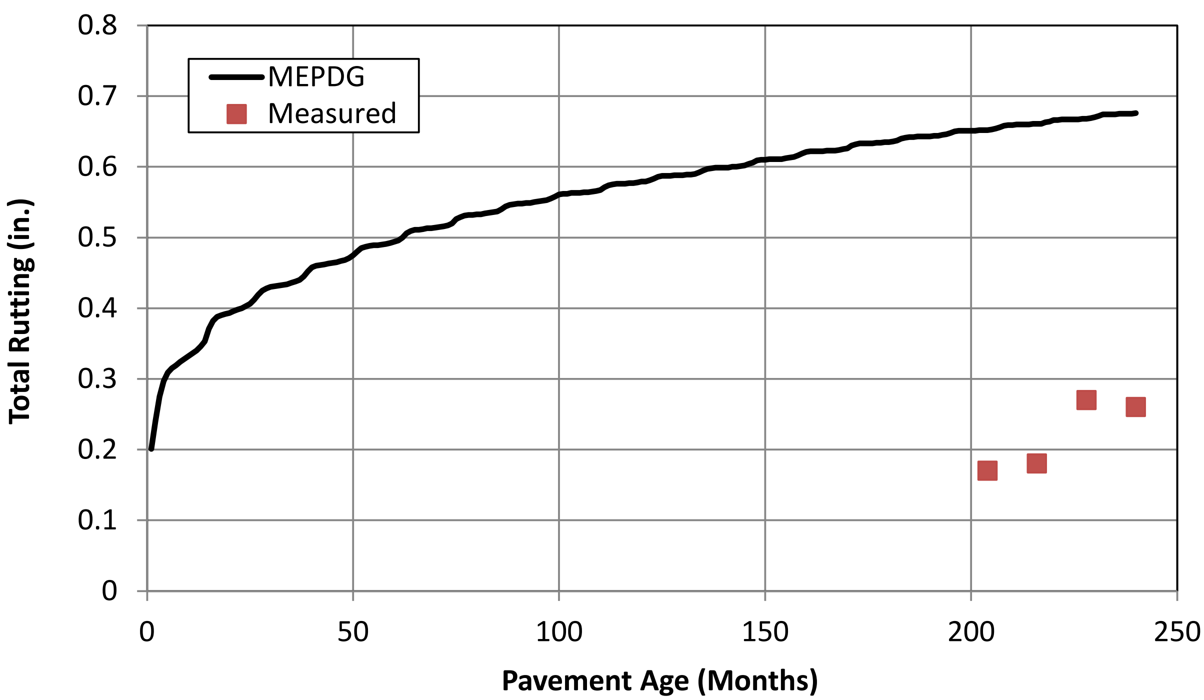

Figure 34. Comparison of MEPDG Predicted Rutting and Field Measured Rutting for SH-95 Project

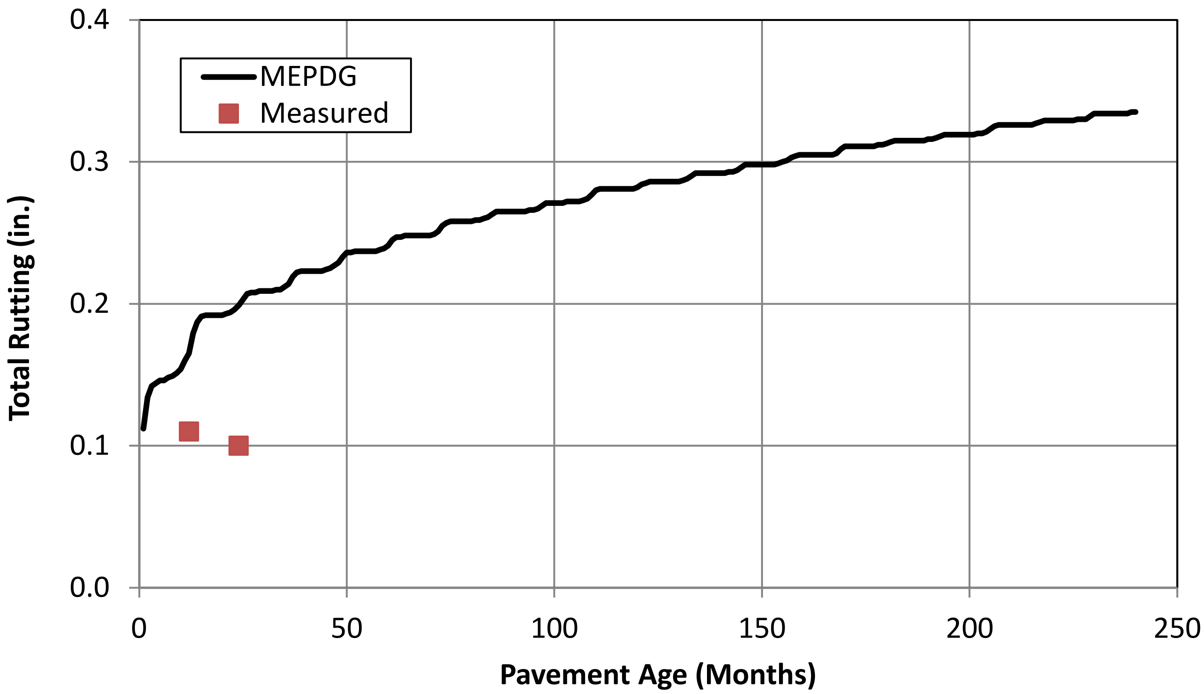

Figure 35. Comparison of MEPDG Predicted Rutting and Field Measured Rutting for SH-93 Project

Table 25 illustrates the contribution of each layer to the total rutting as predicted by MEPDG. All the investigated projects have very thick granular layers above the subgrade; one may surmise from this data that MEPDG overestimates the subgrade rutting. This table also shows some high rutting values in the unbound granular layer(s) predicted by MEPDG. Finally, the AC rutting shown in this table, for most of the investigated projects, also seems high. This reflects a need for local calibration of the MEPDG AC, granular, and subgrade rutting models.

Table 25. MEPDG Predicted Rutting for Each Individual Pavement Layer

ITD Project	MEPDG Predicted Rutting (in.)			
	AC	Unbound Granular Layer(s)	Subgrade	Total
US-2(a)	0.30	0.14	0. 28	0.72
US-2(b)	0.31	0.11	0.12	0.54
US-2(c)	0.30	0.15	0.18	0.63
SH-3	0.24	0.07	0.23	0.54
SH-19	0.25	0.07	0.21	0.54
US-95	0.18	0.23	0.27	0.68
US-93	0.14	0.09	0.10	0.34

A comparison between field measured and MEPDG predicted rutting for all projects combined in shown in Figure 36. Again, MEPDG rutting prediction is highly biased. It predicts significantly higher rutting values compared to the measured ones. This indicates the need for a local calibration of the rutting model in MEPDG. In order to reduce the highly biased MEPDG rutting predictions, a correction (calibration) factor of 0.526 was applied to the total predicted rutting. After the application of the calibration factor, this bias was significantly minimized as shown in Figure 37.

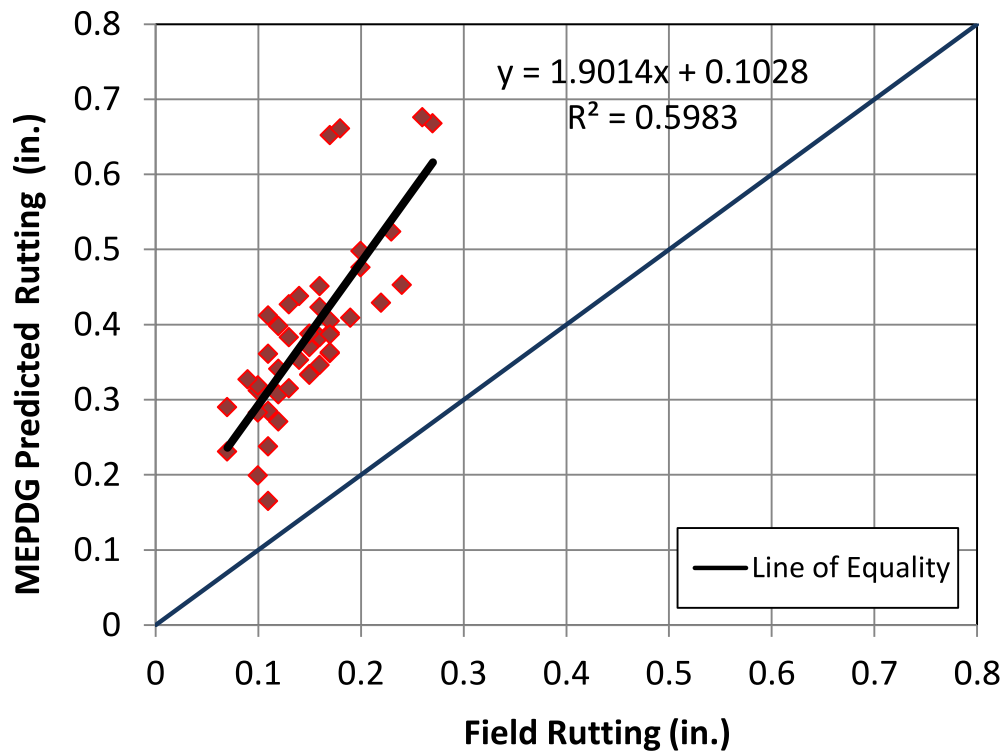

Figure 36. MEPDG Predicted Rutting Versus Field Measured Rutting for All Projects

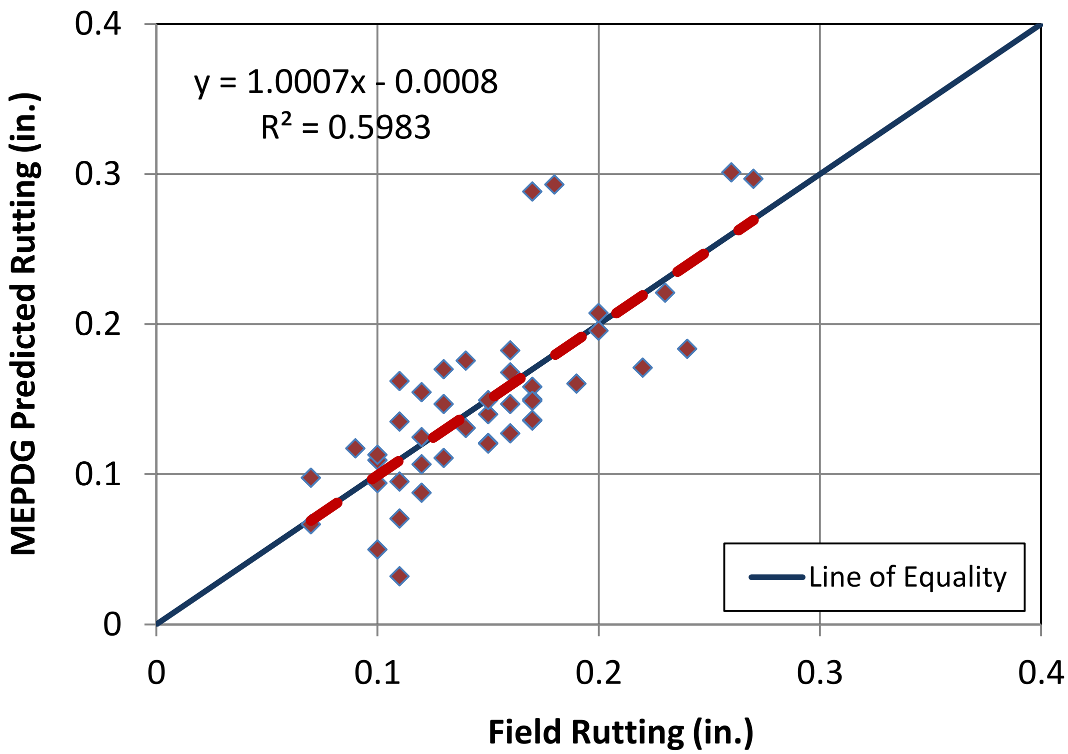

Figure 37. MEPDG Predicted Rutting Versus Field Measured Rutting for All Projects After Applying a Calibration Factor

However, this figure is still showing some scatter in the measured versus predicted rutting. This scatter can be minimized through a comprehensive calibration of MEPDG rutting models.

It should be noted that, this calibration factor is a direct multiplier on the total rutting. However, in order to calibrate the rutting models in MEPDG, separate calibration factors have to be developed for AC, unbound layers, and subgrade rutting.

It should also be noted that, using ALS and allowing EICM to adjust the modulus of the unbound base/subbase and subgrade layers contributed to a significant increase in the total predicted rutting for most of the investigated projects. This is shown in Table 26. EICM predicts moisture changes in the unbound materials and subgrade soils and adjusts the modulus accordingly. Figure 38 shows the change in MEPDG predicted modulus of the base and subbase layers of the US-2(c) project, with time, due to moister changes and freeze-and-thaw. The change in the predicted subgrade modulus with time due to moisture changes for the US-2(c) project is shown in Figure 39. These figures clearly explain the higher rutting occurring when EICM is allowed to adjust the modulus compared to using a representative (constant) modulus value. In addition, ALS is known to yield higher rutting values in the AC layers compared to ESALs.[4]

Table 26. Comparison of MEPDG Predicted Rutting Based on ALS and ESALs

ITD Project	MEPDG Predicted Rutting, in.	
	ALS, EICM	ESALs, Representative Modulus
US-2(a)	0.72	0.49
US-2(b)	0.54	0.43
US-2(c)	0.63	0.49
SH-3	0.54	0.44
SH-19	0.54	0.36
US-95	0.68	0.52
US-93	0.34	0.31

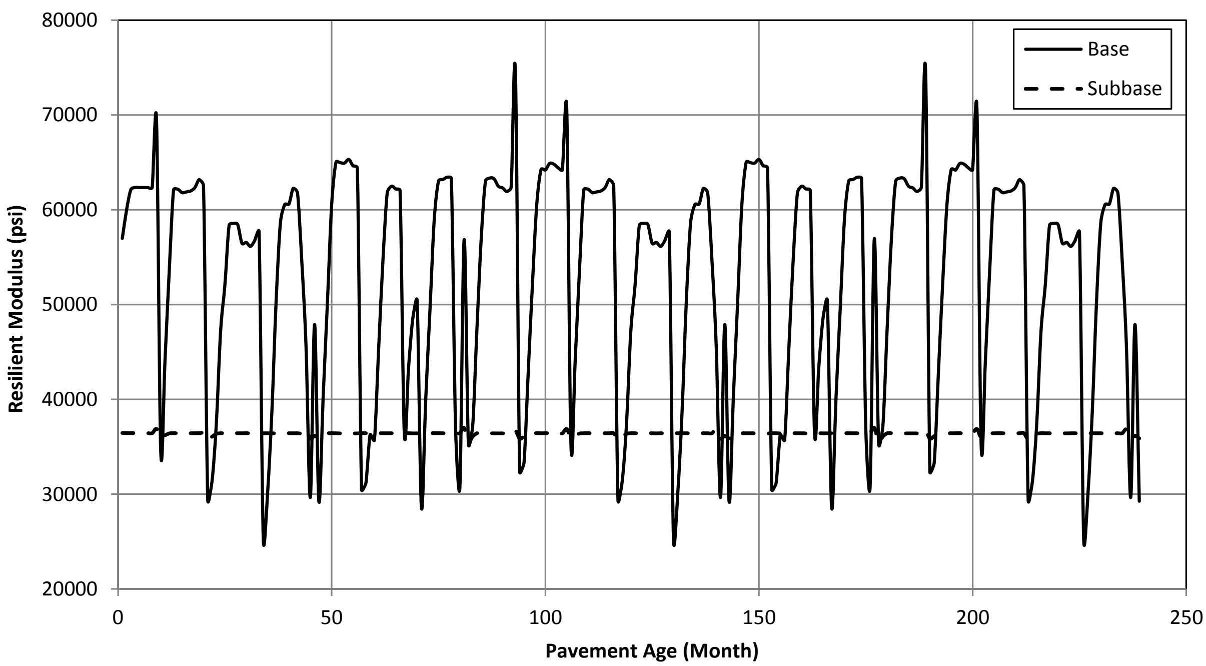

Figure 38. Influence of Moisture Changes and Freeze and Thaw on MEPDG Predicted Resilient Modulus of the Base and Subbase Layers of the US-2(c) Project

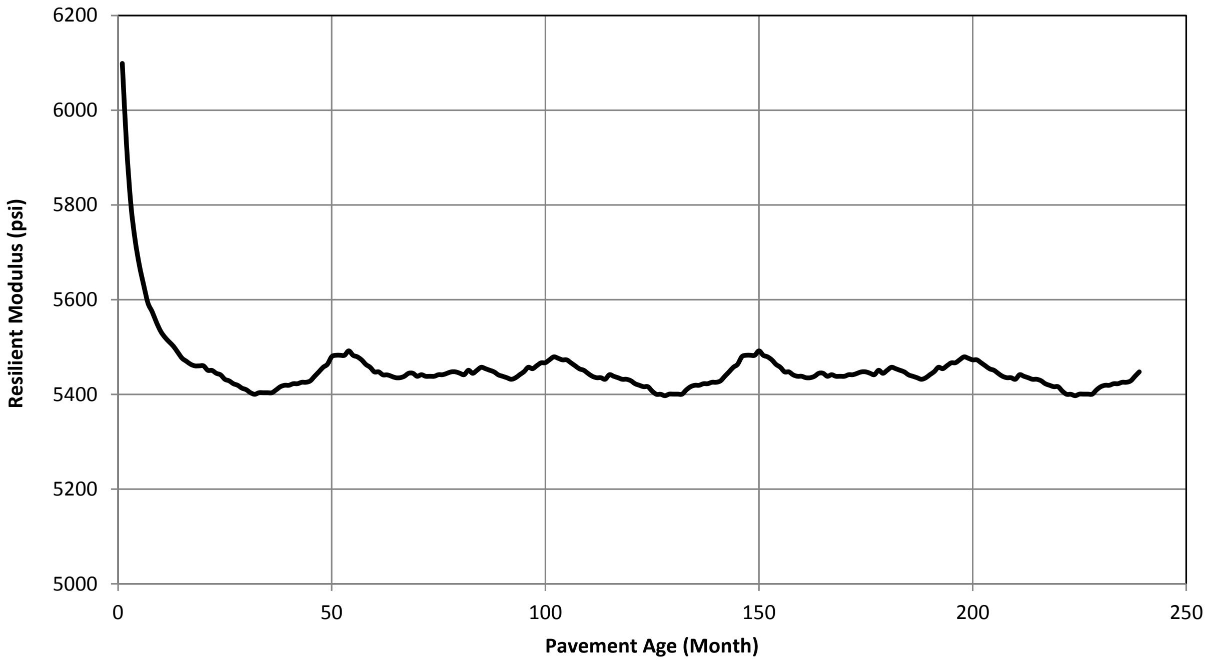

Figure 39. Influence of Moisture Changes on MEPDG Predicted Resilient Modulus of the Subgrade Soil of the US-2(c) Project

Cracking Performance of the Investigated Projects

MEPDG predicted top-down longitudinal cracking and bottom-up alligator fatigue cracking of the investigated projects are depicted in Figure 40 and Figure 41, respectively. No thermal cracking was predicted for any of the investigated projects. These figures indicate that in general the predicted cracking for the investigated projects was very little compared to MEPDG recommended threshold values.

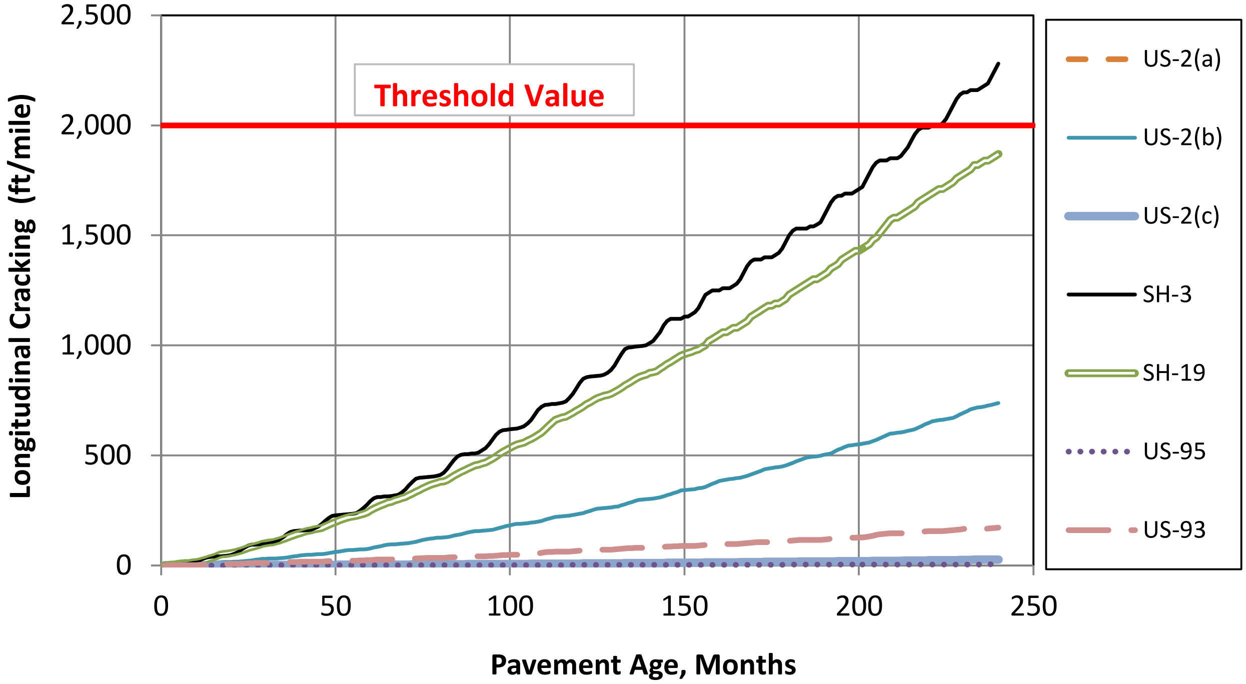

Figure 40. MEPDG Predicted Top-Down Longitudinal Cracking

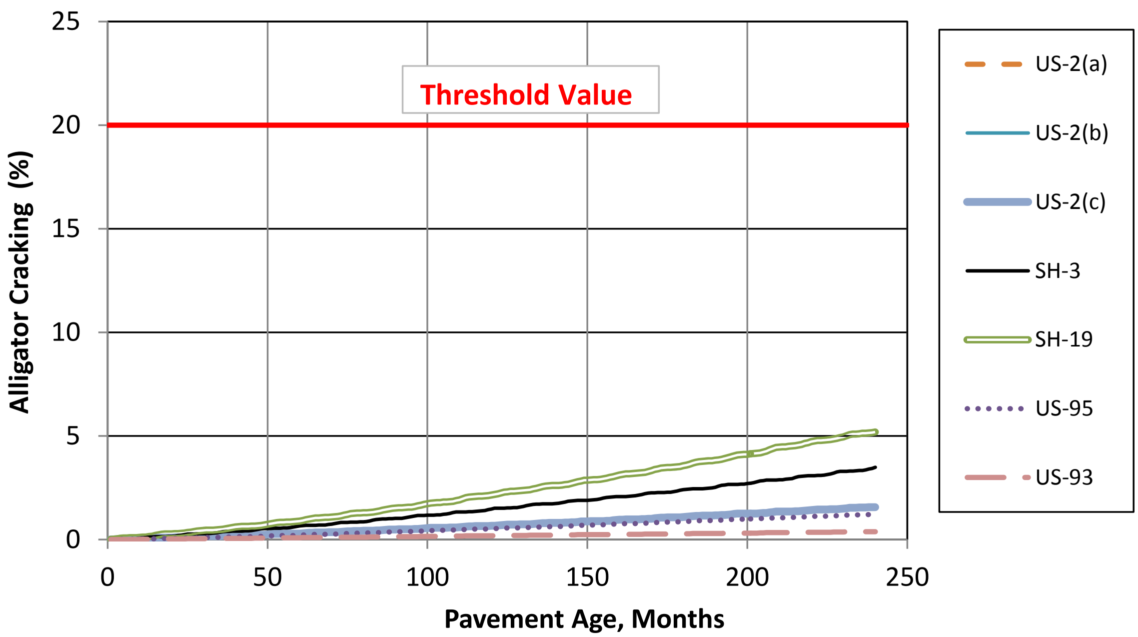

Figure 41. MEPDG Predicted Bottom-Up Fatigue Cracking

The ITD's *Pavement Rating Manual* explains in detail how surface distresses are to be measured.(33) Cracking is rated by severity and extent. A summary of ITD's distress evaluation method is shown in Table 27. In contrast, MEPDG cracks prediction models were calibrated based on LTPP data. This data was collected according to the LTPP distress survey method which is summarized in Table 28.(34) A comparison between these 2 tables reveals that ITD and LTPP cracking evaluation methods are different.

Table 27. ITD Distress Evaluation Method(33)

Distress	ITD Distress Evaluation	
	Severity	**How to Measure**
Alligator (Fatigue) Cracking	**Slight Severity:** Smaller than 1 ft in size **Moderate Severity:** 1 ft to 2 ft in size **Heavy Severity:** 3 ft or more in size	**Light Extent:** 10% or less of the total evaluation section having cracking. **Moderate Extent:** 10-40% of the total evaluation section having cracking. **Heavy Extent:** more than 40% of the total evaluation section having cracking.
Longitudinal Cracking	**Slight Severity:** Crack width is hairline up to ⅛ in. **Moderate Severity:** Crack width is ⅛ to ¼ in. or there is a dip 3 to 6 in. wide at the crack. **Heavy Severity:** Crack width is more than ¼ in. or there is a distinct dip of 6 to 8 in. wide or there is vegetation in the crack.	**Light Extent:** 100 ft or less of cracking per 500 ft. **Moderate Extent**: 100-500 ft of cracking per 500 ft. **Heavy Extent:** more than 500 ft of cracking per 500 ft.
Transverse Cracking	**Slight Severity:** Crack width is hairline up to ⅛ in. **Moderate Severity:** Crack width is ⅛ to ¼ in. or there is a dip 3 to 6 in. wide at the crack. **Heavy Severity:** Crack width is more than ¼" or there is a distinct dip of 6 to 8 in. wide or there is vegetation in the crack.	**Light Extent:** 1-4 cracks per 500 ft. **Moderate Extent:** 4-10 cracks per 500 ft. **Heavy Extent:** more than 10 cracks in 500 ft, or less than 50 ft in between cracks.

Table 28. LTPP Distress Evaluation Method[34]

<table>
<tr><th rowspan="2">Distress</th><th colspan="2">LTPP Distress Evaluation</th></tr>
<tr><th>Severity</th><th>How to Measure</th></tr>
<tr><td>Alligator (Fatigue) Cracking</td><td>Low Severity: An area of cracks with no or only a few connecting cracks; cracks are not spalled or sealed; pumping is not evident.

Moderate Severity: An area of interconnected cracks forming a complete pattern; cracks may be slightly spalled; cracks may be sealed; pumping is not evident.

High Severity: An area of moderately or severely spalled interconnected cracks forming a complete pattern; pieces may move when subjected to traffic; cracks may be sealed; pumping may be evident.</td><td>Record square meters of affected area at each severity level. If different severity levels existing within an area cannot be distinguished, rate the entire area at the highest severity present.</td></tr>
<tr><td>Longitudinal Cracking</td><td>Low Severity: Crack mean width is hairline up to ¼ in. or a sealed crack with sealant material in good condition and with a width that cannot be determined.

Moderate Severity: Any crack with a mean width > ¼ in. and ≤ ¾ in.; or any crack with a mean width ≤ ¾ in. and adjacent low severity random cracking.

High Severity: Any crack with a mean width > ¾ in. or any crack with a mean width ≤ ¾ in. and adjacent moderate to high severity random cracking.</td><td>Record separately:

Wheel Path Longitudinal Cracking
Record the length in meters of longitudinal cracking within the defined wheel paths at each severity level.
Record the length in meters of longitudinal cracking with sealant in good condition at each severity level.

Non-Wheel Path Longitudinal Cracking
Record the length in meters of longitudinal cracking not located in the defined wheel paths at each severity level.
Record the length in meters of longitudinal cracking with sealant in good condition at each severity level.</td></tr>
<tr><td>Transverse Cracking</td><td>Low Severity: Crack mean width is hairline up to ¼ in., or a sealed crack with sealant material in good condition and with a width that cannot be determined.

Moderate Severity: Any crack with a mean width > ¼ in. and ≤ ¾ in.; or any crack with a mean width ≤ ¾ in. and adjacent low severity random cracking.

High Severity: Any crack with a mean width > ¾ in.; or any crack with a mean width ≤ ¾ in. and adjacent moderate to high severity random cracking.</td><td>Record number and length of transverse cracks at each severity level.

Also record length in meters of transverse cracks with sealant in good condition at each severity level.</td></tr>
</table>

Thus, a direct comparison of MEPDG predicted cracks with ITD measured cracks is not feasible. However, trends in cracking between the different projects can be compared. In addition, comparison can also be made with respect to the closeness of the MEPDG predicted cracks to MEPDG threshold values and the closeness of the CI to the ITD threshold value at the year of interest. This comparison is shown in Table 29. Data in this table shows that, for most of the investigated projects, the higher the MEPDG predicted alligator cracking the lower is the CI. Furthermore, most of the investigated projects showed CI values close to 5 meaning no noticeable cracking. This agrees reasonably well with MEPDG predicted alligator cracking values for these projects which were in general very low compared to MEPDG recommended threshold values. On the other hand, the MEPDG predicted longitudinal cracking for the investigated projects did not agree well with ITD CI of these projects. This is clearly shown in Table 29 by the very high values of MEPDG predicted longitudinal cracking as well as the not matching trends in the CI and MEPDG predicted cracking between projects. These results indicate that the MEPDG longitudinal cracking model needs recalibration for Idaho conditions.

Table 29. Comparison of MEPDG Predicted Cracking and Cracking Index from ITD PPMIS

Design Project	US-2(a)	US-2(b)	US-2(c)	SH-3	SH-19	US-95	US-93
Years in Service	6	6	6	10	11	18	2
CI, from ITD's PPMIS	4.7	5.0	5.0	5.0	5.0	3.0	5.0
MEPDG Alligator Cracking (%)	0.37	0.35	0.36	1.48	2.41	1.08	0.04
MEPDG Longitudinal Cracking (ft/mile)	4.4	104.0	3.7	829.0	808.0	4.0	8.7
MEPDG Thermal Cracking (ft/mile)	0	0	0	0	0	0	0

International Roughness Index

MEPDG predicts the IRI of the pavement over time as a function of the initial pavement IRI, fatigue cracking, transverse cracking, average rut depth, and site factors. MEPDG predicted rutting and cracking of the investigated projects indicated that these models need recalibration for Idaho conditions. Once these models are recalibrated, the IRI model will also need to be recalibrated as it is dependent on the predicted rutting and cracking.

Performance at Different Reliability Levels and Longer Pavement Age

The performance of the investigated projects using MEPDG software was evaluated at 2 reliability levels of 50 and 85 percent. Longer service life of 40 years was used in this analysis. This was done to investigate the failure age of these projects. Figure 42 shows the predicted rutting of the investigated projects at 50 percent reliability level. It should be noted that, a local calibration factor of 0.526 was applied to the MEPDG predicted rutting values shown in this figure. MEPDG predicted alligator cracking and IRI at 50 percent reliability level are shown in Figure 43 and Figure 44, respectively. The data in these figures show that all investigated projects performed well according to MEPDG failure criteria. None of the projects failed even after 40-years of service.

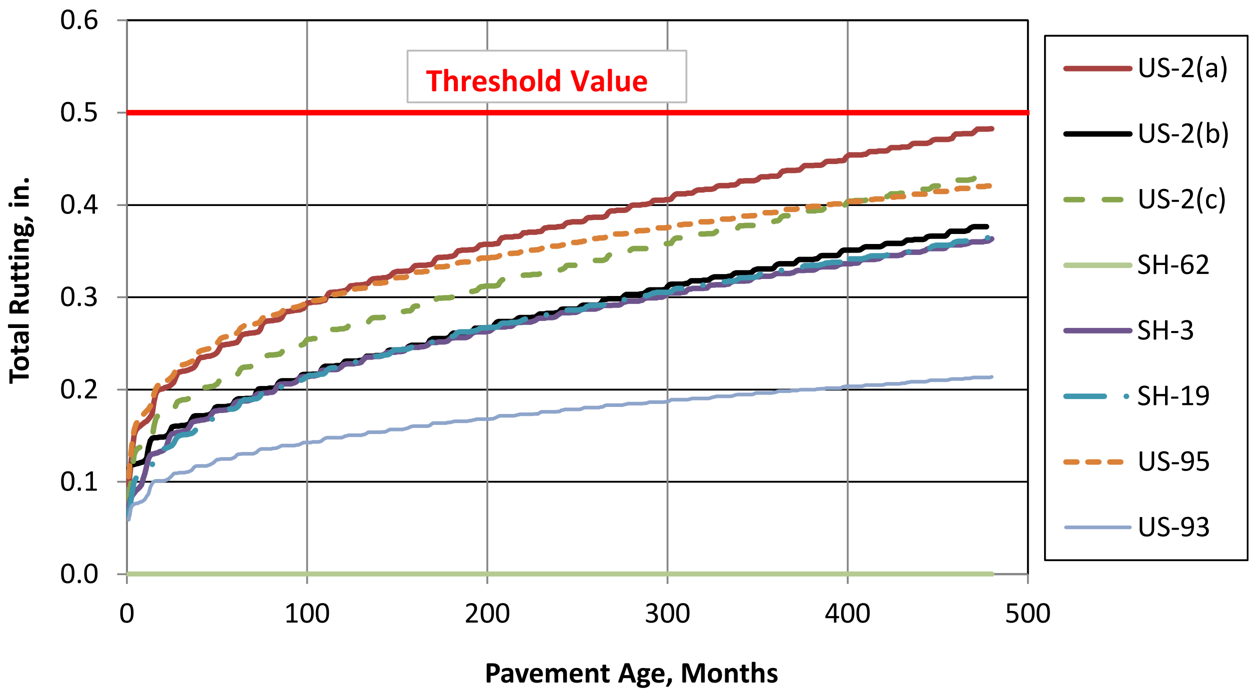

Figure 42. MEPDG Predicted Total Rutting at 50 Percent Reliability Level for 40-Years Service Life

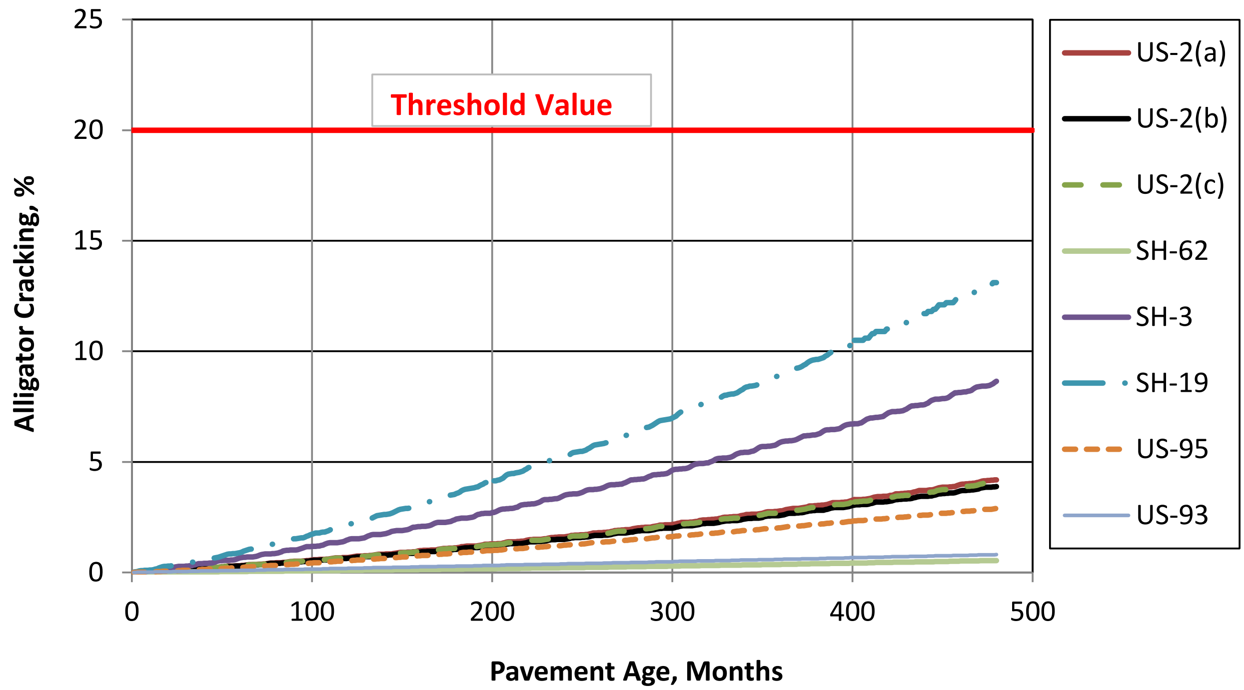

Figure 43. MEPDG Predicted Alligator Cracking at 50 Percent Reliability Level for 40-Years Service Life

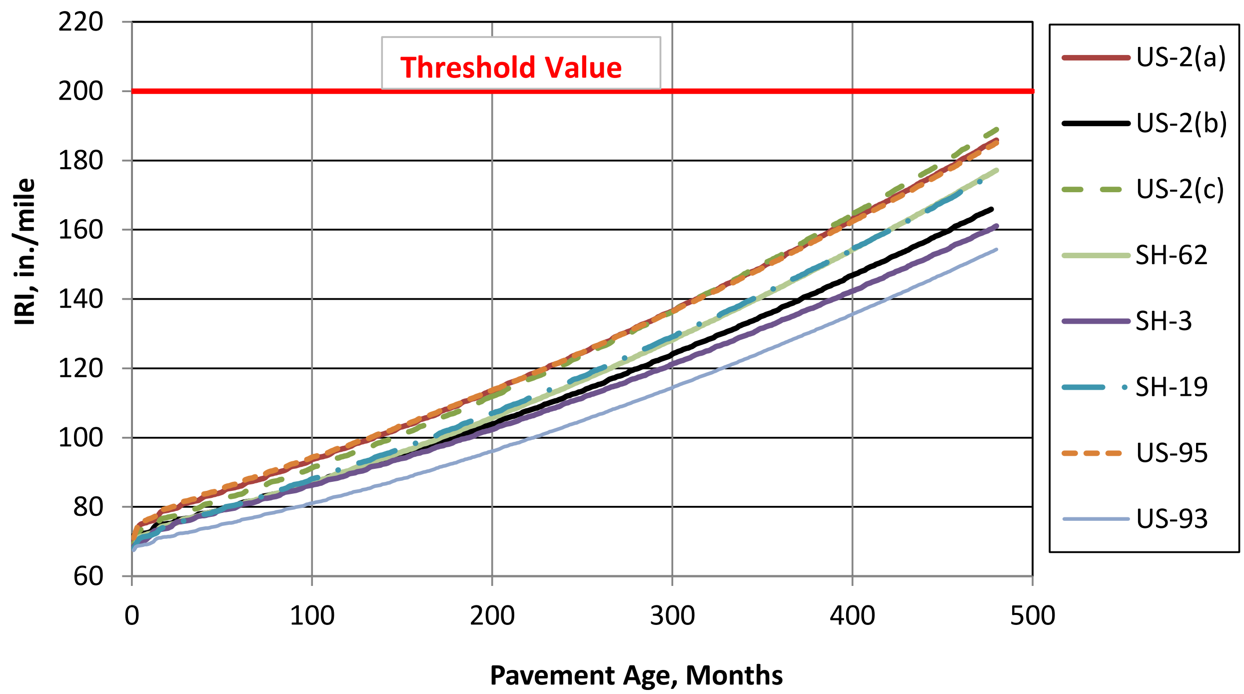

Figure 44. MEPDG Predicted IRI at 50 Percent Reliability Level for 40-Years Service Life

MEPDG predicted rutting, alligator cracking and IRI at 85 percent reliability level are shown in Figure 45 through Figure 47, respectively. These figures show that, even at the higher reliability level, most of the investigated projects performed well beyond their design life (20-years). Few projects failed due to alligator cracking and rutting but after the 20-years design life. Most of the projects failed due to IRI. However, the earliest failure was predicted to occur after 31-years.

It should be noted that alligator cracking and IRI predictions are based on the nationally calibrated models. These results may change after recalibrating these models for Idaho conditions.

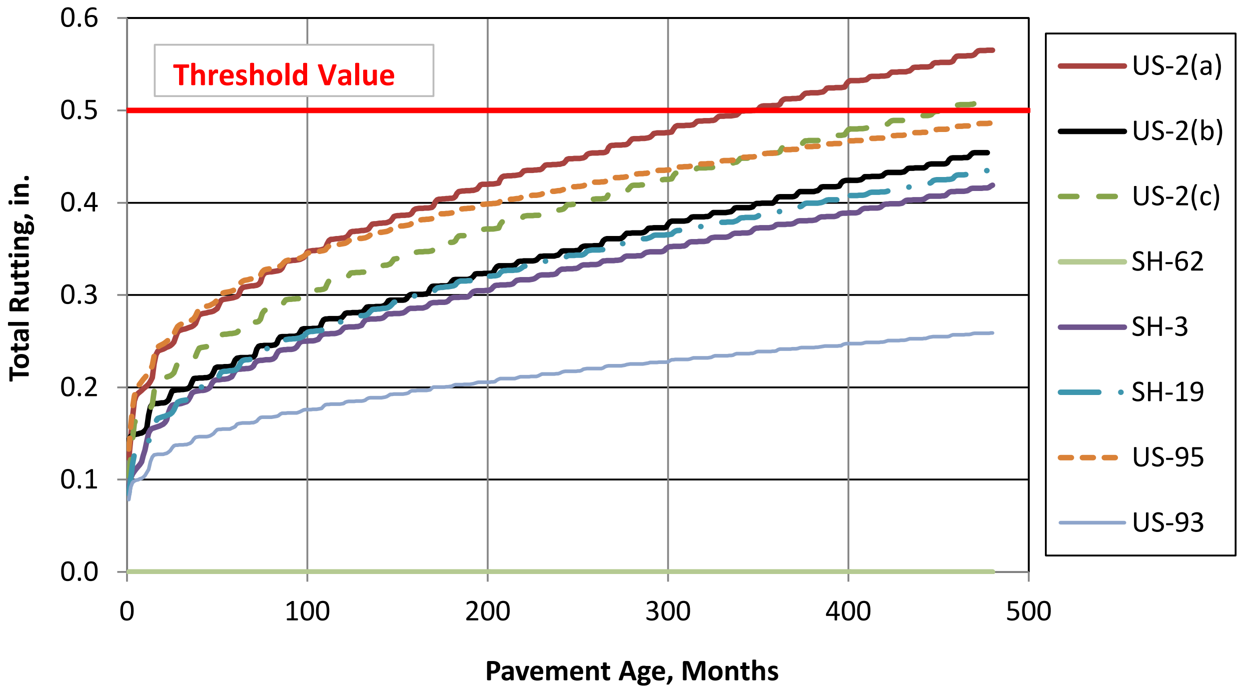

Figure 45. MEPDG Predicted Total Rutting at 85 Percent Reliability Level for 40-Years Service Life

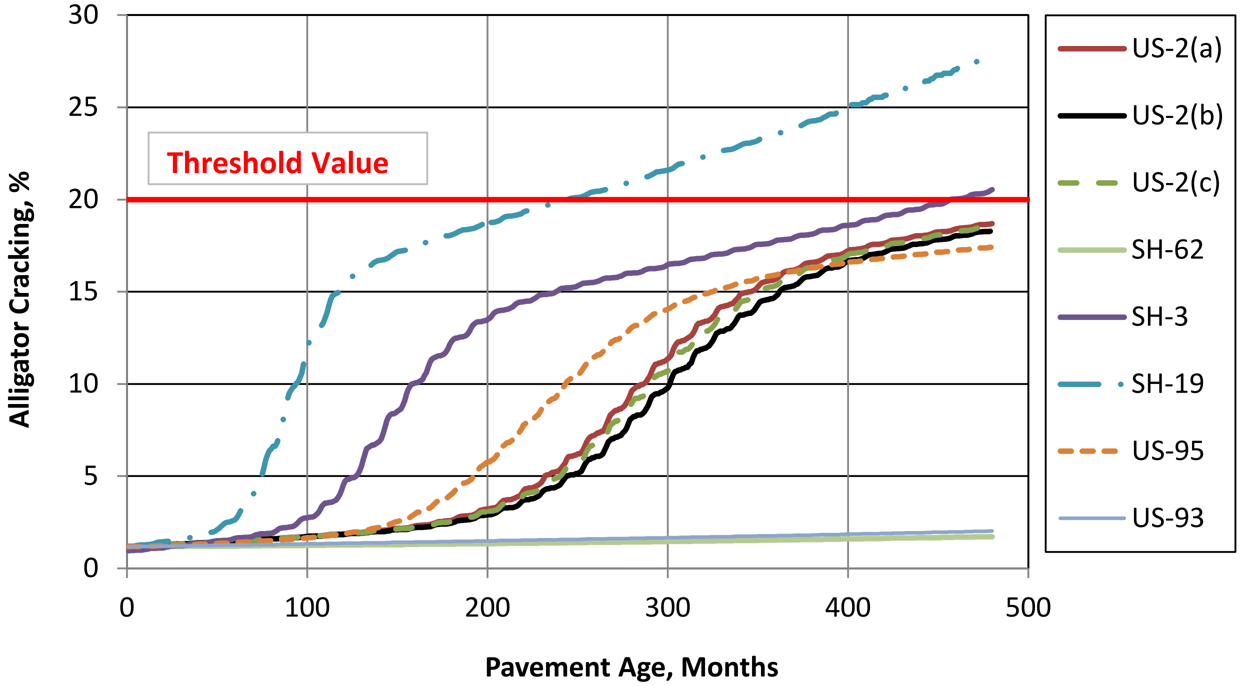

Figure 46. MEPDG Predicted Alligator Cracking at 85 Percent Reliability Level for 40-Years Service Life

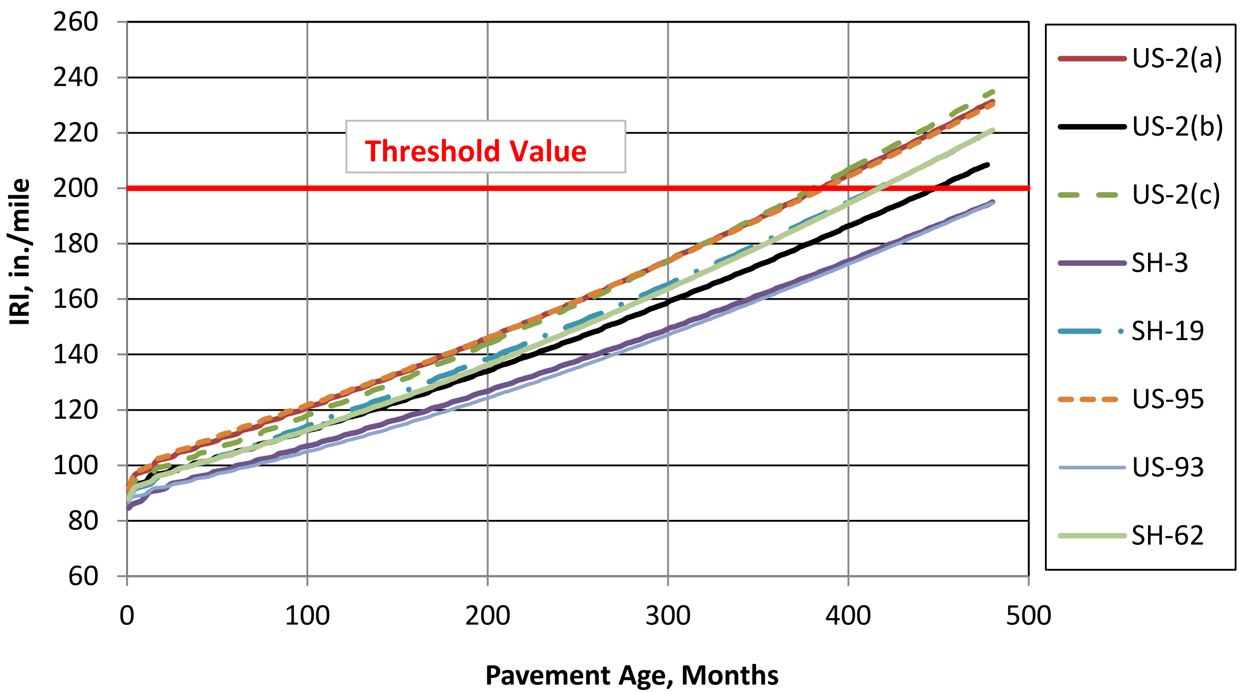

Figure 47. MEPDG Predicted IRI at 85 Percent Reliability Level for 40-Years Service Life

Chapter 5
Comparison of ITD and Other States ESAL Calculation Methods

Traffic is one of the primary inputs to any pavement design methodology. ITD's design method is using TI as the primary traffic load input. The Traffic Index is a function of ESALs which is widely used to express traffic loading by many pavement design procedures. ESAL was developed based on the AASHO road test results.(3) It is a way to convert axle loads of various magnitudes and repetitions to an equivalent number of repetitions of the standard axle which is 18,000 lb single axle.

This chapter evaluates the current ITD factors to estimate traffic ESALs. It also compares between ESALs calculated by ITD's factors and ESALs calculated by factors from states adjacent to Idaho in addition to the AASHTO factors. Furthermore, this chapter presents the details of the development of new truck factors for ESAL calculation developed for Idaho based on the analysis of traffic WIM data. Finally, an evaluation of the current ITD traffic projection method is also presented.

AASHTO 1993 ESAL Calculation

In general, ITD and almost all state DOTs follow the AASHTO 1993 general equation for ESAL calculation which is shown in Figure 48.

$$\text{ESAL} = \left[\sum_{i=1}^{m}(p_i)(\text{LEF})\right](\text{ADT})(\text{T})(365)(\text{D})(\text{F})(\text{G})$$

where:

ESAL	= 18-kip equivalent single axle load
p_i	= percentage of total repetitions for the i^{th} axle load group
LEF	= load equivalency factor for the load group
ADT	= base year average daily traffic for each truck class
T	= percentage of trucks in the ADT
D	= directional distribution factor
F	= lane distribution factor
G	= growth factor

Figure 48. AASHTO 1993 Formula for ESAL Calculation

The load equivalency factors (LEFs) are based on the axle weights of the axle group and the structural number (SN) and terminal serviceability of the pavement. The set of equations shown in Figure 49 can be used to determine the LEF.(35)

$$\mathrm{LEF} = \frac{W_{t18}}{W_{tx}}$$

$$\log\left(\frac{W_{tx}}{W_{t18}}\right) = 4.79\log(18+1) - 4.79\log(L_x + L_2) + 4.33\log(L_2) + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}}$$

$$G_t = \log\left(\frac{4.2 - p_t}{4.2 - 1.5}\right)$$

$$\beta_x = 0.40 + \frac{0.08(L_x + L_2)^{3.23}}{(SN+1)^{5.19} L_2^{\ 3.23}}$$

where:

W_{tx} = number of applications of given axle

W_{t18} = number of standard axle passes (18-kip single axle)

L_x = load of axle group being evaluated, kip

L_2 = axle code (1 for single axle, 2 for tandem axle, and 3 for tridem axle)

β_{18} = value of β_x when L_x = 18 and L_2 = 1

p_t = terminal serviceability index (point at which the pavement is considered to be at the end of its useful life)

SN = structural number

Figure 49. AASHTO 1993 Equation to Calculate the Load Equivalency Factor

The sum of all LEFs for each truck is called the truck factor (TF). If the TF is known, the ESAL can be computed using the formula in Figure 50.

$$\mathrm{ESAL} = (\mathrm{ADT})(\mathrm{T})(365)(\mathrm{D})(\mathrm{F})(\mathrm{G})(\mathrm{TF})$$

Where: all terms in the above equation are defined in Figure 49.

Figure 50. Equation to Determine ESALs as a Function of Truck Factor

The recommended truck factors for rural and urban roadway systems in the U.S. are shown in Table 30 and Table 31, respectively.

The main difference between state DOTs and AASHTO ESAL calculation is that each state has developed its own factors (D, F, G, and TF) based on their local conditions.

Table 30. Truck Factors for Different Rural Highways and Vehicles in the U.S.[35]

Vehicle Type	Interstate	Other Principal	Minor Arterial	Major Collectors	Minor Collectors	Range
Single-Unit Trucks						
2-axle, 4-tire	0.003	0.003	0.003	0.017	0.003	0.003-0.017
2-axle, 6-tire	0.21	0.25	0.28	0.41	0.19	0.19-0.41
3-axle or More	0.61	0.86	1.06	1.26	0.45	0.45-1.26
All Single Units	0.06	0.08	0.08	0.12	0.03	0.03-0.12
Tractor Semi-Trailers						
4-axle or Less	0.62	0.92	0.62	0.37	0.91	0.37-0.91
5-axle	1.09	1.25	1.05	1.67	1.11	1.05-1.67
6-axle or More	1.23	1.54	1.04	2.21	1.35	1.04-2.21
All Multiple Units	1.04	1.21	0.97	1.52	1.08	0.97-1.52
All Trucks	0.52	0.38	0.21	0.30	0.12	0.12-0.52

Table 31. Truck Factors for Different Urban Highways and Vehicles in the U.S.[35]

Vehicle Type	Interstate	Other Freeways	Other Principal	Minor Arterial	All Collectors	Range
Single-Unit Trucks						
2-axle, 4-tire	0.002	0.015	0.002	0.006	-	0.006-0.015
2-axle, 6-tire	0.17	0.13	0.24	0.23	0.13	0.12-0.24
3-axle or More	0.61	0.74	1.02	0.76	0.72	0.61-1.02
All Single Units	0.05	0.06	0.09	0.04	0.16	0.04-0.16
Tractor Semi-Trailers						
4-axle or Less	0.98	0.48	0.71	0.46	0.40	0.40-0.98
5-axle	1.07	1.17	0.97	0.77	0.63	0.63-1.17
6-axle or More	1.05	1.19	0.90	0.64	-	0.64-1.19
All Multiple Units	1.05	0.96	0.91	0.67	0.53	0.53-1.05
All Trucks	0.39	0.23	0.21	0.07	0.24	0.07-0.39

ITD ESAL Calculation

ITD usually designs flexible pavements for 20 years. In order to calculate ESALs, ITD first classifies traffic into passenger traffic and commercial traffic. Commercial traffic includes all types of trucks (Truck Class 4 to 14). In order to forecast the future 20 years of traffic volume, ITD uses a simple technique. ITD assumes that past trends in percent increase in traffic volume each year will continue in the future. This applies on both types of traffic (passenger and commercial) separately.

To estimate the traffic volume in the design lane, ITD assumes a directional distribution of 50/50 or 60/40 and a lane distribution factor based on the number of lanes in each direction as shown in Table 32.

Table 32. ITD Lane Distribution Factors[2]

Number of Lanes Per Direction	Percent CADT* in Design Lane
1	100
2	70 – 100
3	60 – 80
4	50 - 75

* CADT = Commercial Average Daily Traffic

Once the cumulative commercial traffic in the design lane during the service life of the pavement is determined, ESALs are then calculated by multiplying the percent Commercial Average Daily Traffic (CADT) by the TF. ITD's TF vary by commercial traffic class and the year of interest. According to ITD, commercial traffic is classified into "Heavy," "Medium," and "Light" based on the percentages of two- and five-axle trucks within the CADT according to Table 33.

Table 33. ITD Commercial Traffic Classification[2]

Classification	Percent of Commercial Truck Volume (CADT)	
	Two-Axle	**Five-Axle**
Heavy	30 - 50	25 - 40
Medium	50 - 70	10 – 25
Light	70 - 100	0 - 10

Notes: If the two-axle classification differs from the five-axle, use the higher classification for design. Interstate highways are always classified as 'Heavy"

It should be noted that, this classification was first developed in 1964 and was slightly revised in 1984.[17] It was developed originally based on relating the 5 kip-EWL to the distribution of the 2-axle and 5-axle commercial vehicles.[16, 17, 41] Current ITD truck factors for each commercial traffic class and year of interest (1970 through 2060) are shown graphically in Figure 51. One can see that both passenger cars and light commercial traffic class truck factors are constant, while the truck factors for all other commercial traffic classes increase linearly from year to year (TF growth). There is also a shift in the TF starting in 1991, which may reflect an update in these factors around that time.

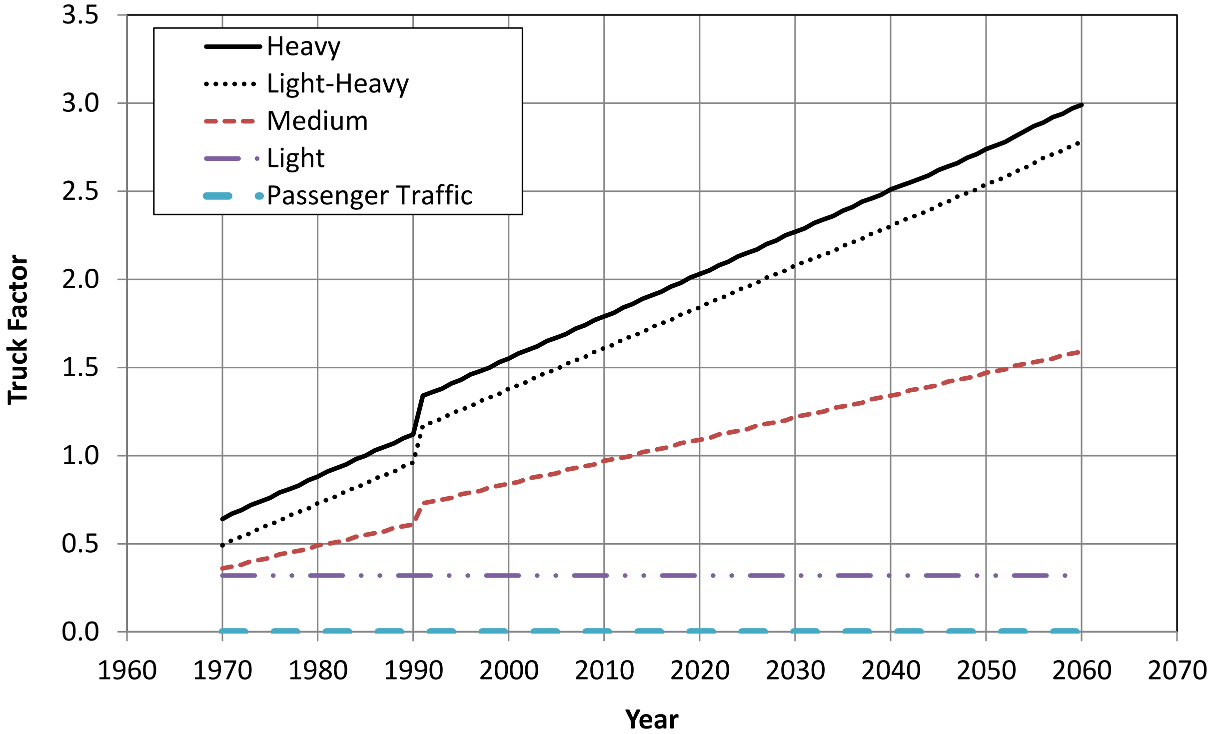

Figure 51. ITD Current Truck Factors

Washington State Department of Transportation ESAL Calculation

Washington State Department of Transportation (WSDOT) estimates their design ESALs for specific pavement section by first forecasting the traffic the pavement will be subjected to over its design life. Then, they convert the traffic volume to ESALs based on truck factors developed for each truck class. WSDOT classifies trucks into three categories; single units, double units, and trains.(36) These truck categories are simplified version of the FHWA truck classes shown in Figure 52.(37) WSDOT truck factors from their Pavement Management System (PMS) and the equivalent FHWA truck classes are illustrated in Table 34.(36) This table shows the TF developed based on the initial analysis of 10 WIM site data in Washington. A comparison between these TF shows that TF based on the WIM analysis are a little lower than those from WSDOT PMS.

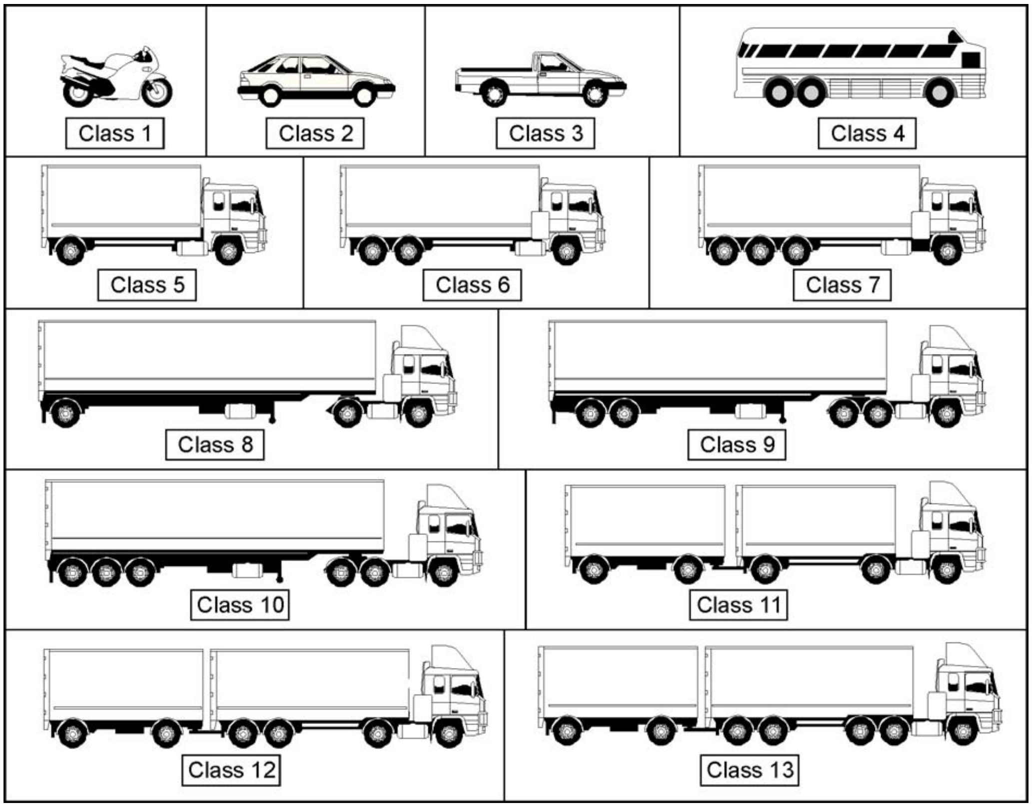

Figure 52. FHWA Vehicle Classes Used for Collecting Traffic Data[37]

Table 34. WSDOT Truck Factors from PMS and WIM Analysis[36]

WSDOT Category	FHWA Classes	TF from PMS	TF from WIM Analysis
Single Units	4, 5, 6, 7	0.40	0.37
Double Units	8, 9, 10	1.00	1.02
Trains	11, 12, 13	1.75	1.22

WSDOT also developed truck factors based on data collected between 1960 and 1983. These truck factors are shown in Table 35. These factors were calculated based on data from limited number of weigh stations using the AASHTO 1993 equation for LEF (Figure 49 in this report) at a structure number (SN = 5) and a terminal serviceability (p_t = 2.5). It was noted that these truck factors may be biased.[36]

Table 35. Typical WSDOT Truck Factors Based on Measurement[36]

Highway System	Truck Factors				
	Single Units	Combination Units	Buses	Individual Axle	Overall Trucks (Excludes Buses)
Interstate	0.30	1.20	1.60	0.25	1.10
Non-Interstate Rural	0.50	1.40	1.60	0.25	1.40
Non-Interstate Urban	0.25	1.20	1.60	0.25	1.00

WSDOT's applies ESAL growth rate using the equation in Figure 53.

$$\text{Total ESAL Growth Rate} = (1 + G)(1 + 0.016) - 1.0$$

where:

G = traffic growth rate (2% minimum)

0.016 = additional growth rate assumed to account for the increase in per-tire load.

Figure 53. WSDOT Equation for ESAL Growth Rate

Utah Department of Transportation ESAL Estimation

Utah Department of Transportation (UDOT) uses the following sampling process to produce the design factors and growth rates for volume, vehicle classifications and WIM data for ESALs for each vehicle classification to be used in the UDOT design process.[14] Traffic volume is collected at over 5,000 sites, with one third (1,667) of the sites collected annually. At 3 sites, volume is collected for a minimum of 48 hours. From the volume sites, 300 random sites were selected for a traffic classification count. These counts are performed over a 3-year period, with 100 of the classification counts performed annually. A total of 90 WIM sites were selected out of the 300 classification sites in order to calculate TF for different roadway functional classes. For sites with no traffic data, manual counts are performed. The AASHTO 1993 equation (Figure 48) is used to calculate ESALs in Utah. The growth factor (G) is calculated using the equation in Figure 54:

$$G = \frac{(1+r)^n - 1}{r}$$

where:

r = growth rate

n = design period (years)

Figure 54. Equation to Calculate the Growth Factor

UDOT's directional distribution factors are 0.5 for two-way and 1.0 for one-way traffic. UDOT recommended lane distribution factors are a function of the number of lanes and the Annual Average Daily Traffic (AADT) as shown in Table 36.

Table 36. UDOT Recommended Lane Distribution Factors[14]

Number of Lanes	Lane Distribution Factor
3 or Less	1
4 to 5	$2.275\,(AADT)^{-0.1054}$
6 or More	$2.484\,(AADT)^{-0.1312}$

The truck factors for UDOT are determined based on the functional classification of the roadway as shown in Table 37. UDOT default truck factors for rural and urban highways are shown in Table 38 and Table 39 respectively. An ESAL growth rate on the TF is applied as illustrated in Table 40.

Table 37. UDOT Functional Classification Code[14]

Code	Rural Functional Class	Code	Urban Functional Class
01	Interstate System	**11**	Interstate System
02	Other Principal Arterials	**12**	Other Freeways and Expressways
06	Minor Arterial Systems	**14**	Other Principal Arterials
07	Major Collector	**16**	Minor Arterial Systems
08	Minor Collector	**17**	Collector System
09	Local System	**19**	Local System

Table 38. Truck Factors for the Rural Functional Classes[14]

Axle Class	Functional Class					
	01	02	06	07	08	09
1-2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
3	0.03	0.03	0.03	0.03	0.03	0.03
4	0.88	0.88	0.88	0.88	0.88	0.88
5-7	0.4718	0.1996	0.2896	0.2798	0.2798	0.2798
8-10	2.8744	1.7796	1.641	1.0079	1.0079	1.0079
11-13	3.6942	1.3596	1.7199	0.6400	0.6400	0.6400

Table 39. Truck Factors for the Urban Functional Classes[14]

Axle Class	Functional Class					
	11	12	14	16	17	19
1-2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
3	0.03	0.03	0.03	0.03	0.03	0.03
4	0.88	0.88	0.88	0.88	0.88	0.88
5-7	0.3529	0.3529	0.1912	0.3529	0.3529	0.1912
8-10	1.6884	1.6884	1.8133	2.6028	2.6028	2.6028
11-13	2.5203	2.5203	1.9288	3.3584	3.3584	3.3584

Table 40. ESAL Growth Rate[14]

FHWA Truck Class	Truck Factor Adjustment
1 to 4	TF
5 to 7	TF+0.1
8 to 13	TF+0.3

California Department of Transportation ESAL Calculation

Caltrans also follows the AASHTO 1993 equation for ESAL calculation. The main difference is in the TF used by Caltrans. The default truck factors used by Caltrans for ESAL calculation are shown in Table 41.

Table 41. Caltrans Default Truck Factors[38]

Vehicle Type (By Axle Classification)	TF
2-Axle Trucks or Buses	0.189
3-Axle Trucks or Buses	0.504
4-Axle Trucks	0.805
5 or More-Axle Trucks	1.888

Comparison of ITD and Other States ESAL Calculation Methods

Classified 2009 traffic count data from 3 different WIM sites in Idaho was used to calculate ESAL using ITD's, UDOT's, WSDOT's, Caltrans', and AASHTO 1993 methods. Passenger cars were not taken into account when ESALs were calculated because their contribution to ESALs is insignificant.

Table 42 provides the location, and highway functional classification for each WIM site.

Table 42. WIM Locations

WIM Number	Functional Class	Route	Mile Post	Closest City
134	Principal Arterial - Other (Rural)	US-30	425.785	Georgetown
137	Principal Arterial - Other (Rural)	US-95	37.075	Homedale
192	Principal Arterial - Other (Rural)	US-93	16.724	Rogerson

The ADTT per truck class for the 3 investigated WIM sites is given in Table 43. This data shows that FHWA Truck Class 9 is the predominant truck class at each of these WIM sites.

Table 43. Average Daily Truck Traffic by Truck Class

FHWA Truck Class	ADTT for WIM Site No.		
	134	**137**	**192**
4	19	22	19
5	184	35	27
6	16	47	11
7	3	1	3
8	48	28	39
9	526	214	410
10	29	36	20
11	2	3	2
12	2	1	2
13	34	26	9

ESALs were calculated based on this data using the prescribed methods assuming a design period of 20 years and with no growth in traffic. When ITD's method was used for ESAL calculation, the design period was assumed to start at 2010 and end at 2030. This is important to note as ITD truck factors increase from year to year i.e., if the design period is from 2015 to 2035, ITD's method will result in different ESALs. The directional distribution factor was assumed as 0.5, while the lane distribution factor was taken as 1.0. Comparison between ESALs calculated using ITD, WSDOT, UDOT, Caltrans, and AASHTO 1993 using data corresponding to the prescribed WIM sites are shown in Figure 55 through Figure 57.

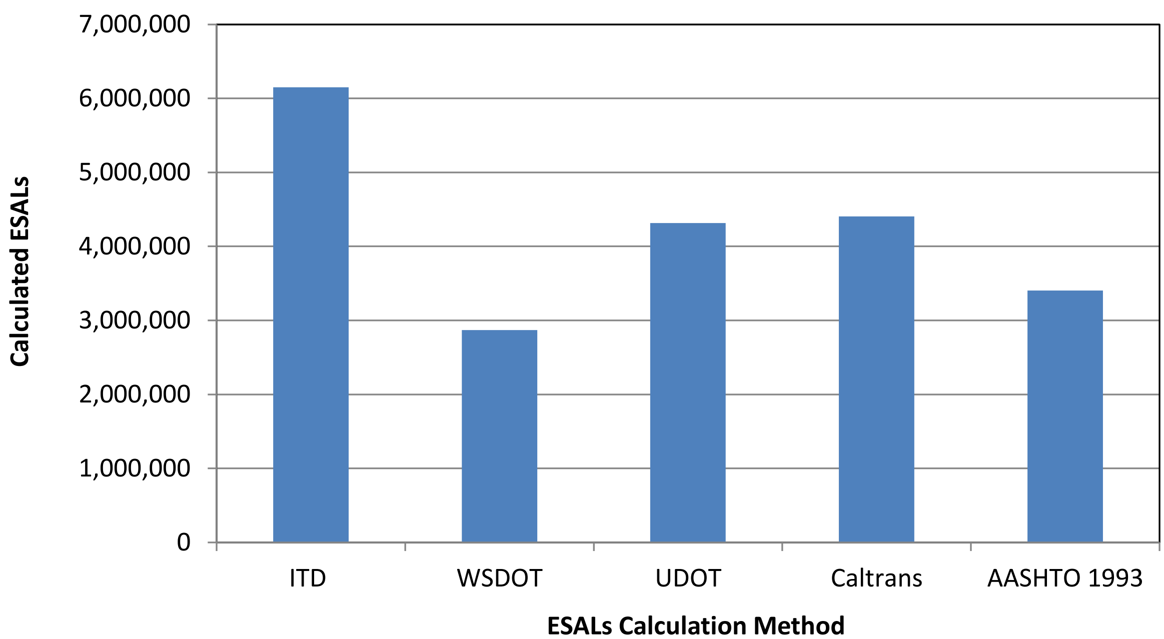

Figure 55. Comparison of ITD, WSDOT, UDOT, Caltrans, and AASHTO 1993 Calculated ESALs for Traffic Data Corresponding to WIM Site 134

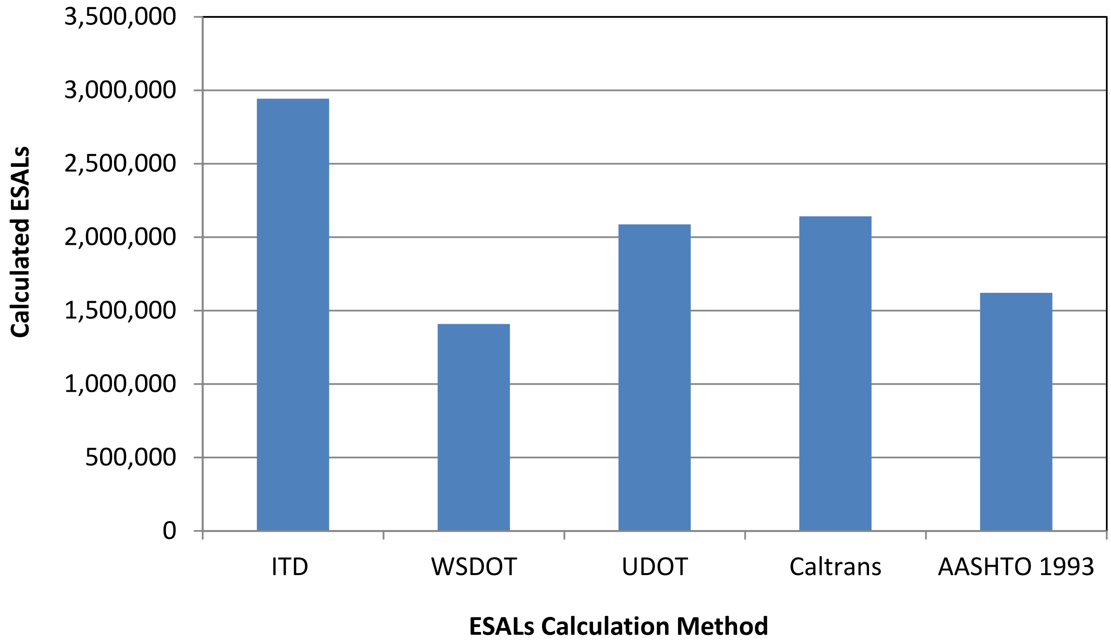

Figure 56. Comparison of ITD, WSDOT, UDOT, Caltrans, and AASHTO 1993 Calculated ESALs for Traffic Data Corresponding to WIM Site 137

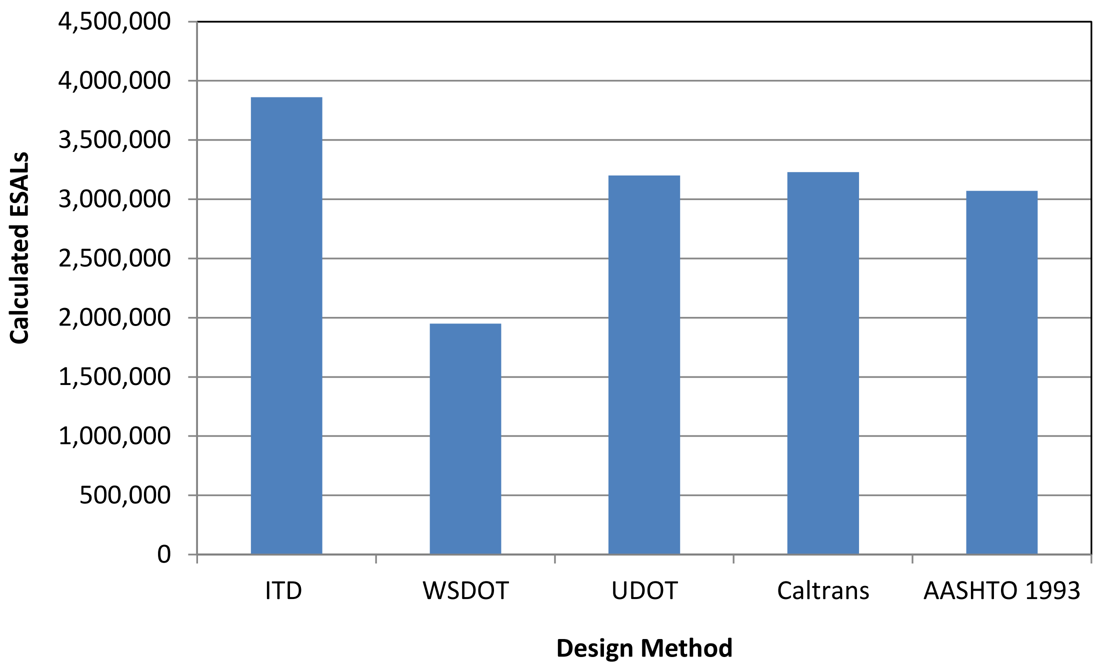

Figure 57. Comparison of ITD, WSDOT, UDOT, Caltrans, and AASHTO 1993 Calculated ESALs for Traffic Data Corresponding to WIM Site 192

These figures clearly show that ITD's truck factors are more conservative when compared to WSDOT's, UDOT's, Caltrans', and AASHTO 1993. These figures suggest that current ITD's TF and truck classification system needs to be revised.

Development of Truck Factors from MEPDG Axle Load Spectra

Axle Load Spectra (ALS) present the percentage of the total axle applications within each load interval for each axle type (single, tandem, tridem, and quad) and vehicle class (FHWA vehicle class 4 to 13).(4) ALS can only be determined from WIM data. As a part of the ITD's RP 193 Research Project, a total of 14 WIM site data located on different highways in Idaho were analyzed in order to develop ALS for MEPDG.(8) These WIM Sites are shown in Table 44.

Table 44. WIM Sites Used for the Development of ITD Axle Load Spectra

WIM Site Number	Functional Classification	Route	Mile Post	Closest City
79	Principal Arterial-Interstate (Rural)	I-15	27.7	Downey
93	Principal Arterial-Interstate (Rural)	I-86	25.05	Massacre Rocks
96	Principal Arterial-Other (Rural)	US-20	319.2	Rigby
117	Principal Arterial-Interstate (Rural)	I-84	231.7	Cottrell
129	Principal Arterial-Other (Rural)	US-93	59.8	Jerome
134	Principal Arterial-Other (Rural)	US-30	425.785	Georgetown
137	Principal Arterial-Other (Rural)	US-95	37.075	Homedale
138	Principal Arterial-Other (Rural)	US-95	22.72	Marsing
148	Principal Arterial-Other (Rural)	US-95	363.89	Potlatch
155	Minor Arterial (Rural)	US-30	229.62	Hansen
156	Minor Arterial (Rural)	SH-33	21.94	Howe
169	Principal Arterial-Other (Rural)	US-95	56.002	Parma
185	Principal Arterial-Other (Rural)	US-12	163.01	Powell
192	Principal Arterial-Other (Rural)	US-93	16.724	Rogerson

In order to develop site-specific ALS for each WIM site, all truck weight record files for all 12 months of 2009 were uploaded and ran with the TrafLoad software. The software, which was developed as a part of the NCHRP 1-39 project to process traffic data from WIM data into MEPDG, outputs the load spectrum for each axle type and vehicle class per month of the analysis year.(39)

Site specific and statewide ALS values were developed for Idaho based on the analysis of the traffic data from the 14 WIM sites. In addition, these 14 WIM sites were further classified, based on similarity of load groups, into 3 different truck weight road groups (TWRGs). The TWRGs representing Idaho traffic loading characteristics are as follows:

- Primarily loaded- in which there is bimodal distribution of the axle weights with a large percentage of the trucks are heavily loaded.
- Moderately Loaded-in which there is a bimodal distribution of the axle weights with almost similar percentages of the heavy and light axle weights.
- Lightly loaded-in which there is a bimodal distribution of the axle weights with a large percentages of the trucks are empty or partially loaded.

Table 45 illustrates the WIM sites belonging to each of the developed TWRG.

Table 45. WIM Sites Associated with Idaho Truck Weight Road Groups

Idaho Truck Weight Road Groups (TWRGs)	WIM Site
Primarily Loaded	79, 117, 134, 148, 155
Moderately Loaded	93, 137, 138, 156, 169, 185
Lightly Loaded	96, 129, 192

The set of equations in Figure 49 along with the average number of axles per truck type and axle group developed from the ITD's WIM data shown in Table 46 were used to develop the TF for each of the 3 WIM data. They were also used to compute the TF for the statewide and primary, moderately, and lightly loaded TWRGs. A structure number (SN = 5) and terminal serviceability index (p_t = 2.5) were used in the development of the TF. It should be noted that the AASHTO 1993 equation for ESAL calculation did not include a code for Quad axles. Because this study has quad axles, a code of four was assumed for this axle type. This assumption may not significantly affect the final results because there were very few quad axles in comparison to the other axle types.

Table 46. Average Number of Axles per Truck and Axle Group Based on ITD's WIM Data

Truck Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
4	1.59	0.34	0.00	0.00
5	2.00	0.00	0.00	0.00
6	1.00	1.00	0.00	0.00
7	1.00	0.22	0.83	0.10
8	2.52	0.60	0.00	0.00
9	1.25	1.87	0.00	0.00
10	1.03	0.85	0.95	0.26
11	4.21	0.29	0.01	0.00
12	3.24	1.16	0.07	0.01
13	3.32	1.79	0.14	0.02

The developed TF for the 3 ITD TWRGs and the developed statewide TF are shown in Table 47. Figure 59 to Figure 61 compares the ESAL calculations based on the current ITD factors, TF from actual WIM site ALS, statewide ALS and the TWRG ALS for WIM sites 134, 137, and 192, respectively. The error in the ESAL calculation normalized based on the ESALs calculated from truck factors corresponding to the actual load spectra of the site was calculated for the current and the statewide and TWRG truck factors. This is shown in Figure 58.

$$NE = \frac{|x_a - x_s|}{x_a}.100$$

where:

NE = absolute value of the normalized error, percent

X_a = ESALs calculated based on the TF from actual ALS of the site

X_s = ESALs calculated based on the TF from statewide or TWRG ALS

Figure 58. Equation to Calculate the Normalized Error

The normalized error plots for the 3 WIM sites are shown in Figure 62 to Figure 64. These figures show that the current ITD's ESAL calculation method produces high errors (66 percent to 74 percent) on the conservative side for the moderately and lightly loaded sites. The minimum normalized error for the primarily loaded site was 9.7 percent. The statewide developed ALS produced normalized error in the range of 20 percent to 39 percent compared to the ESALs calculated based on the actual ALS for a specific site. The minimum normalized error found when using the TF corresponding to the specific TWRG was 1 percent to less than 9 percent.

Table 47. Developed Truck Factors Based on WIM Data Analysis for ITD

FHWA Vehicle Class	Truck Weight Road Group (TWRG)			Statewide
	Lightly Loaded	Moderately Loaded	Primarily Loaded	
4	0.644	0.677	1.611	1.016
5	0.182	0.491	0.995	0.620
6	0.436	0.723	1.360	0.903
7	0.654	0.890	1.735	1.052
8	0.404	0.532	1.415	0.852
9	1.209	1.454	2.217	1.704
10	0.809	1.076	2.962	1.726
11	0.615	1.389	3.678	2.043
12	1.148	0.818	2.523	1.508
13	2.150	1.520	2.901	2.119

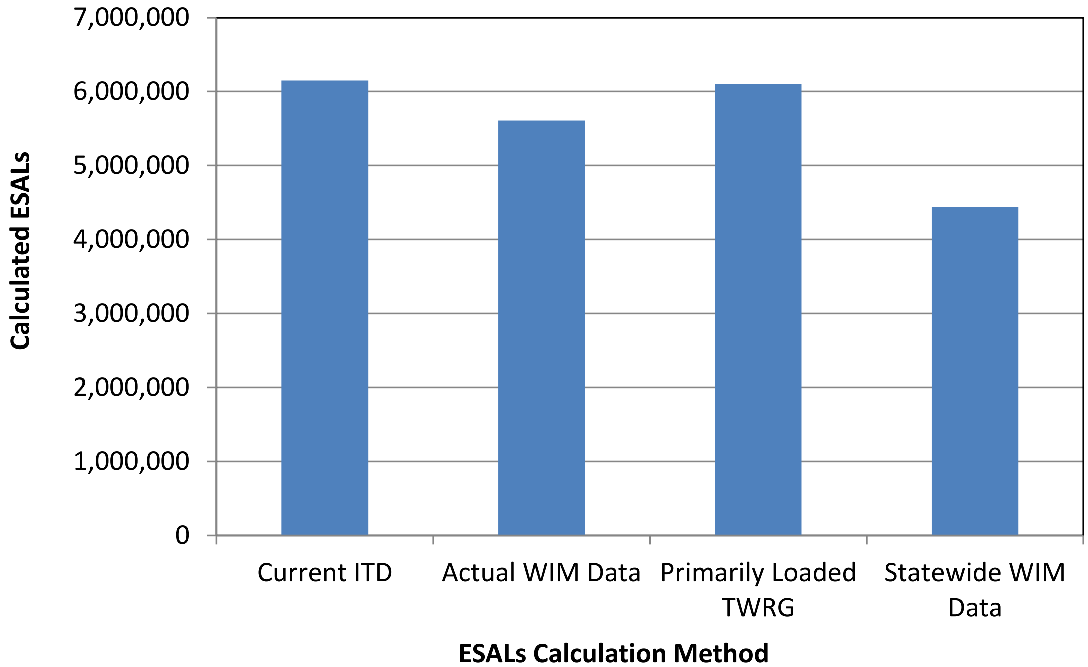

Figure 59. Comparison of ESALs Based on Current ITD, Actual, Statewide, and Primarily Loaded TWRG Truck Factors from WIM Site 134

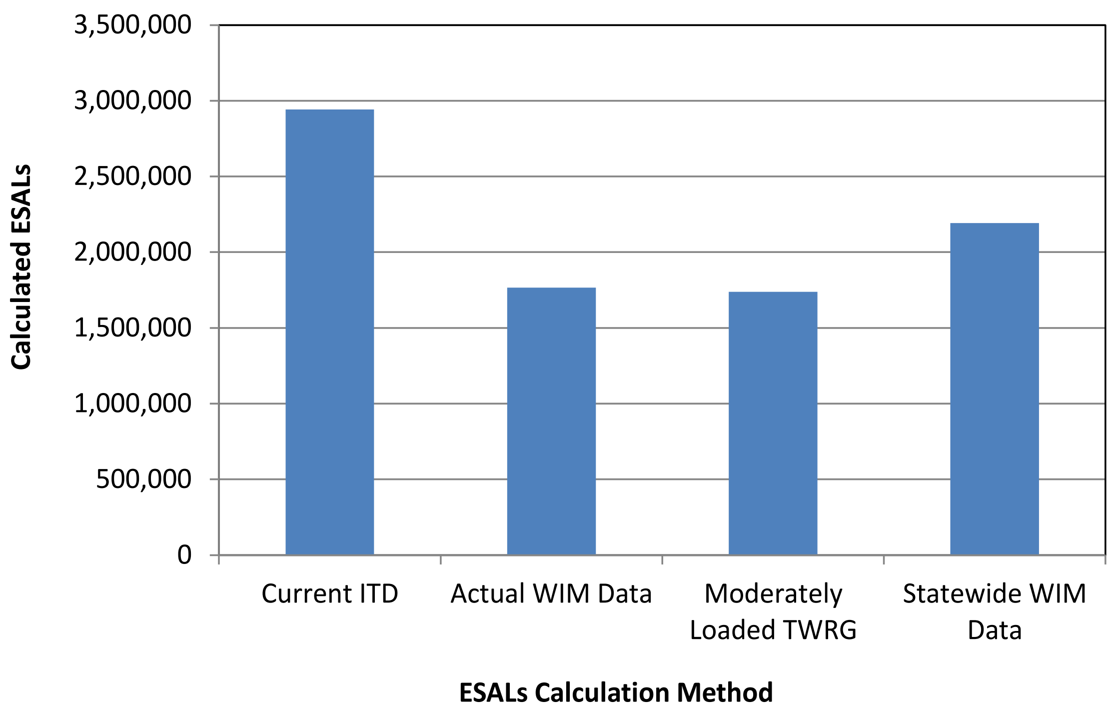

Figure 60. Comparison of ESALs Based on Current ITD, Actual, Statewide, and Moderately Loaded TWRG Truck Factors from WIM Site 137

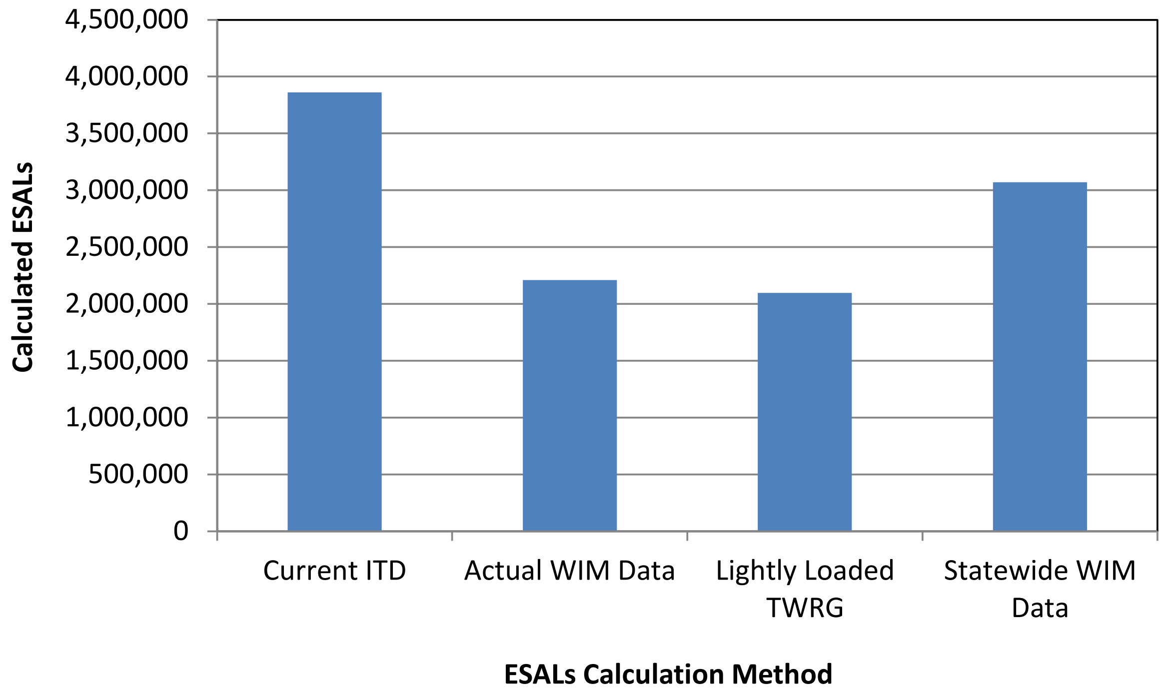

Figure 61. Comparison of ESALs Based on Current ITD, Actual, Statewide, and Lightly Loaded TWRG Truck Factors from WIM Site 192

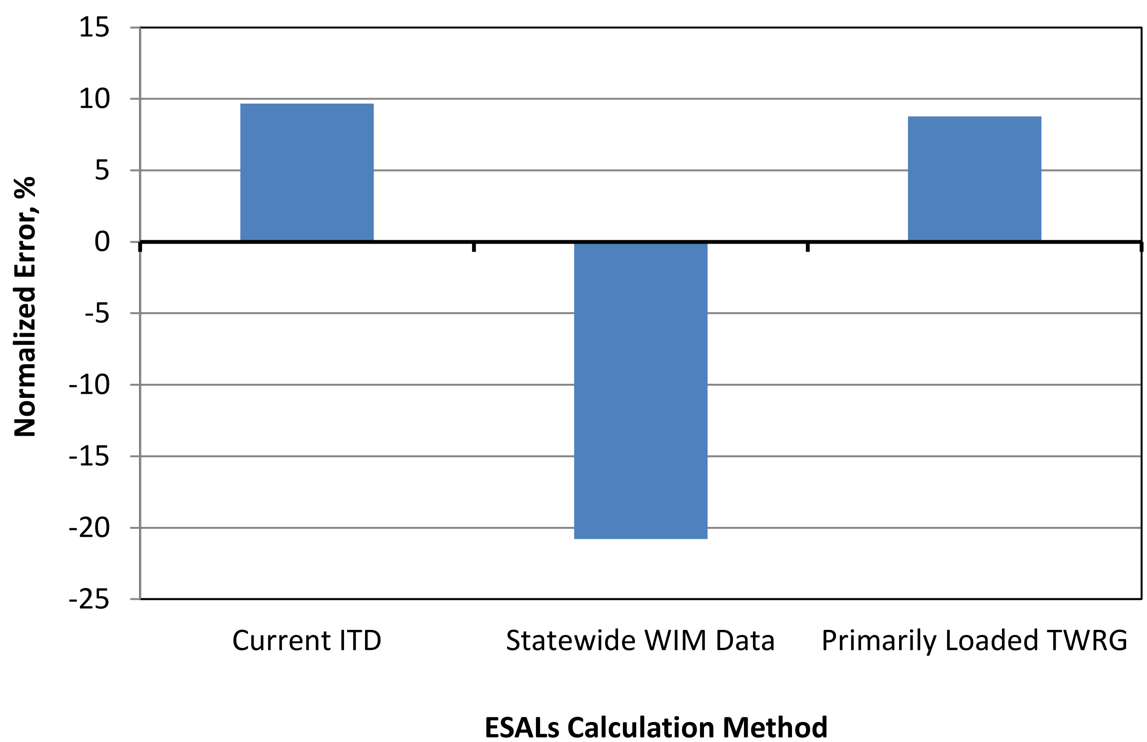

Figure 62. ESALs Normalized Error for WIM Site 134

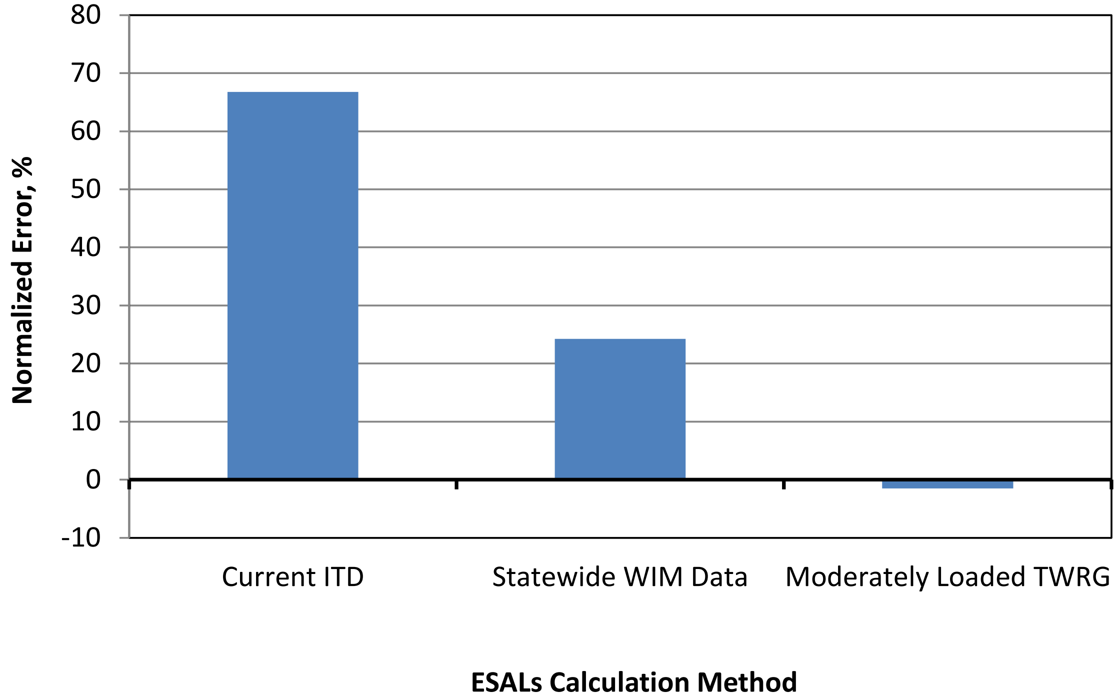

Figure 63. ESALs Normalized Error for WIM Site 137

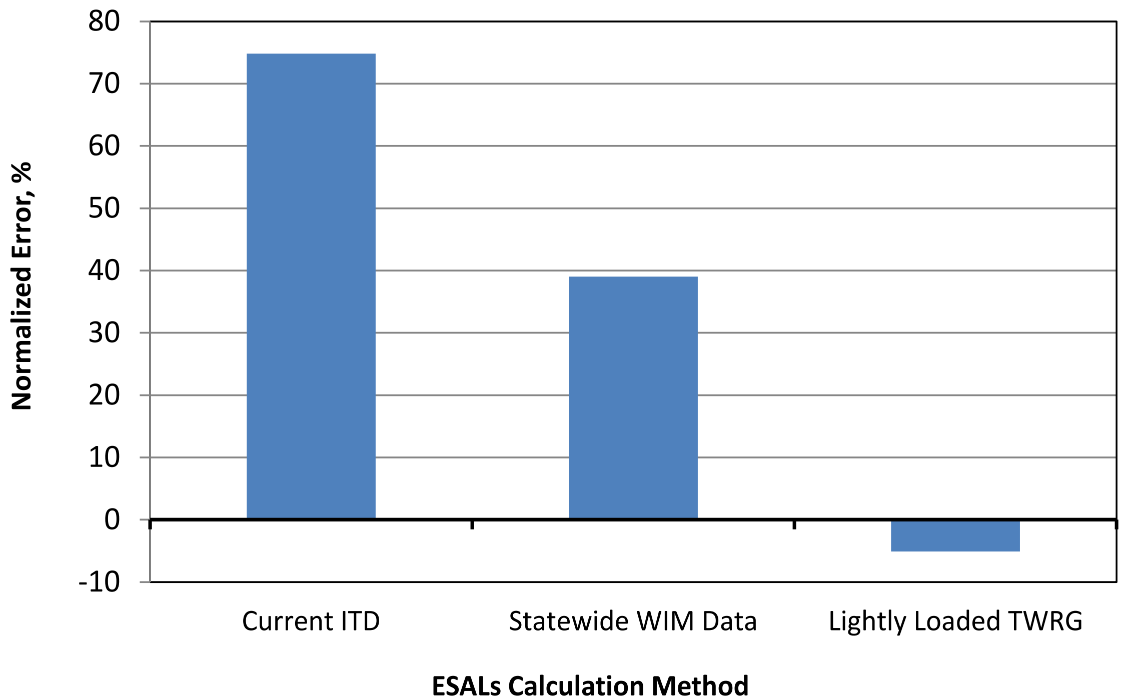

Figure 64. ESALs Normalized Error for WIM Site 192

Development of Truck Factors for a Simplified Truck Classification

In order to facilitate the implementation of the developed TF for ITD, a simplified truck traffic classification similar to the one used by WSDOT is suggested. This simplified classification divides trucks into three classes; single units, double units, and trains. The FHWA truck classes that fall into the 3 simple classes are shown in Table 48. Based on this simplified truck traffic classification, TF were established using the weighted average of the truck factor for each FHWA class. These developed TF for the simplified classification are also summarized in Table 48.

Table 48. Developed ITD Truck Factors Based on WIM Data Analysis for Simplified Truck Classification

FHWA Vehicle Class	Simplified Truck Class	Light	Medium	Heavy	Statewide
4	Single Units	0.269	0.559	1.230	0.743
5					
6					
7					
8	Double Units	1.041	1.302	2.270	1.623
9					
10					
11	Trains	1.991	1.494	2.948	2.063
12					
13					

Current ITD Traffic Projection Method

In pavement design, forecasted values of AADT directly affect the estimation of design ESALs. A direct result of this is either under or over design of pavement. An under-designed project may lead to a premature failure of the pavement. An over-designed project may lead to a waste in funds that could be used on different projects.

Flexible pavement design life in Idaho is always 20 years, and traffic volume is projected 20 years in the future. Forecast traffic 20 years in the future, ITD assumes that past trends will continue in the future as a percent increase. A graphic explanation of ITD's current traffic projection method is shown in Figure 65.

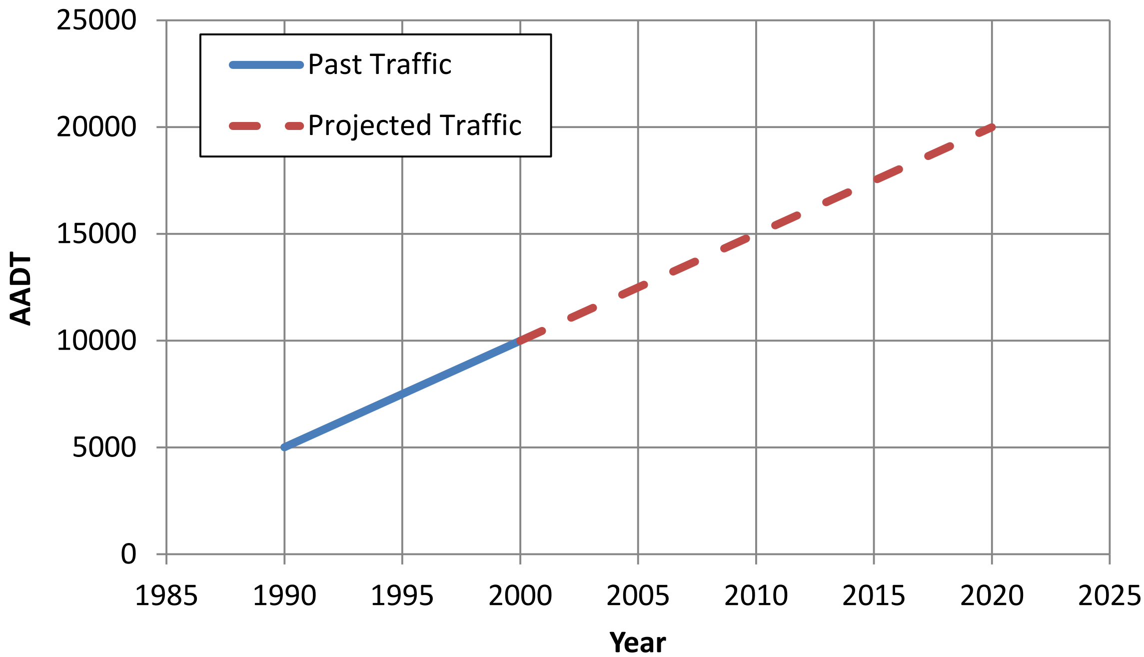

Figure 65. Example Showing Current ITD Traffic Projection Method

Idaho Traffic Volume Data

Traffic volume data is one of the primary components of the traffic data. ITD collects traffic volume data in terms of AADT and CAADT. Traffic counts are performed in Idaho using different methods such as Automatic Traffic Recorders (ATR) and WIM. The data used to investigate the accuracy of the current ITD traffic projection method were from ATRs and WIM stations. In Idaho, there are approximately 173 ATR sites and 25 WIM sites.

Accuracy of the Current ITD Traffic Projection Method

In order to investigate the accuracy of the current ITD traffic volume projection method, traffic volume data expressed in AADT from about 92 different ATRs and WIM stations located statewide were gathered. The source of the data is ITD's Planning Website.[40] The data covers from 1990 through 2009. Only 92 ATR and WIM sites were used in this analysis. Many of the ATR/WIM sites had data missing and could not be used for analysis. The ATR site AADT database used in this analysis is compiled in Appendix E.

The AADT used in this analysis contained data for the years between 1990 and 2009. The trends in the traffic between the years 1990 and 1995 were used to forecast the AADT for the year 2009. In general, 5 years of known traffic volume were used to project the traffic 14 years in the future. Some ATR sites did not have the traffic volume recorded in 2009; in such a case, traffic volume from 2008 was used in the analysis.

A comparison was made between the projected traffic and the actual recorded (observed) traffic from 2009. This comparison is depicted in Figure 66. This figure shows that current ITD traffic projection method significantly overestimates future traffic. The average absolute error between actual and projected traffic was found to be 42.11 percent. If traffic projections are for 20 years in the future, this error is expected to be higher. This analysis shows that the current ITD projection method produces highly significant errors in the projected traffic volume. There are several traffic forecasting methods that ITD may investigate. These methods include: time series forecasting, regression, clustering, and neural networks. More details regarding these methods and other models developed for Idaho can be found in studies done by Dixon and Kyte.[42, 43]

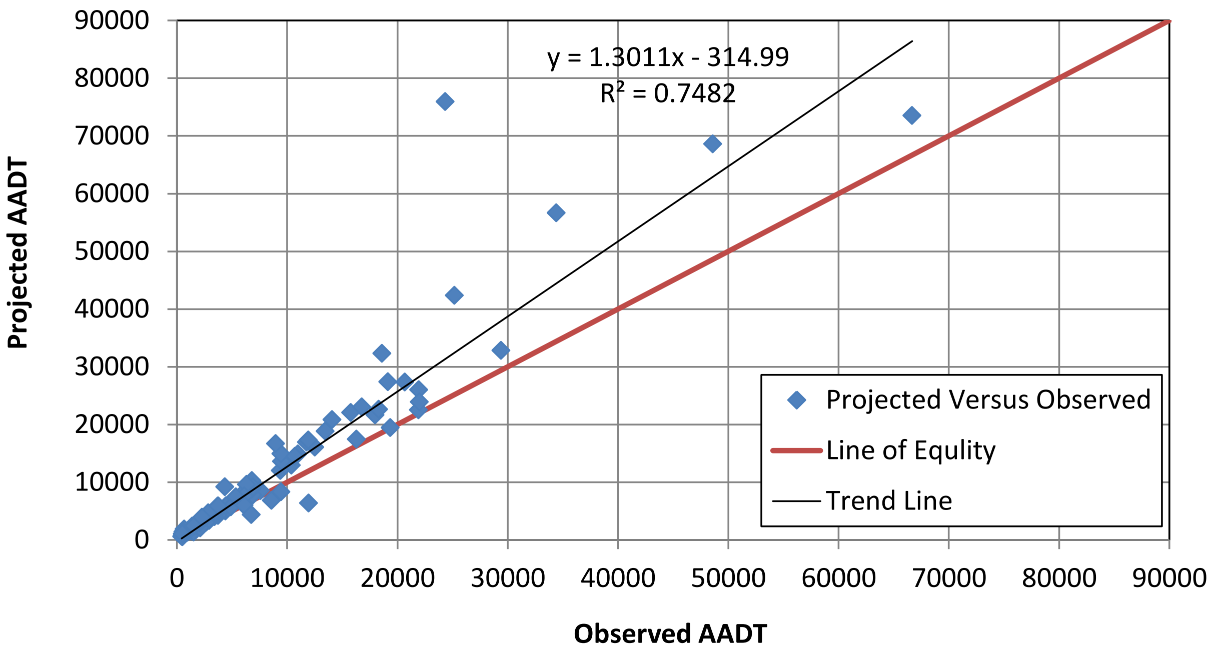

Figure 66. Observed Versus Projected AADT Based on Current ITD Traffic Projection Method

Chapter 6
Analysis of ITD Climatic Factors

Pavement performance is significantly affected by environmental conditions. The two environmental characteristics that greatly affect the pavement performance are temperature and moisture. Both AASHO and the Western Association of State Highway Officials (WASHO) road tests showed great seasonal variations in the deflection measurements under wheel loads. During hot summer months, asphalt layer(s) modulus can reach 100,000 psi, while during cold weather it can reach up to 2 to 3 million psi.(4) This has a direct effect on the state of stress within the asphalt layer(s) and all layers beneath them. Unbound base/subbase layers and subgrade soil resilient modulus values are affected by the moisture variation. In freezing temperatures, water in the soil freezes and its resilient modulus could reach values 20 to 120 times higher than the value of the modulus before freezing.(4) In addition, during the thawing process, a huge reduction in the pavement strength occurs.

This chapter presents how environmental conditions are addressed in ITD's design method and compares it with AASHTO 1993 and MEPDG methods.

ITD Climatic and Environmental Factors

Thirty years of Idaho's monthly data concerning temperature, precipitation, and freeze periods were used to create different climactic regions (zones) in Idaho.(41) An evaluation of the spring breakup periods by Idaho District Maintenance Engineers was also used in creating these climatic zones. Each of these zones was assigned a regional (climatic) factor. These climatic factors represent the percentage increase (from 0 to 15 percent) in the total required pavement structure thickness. The factors were selected based on the severity of the climatic or environmental conditions in each zone to provide additional protection during winter and spring conditions. These climatic factors were developed using the recommendations from AASHO Committee on Design.(41) The Idaho climatic zones map is divided into 4 zones (1 to 4) with each zone assigned a regional factor (1 to 1.15). The map is shown in Figure 67.

AASHTO 1993 Climatic and Environmental Factors

AASHTO's 1993 guide handles the environmental effects through the influence of the seasonal temperature and moisture variation on material properties.(3) It incorporates the effective roadbed soil resilient modulus to account for the influence of seasonal variation in moisture on the modulus of the subgrade using relative damage approach. It also incorporates drainage coefficients for the unbound base and subbase granular layers to account for the subsurface water drainage.

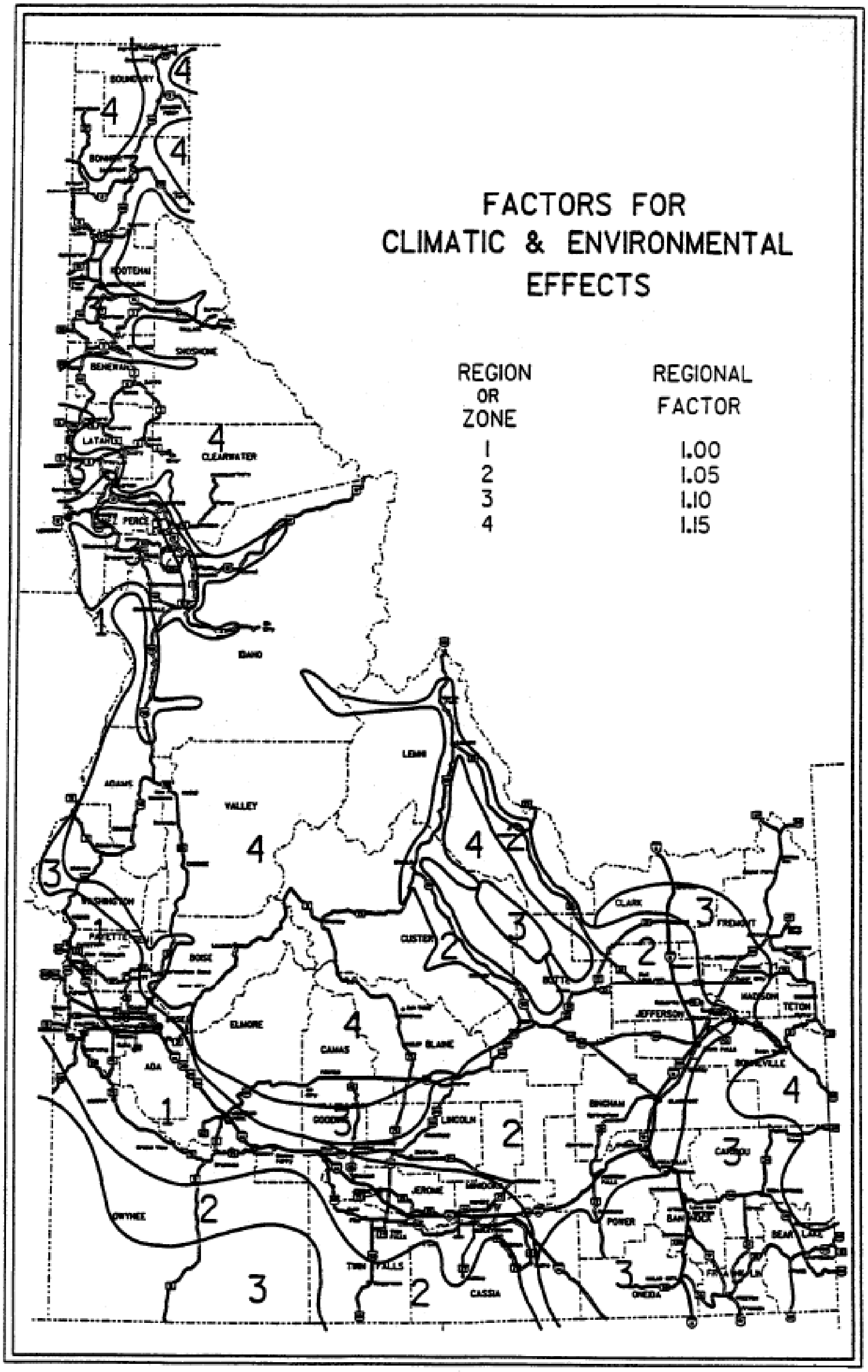

Figure 67. Current Idaho Climatic Zones Along with Climatic Factors[(2)]

MEPDG Climatic and Environmental Factors

MEPDG utilizes an advanced climatic modeling tool called the Enhanced Integrated Climatic Model (EICM). EICM is a one-dimensional coupled heat and moisture flow program.(4) It consists of 3 major components:

- Climatic-Materials-Structural Model (CMS Model).
- CRREL Frost Heave and Thaw Settlement Model (CRREL Model).
- Infiltration and Drainage Model (ID Model).

EICM predicts the hourly temperature and moisture variations within each pavement layer and foundation over the entire design life of the pavement. It also estimates resilient modulus adjustment factors, pore water pressure, water content, frost and thaw depth, frost heave, and drainage performance. Hourly weather-related data required by EICM to perform analysis are: air temperature, precipitation, wind speed, percentage sunshine, and relative humidity. This data is available from 851 weather stations across the U.S. for locations where weather station is not available, a virtual weather station is created to provide the climatic data needed by MEPDG.

The water table depth is another important climatic input required by MEPDG. This input is important for the overall accuracy of the foundation/pavement moisture contents.

MEPDG Climatic and Environmental Factors for Idaho

In Idaho, a total of 12 weather station sites are included in the MEPDG national database. Weather stations in bordering states may also be used. The location information and the number of months of available data and missing data for the stations in Idaho are summarized in Table 49.

A summary of the mean annual air temperature and rainfall, average annual number of freeze-thaw cycles, average wind speed, and percent average sunshine for MEPDG climatic locations in Idaho is shown in Table 50.

Table 49. Summary of Idaho Weather Stations Currently Available in MEPDG Software Version 1.1[4]

Weather Station	Station Location	Latitude (Degree.Minutes)	Longitude (Degree.Minutes)	Elevation (ft)	Months of Available Data
Boise	Boise Air Terminal/ Gowen Field Airport	43.34	-116.13	2861	116
Burley	Burley Municipal Airport	42.32	-113.46	4151	64
Challis	Challis Airport	41.31*	-114.13	5042	90
Idaho Falls	Idaho Falls Regional Airport	43.31	-112.04	4768	97
Jerome	Jerome County Airport	42.44	-114.28	4012	109
Lewiston	Lewiston-Nez Perce County Airport	46.22	-117.01	1447	116
McCall	McCall Municipal Airport	44.53	-116.06	5032	101
Mullan Pass VOR	Mullan Pass	47.28	-115.38	6074	116
Pocatello	Pocatello Regional Airport	42.55	-112.34	4454	116
Rexburg	Rexburg-Madison County Airport	43.5	-111.53	4875	97
Twin Falls	Joslin Field- Magic Valley Regional Airport	42.29	-114.29	4148	105
Pullman /Moscow	Pullman/Moscow Regional Airport	46.44	-117.07	2540	93

* The latitude for Challis Airport should be 44.3 according to Google Earth and www.airnav.com. This should be corrected in MEPDG

Table 50. Summary of the Climatic Data for the MEPDG Weather Stations Located in Idaho

Location	Mean Annual Air Temperature (ºF)	Mean Annual Rainfall (in.)	Air-Freezing Index (ºF-days)	Average Annual Number of Freeze/Thaw Cycles	Average Wind Speed (mph)	Average Sunshine (%)
Boise	53.26	11.20	229.86	75	6.6	72.27
Burley	48.09	9.38	592.93	98	7.3	71.72
Challis	44.08	6.70	1400.51	119	3.7	67.69
Idaho Falls	44.93	8.57	1132.89	109	7.6	62.35
Pullman/Moscow	48.01	12.40	272.8	75	6.7	60.47
Lewiston	53.46	13.97	121.38	47	4.8	62.61
McCall	39.68	24.64	1471.71	140	3.5	57.43
Mullan Pass	37.62	37.67	1419.06	59	5.3	45.04
Pocatello	47.74	10.89	730.58	108	8.3	64.99

Investigating the Accuracy of the ITD Climatic Zones and Factors

To investigate the accuracy of the current ITD climatic zones, six different MEPDG weather station data were selected and MEPDG was run for a typical pavement section. The US-93 section data presented in Chapter 4 was used for this analysis. All inputs were kept constant except the weather station. Depth to groundwater table was also held constant in all MEPDG simulation runs. It is important to note that, the 6 weather stations were selected such that 2 are located in each of 3 ITD climatic zones. This is shown in Table 51. The Logan-Cache airport weather station is located in Utah but is very close to the southern border of Idaho.

Table 51. Selected MEPDG Weather Stations and Corresponding ITD Climatic Zone

Weather Station	Station Location	ITD Climatic Zone
Pocatello	Pocatello Regional Airport	2
Idaho Falls	Idaho Falls Regional Airport	2
Logan, Utah	Logan-Cache Airport	3
Pullman/Moscow	Pullman/Moscow Regional Airport	3
McCall	McCall Municipal Airport	4
Mullan Pass	Mullan Pass	4

Figure 68 through Figure 74 present a comparison of MEPDG predicted distresses and smoothness for the 4 MEPDG computer simulations runs using the 2 different weather stations located in ITD Climatic Zone 3. Figure 75 through Figure 81 present a comparison of MEPDG predicted distresses and smoothness for the conducted MEPDG runs using the 4 different weather stations located in Climatic Zones 2 and 4. Figure 68 to Figure 70 show that although Logan and Pullman/Moscow weather stations are located within the same ITD Climatic Zone 3, there is a significant difference in the predicted longitudinal cracking and total rutting. Furthermore, Figure 75 and Figure 78 show that although McCall and Mullan Pass weather stations are located in ITD Climatic Zone 4, there is a significant difference in the predicted longitudinal cracking and AC rutting. Idaho Falls and Pocatello weather stations which are located in ITD Climatic Zone 2 yielded fairly similar distresses and smoothness. This analysis shows that the current climatic zones developed for Idaho are not consistent and need revisions. Redefining these climatic zones is important for accurate design. In addition, as previously explained, the current ITD climatic factors are empirical factors developed to provide additional protection during winter and spring conditions. MEPDG takes into account the influence of hourly change in moisture and temperature on the fundamental materials properties.

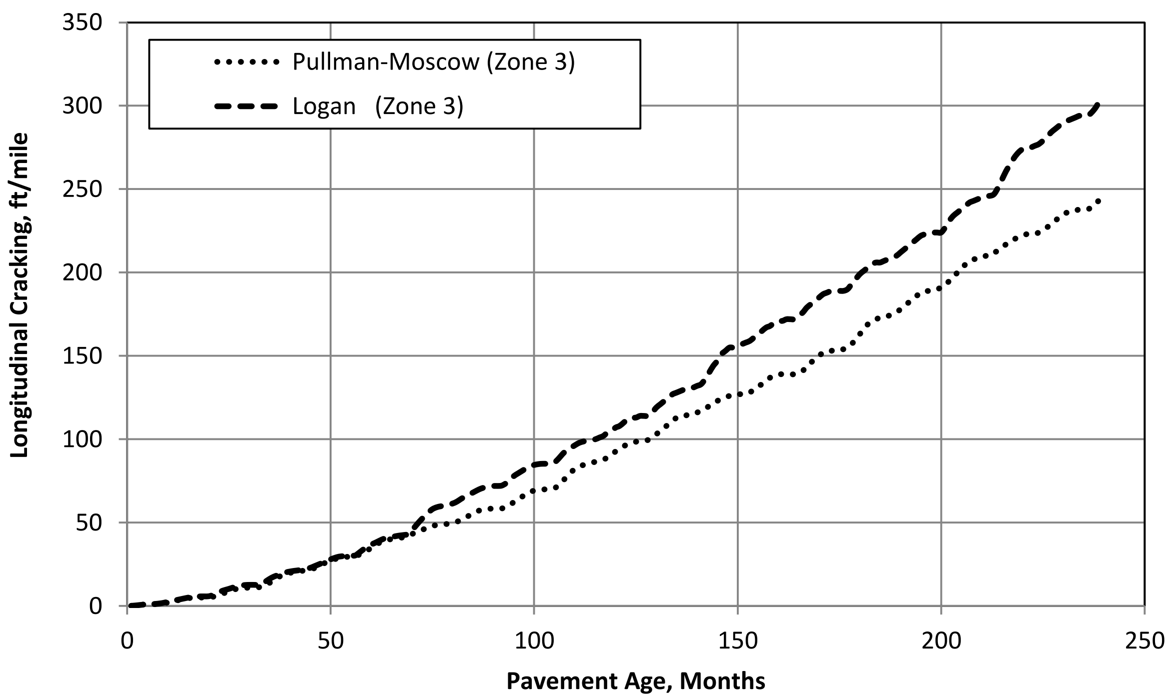

Figure 68. MEPDG Predicted Longitudinal Cracking for the Investigated Climatic Locations in ITD Zone 3

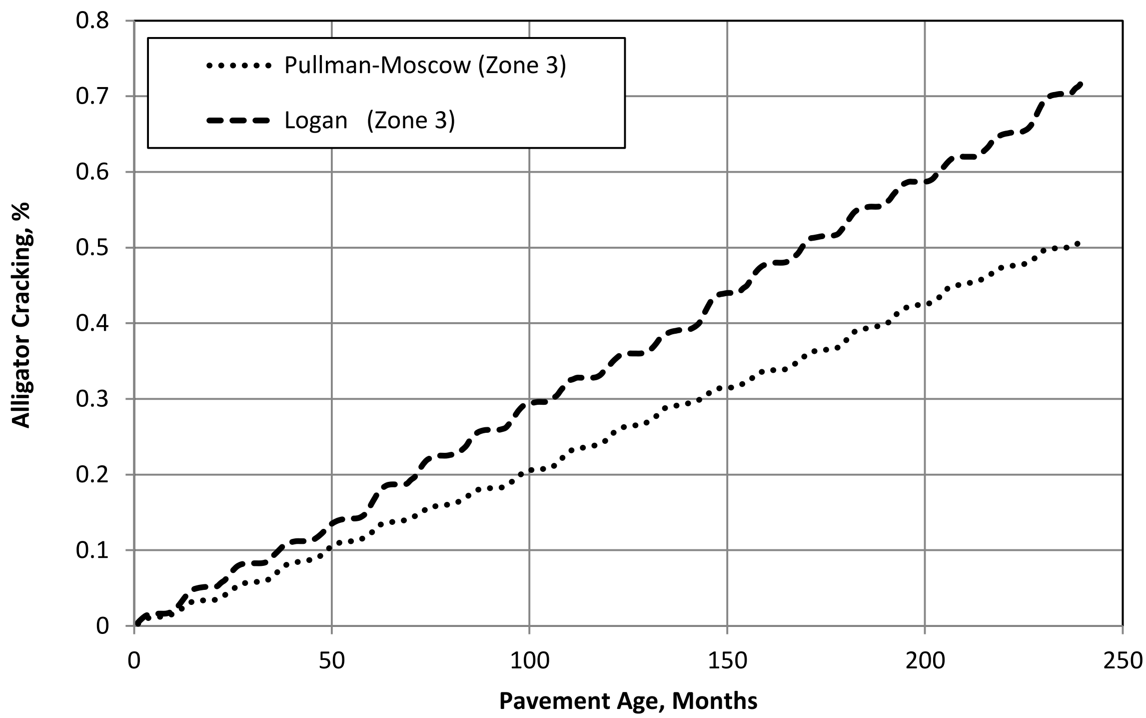

Figure 69. MEPDG Predicted Alligator Cracking for the Investigated Climatic Locations in ITD Zone 3

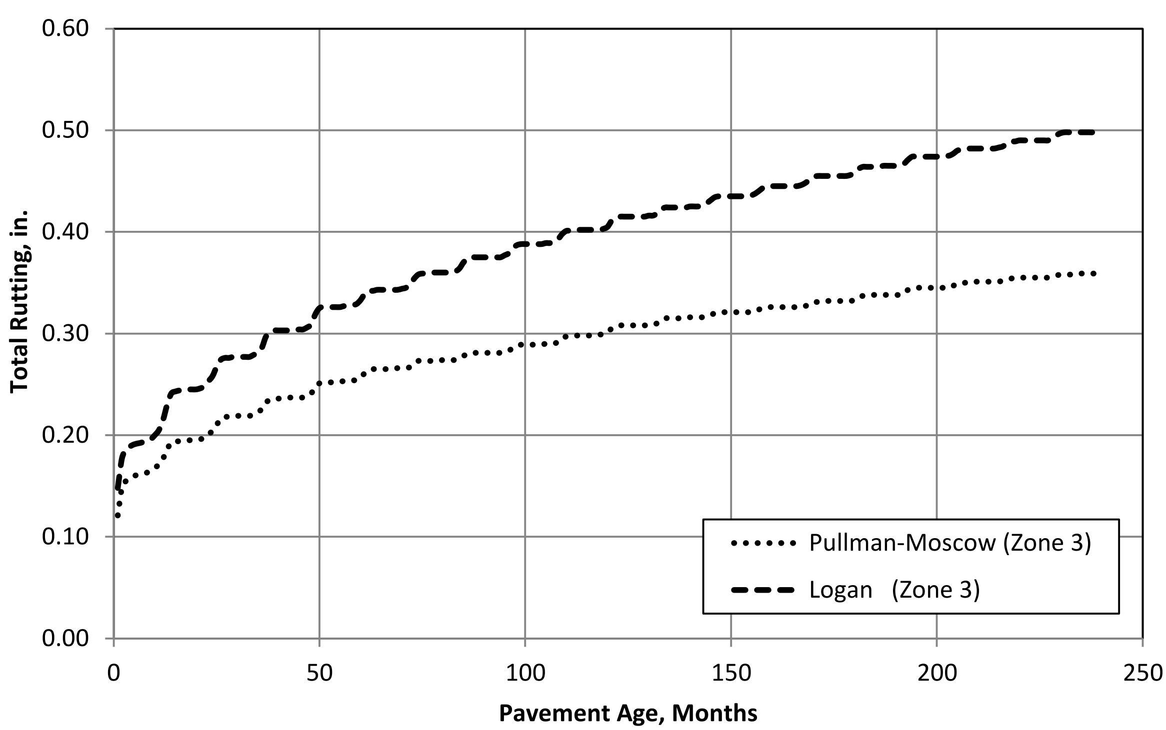

Figure 70. MEPDG Predicted Total Rutting for the Investigated Climatic Locations in ITD Zone 3

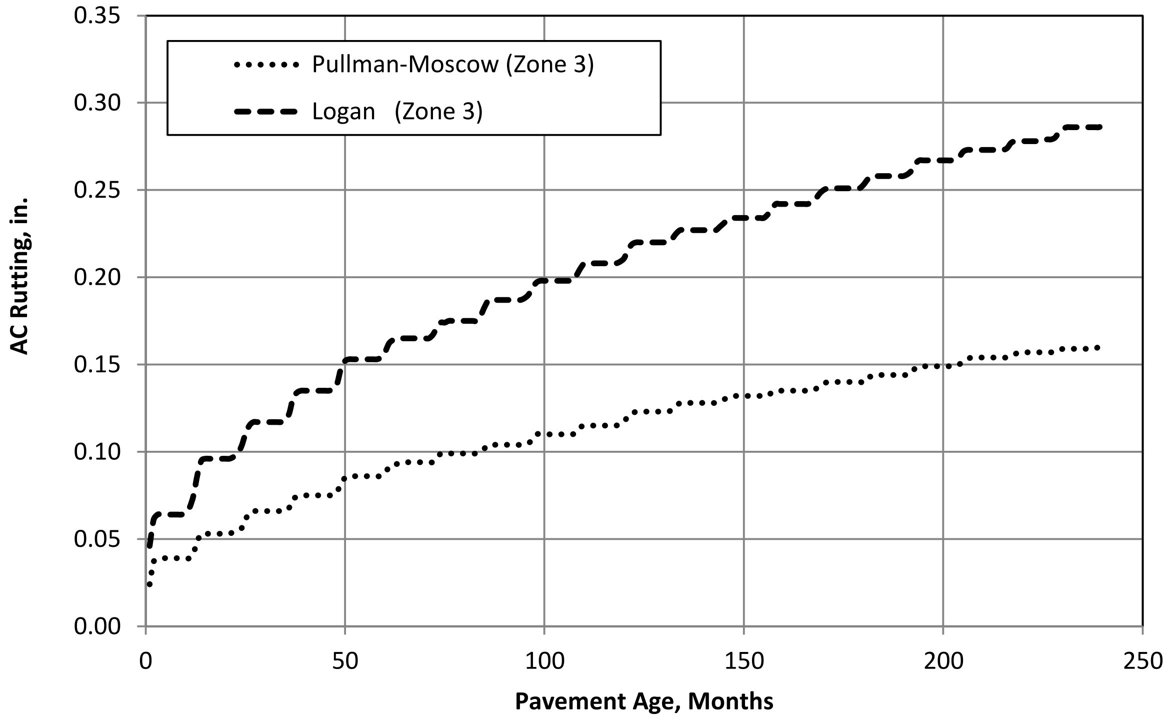

Figure 71. MEPDG Predicted AC Rutting for the Investigated Climatic Locations in ITD Zone 3

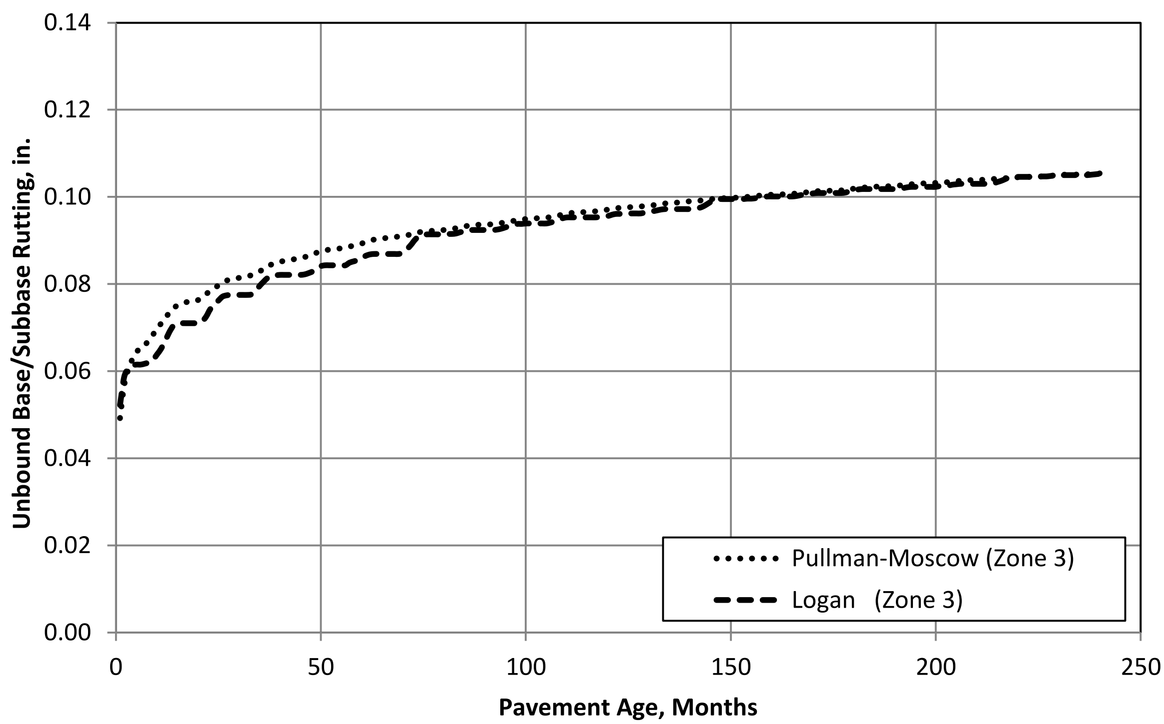

Figure 72. MEPDG Predicted Unbound Granular Layers Rutting For the Investigated Climatic Locations in ITD Zone 3

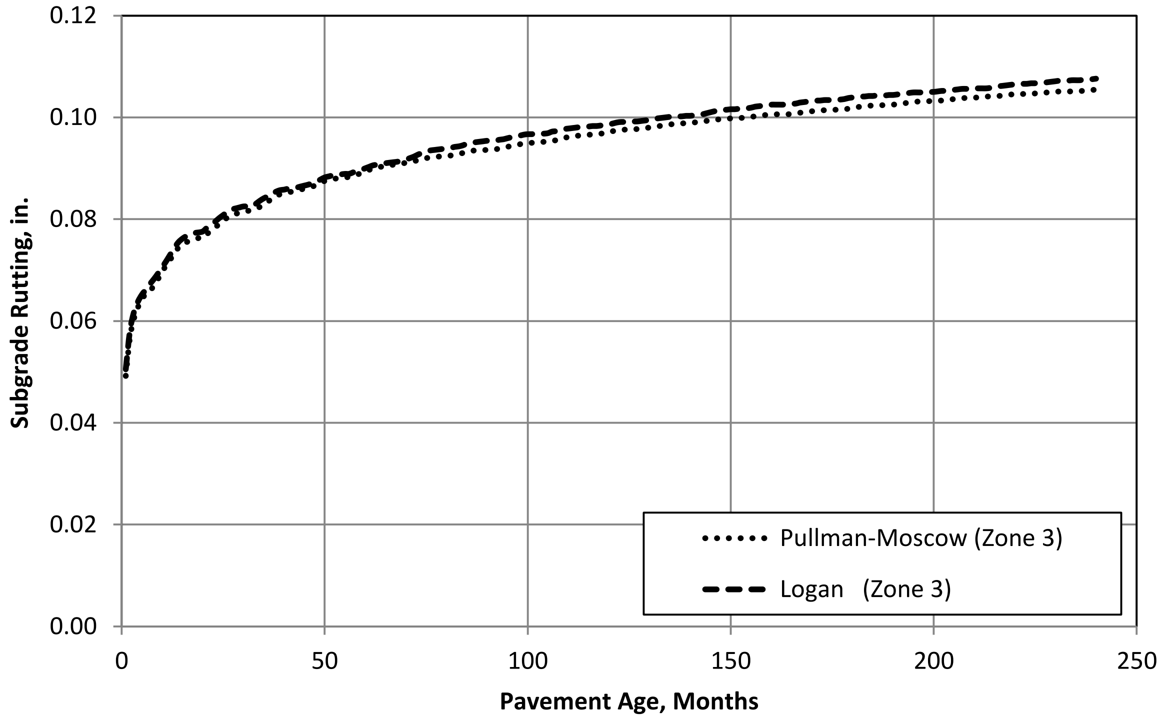

Figure 73. MEPDG Predicted Subgrade Rutting for the Investigated Climatic Locations in ITD Zone 3

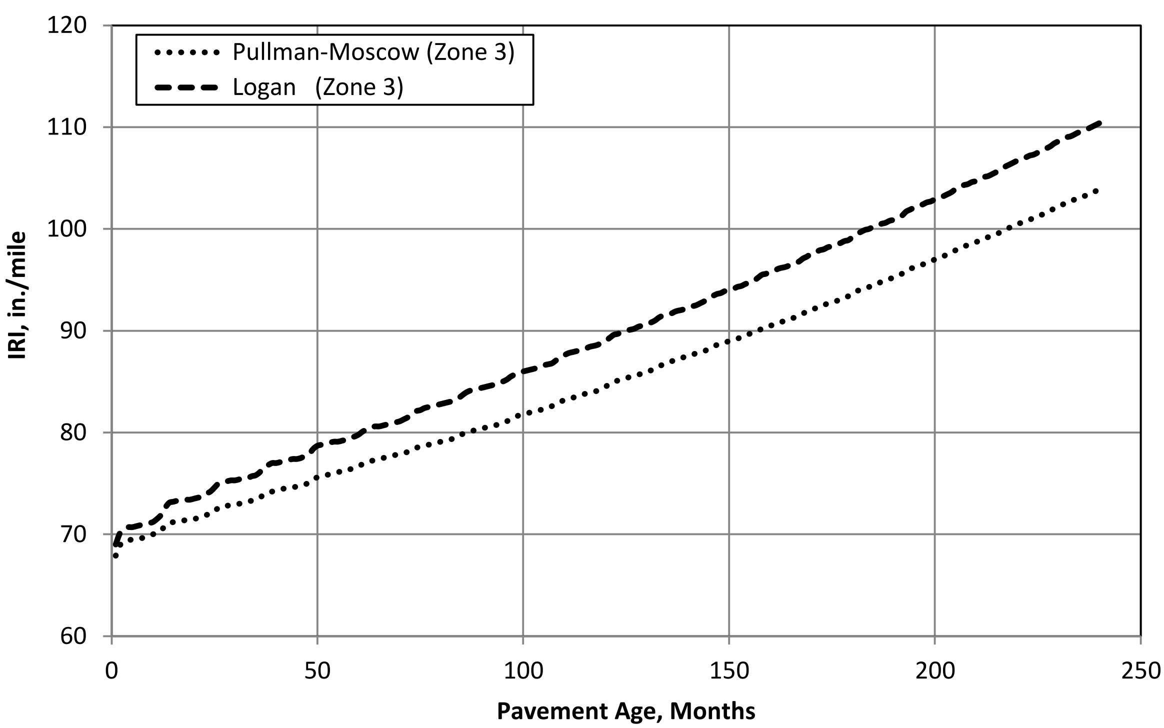

Figure 74. MEPDG Predicted IRI for the Investigated Climatic Locations in ITD Zone 3

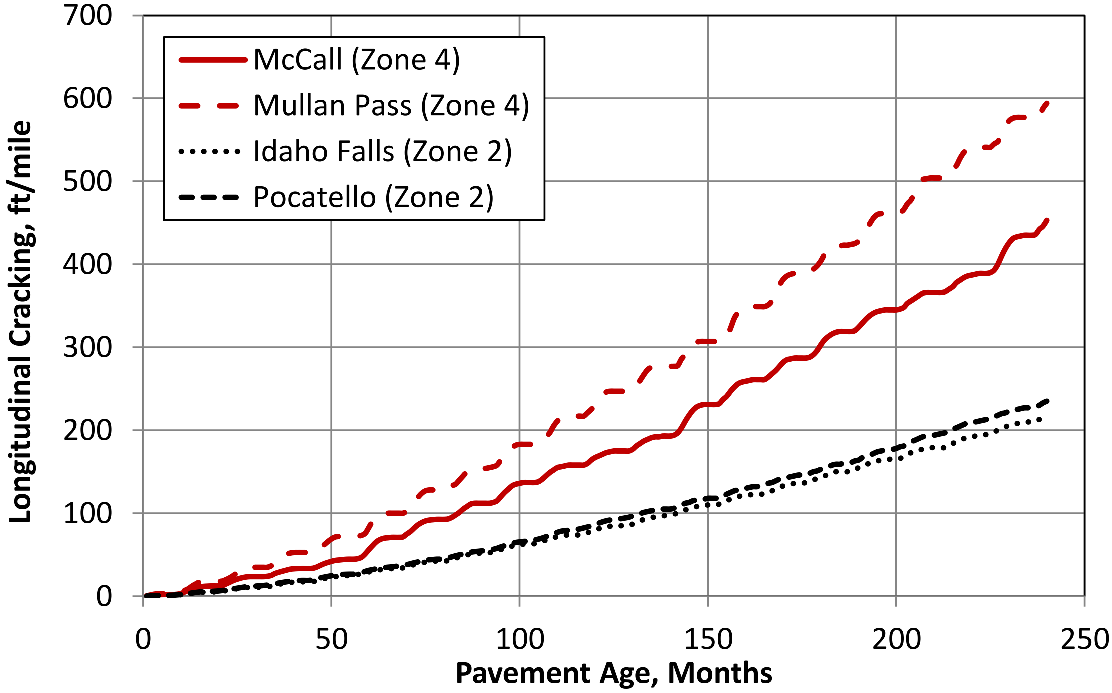

Figure 75. MEPDG Prediceted Longitudinal Cracking for the Investigated Climatic Locations in Zones 2 and 4

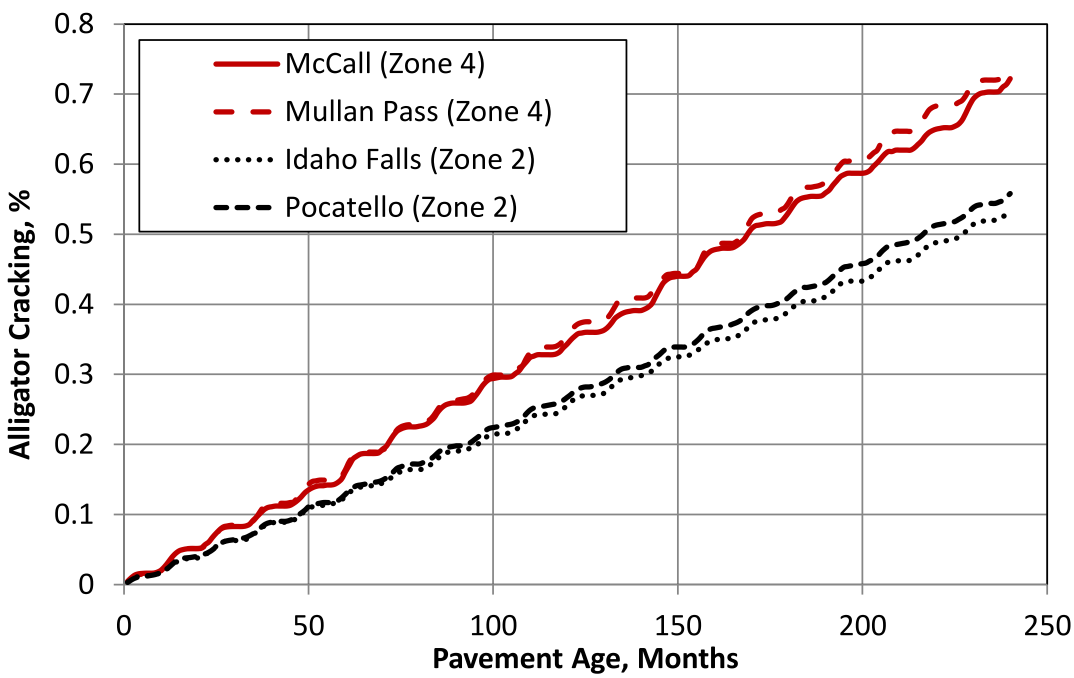

Figure 76. MEPDG Predicted Alligator Cracking for the Investigated Climatic Locations in Zones 2 and 4

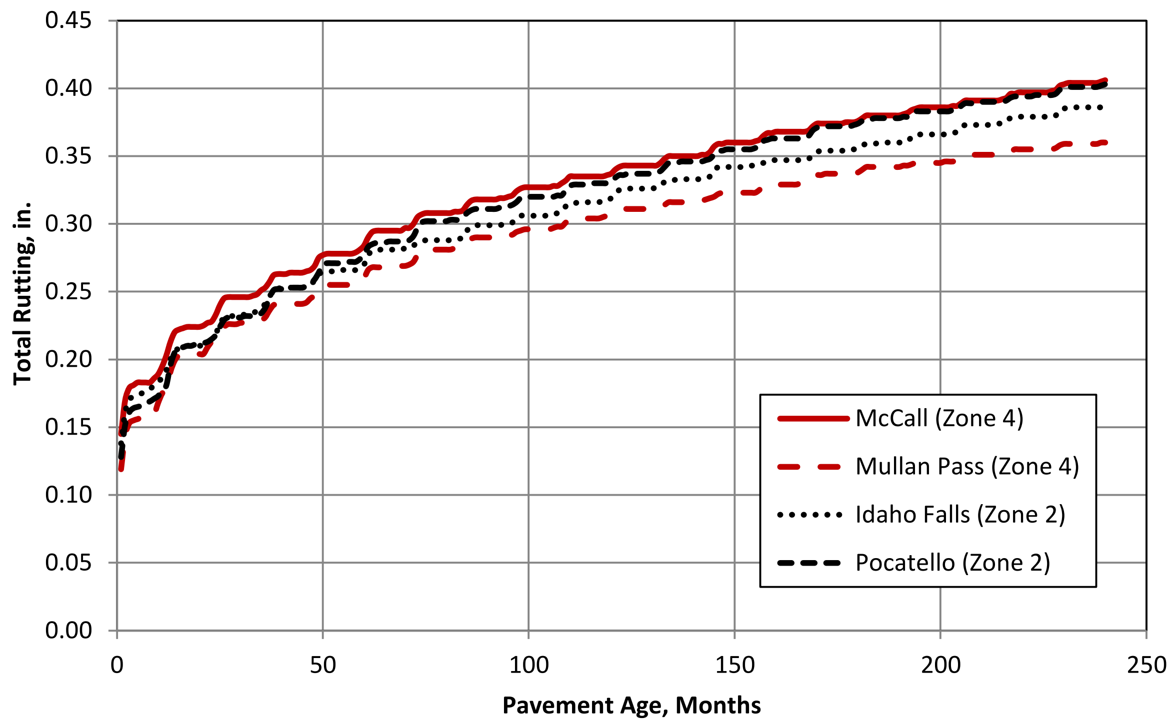

Figure 77. MEPDG Predicted Total Rutting for the Investigated Climatic Locations in Zones 2 and 4

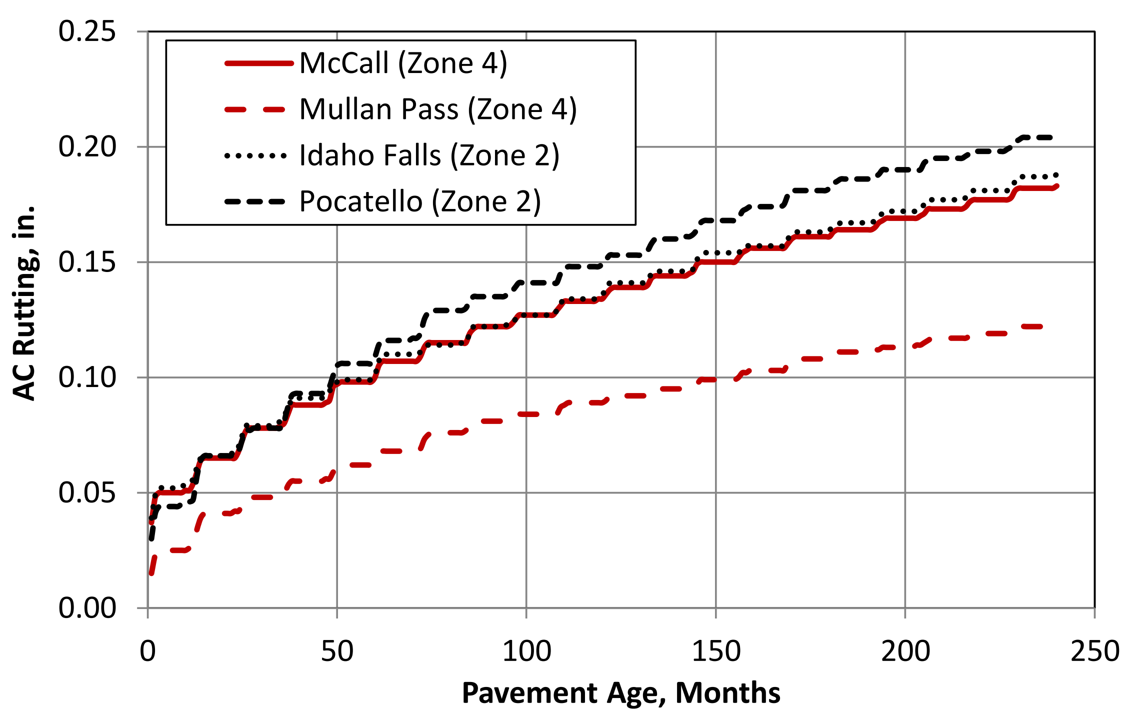

Figure 78. MEPDG Predicted AC Rutting for the Investigated Climatic Locations in ITD Zones 2 and 4

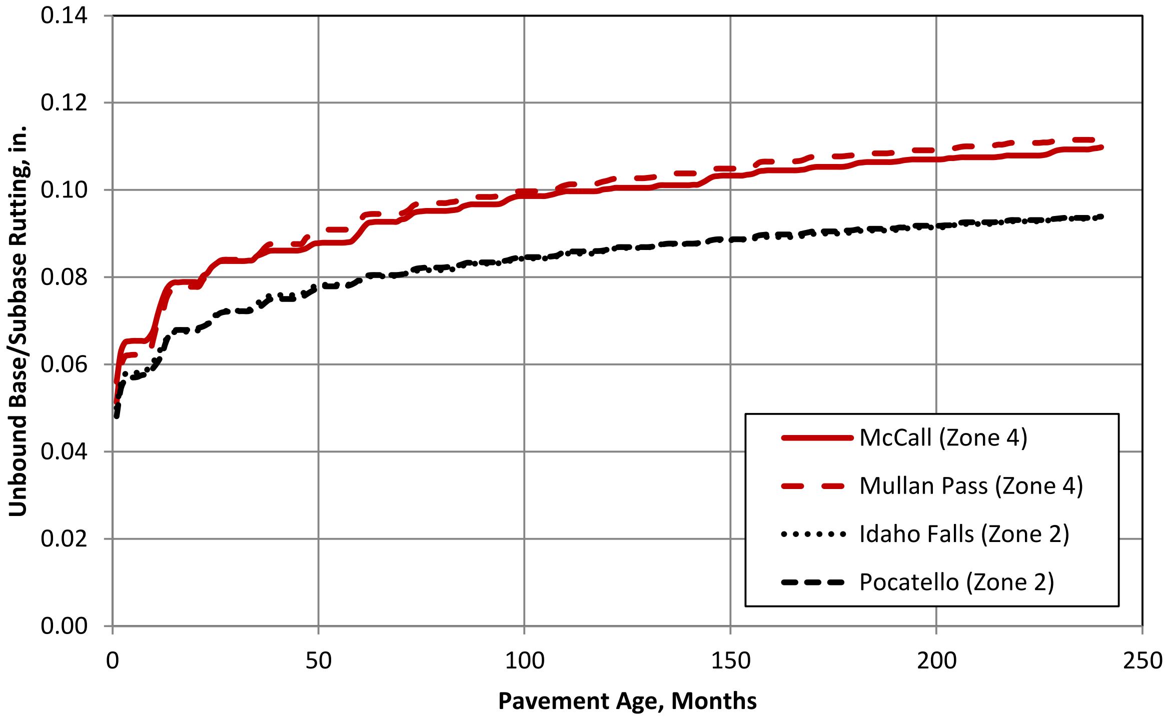

Figure 79. MEPDG Predicted Unbound Granular Layers Rutting for the Investigated Climatic Locations in ITD Zones 2 and 4

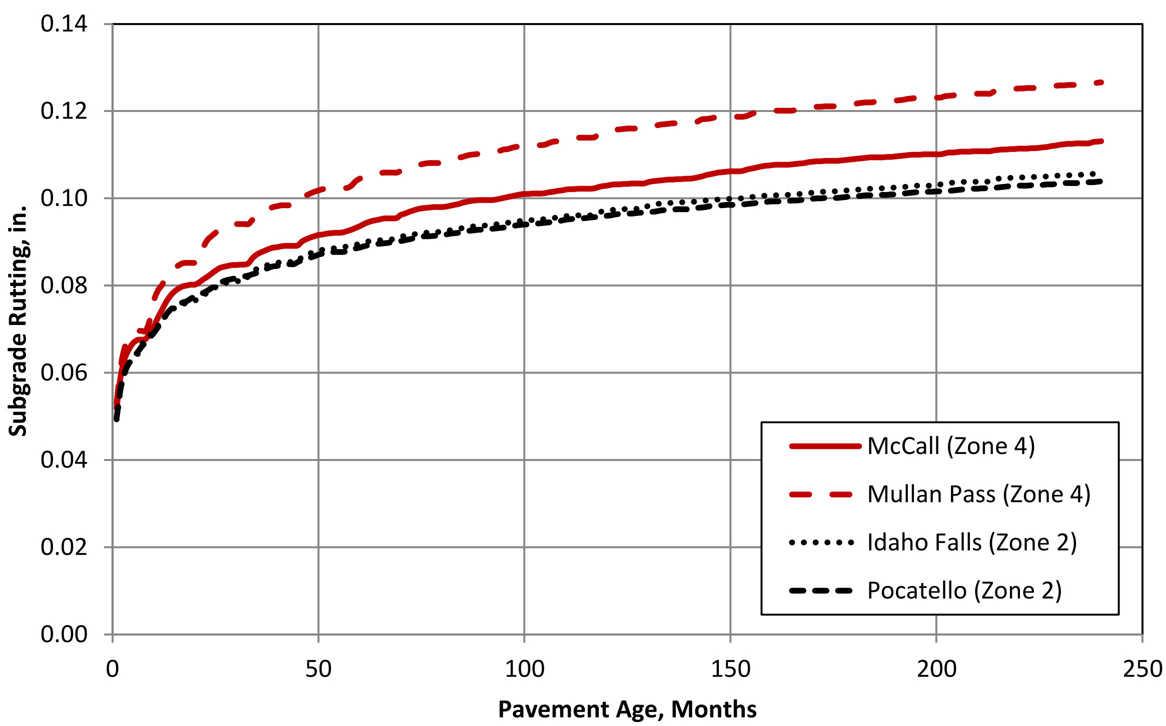

Figure 80. MEPDG Predicted Subgrade Rutting for the Investigated Climatic Locations in ITD Zones 2 and 4

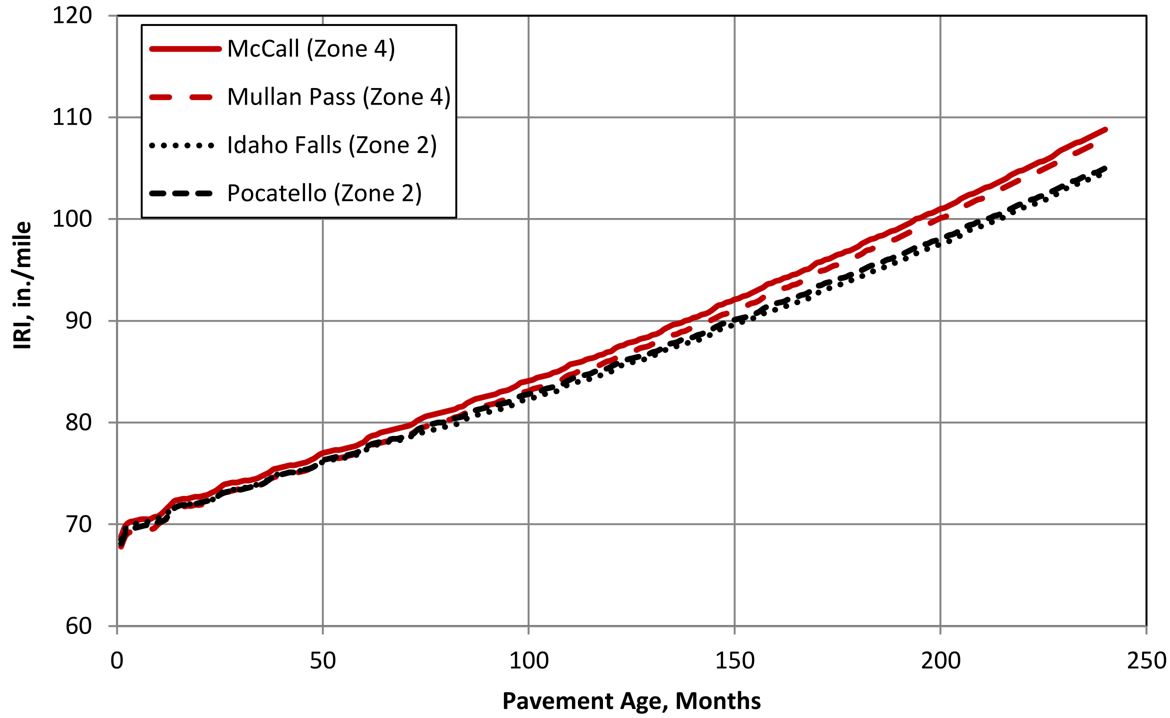

Figure 81. MEPDG Predicted IRI for the Investigated Climatic Locations in ITD Zones 2 and 4

Chapter 7
Summary, Conclusions, and Recommendations

Summary

In this study, researchers evaluated ITD's pavement design and compared it to the American Association of State Highway and Transportation Officials (AASHTO) 1993 design guide and to Mechanistic-Empirical Pavement Design Guide (MEPDG). This study also included a review of Equivalent Single Axle Loads (ESAL) calculation methods and an evaluation of ITD climatic factors. To conduct the analysis, eight in-service pavement sections located in different regions of Idaho were selected. All sections were redesigned using ITD, AASHTO 1993, and MEPDG procedures. The designs by AASHTO 1993 and MEPDG were conducted at 50 and 85 percent reliability levels. The nationally calibrated MEPDG (Version 1.1) was used to predict the performance of the three design methods. Input data required by the three design methods were collected from several sources. Level 2 Hot Mix Asphalt (HMA) and subgrade material characterization inputs were used in MEPDG analysis. All other MEPDG inputs were Level 3. Performance indicators predicted using MEPDG related to the 3 design methods were compared to each other. MEPDG performance predictions were also compared to measured field performance. ITD ESAL calculation and traffic volume projection methods were studied and compared to other states. New truck factors were developed for ITD based on the analysis of Idaho's Weigh-In-Motion (WIM) data. ITD climatic factors were also analyzed and compared with MEPDG.

Conclusions

Based on the results and analyses presented in this research, the following observations and conclusions are made:

Evaluation of ITD Design Method for Flexible Pavement

- In general, both ITD and AASHTO 1993 flexible pavement design methods yielded pavement structures that conform to MEPDG recommended design criteria.
- The unbound granular layer thicknesses resulting from ITD's design method were much greater (2 to 4.5 times as thick) than those recommended using AASHTO 1993 and MEPDG design methods.
- ITD's design method is more sensitive to the subgrade strength compared to AASHTO 1993 and MEPDG methods.
- The 3 design methods yielded reasonably similar asphalt layer thicknesses at 50 percent reliability. However, at higher reliability levels, MEPDG yielded greater asphalt concrete (AC) thickness compared to the other two methods, especially in cases of very weak subgrade strength.

- At 50 percent reliability, a reasonable agreement was found between the AASHTO 1993 and MEPDG design methods regarding the resulting pavement structure.
- No thermal cracking was predicted for any of the studied pavement sections using MEPDG.
- The resulting structural design for each of the investigated pavement sections, using MEPDG with national calibration factors, was found to be governed by the predicted total pavement rutting. Predicted alligator fatigue cracking, thermal cracking, and IRI were much lower than MEPDG recommended threshold values for the investigated sections.
- At the 85 percent reliability level, MEPDG designs generally yielded the least amount of cracking for sections with thicker AC. This indicates that AC layer thickness has more significant influence on the alligator cracking than does unbound layer thickness.

ITD ESAL Calculation Method

- ITD's Truck Factors (TF) used in ESAL calculations are conservative when compared to other state DOTs and AASHTO factors.
- Current ITD TF are highly conservative compared to the regional and statewide factors developed based on the analysis of WIM site data from various Idaho locations.
- Current ITD truck classification is based on equivalent wheel load (EWL) and does not accurately represent the current truck traffic.
- Current ITD traffic volume projection method yields highly conservative traffic volumes.

ITD Pavement Performance Management Information System

- ITD's current method of pavement evaluation combines thermal cracking, bottom-up alligator fatigue cracking, and top-down longitudinal cracking into one Cracking Index. In addition, ITD's crack ratings are measured and reported differently compared to MEPDG cracking measurement requirements. MEPDG calibration requires a separate value for each one of these distresses. Longitudinal and thermal cracking in MEPDG are predicted in units of ft/mile. Alligator fatigue cracking is computed as the area of cracking (ft^2/500 ft/lane width) and expressed as a percentage.
- For rutting evaluation, ITD measures the total rutting at the AC surface. Total rutting includes, HMA and rutting in the unbound and subgrade layers. MEPDG predicts the rutting within each layer separately.

MEPDG Predicted Performance for Idaho Conditions

- Comparison between total rutting predicted using the nationally calibrated MEPDG and actual measured field rutting for the investigated projects revealed that MEPDG's nationally calibrated rutting models are significantly over predicting total rutting. This is mostly true with predicted subgrade rutting.
- MEPDG traffic characterization through axle load spectra (ALS) yielded significantly higher cracking and rutting compared to traffic characterization in terms of ESALs.
- MEPDG predicted alligator cracking was found very low compared to the threshold values. These results agree with the reported crack index (CI) for the investigated pavement sections.
- The trends in MEPDG predicted alligator cracking were found to agree with the trends in the reported CI for the investigated pavement sections.
- MEPDG distress prediction and smoothness models need to be locally calibrated for Idaho conditions.

ITD Climatic Zones and Factors

- Analysis of pavement sections in the same climatic zones did not reveal similar performance in some zones while it revealed similar performance in others. The analysis indicates that Idaho's current climatic zones are not consistent and need to be revised. Current ITD climatic factors are empirical factors developed to provide additional protection during winter and spring conditions. MEPDG takes into account the influence of the hourly change in moisture and temperature on the fundamental materials properties.

Recommendations

Based on the findings of this research the following are recommended:

- It is recommended that ITD proceed with the implementation and calibration of the MEPDG for Idaho to replace the ITD's current design method as soon as practical.
- To ensure consistency with MEPDG distress prediction, it is recommended that ITD perform pavement condition surveys and update their Pavement Performance Management Information System (PPMIS) in accordance with LTPP method of data collection.
- It is recommended to replace the current ITD truck classification system with the FHWA truck classification system or a simplified system derived from it.
- It is recommended to replace Idaho's current TF with the newly developed truck factors based on the analysis of Idaho WIM site data.

- It is recommended that ITD's current method for projecting future traffic volume needs to be changed as it consistently over predicts traffic volume. There are several traffic forecasting methods that ITD may investigate. These methods include: time series forecasting, regression, clustering, and neural networks.

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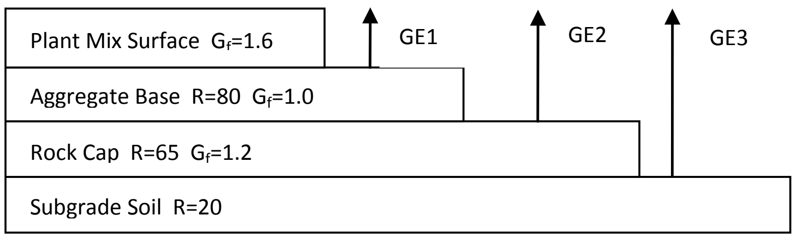
Appendix A
Idaho R-Value Thickness Design

US-2 (a)



<u>Calculate the Design ESALs</u>

$TI = 9.0\left(\frac{ESALs}{10^6}\right)^{0.119}$ $TI = 9.0\left(\frac{7{,}920{,}000}{10^6}\right)^{0.119}$ $TI = 11.51,\ use\ 11.5$

<u>Calculate the ballast requirement for the plant mix surface, including climate adjustment.</u>

$GE = 0.0032(TI)(100 - R)(CF)$
$GE = 0.0032(11.5)(100 - 80)(1.1)$
$GE = 0.81\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for the plant mix pavement.</u>

$T = {}^{0.81}/_{1.6} = 0.51\ ft, use\ 0.50\ ft. plant\ mix$
$GE_{1(actual)} = 0.50 \times 1.6$
$GE_{1(actual)} = 0.80$

<u>Calculate the ballast requirement for the crushed aggregate base course.</u>

$GE = 0.0032(11.5)(100 - 65)(1.1)$
$GE = 1.42\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for aggregate base.</u>

$T = {}^{(1.42 - 0.80)}/_{1.0} = 0.61\ ft, use\ 0.60\ ft. aggregate\ base.$
$GE_{2(actual)} = (0.60 \times 1.0) + 0.80$
$GE_{2(actual)} = 1.4\ ft$

<u>Calculate the ballast requirement for the rock cap.</u>

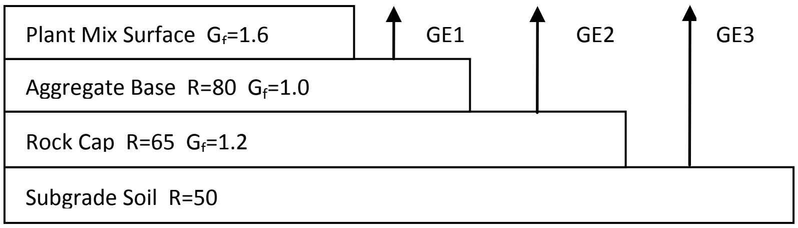
$GE = 0.0032(11.5)(100 - 20)(1.1)$
$GE = 3.24\ ft., use\ 3.3\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for rock cap.</u>

$T = {}^{(3.3 - 1.4)}/_{1.2} = 1.58\ ft, use\ 1.60\ ft. Rock\ cap.$
$GE_{3(actual)} = (1.6 \times 1.2) + 1.40$
$GE_{3(actual)} = 3.32\ ft$

The designed typical section is composed of:	The constructed typical section is composed of:
0.5’ (6”) plant mix pavement	0.5’ (6”) plant mix pavement
0.6’ (7.2”) aggregate base	0.5’ (6”) aggregate base
1.6’ (19.2”) rock cap	2.2’ (26.4”) rock cap

US-2 (b)



Calculate the Design ESALs

$TI = 9.0\left(\frac{ESALs}{10^6}\right)^{0.119}$ $TI = 9.0\left(\frac{7{,}920{,}000}{10^6}\right)^{0.119}$ $TI = 11.51,\ use\ 11.5$

Calculate the ballast requirement for the plant mix surface, including climate adjustment.

$GE = 0.0032(TI)(100 - R)(CF)$

$GE = 0.0032(11.5)(100 - 80)(1.1)$

$GE = 0.81\ ft.$

Calculate the layer thickness by applying the substitution ratio for the plant mix pavement.

$T = {}^{0.81}/_{1.6} = 0.51\ ft, use\ 0.50\ ft. plant\ mix$

$GE_{1(actual)} = 0.50 \times 1.6$

$GE_{1(actual)} = 0.80$

Calculate the ballast requirement for the crushed aggregate base course.

$GE = 0.0032(11.5)(100 - 65)(1.1)$

$GE = 1.42\ ft.$

Calculate the layer thickness by applying the substitution ratio for aggregate base.

$T = {}^{(1.42 - 0.80)}/_{1.0} = 0.61\ ft, use\ 0.60\ ft. aggregate\ base.$

$GE_{2(actual)} = (0.60 \times 1.0) + 0.80$

$GE_{2(actual)} = 1.4\ ft$

Calculate the ballast requirement for the rock cap.

$GE = 0.0032(11.5)(100 - 50)(1.1)$

$GE = 2.02\ ft.$

Calculate the layer thickness by applying the substitution ratio for rock cap.

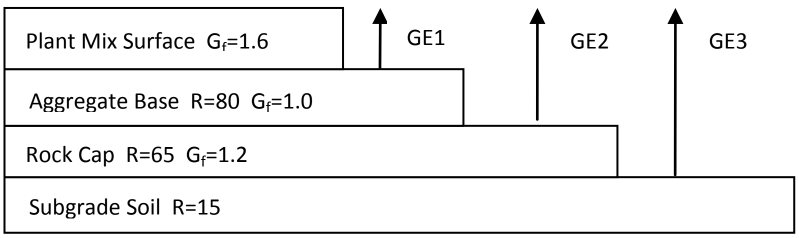
$T = {}^{(2.02 - 1.4)}/_{1.2} = 0.62\ ft, use\ 0.60\ ft. Rock\ cap.$

$GE_{3(actual)} = (0.6 \times 1.2) + 1.40$

$GE_{3(actual)} = 2.12\ ft$

The designed typical section is composed of:	The constructed typical section is composed of:
0.5’ (6”)plant mix pavement 0.6’ (7.2”) aggregate base 0.6’ (7.2”) rock cap	0.5’ (6”) plant mix pavement 0.5’ (6”) aggregate base 0.75’ (9”) rock cap

US-2 (c)



Calculate the Design ESALs

$TI = 9.0\left(\frac{ESALs}{10^6}\right)^{0.119}$ $TI = 9.0\left(\frac{7{,}920{,}000}{10^6}\right)^{0.119}$ $TI = 11.51,\ use\ 11.5$

Calculate the ballast requirement for the plant mix surface, including climate adjustment.

$GE = 0.0032(TI)(100 - R)(CF)$
$GE = 0.0032(11.5)(100 - 80)(1.1)$
$GE = 0.81\ ft.$

Calculate the layer thickness by applying the substitution ratio for the plant mix pavement.

$T = {}^{0.81}/_{1.6} = 0.51\ ft, use\ 0.50\ ft. plant\ mix$
$GE_{1(actual)} = 0.50 \times 1.6$
$GE_{1(actual)} = 0.80$

Calculate the ballast requirement for the crushed aggregate base course.

$GE = 0.0032(11.5)(100 - 65)(1.1)$
$GE = 1.42\ ft.$

Calculate the layer thickness by applying the substitution ratio for aggregate base.

$T = {}^{1.42 - 0.80}/_{1.0} = 0.61\ ft, use\ 0.60\ ft. aggregate\ base.$
$GE_{2(actual)} = (0.60 \times 1.0) + 0.80$
$GE_{2(actual)} = 1.4\ ft$

Calculate the ballast requirement for the rock cap.

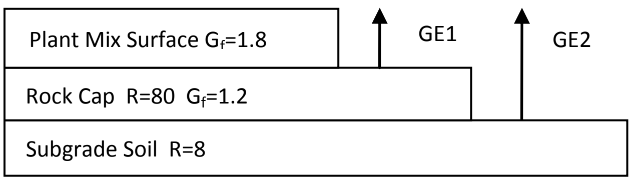
$GE = 0.0032(11.5)(100 - 15)(1.1)$
$GE = 3.44\ ft., use\ 3.5\ ft.$

Calculate the layer thickness by applying the substitution ratio for rock cap.

$T = {}^{3.5 - 1.4}/_{1.2} = 1.75\ ft, use\ 1.75\ ft. Rock\ cap.$
$GE_{3(actual)} = (1.75 \times 1.2) + 1.40$
$GE_{3(actual)} = 3.5\ ft$

The designed typical section is composed of:	The constructed typical section is composed of:
0.5’ (6”) plant mix pavement 0.6’ (7.2”) aggregate base 1.75’ (21”) rock cap	0.5’ (6”) plant mix pavement 0.5’ (6”) aggregate base 2.5’ (30”) rock cap

SH-62



<u>Calculate the Design ESALs</u>

$$TI = 9.0\left(\frac{ESALs}{10^6}\right)^{0.119} \quad TI = 9.0\left(\frac{816{,}000}{10^6}\right)^{0.119} \quad TI = 8.78$$

<u>Calculate the ballast requirement for the plant mix surface, including climate adjustment.</u>

$GE = 0.0032(TI)(100 - R)(CF)$

$GE = 0.0032(8.78)(100 - 80)(1.1)$

$GE = 0.62\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for the plant mix pavement.</u>

$T = {}^{0.62}/_{1.8} = 0.34\ ft, use\ 0.35\ ft. plant\ mix$

$GE_{1(actual)} = 0.35 \times 1.8$

$GE_{1(actual)} = 0.63$

<u>Calculate the ballast requirement for the rock cap.</u>

$GE = 0.0032(8.78)(100 - 8)(1.1)$

$GE = 2.84\ ft., use\ 2.85\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for rock cap.</u>

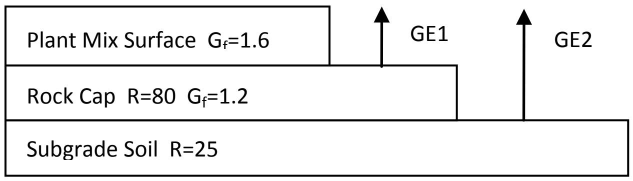
$T = {}^{(2.85 - .63)}/_{1.2} = 1.85\ ft, use\ 1.85\ ft. Rock\ cap.$

$GE_{2(actual)} = (1.85 \times 1.2) + 0.63$

$GE_{2(actual)} = 2.85\ ft$

The designed typical section is composed of:	The constructed typical section is composed of:
0.35’ (4.2”) plant mix pavement 1.85’ (22.2”) rock cap	0.35’ (4.2”) plant mix pavement 1.85’ (22.2”) rock cap

SH-3



Calculate the Design ESALs

$TI = 9.0\left(\frac{ESALs}{10^6}\right)^{0.119}$ $TI = 9.0\left(\frac{3{,}696{,}000}{10^6}\right)^{0.119}$ $TI = 10.51,\ use\ 10.5$

Calculate the ballast requirement for the plant mix surface, including climate adjustment.

$GE = 0.0032(TI)(100 - R)(CF)$

$GE = 0.0032(10.5)(100 - 80)(1.0)$

$GE = 0.67\ ft.$

Calculate the layer thickness by applying the substitution ratio for the plant mix pavement.

$T = {}^{0.67}/_{1.6} = 0.42\ ft, use\ 0.45\ ft. plant\ mix$

$GE_{1(actual)} = 0.45 \times 1.6$

$GE_{1(actual)} = 0.72$

Calculate the ballast requirement for the rock cap.

$GE = 0.0032(10.5)(100 - 25)(1.0)$

$GE = 2.52\ ft., use\ 2.5\ ft.$

Calculate the layer thickness by applying the substitution ratio for rock cap.

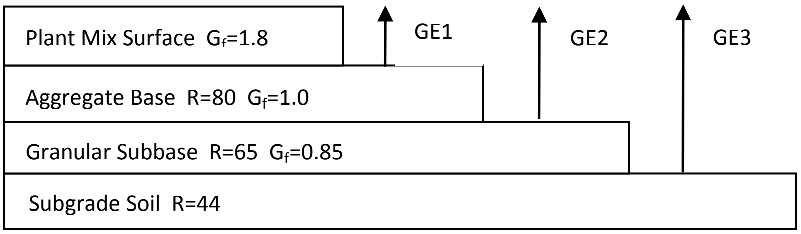
$T = {}^{(2.5 - 0.72)}/_{1.2} = 1.48\ ft, use\ 1.50\ ft. Rock\ cap.$

$GE_{3(actual)} = (1.5 \times 1.2) + 0.72$

$GE_{3(actual)} = 2.52\ ft$

The designed typical section is composed of:	The constructed typical section is composed of:
0.45’ (5.4”) plant mix pavement 1.5’ (18”)rock cap	0.45’ (5.4”) plant mix pavement 1.6’ (19.2) rock cap

SH-19



<u>Calculate the Design ESALs</u>

$TI = 9.0\left(\frac{ESALs}{10^6}\right)^{0.119}$ $TI = 9.0\left(\frac{1{,}677{,}000}{10^6}\right)^{0.119}$ $TI = 9.57,\ use\ 9.6$

<u>Calculate the ballast requirement for the plant mix surface, including climate adjustment.</u>

$GE = 0.0032(TI)(100 - R)(CF)$
$GE = 0.0032(9.6)(100 - 80)(1.0)$
$GE = 0.61\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for the plant mix pavement.</u>

$T = {}^{0.61}/_{1.8} = 0.34\ ft, use\ 0.35\ ft. plant\ mix$
$GE_{1(actual)} = 0.35 \times 1.8$
$GE_{1(actual)} = 0.63$

<u>Calculate the ballast requirement for the crushed aggregate base course.</u>

$GE = 0.0032(9.6)(100 - 65)(1.0)$
$GE = 1.08\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for aggregate base.</u>

$T = {}^{(1.08 - 0.63)}/_{1.0} = 0.45\ ft, use\ 0.45\ ft. aggregate\ base.$
$GE_{2(actual)} = (0.45 \times 1.0) + 0.63$
$GE_{2(actual)} = 1.08\ ft$

<u>Calculate the ballast requirement for the granular subbase.</u>

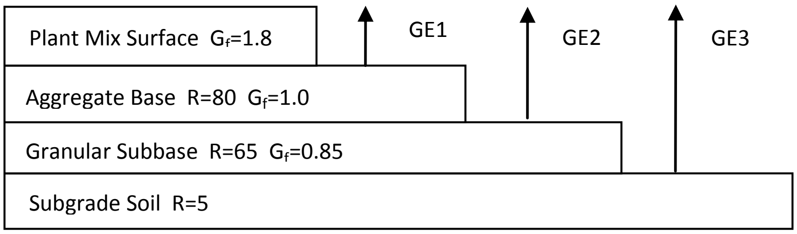
$GE = 0.0032(9.6)(100 - 44)(1.0)$
$GE = 1.72\ ft., use\ 1.8\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for granular subbase.</u>

$T = {}^{(1.8 - 1.08)}/_{0.85} = 0.85\ ft, use\ 0.85\ ft. granular\ subbase.$
$GE_{3(actual)} = (0.85 \times 0.85) + 1.13$
$GE_{3(actual)} = 1.80\ ft$

The designed typical section is composed of:	The constructed typical section is composed of:
0.35’ (4.2”) plant mix pavement 0.45’ (5.4”) aggregate base 0.85’ (10.2”) granular subbase	0.35’ (4.2”) plant mix pavement 0.45’ (5.4”) aggregate base 1.0’ (12”) granular subbase

US-95



<u>Calculate the Design ESALs</u>

$TI = 9.0\left(\frac{ESALs}{10^6}\right)^{0.119}$ $TI = 9.0\left(\frac{2{,}240{,}000}{10^6}\right)^{0.119}$ $TI = 9.911,\ use\ 9.9$

<u>Calculate the ballast requirement for the plant mix surface, including climate adjustment.</u>

$GE = 0.0032(TI)(100 - R)(CF)$

$GE = 0.0032(9.9)(100 - 80)(1.0)$

$GE = 0.63\ ft.$

<u>Calculate the layer thickness by applying the substitution ration for the plant mix pavement.</u>

$T = {}^{0.63}/_{1.8} = 0.35\ ft, use\ 0.35\ ft. plant\ mix$

$GE_{1(actual)} = 0.35 \times 1.8$

$GE_{1(actual)} = 0.63$

<u>Calculate the ballast requirement for the crushed aggregate base course.</u>

$GE = 0.0032(9.9)(100 - 60)(1.0)$

$GE = 1.27\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for aggregate base.</u>

$T = {}^{(1.27 - 0.63)}/_{1.0} = 0.64\ ft, use\ 0.65\ ft. aggregate\ base.$

$GE_{2(actual)} = (0.65 \times 1.0) + 0.63$

$GE_{2(actual)} = 1.28\ ft$

<u>Calculate the ballast requirement for the rock cap.</u>

$GE = 0.0032(9.9)(100 - 5)(1.0)$

$GE = 3.01\ ft., use\ 3.0\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for rock cap.</u>

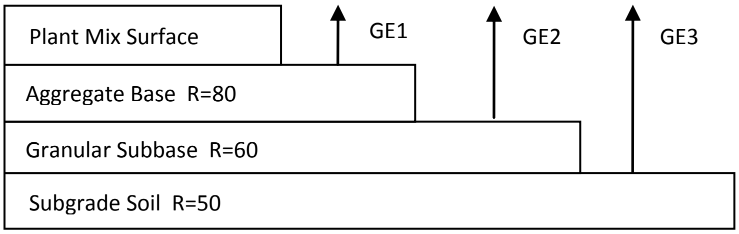
$T = {}^{(3.0 - 1.28)}/_{1.2} = 1.43\ ft, use\ 1.5\ ft. Rock\ cap.$

$GE_{3(actual)} = (1.5 \times 1.2) + 1.28$

$GE_{3(actual)} = 3.08\ ft$

The designed typical section is composed of:	The constructed typical section is composed of:
0.35’ (4.2”) plant mix pavement	0.3’ (3.6”) plant mix pavement
0.65’ (7.8”) aggregate base	0.5’ (6”) aggregate base
1.5’ (18”) rock cap	2.5’ (30”) rock cap

US-93



<u>Calculate the Design ESALs</u>

$TI = 9.0\left(\frac{ESALs}{10^6}\right)^{0.119}$ $TI = 9.0\left(\frac{3{,}034{,}000}{10^6}\right)^{0.119}$ $TI = 10.27,\ use\ 10.3$

<u>Calculate the ballast requirement for the plant mix surface, including climate adjustment.</u>

$GE = 0.0032(TI)(100 - R)(CF)$
$GE = 0.0032(10.3)(100 - 80)(1.05)$
$GE = 0.69\ ft.$

<u>Calculate the layer thickness by applying the substitution ration for the plant mix pavement.</u>

$T = {}^{0.69}/_{1.6} = 0.43\ ft, use\ 0.45\ ft. plant\ mix$
$GE_{1(actual)} = 0.45 \times 1.6$
$GE_{1(actual)} = 0.72$

<u>Calculate the ballast requirement for the crushed aggregate base course.</u>

$GE = 0.0032(10.3)(100 - 60)(1.05)$
$GE = 1.38\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for aggregate base.</u>

$T = {}^{(1.38 - 0.72)}/_{1.0} = 0.66\ ft, use\ 0.65\ ft. aggregate\ base.$
$GE_{2(actual)} = (0.65 \times 1.0) + 0.72$
$GE_{2(actual)} = 1.37\ ft$

<u>Calculate the ballast requirement for the rock cap.</u>

$GE = 0.0032(10.3)(100 - 50)(1.05)$
$GE = 1.73\ ft., use\ 1.75\ ft.$

<u>Calculate the layer thickness by applying the substitution ratio for rock cap.</u>

$T = {}^{(1.75 - 1.37)}/_{0.85} = 0.45\ ft, use\ 0.45\ ft. Rock\ cap.$
$GE_{3(actual)} = (0.45 \times 0.85) + 1.37$
$GE_{3(actual)} = 1.75\ ft$

The designed typical section is composed of:	The constructed typical section is composed of:
0.45’ (5.4”) plant mix pavement 0.65’ (7.8”) aggregate base 0.45’ (5.4”) granular subbase	0.5’ (6”) plant mix pavement 0.75’ (9”)aggregate base 0.5’ (6”) granular subbase

Appendix B
Comparison of MEPDG Predicted Distresses and Smoothness at 50 Percent Reliability

The figures presented in this appendix show a comparison of MEPDG predicted distresses and smoothness for structures designed using ITD, MEPDG, and AASHTO 1993 at 50 percent reliability.

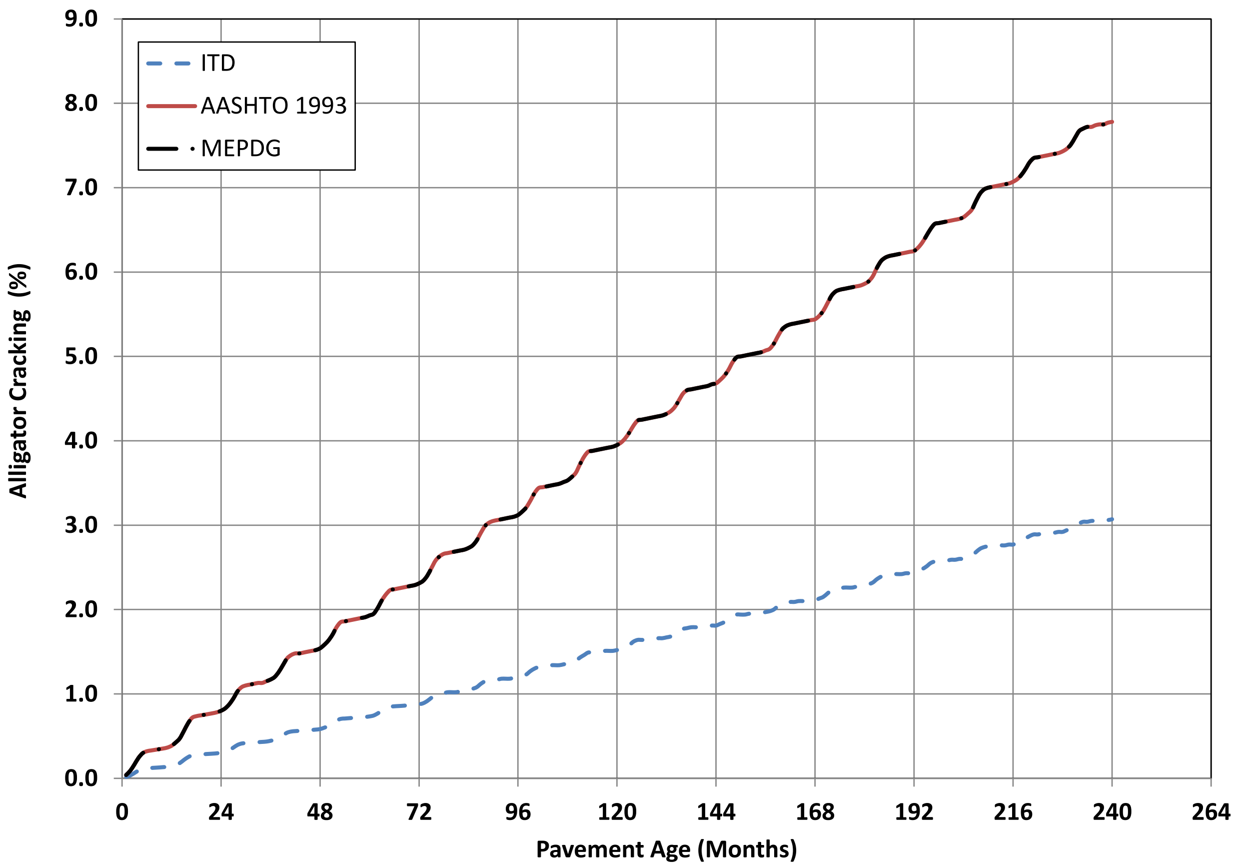


Figure 82. Comparison of the Predicted Alligator Cracking, US-2(a) Project

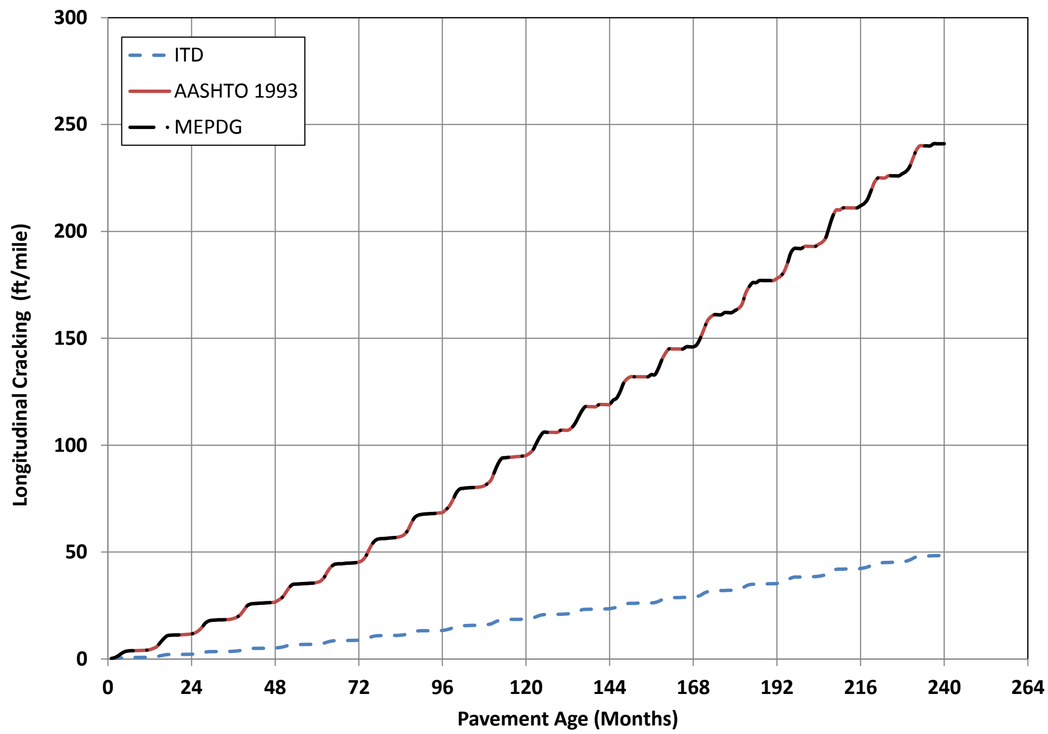


Figure 83. Comparison of the Predicted Longitudinal Cracking, US-2(a) Project

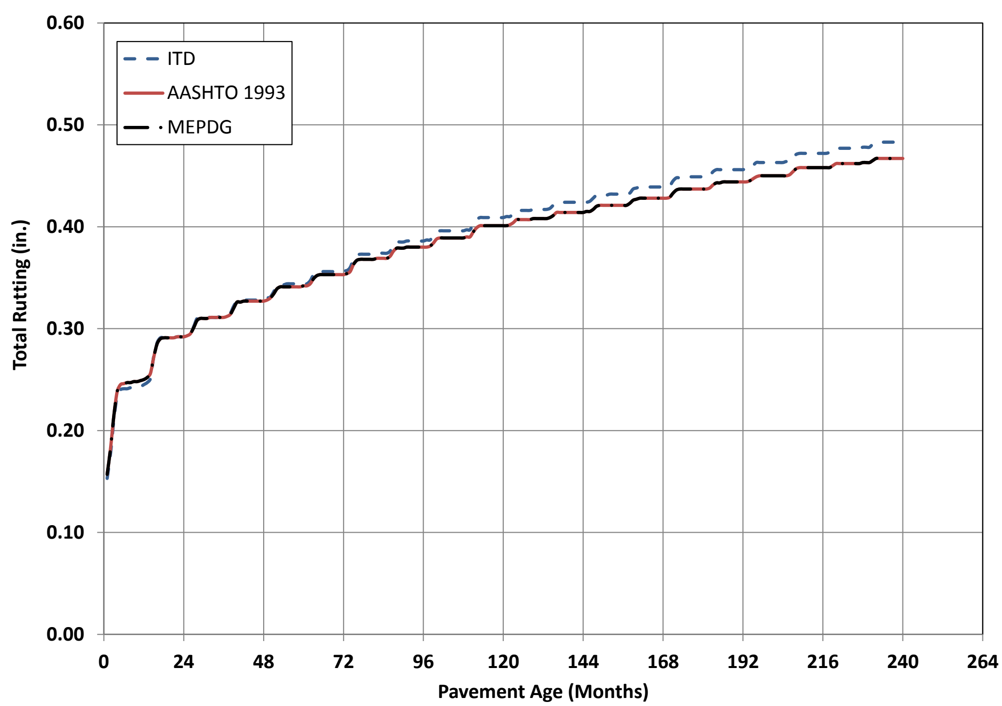


Figure 84. Comparison of the Predicted Total Rutting, US-2(a) Project

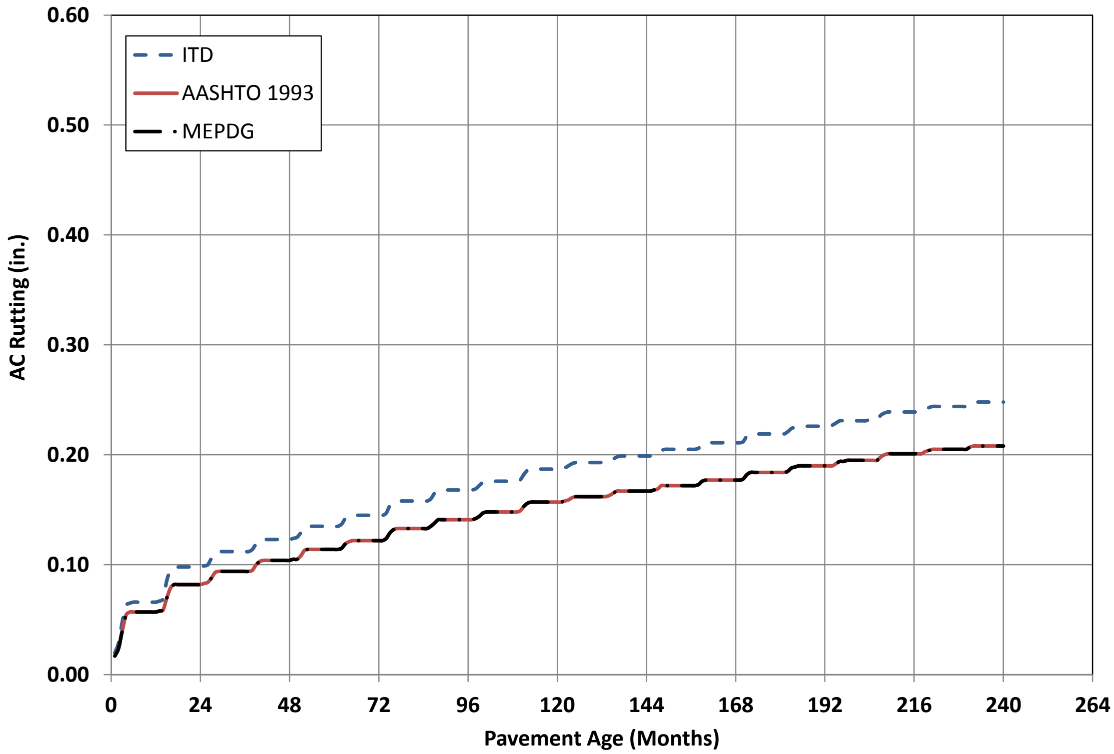


Figure 85. Comparison of the Predicted AC Layer Rutting, US-2(a) Project

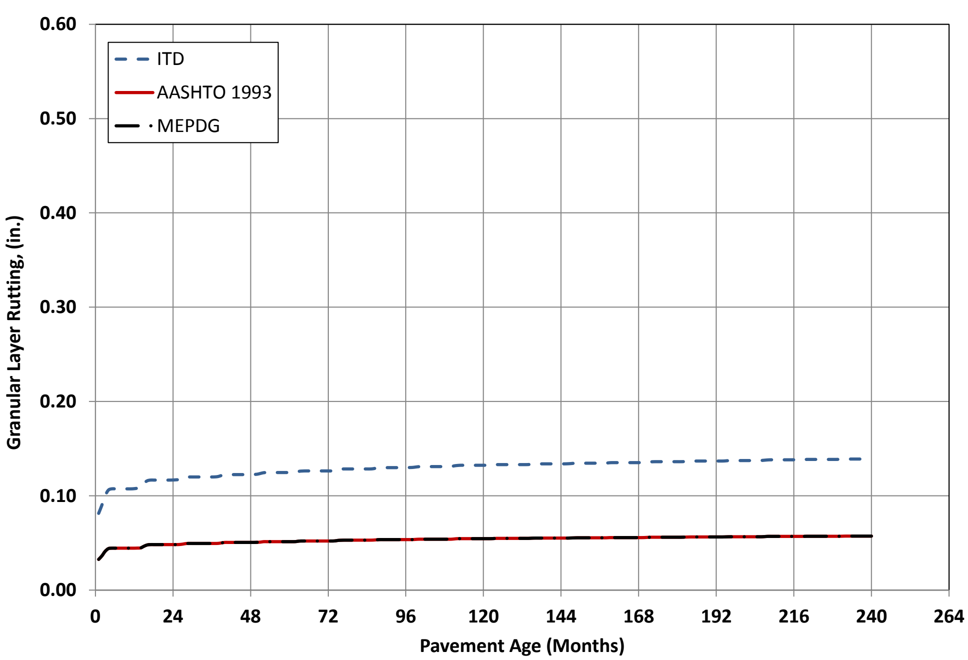


Figure 86. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-2(a) Project

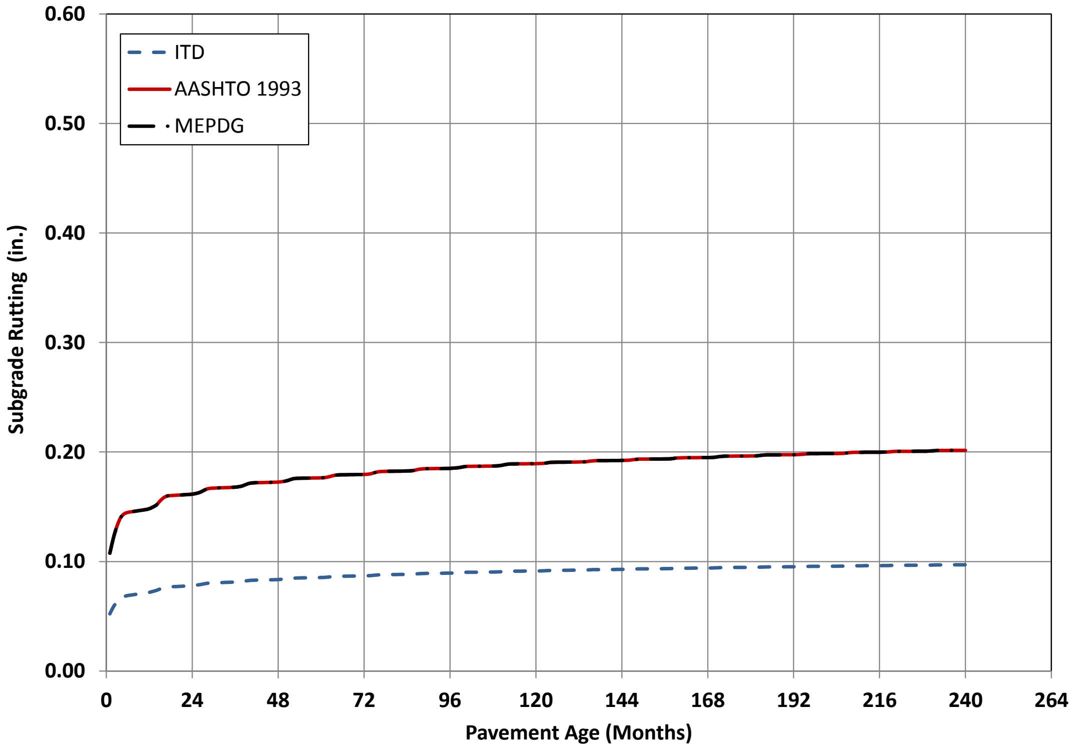


Figure 87. Comparison of the Predicted Subgrade Layer Rutting, US-2(a) Project

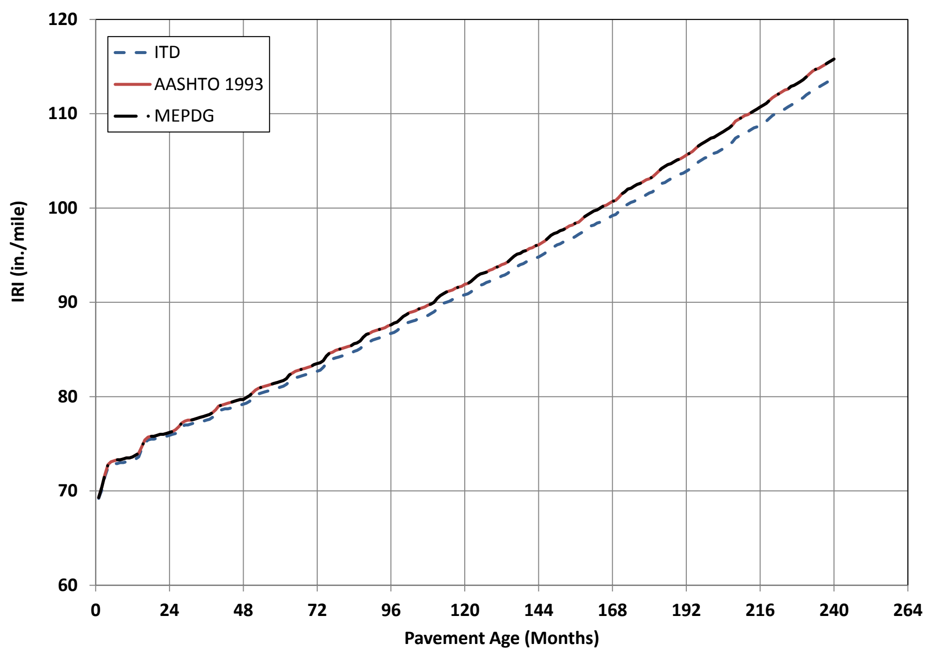


Figure 88. Comparison of the Predicted IRI, US-2(a) Project

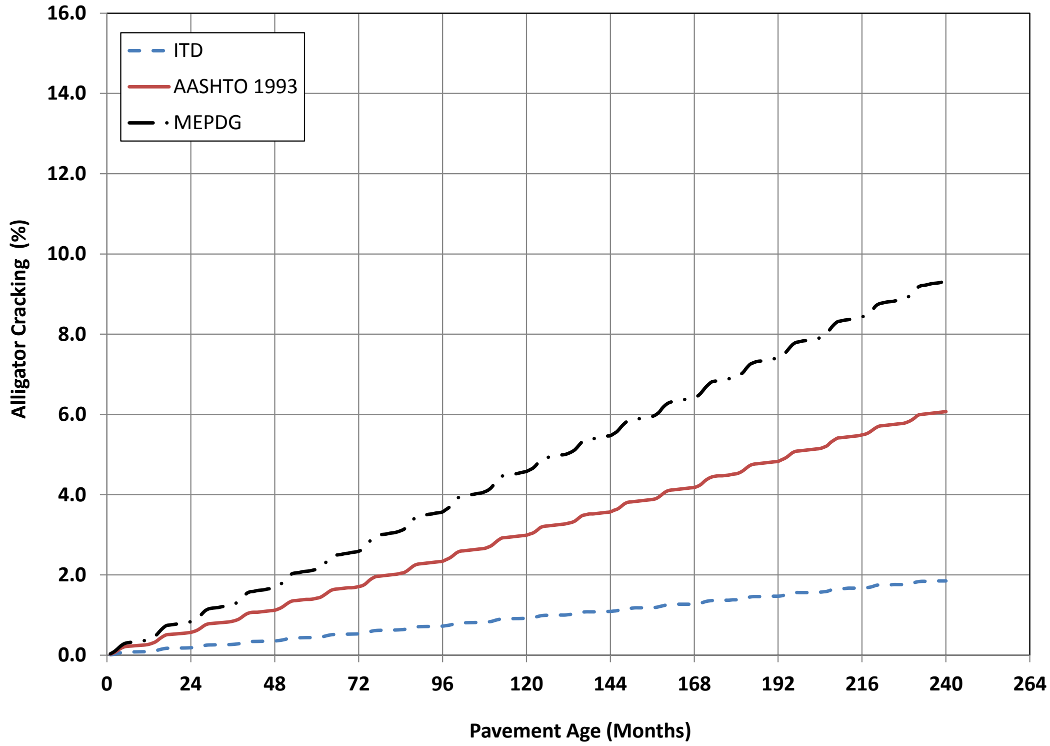


Figure 4. Comparison of the Predicted Alligator Cracking, US-2(b) Project

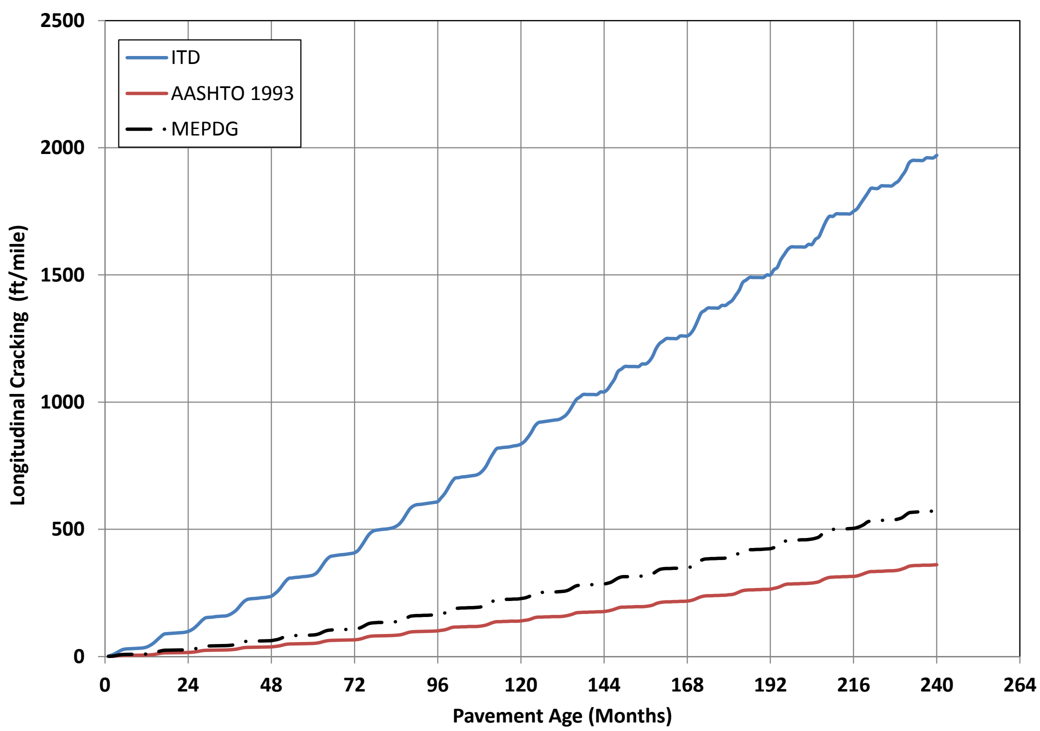


Figure 90. Comparison of the Predicted Longitudinal Cracking, US-2(b) Project

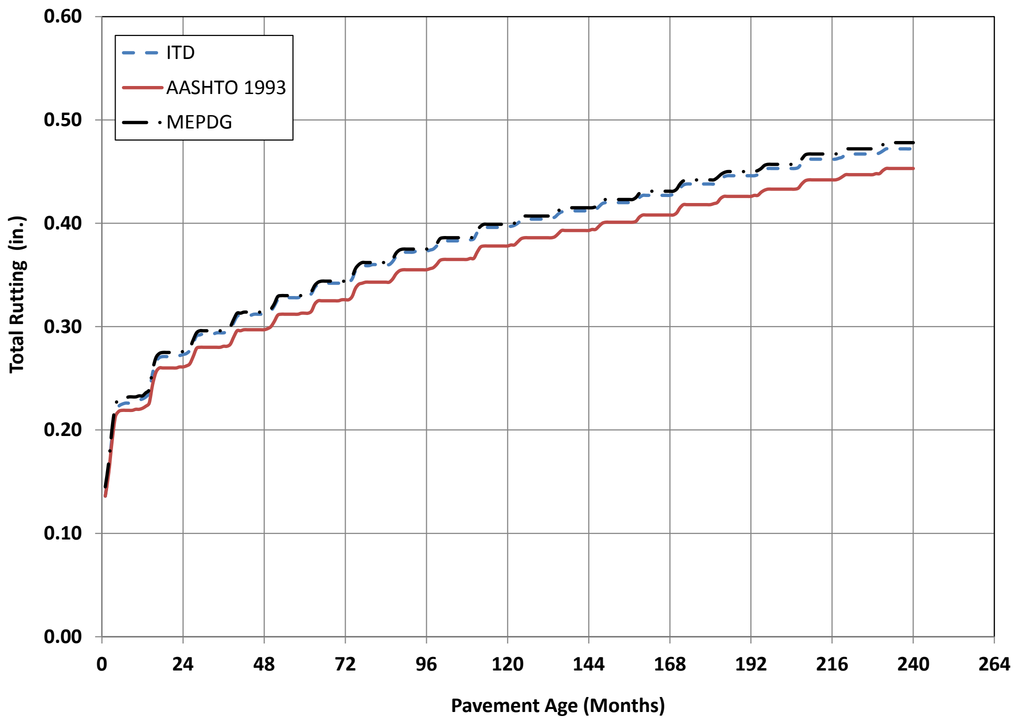


Figure 91. Comparison of the Predicted Total Rutting, US-2(b) Project

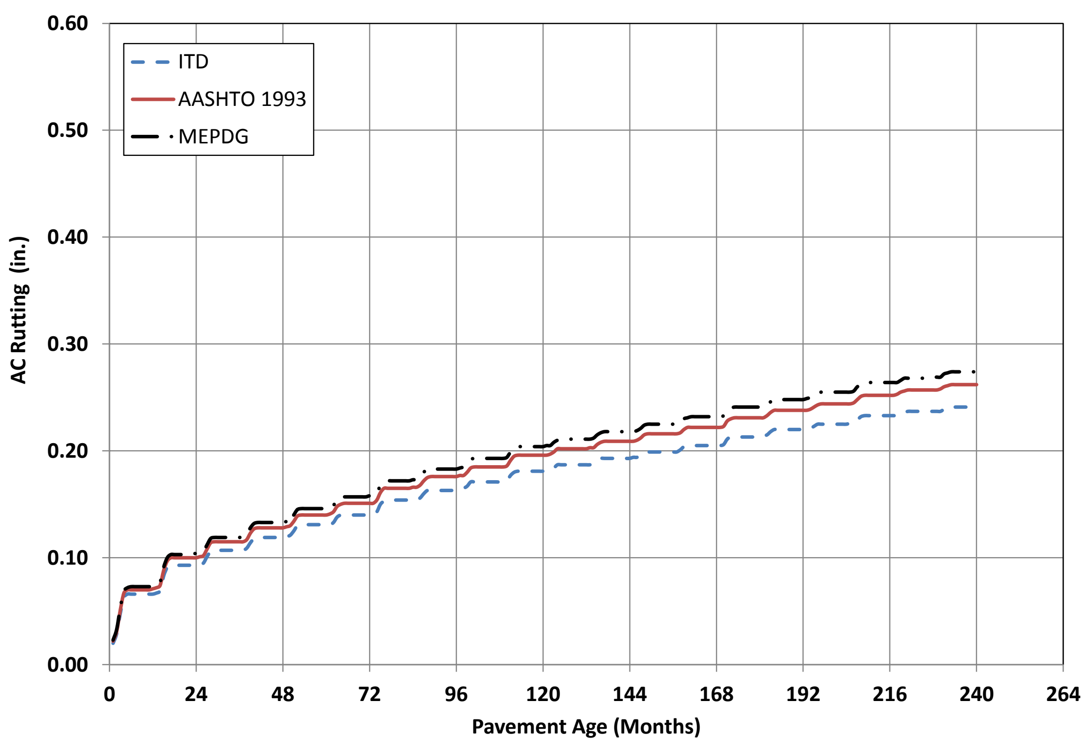


Figure 92. Comparison of the Predicted AC Layer Rutting, US-2(b) Project

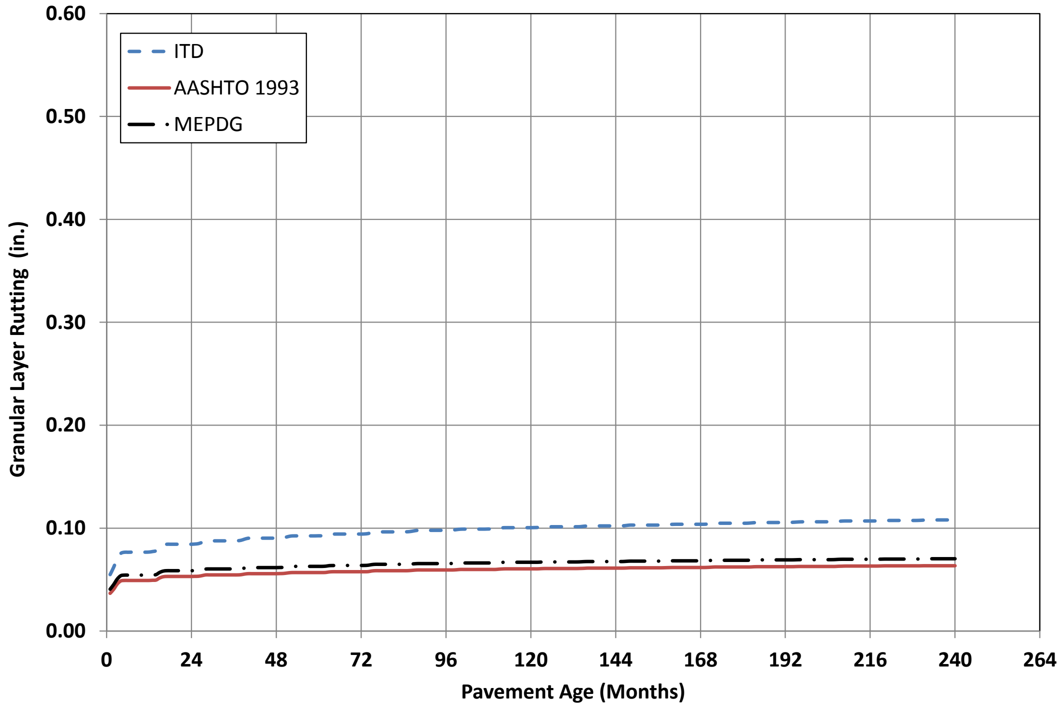


Figure 93. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-2(b) Project

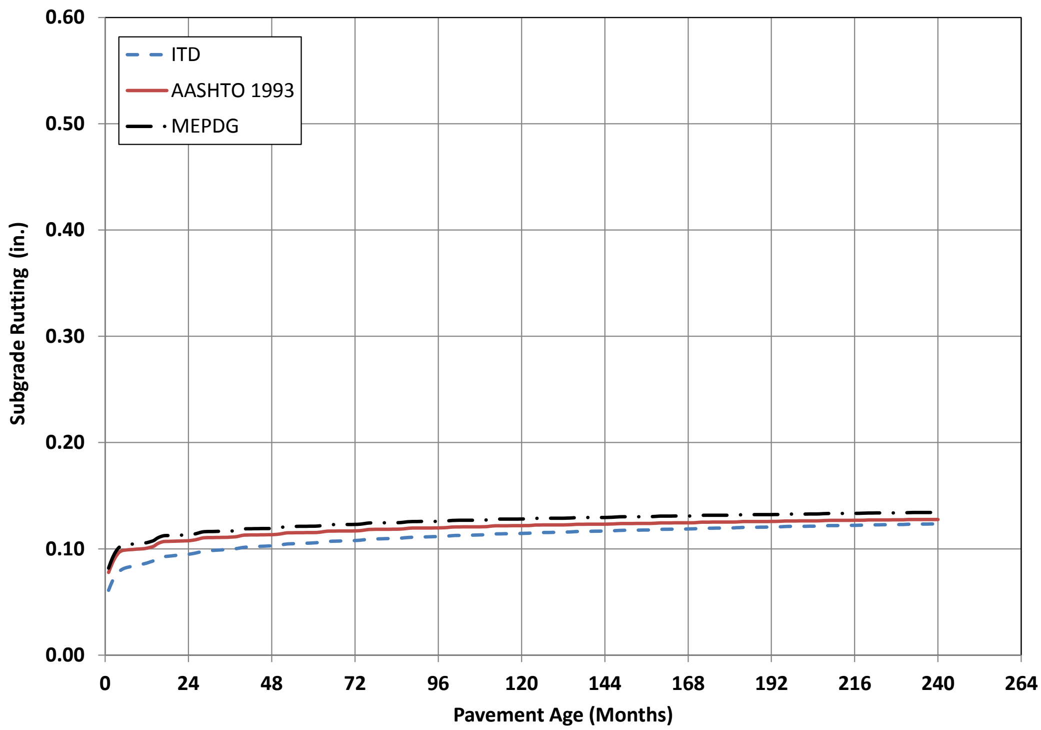


Figure 94. Comparison of the Predicted Subgrade Layer Rutting, US-2(b) Project

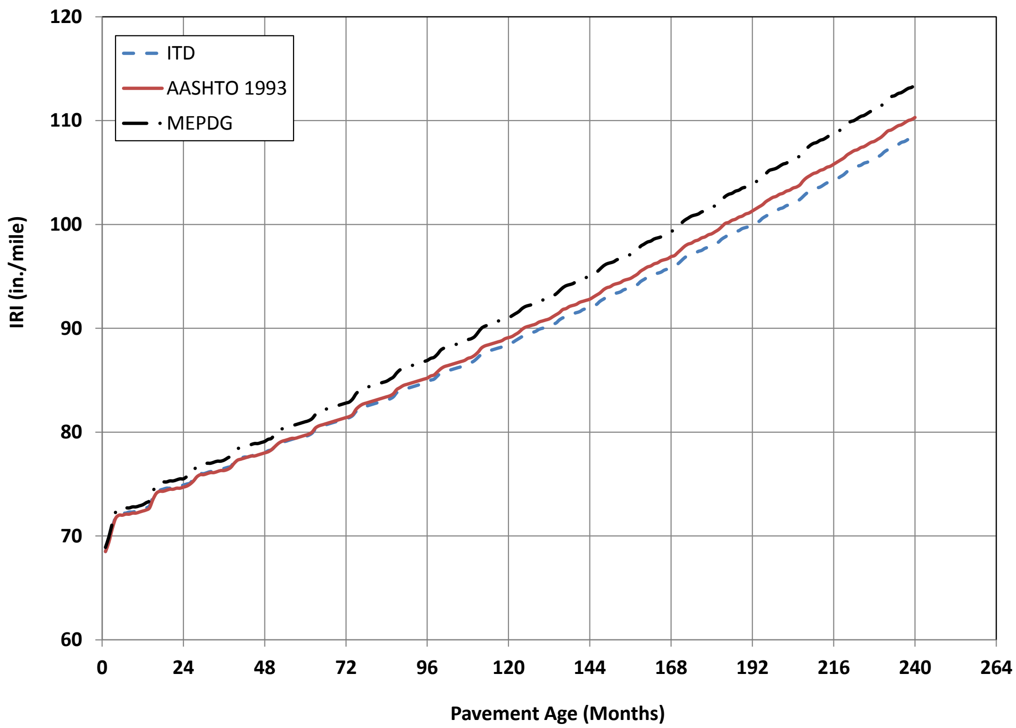


Figure 95. Comparison of the Predicted IRI, US-2(b) Project

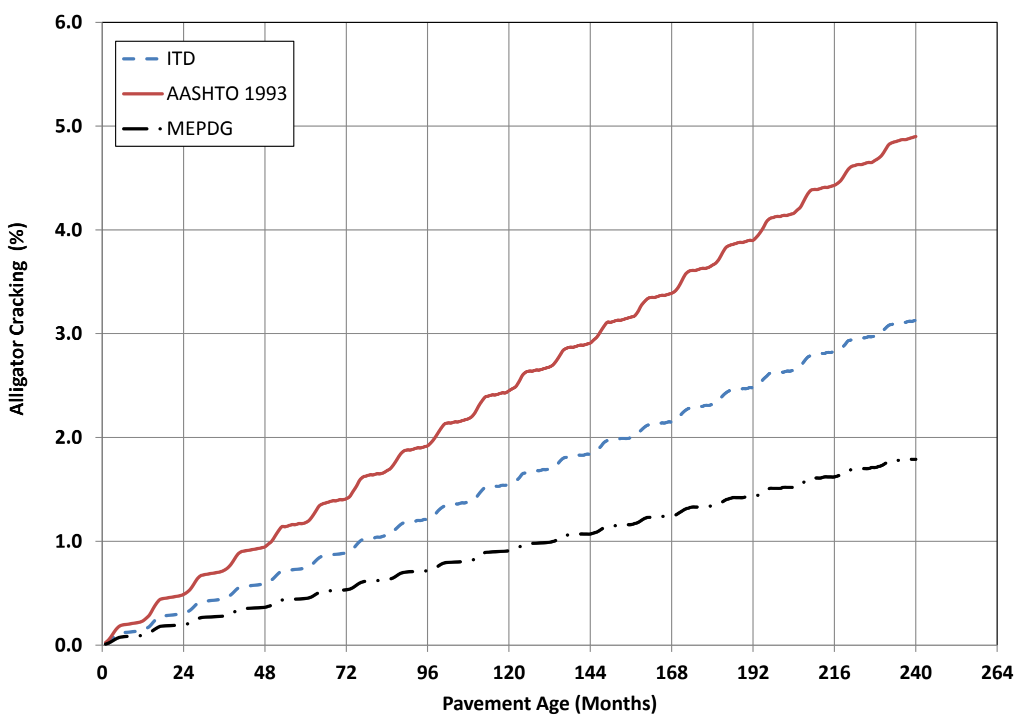


Figure 96. Comparison of the Predicted Alligator Cracking, US-2(c) Project

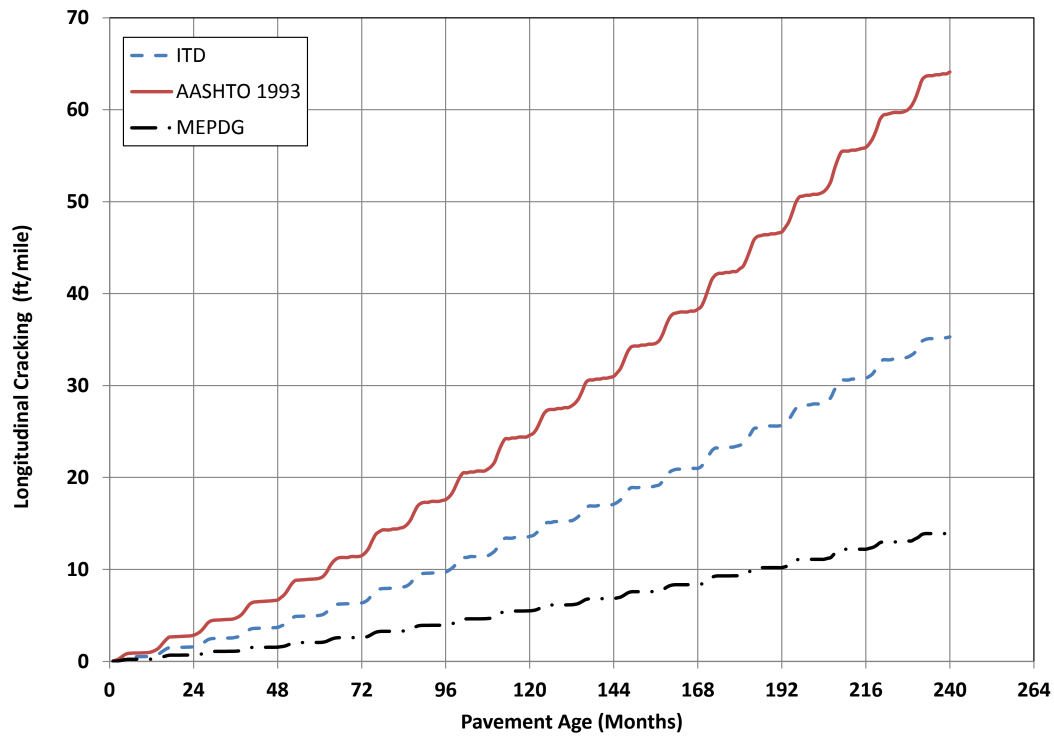


Figure 97. Comparison of the Predicted Longitudinal Cracking, US-2(c) Project

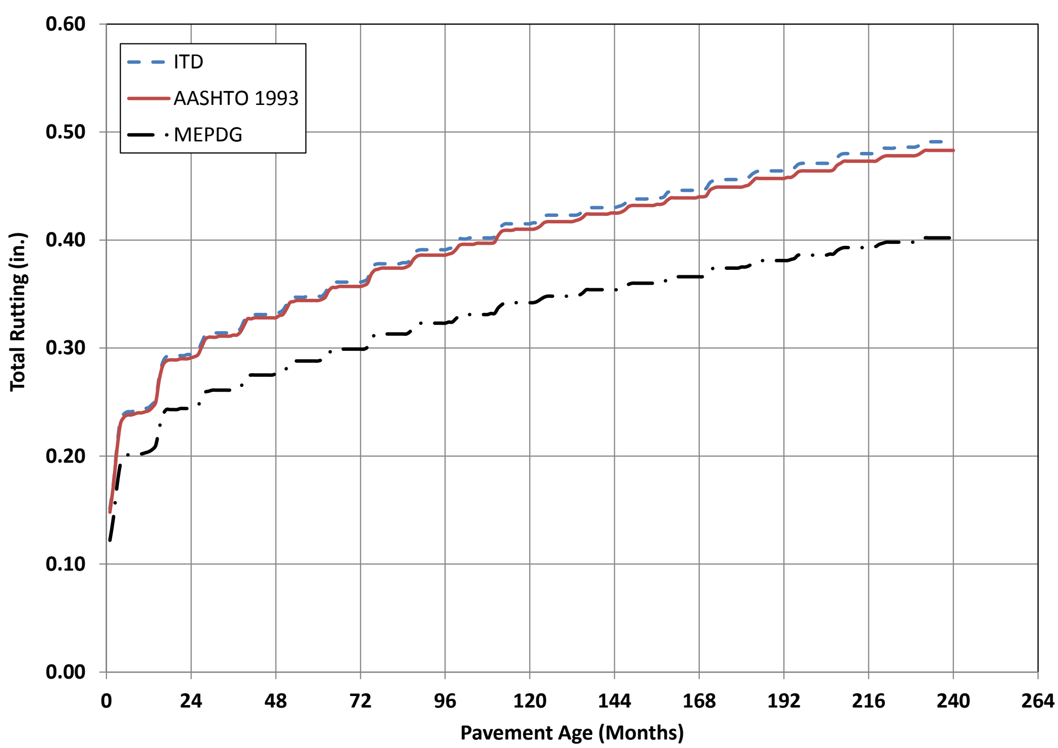


Figure 98. Comparison of the Predicted Total Rutting, US-2(c) Project

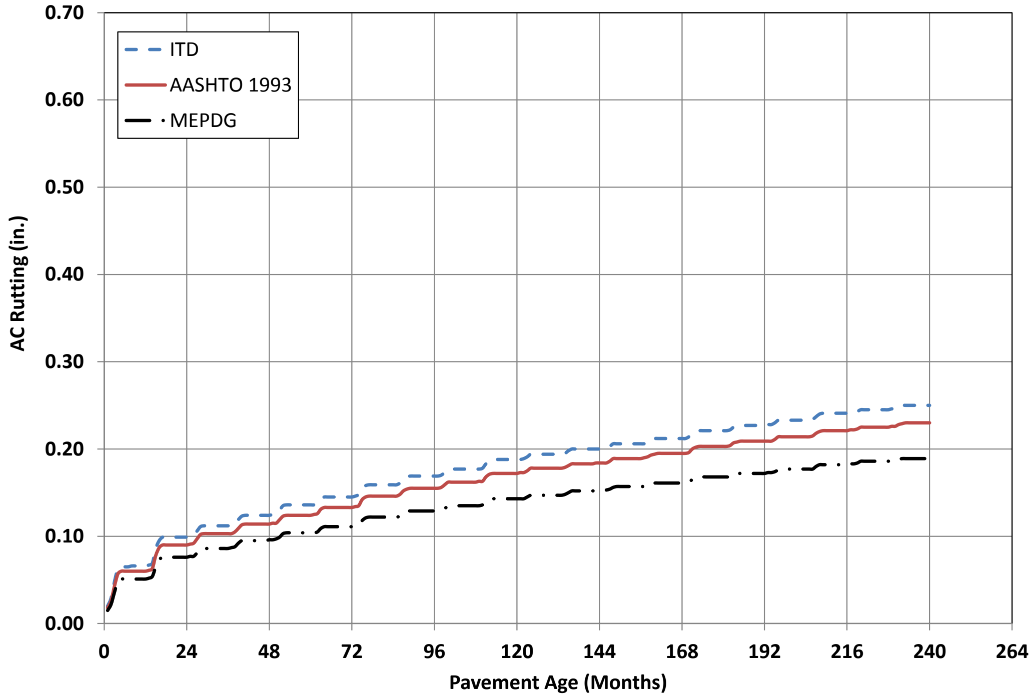


Figure 99. Comparison of the Predicted AC Layer Rutting, US-2(c) Project

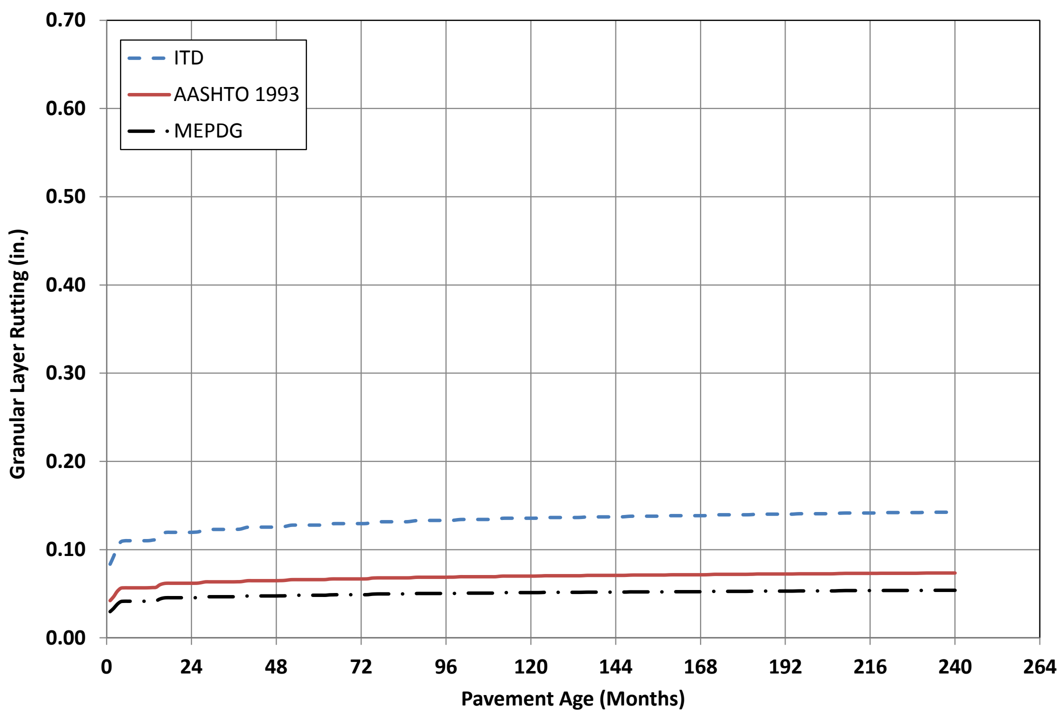


Figure 100. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-2(c) Project

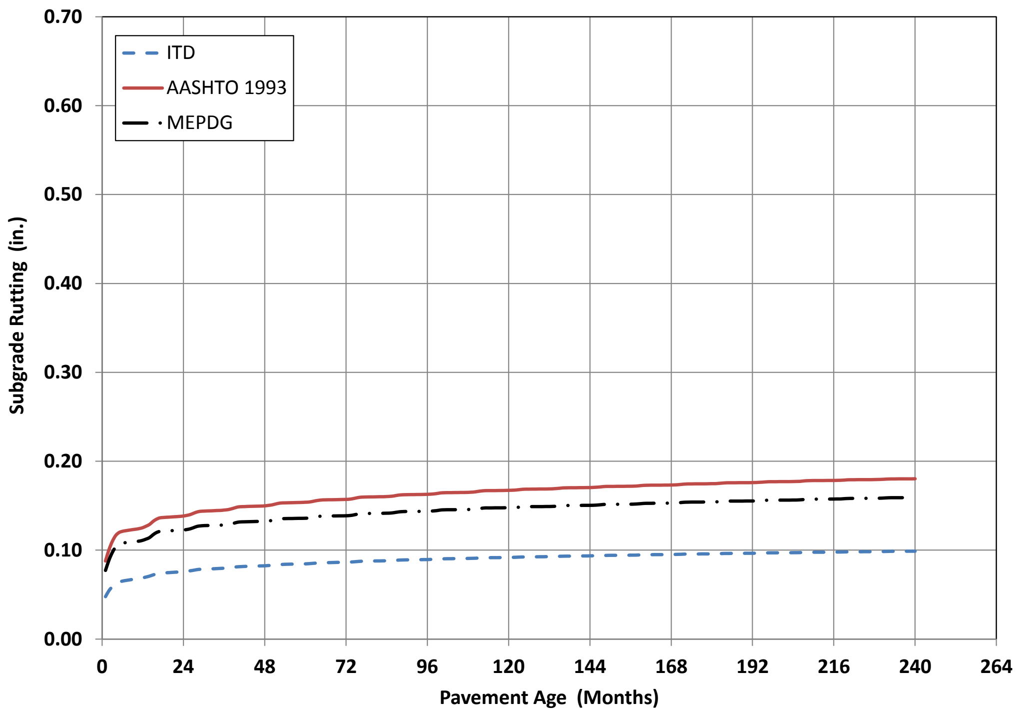


Figure 101. Comparison of the Predicted Subgrade Layer Rutting, US-2(c) Project

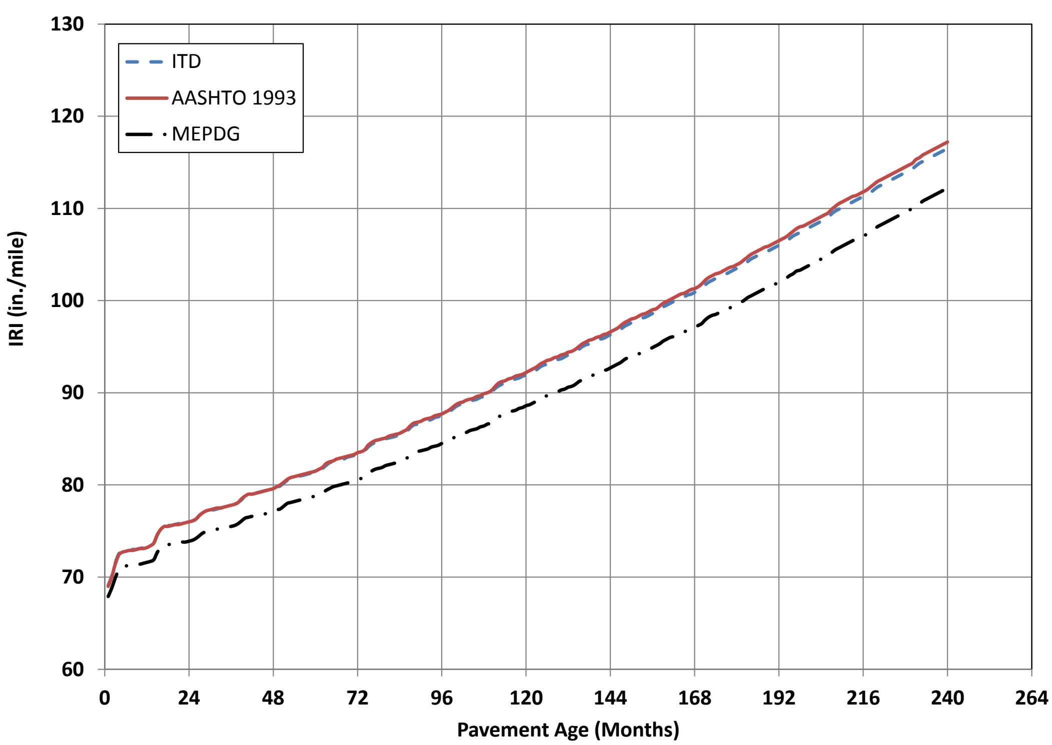


Figure 102. Comparison of the Predicted IRI, US-2(c) Project

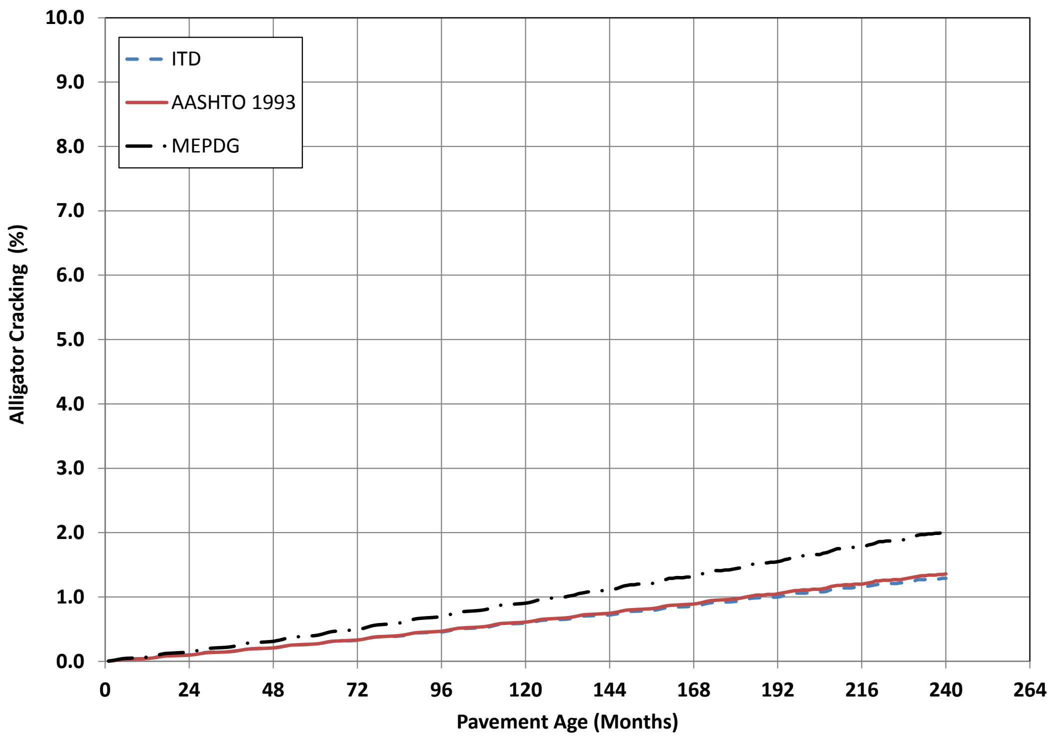


Figure 103. Comparison of the Predicted Alligator Cracking, SH-62 Project

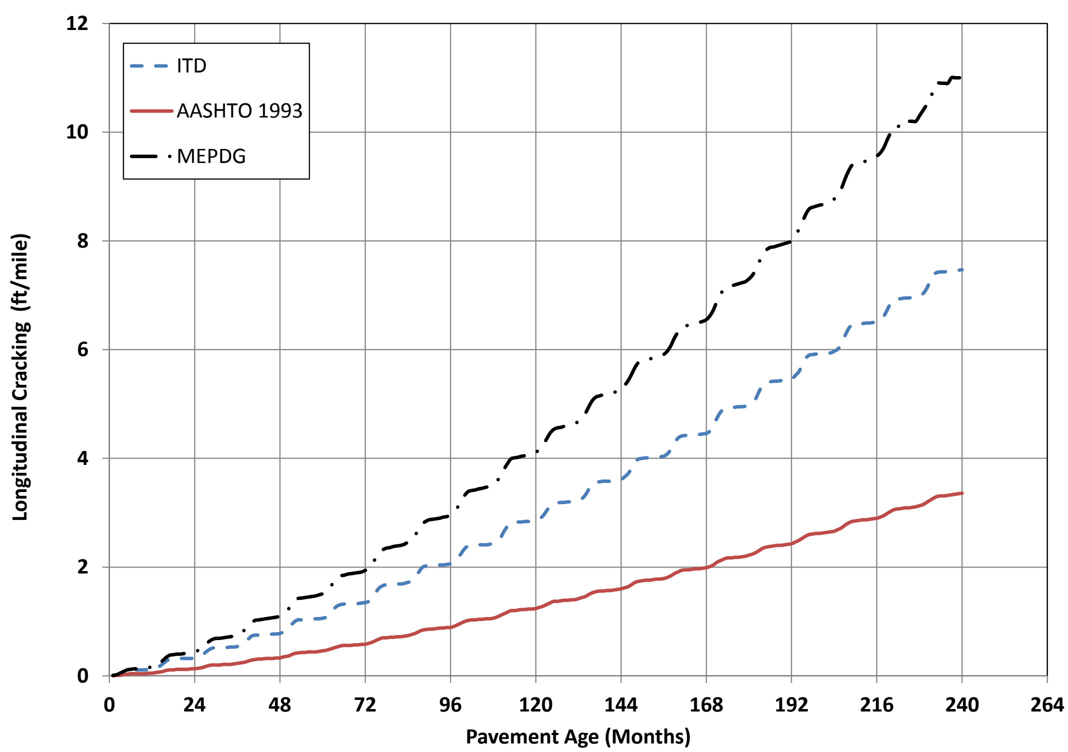


Figure 104. Comparison of the Predicted Longitudinal Cracking, SH-62 Project

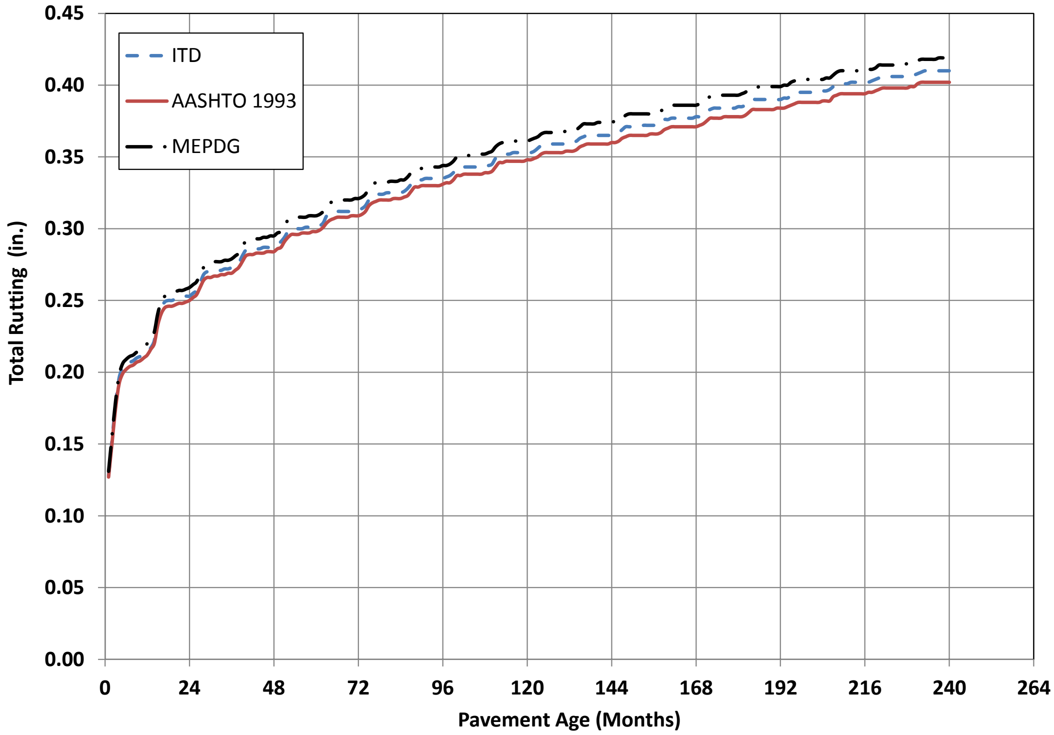


Figure 105. Comparison of the Predicted Total Rutting, SH-62 Project

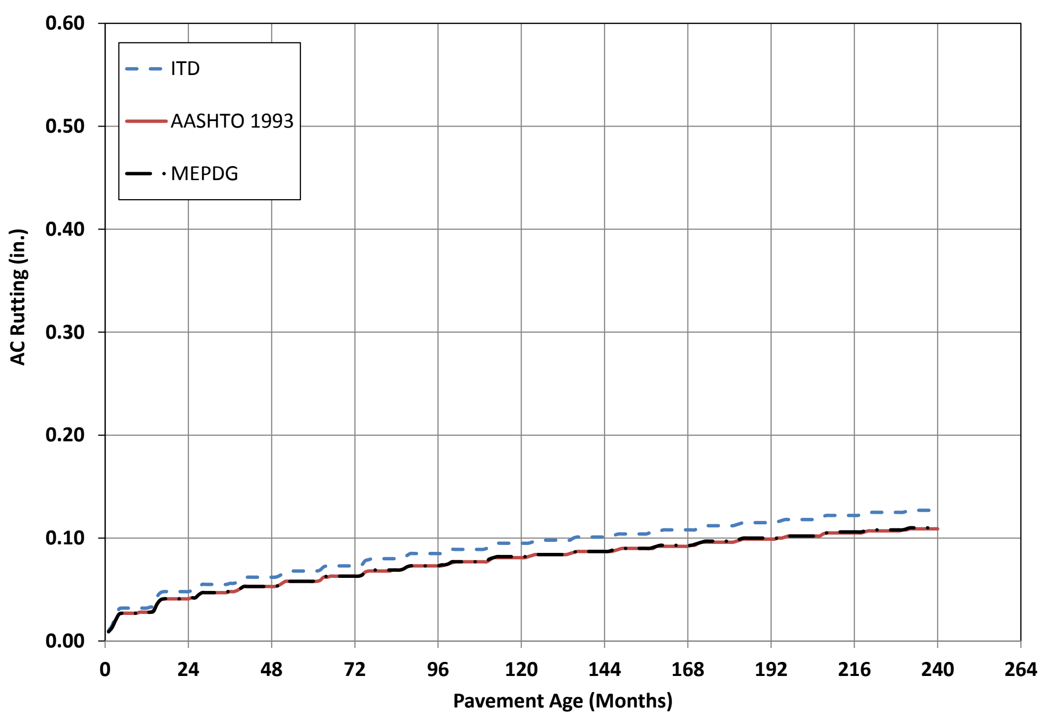


Figure 106. Comparison of the Predicted AC Layer Rutting, SH-62 Project

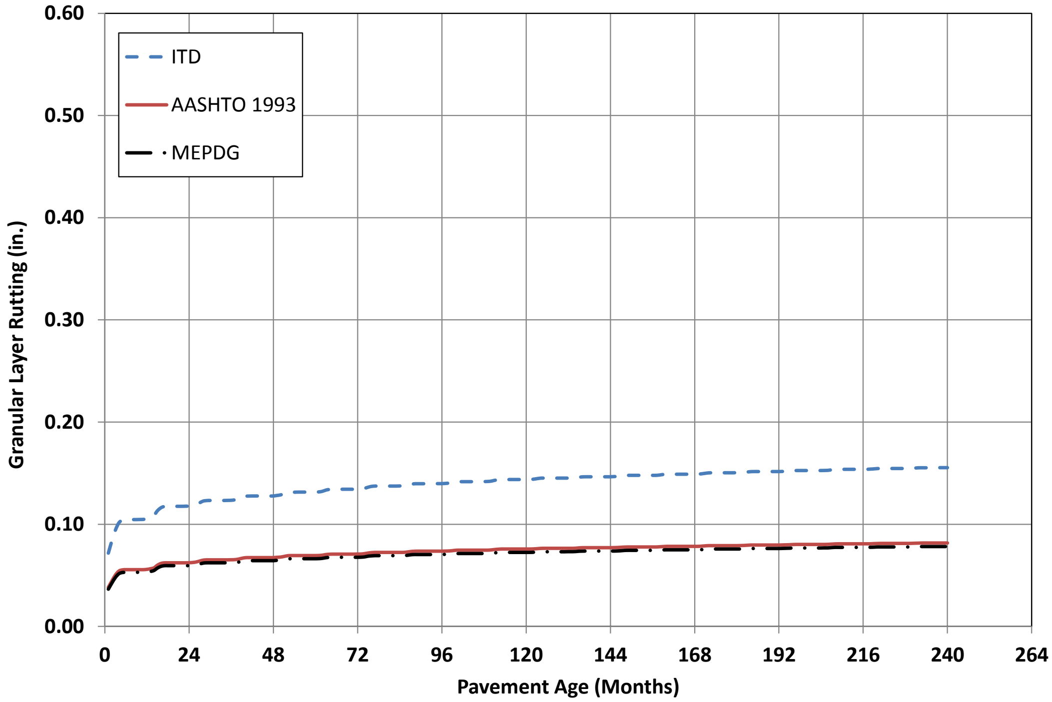


Figure 107. Comparison of the Predicted Unbound Granular Layer(s) Rutting, SH-62 Project

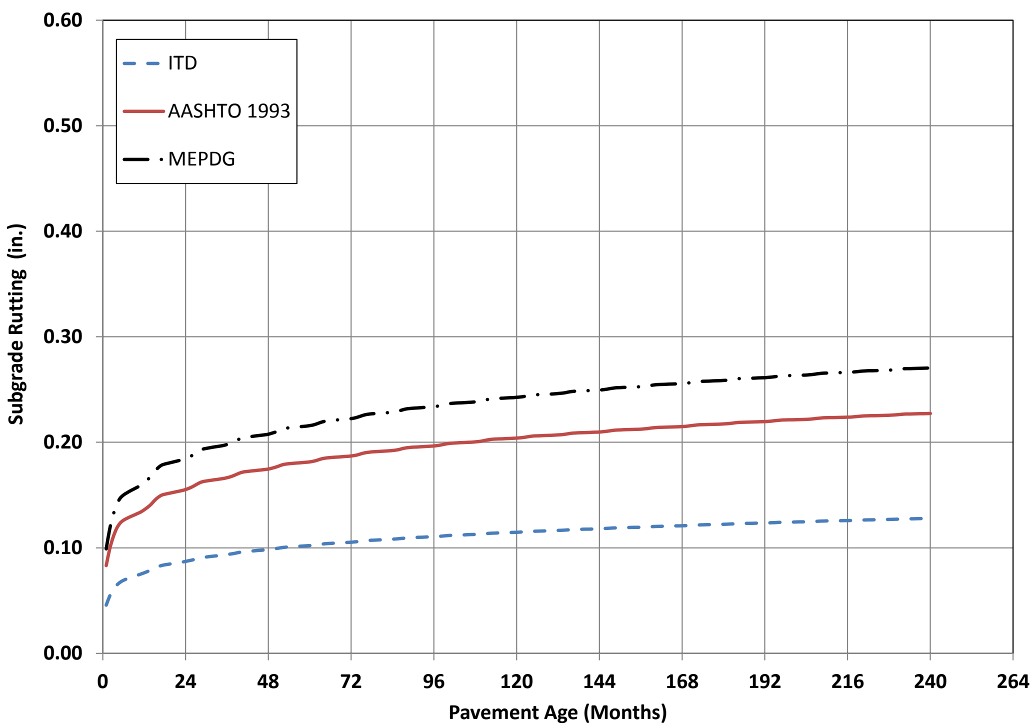


Figure 108. Comparison of the Predicted Subgrade Layer Rutting, SH-62 Project

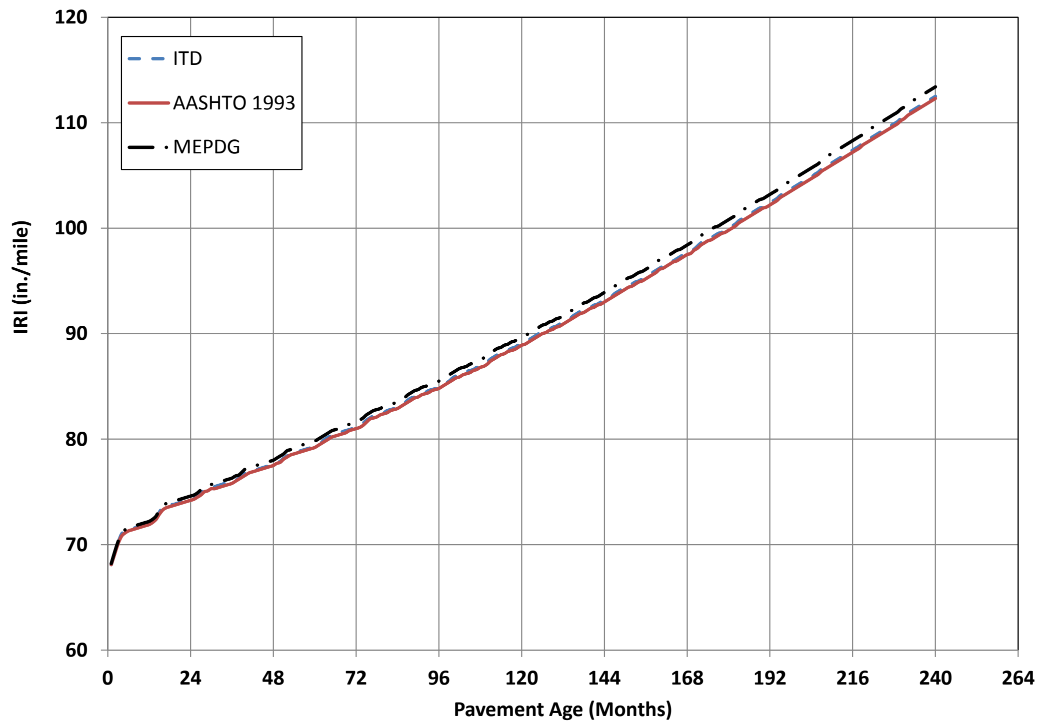


Figure 109. Comparison of the Predicted IRI, SH-62 Project

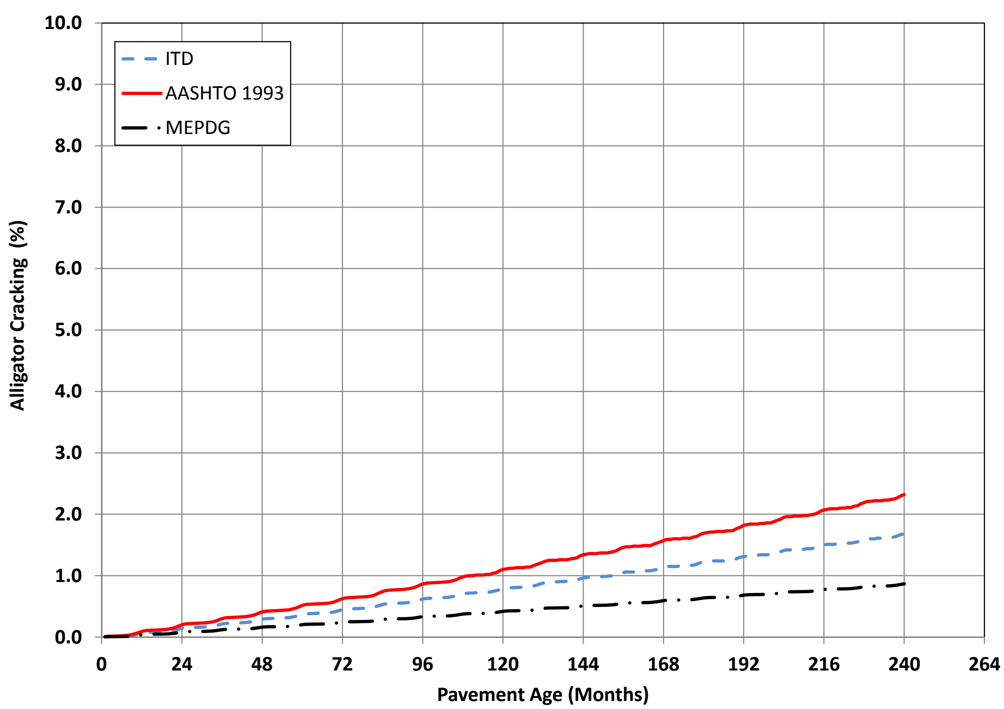


Figure 110. Comparison of the Predicted Alligator Cracking, SH-3 Project

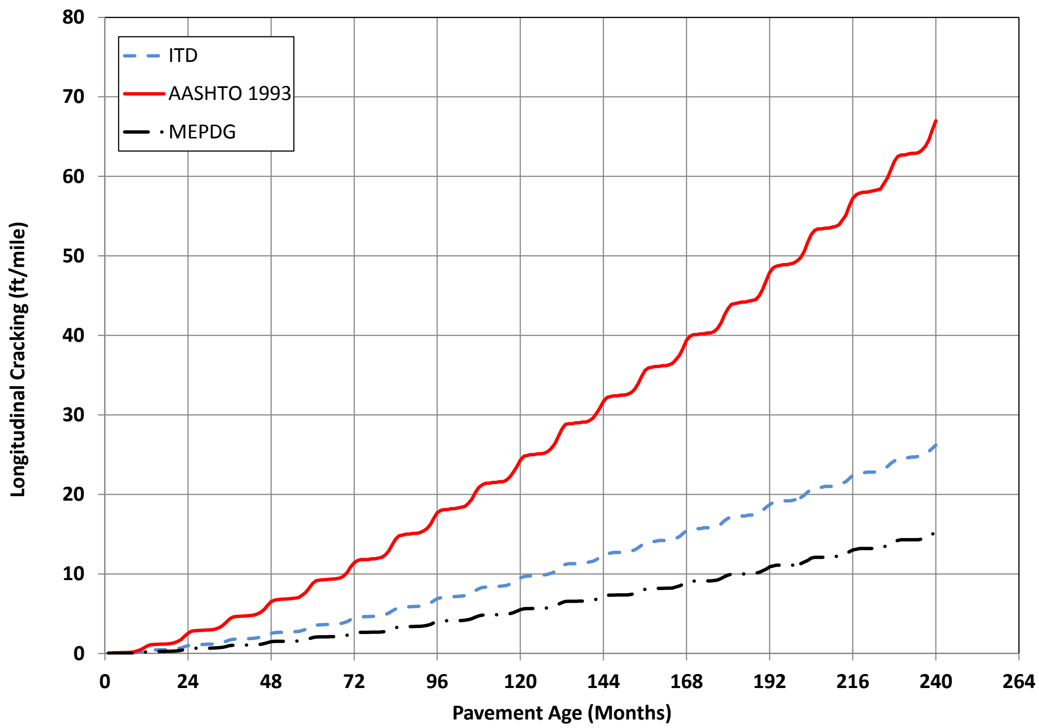


Figure 111. Comparison of the Predicted Longitudinal Cracking, SH-3 Project

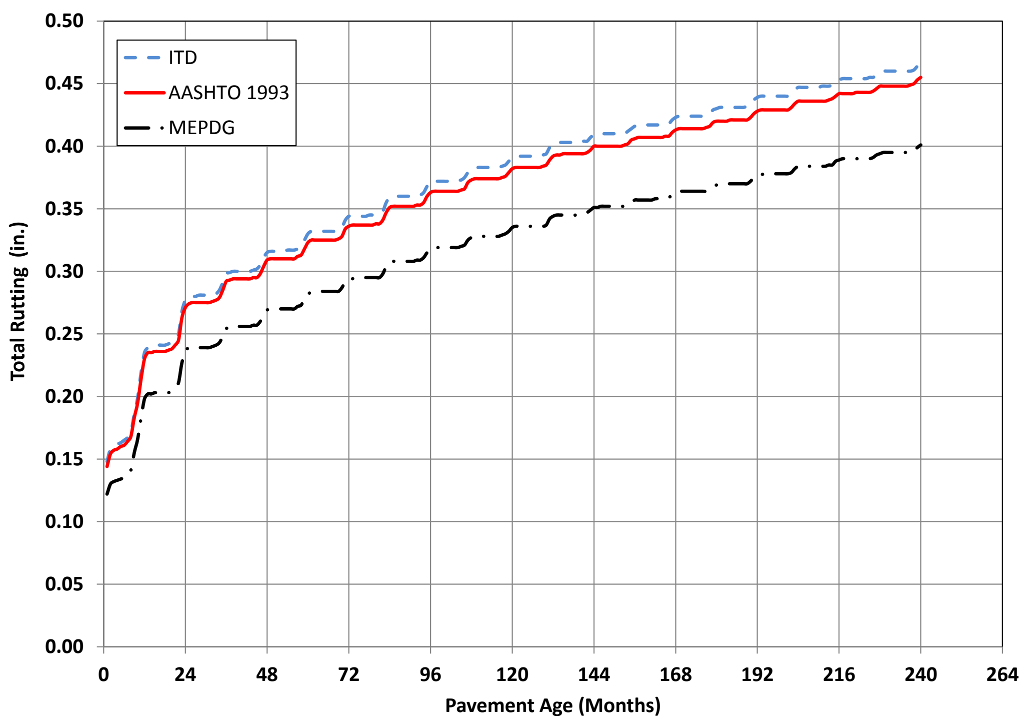


Figure 112. Comparison of the Predicted Total Rutting, SH-3 Project

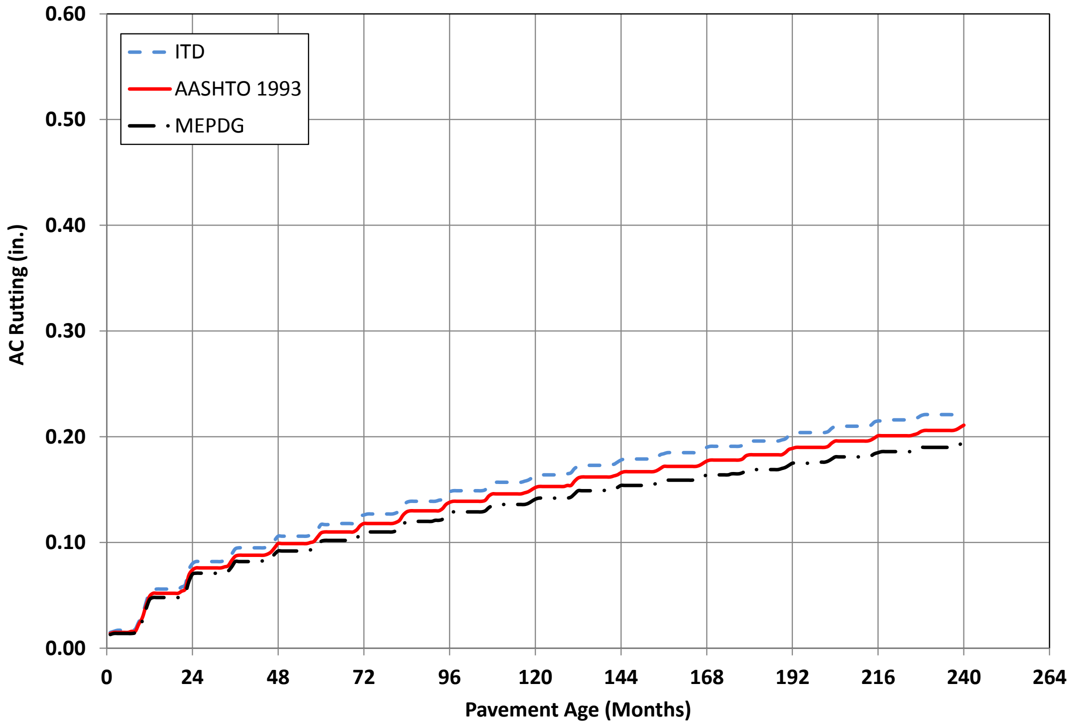


Figure 113. Comparison of the Predicted AC Layer Rutting, SH-3 Project

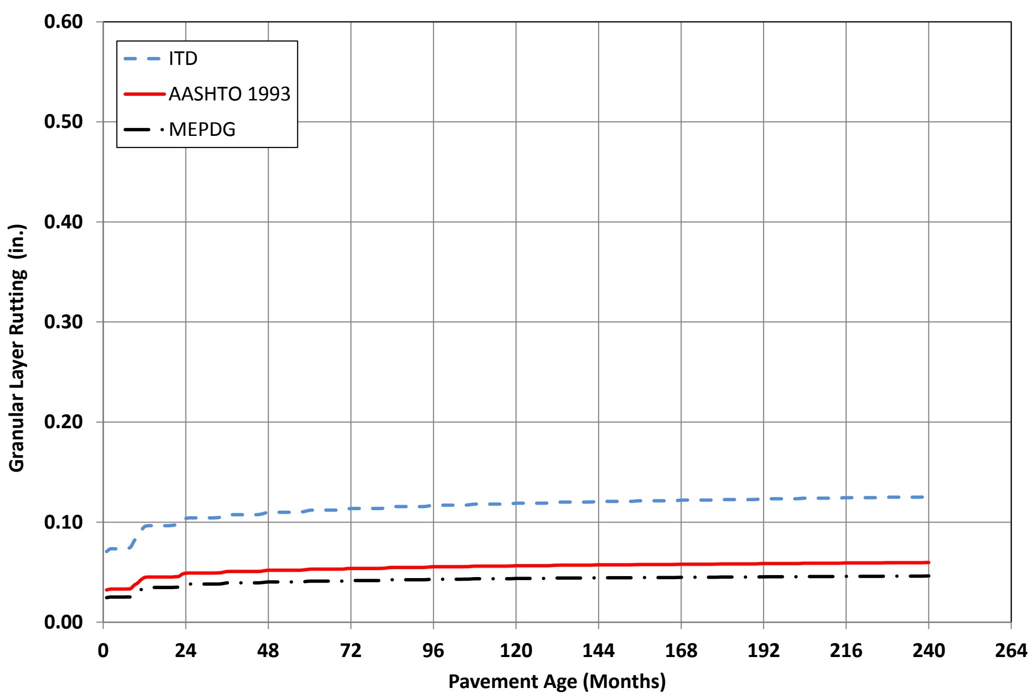


Figure 114. Comparison of the Predicted Unbound Granular Layer(s) Rutting, SH-3 Project

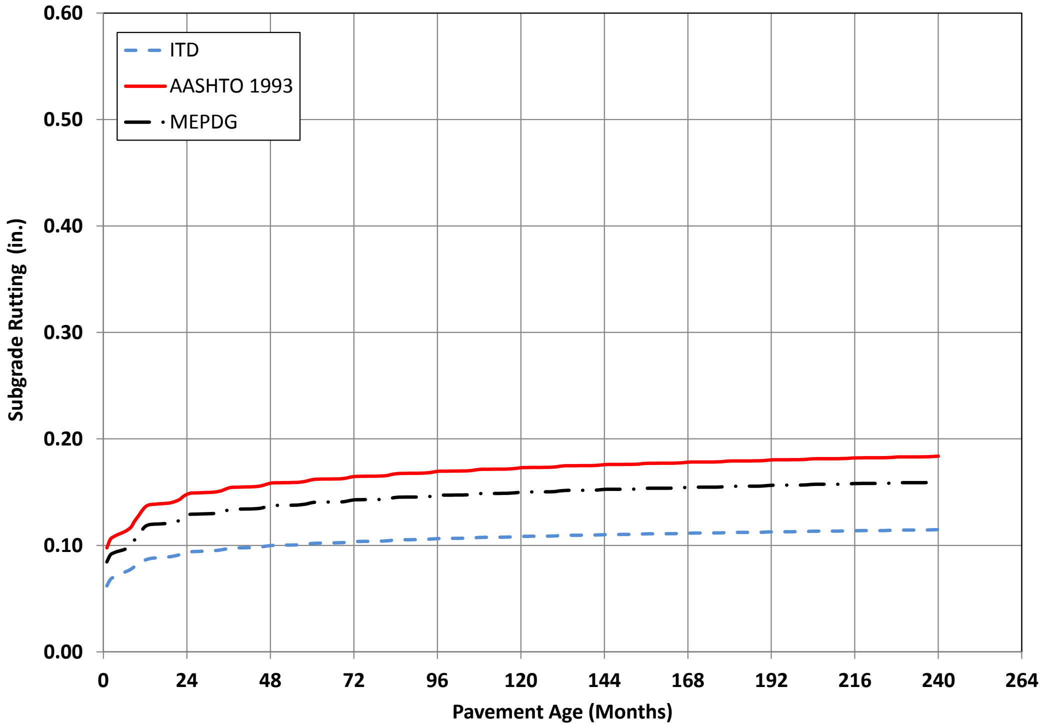


Figure 115. Comparison of the Predicted Subgrade Layer Rutting, SH-3 Project

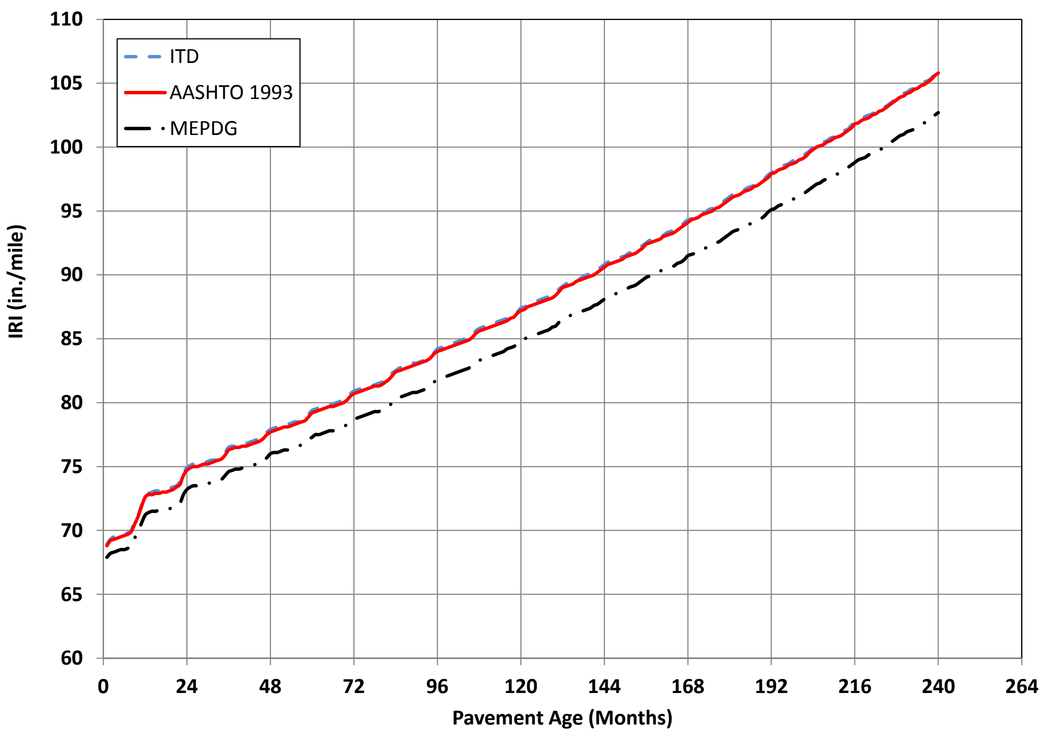


Figure 116. Comparison of the Predicted IRI, SH-3 Project

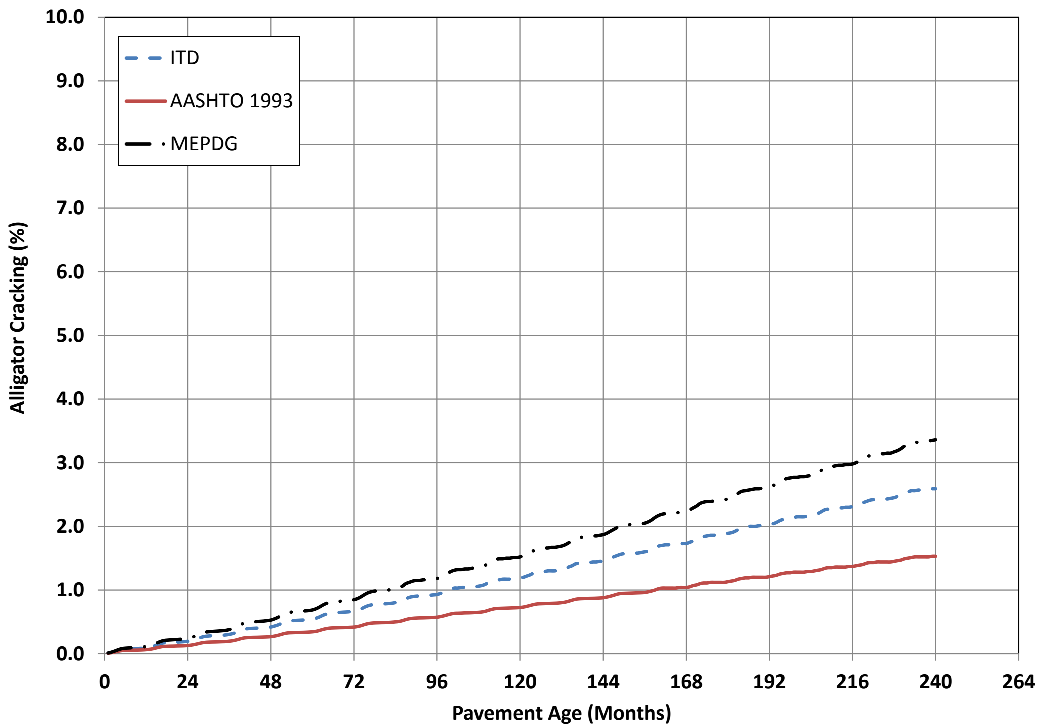


Figure 117. Comparison of the Predicted Alligator Cracking, SH-19 Project

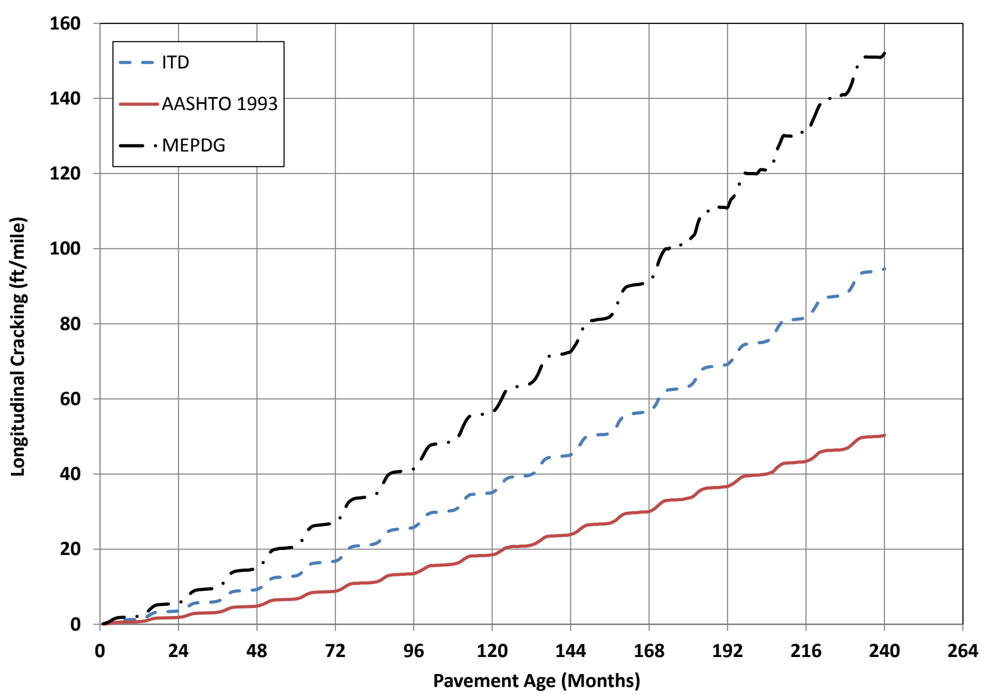


Figure 118. Comparison of the Predicted Longitudinal Cracking, SH-19 Project

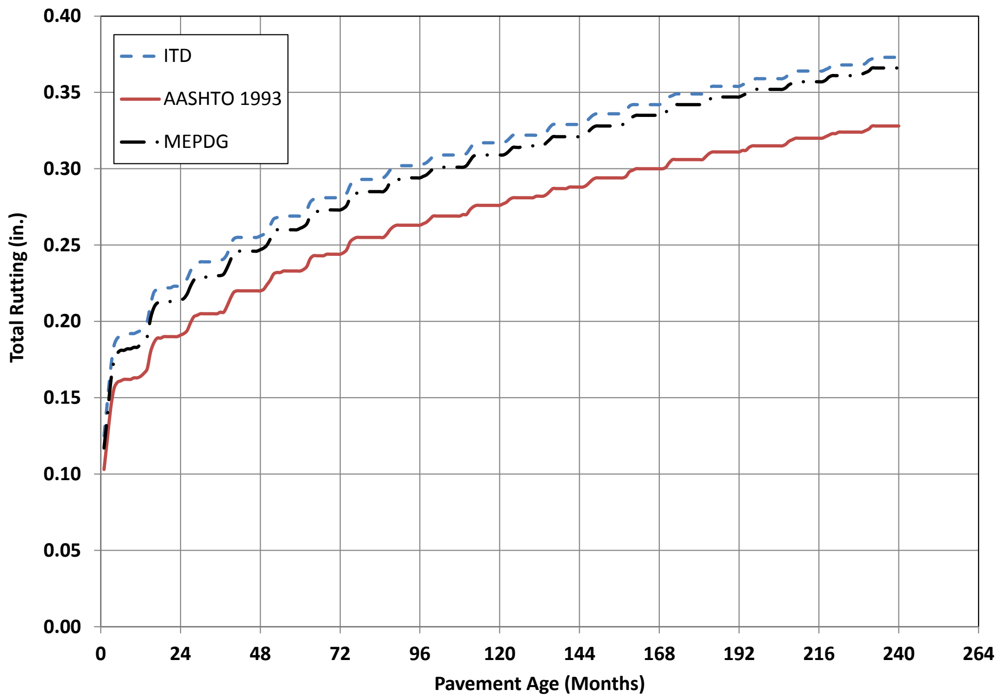


Figure 119. Comparison of the Predicted Total Rutting, SH-19 Project

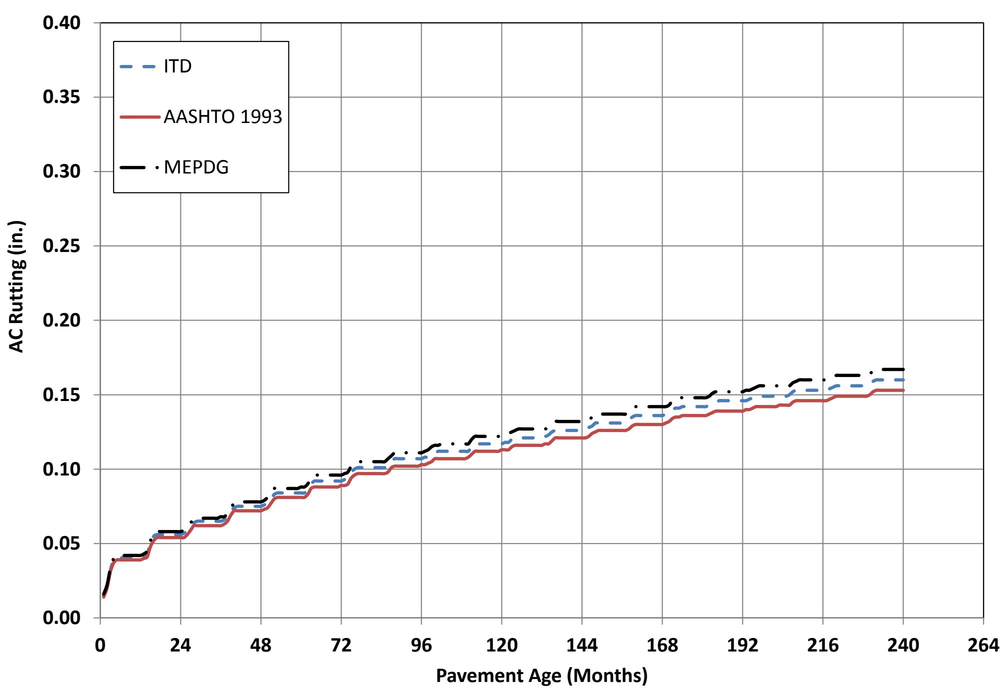


Figure 120. Comparison of the Predicted AC Layer Rutting, SH-19 Project

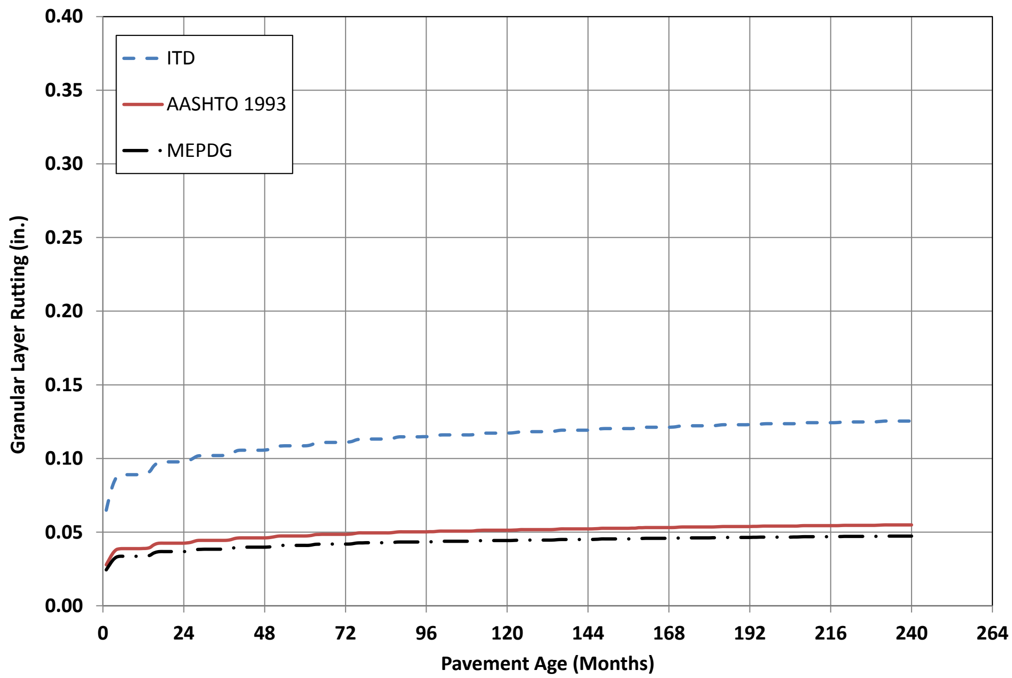


Figure 121. Comparison of the Predicted Unbound Granular Layer(s) Rutting, SH-19 Project

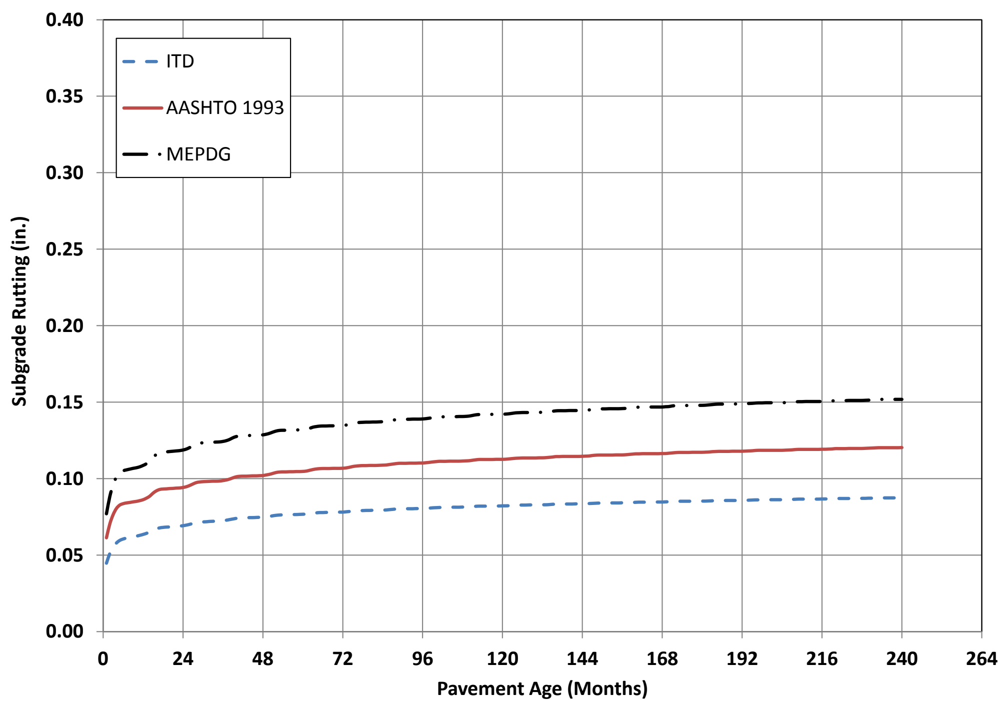


Figure 122. Comparison of the Predicted Subgrade Layer Rutting, SH-19 Project

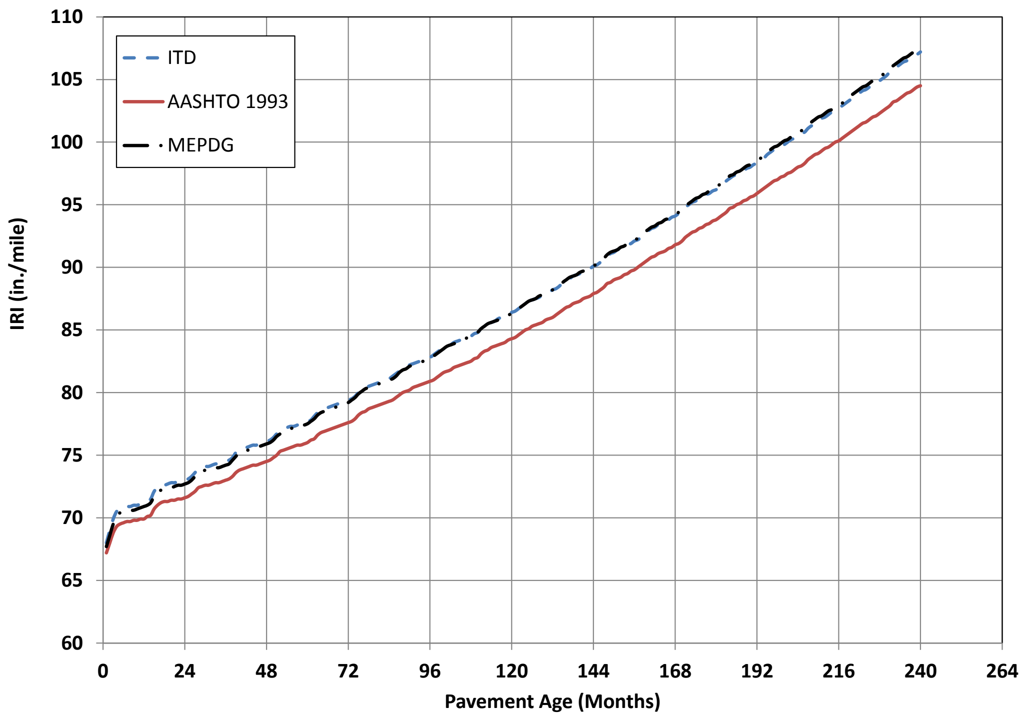


Figure 123. Comparison of the Predicted IRI, SH-19 Project

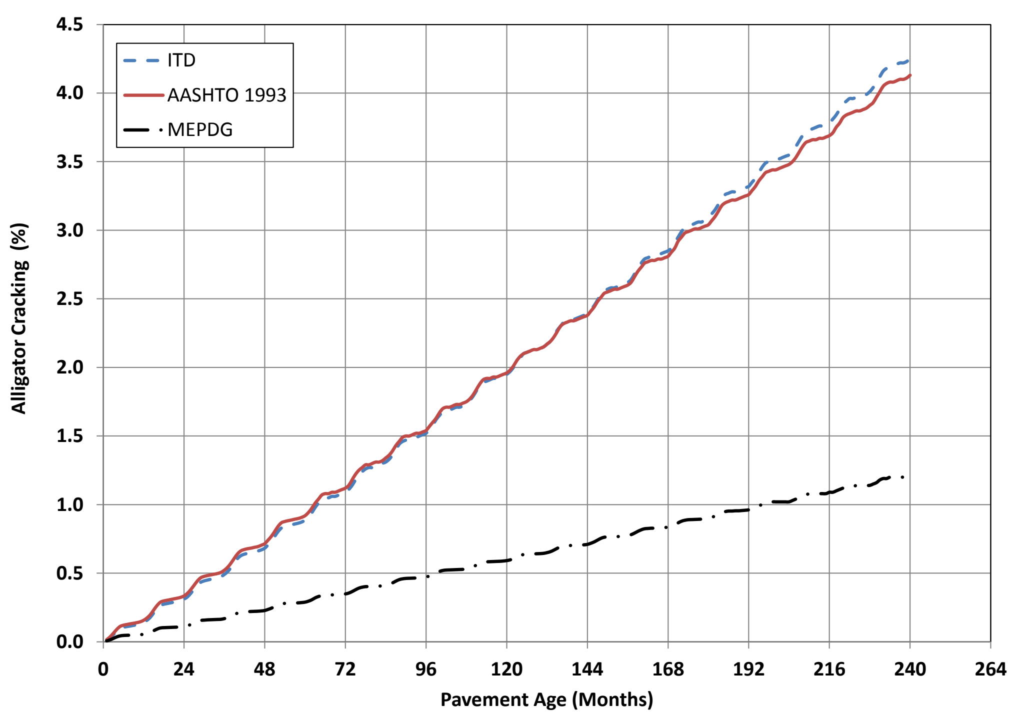


Figure 124. Comparison of the Predicted Alligator Cracking, US-95 Project

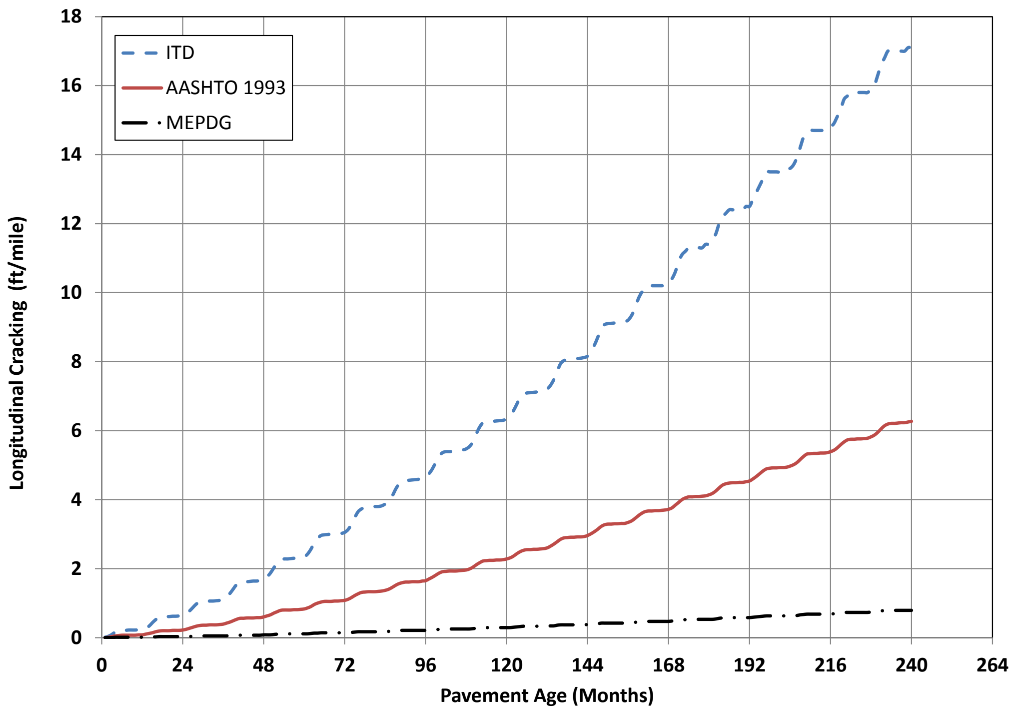


Figure 125. Comparison of the Predicted Longitudinal Cracking, US-95 Project

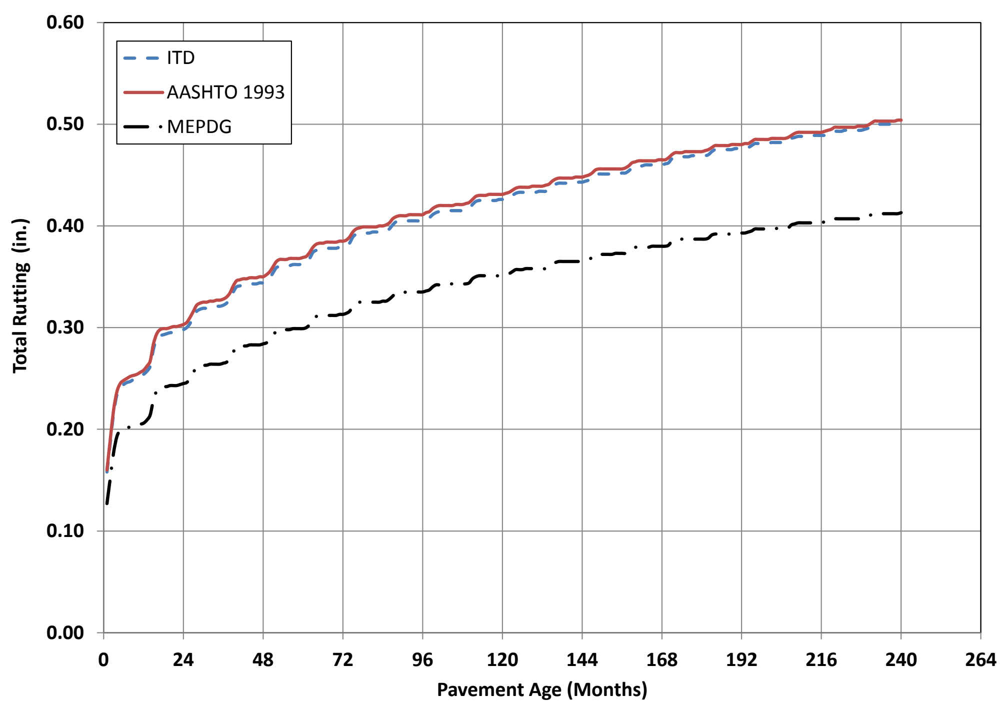


Figure 126. Comparison of the Predicted Total Rutting, US-95 Project

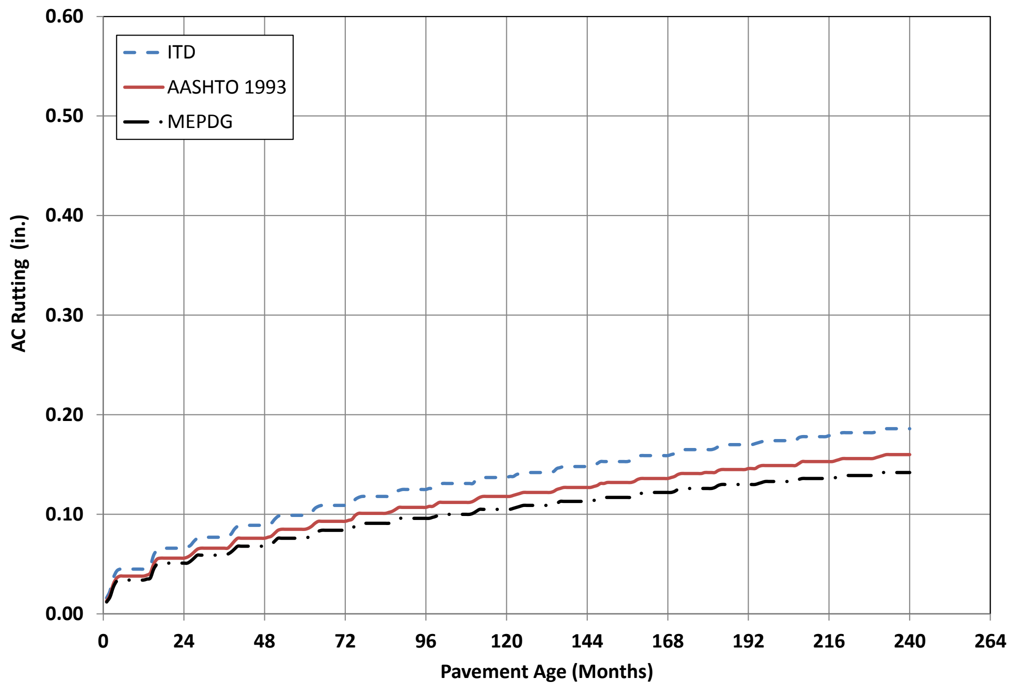


Figure 127. Comparison of the Predicted AC Layer Rutting, US-95 Project

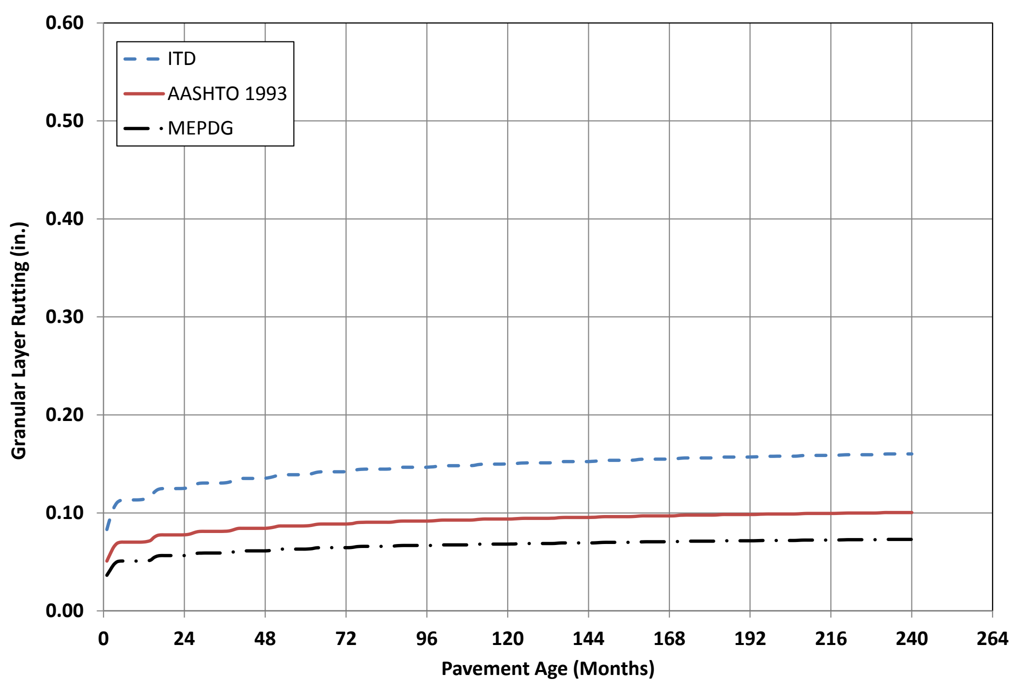


Figure 128. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-95 Project

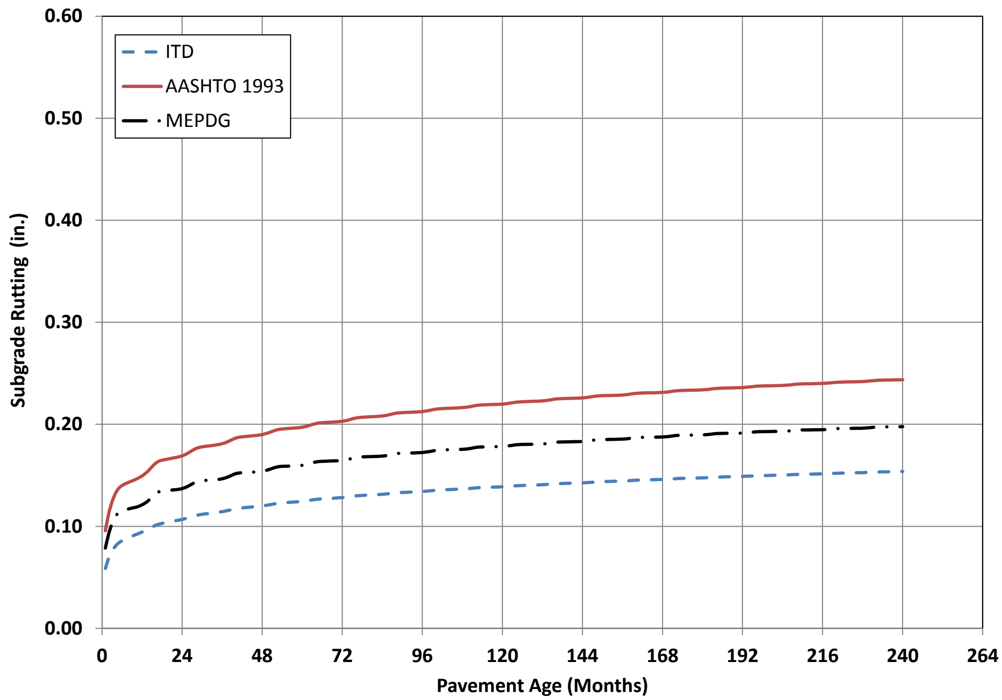


Figure 129. Comparison of the Predicted Subgrade Layer Rutting, US-95 Project

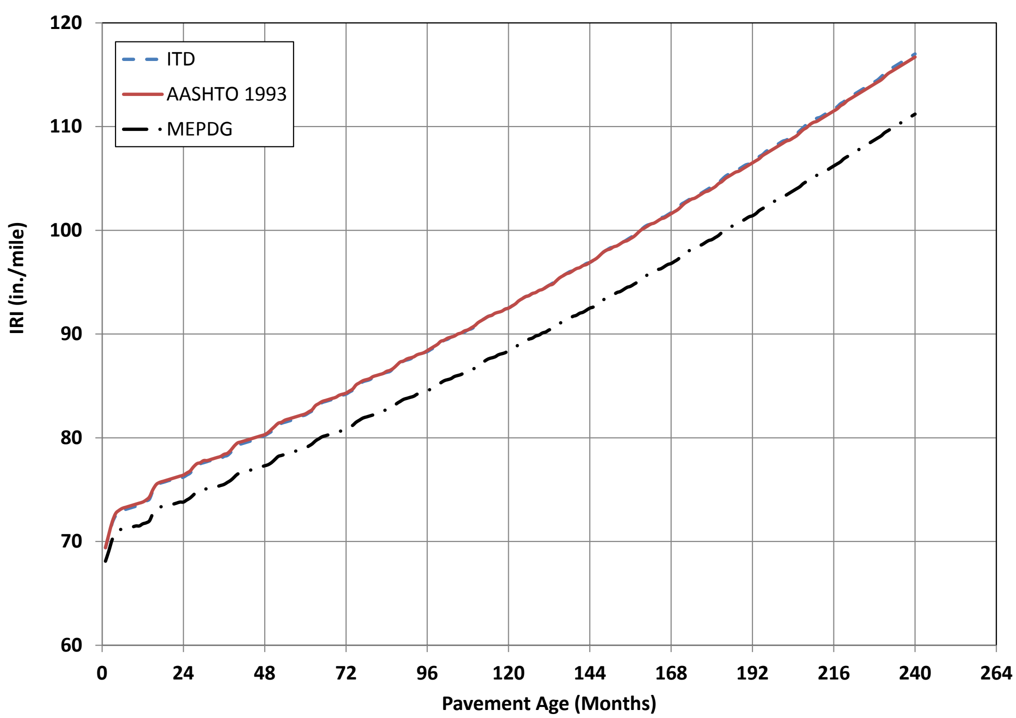


Figure 130. Comparison of the Predicted IRI, US-95 Project

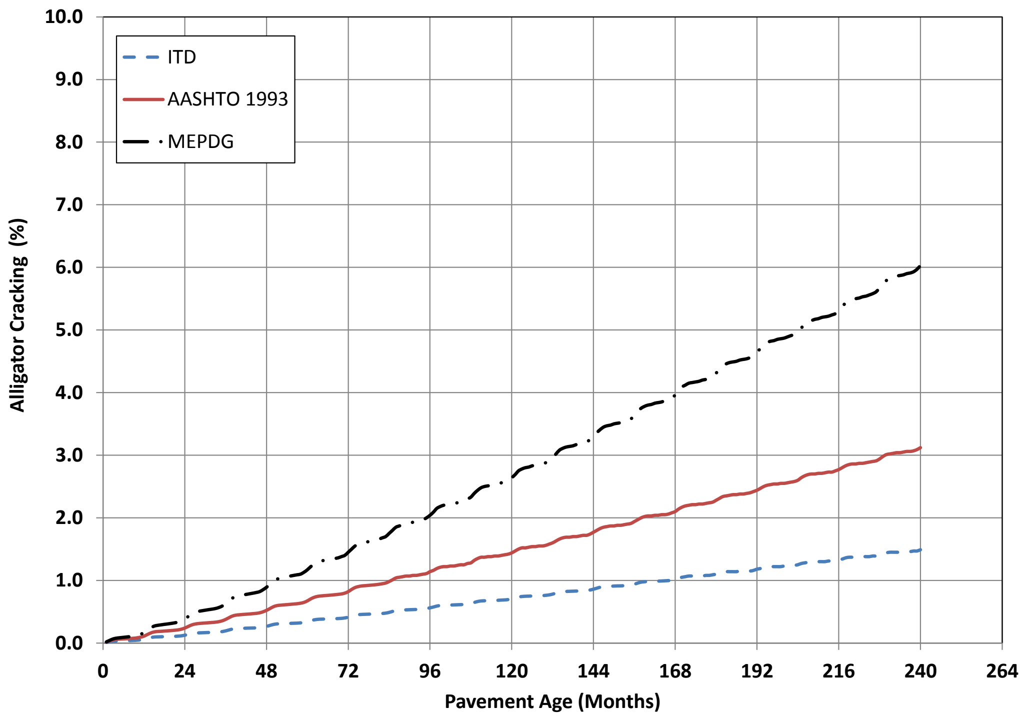


Figure 131. Comparison of the Predicted Alligator Cracking, US-93 Project

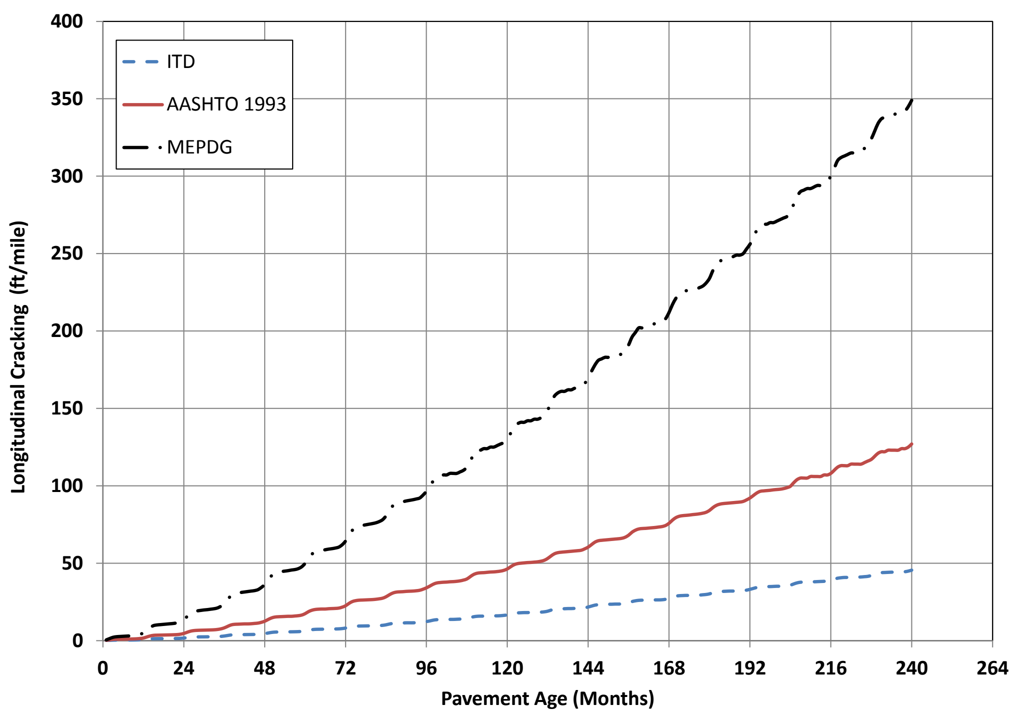


Figure 132. Comparison of the Predicted Longitudinal Cracking, US-93 Project

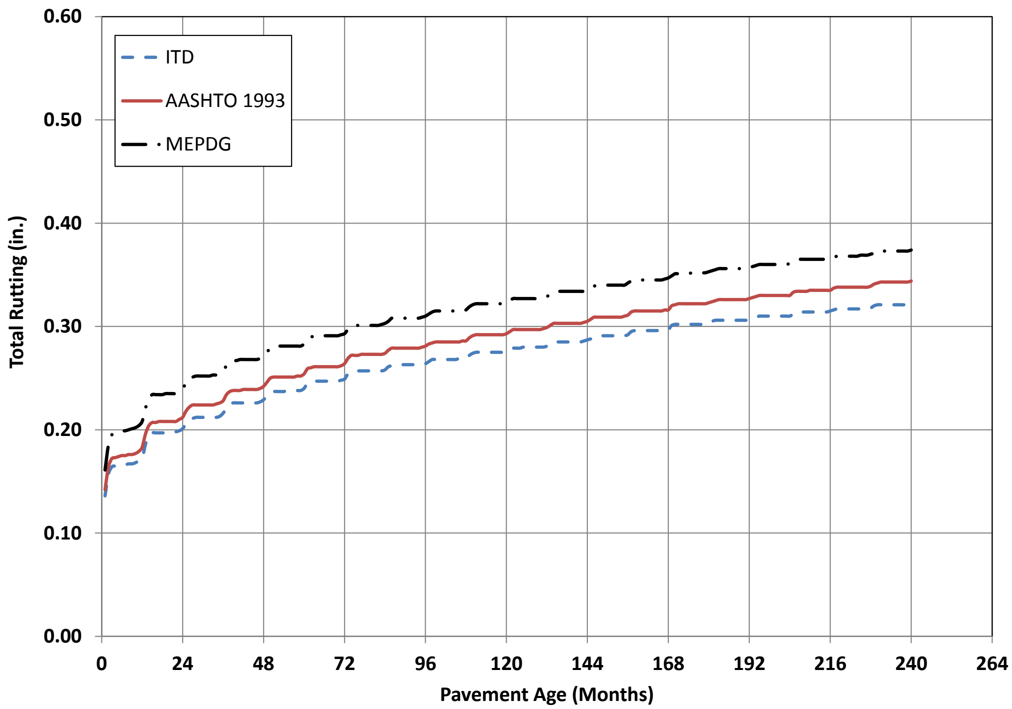


Figure 133. Comparison of the Predicted Total Rutting, US-93 Project

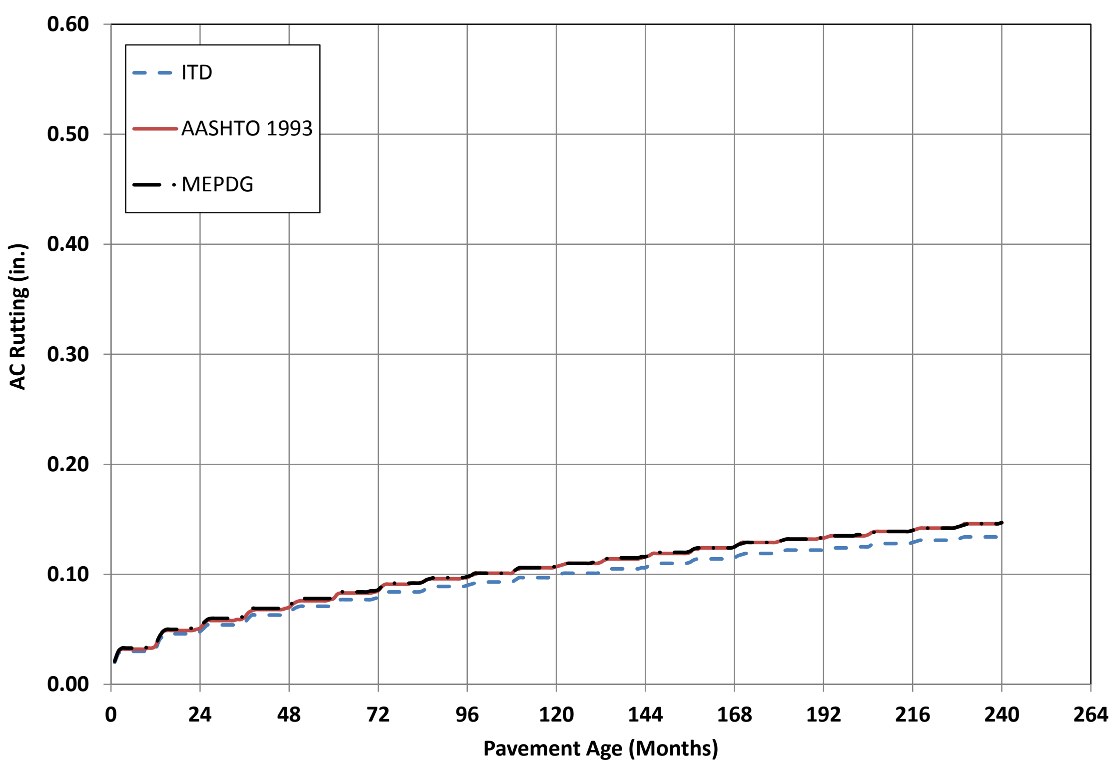


Figure 134. Comparison of the Predicted AC Layer Rutting, US-93 Project

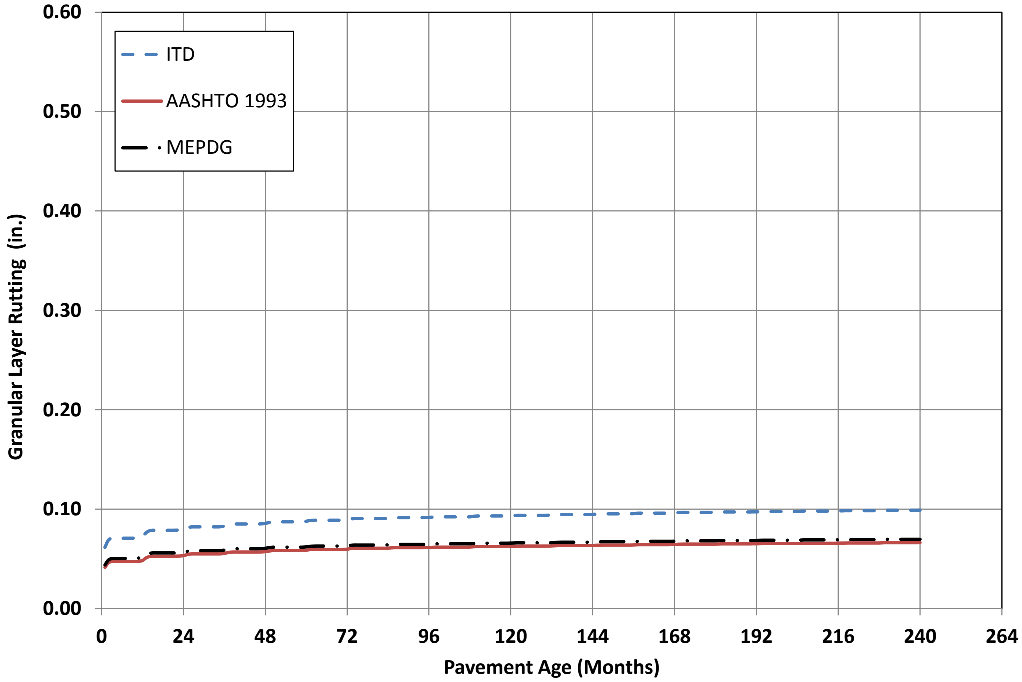


Figure 135. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-93 Project

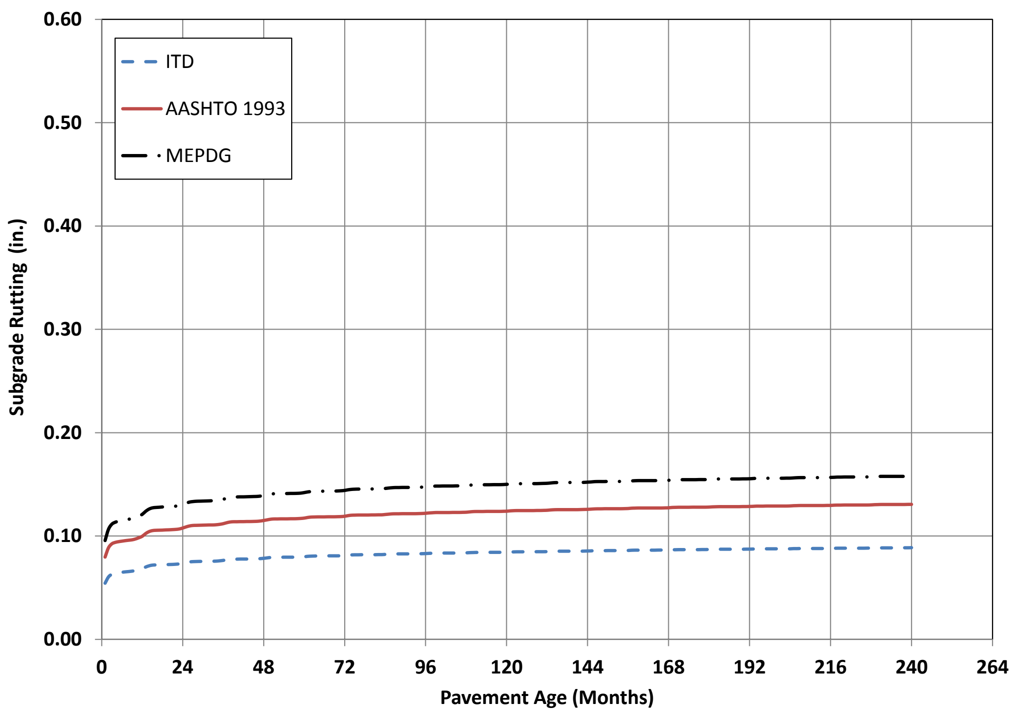


Figure 136. Comparison of the Predicted Subgrade Layer Rutting, US-93 Project

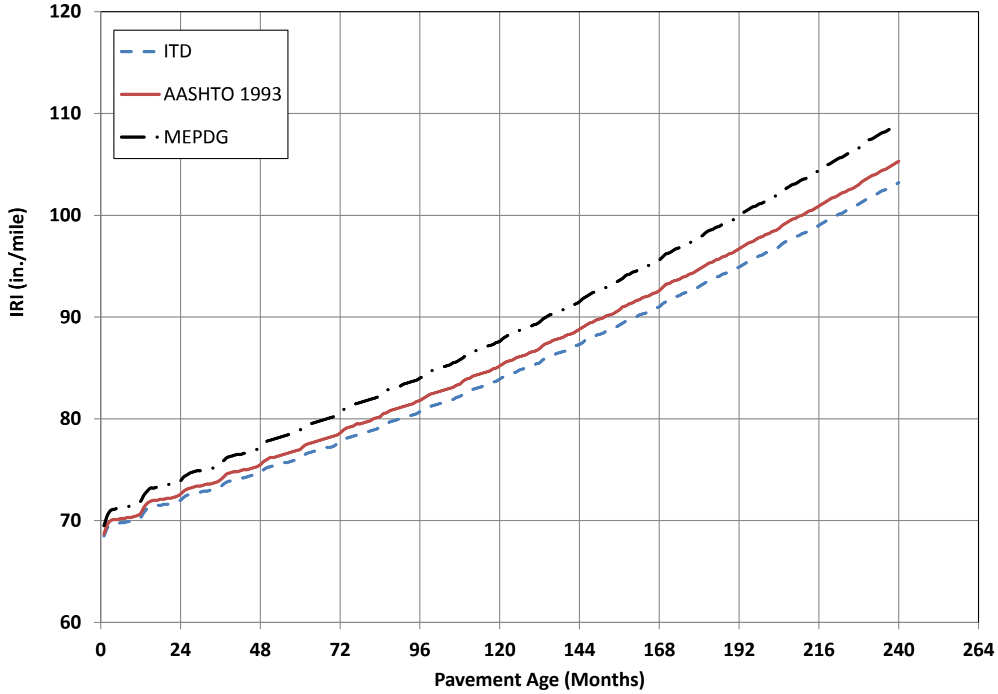


Figure 137. Comparison of the Predicted IRI, US-93 Project

Appendix C
Comparison of MEPDG Predicted Distresses and Smoothness at 85 Percent Reliability

The figures presented in this appendix show a comparison of MEPDG predicted distresses and smoothness for structures designed using MEPDG and AASHTO 1993 at 85 percent reliability compared to structures designed using ITD design method which does not incorporate reliability.

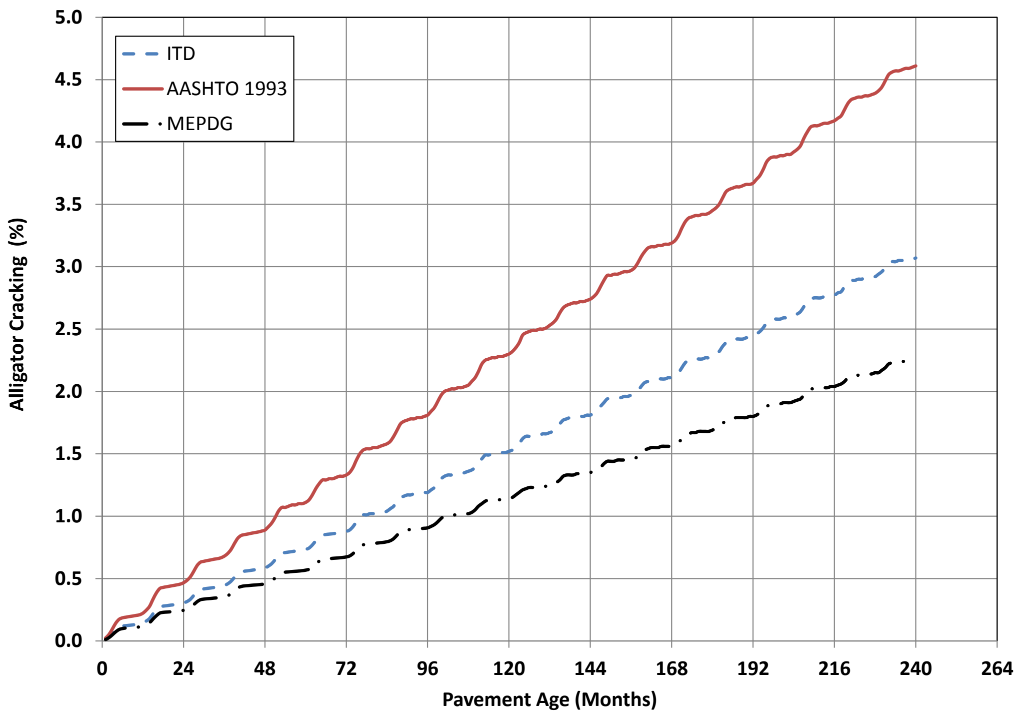


Figure 138. Comparison of the Predicted Alligator Cracking, US-2(a) Project

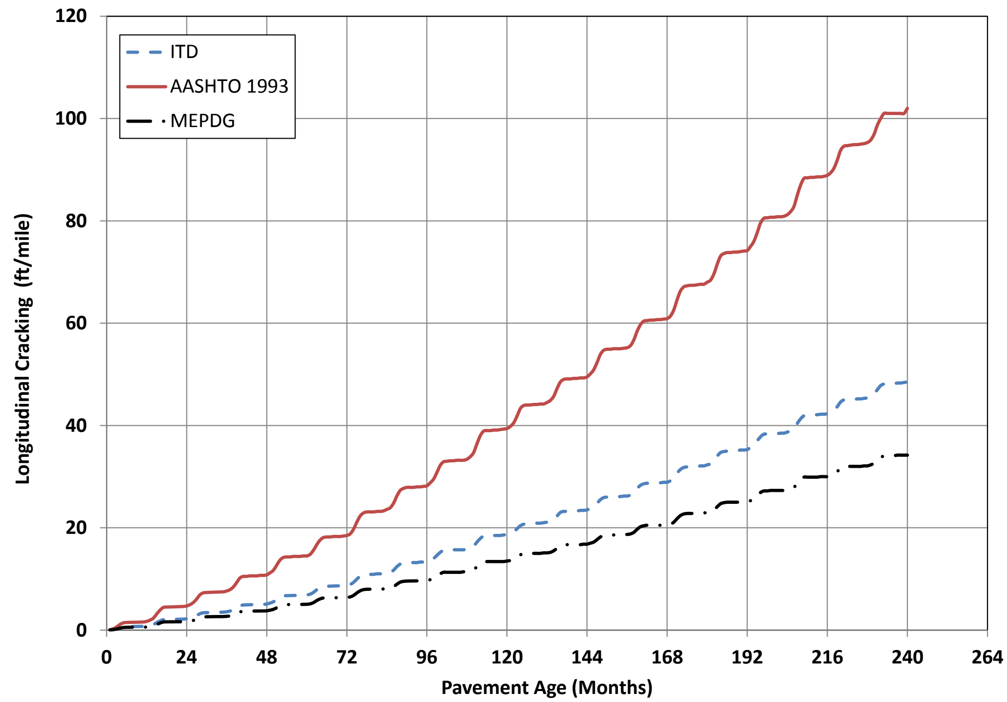


Figure 139. Comparison of the Predicted Longitudinal Cracking, US-2(a) Project

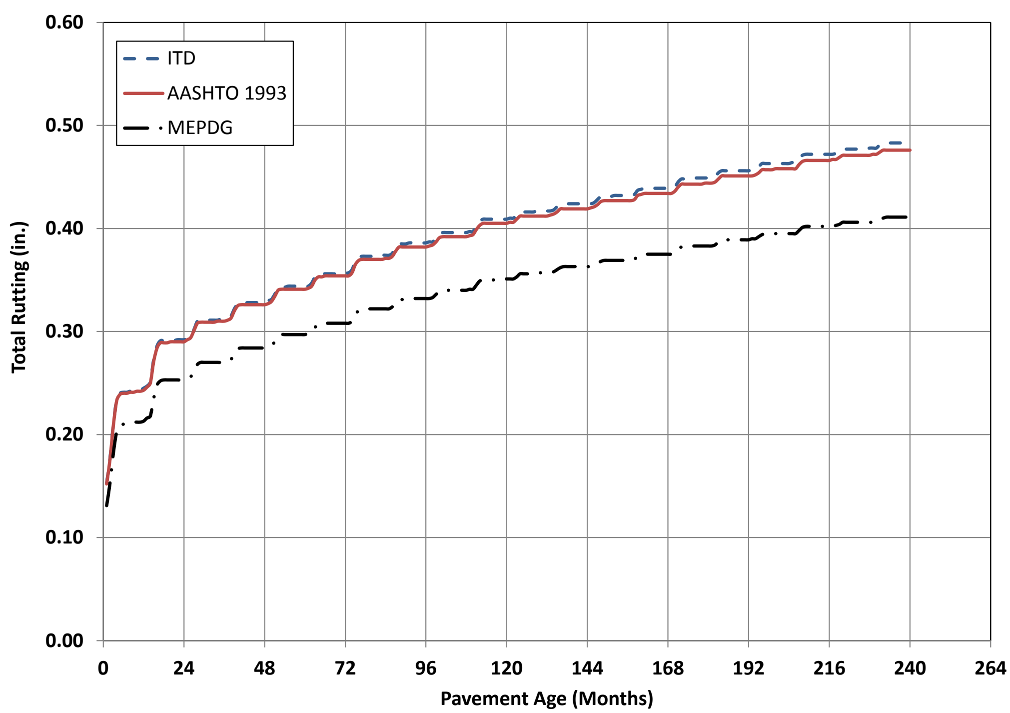


Figure 140. Comparison of the Predicted Total Rutting, US-2(a) Project

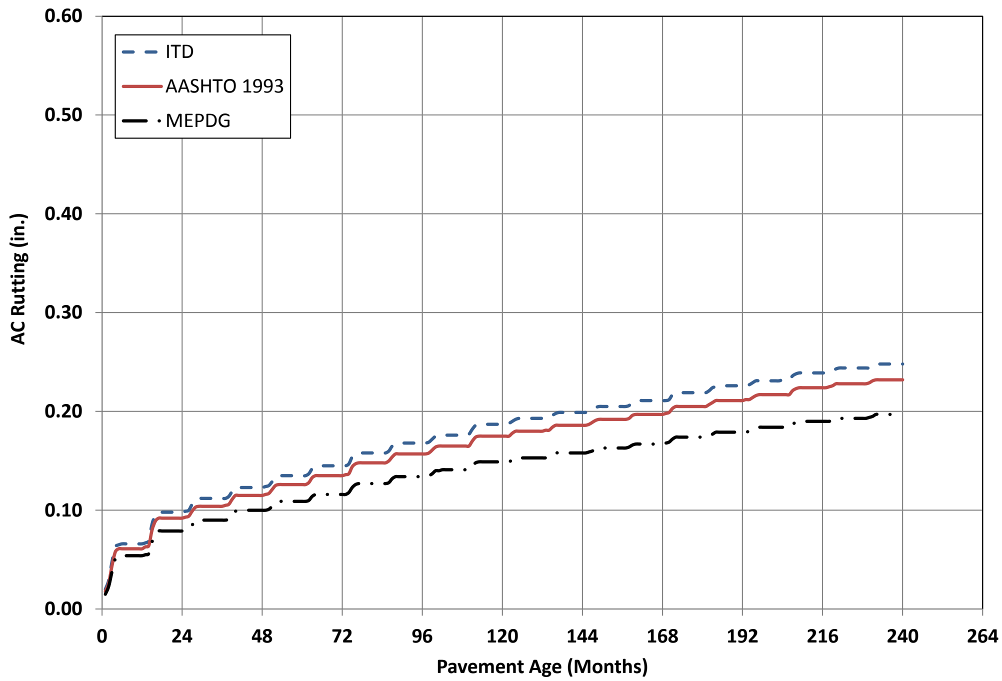


Figure 141. Comparison of the Predicted AC Layer Rutting, US-2(a) Project

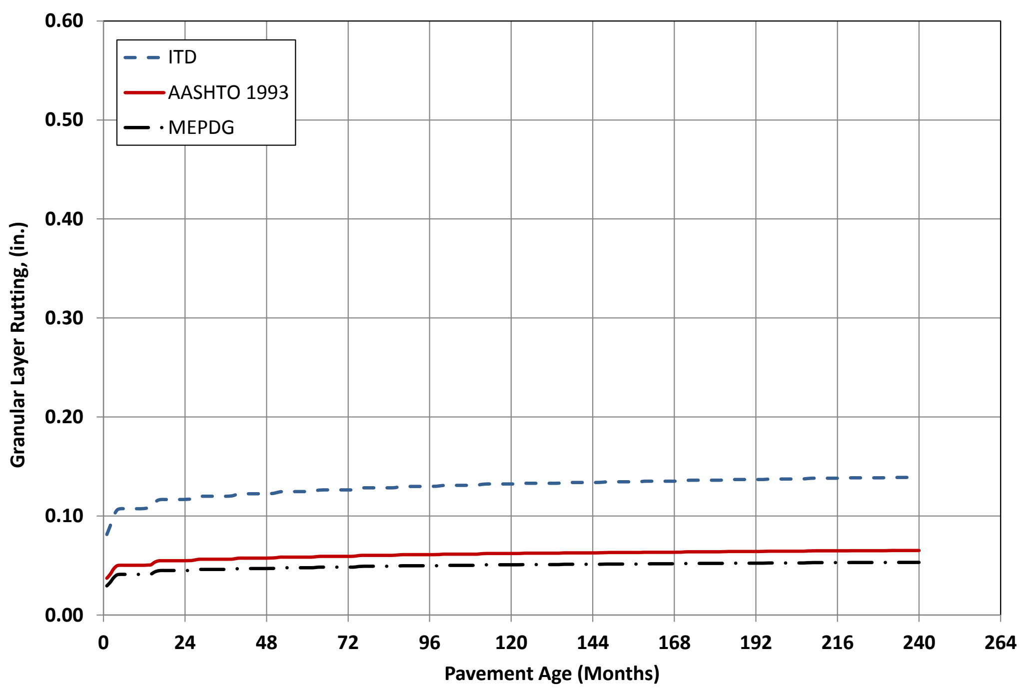


Figure 142. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-2(a) Project

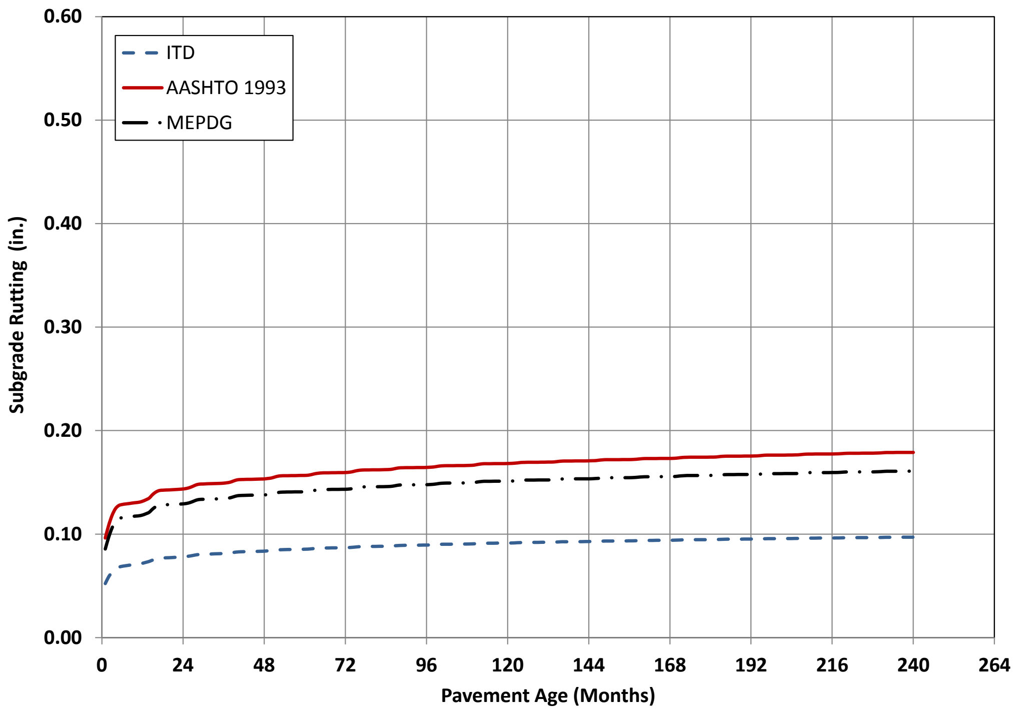


Figure 143. Comparison of the Predicted Subgrade Layer Rutting, US-2(a) Project

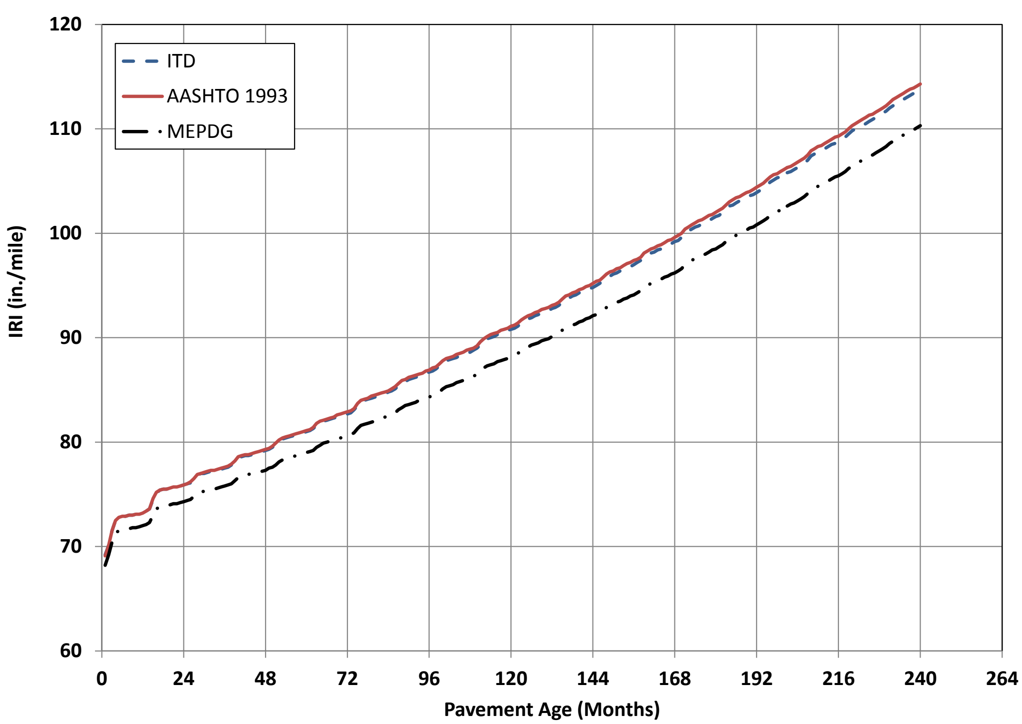


Figure 144. Comparison of the Predicted IRI, US-2(a) Project

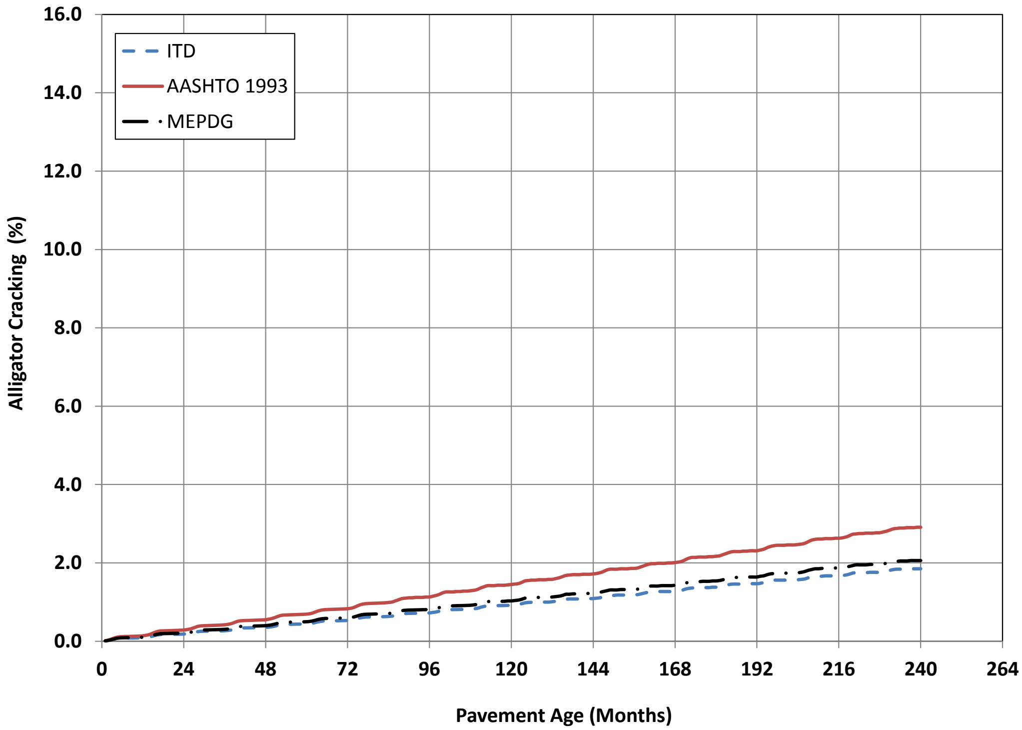


Figure 145. Comparison of the Predicted Alligator Cracking, US-2(b) Project

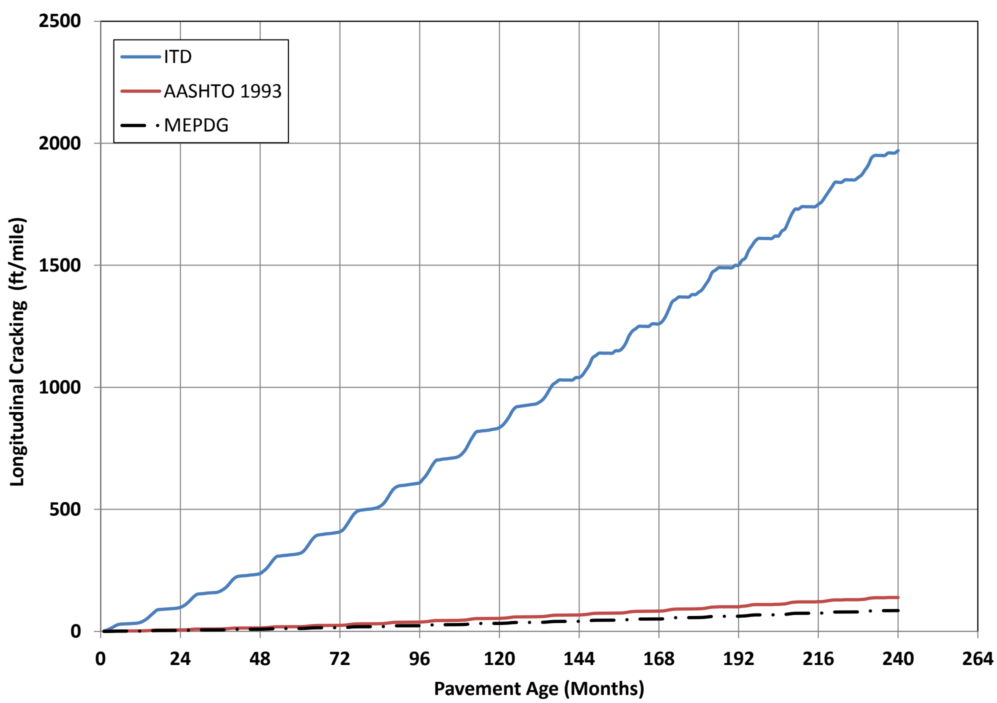


Figure 146. Comparison of the Predicted Longitudinal Cracking, US-2(b) Project

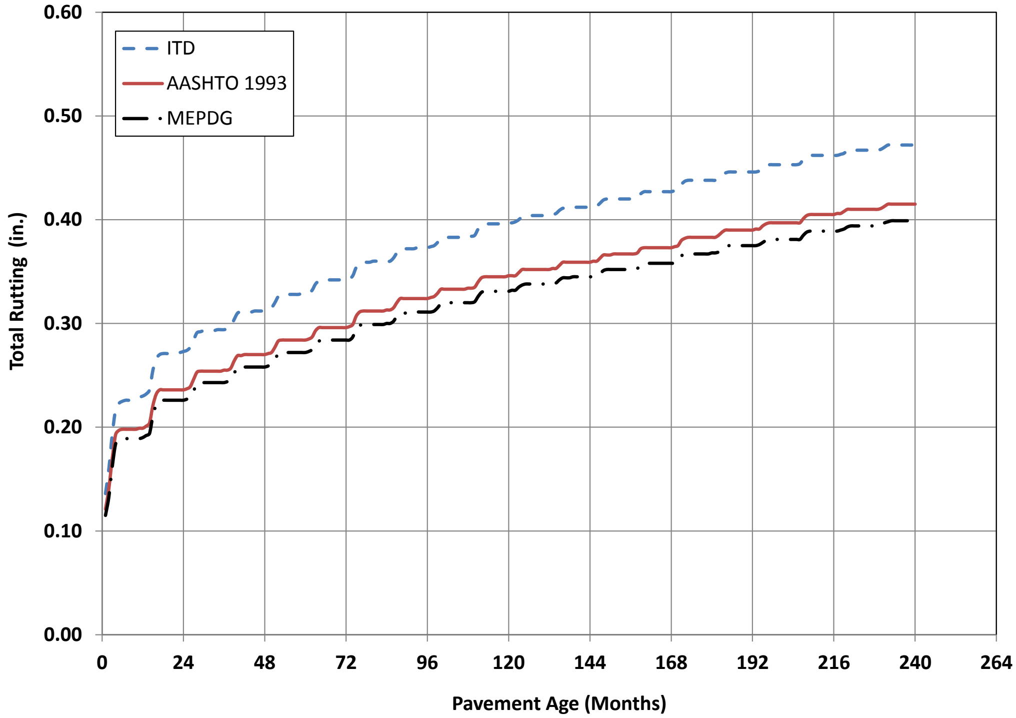


Figure 147. Comparison of the Predicted Total Rutting, US-2(b) Project

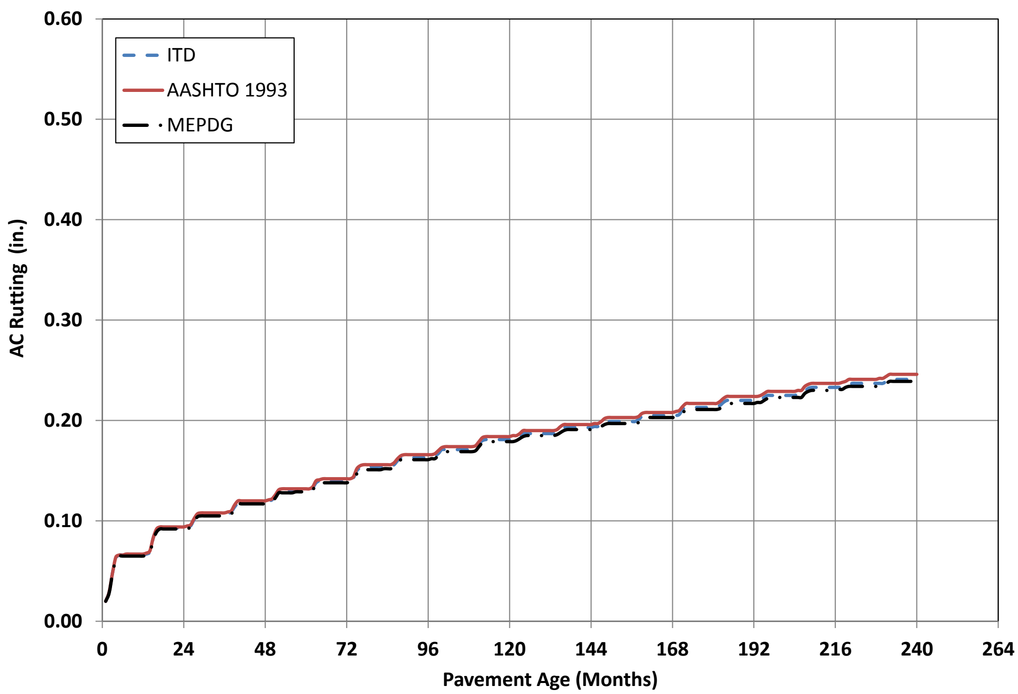


Figure 148. Comparison of the Predicted AC Layer Rutting, US-2(b) Project

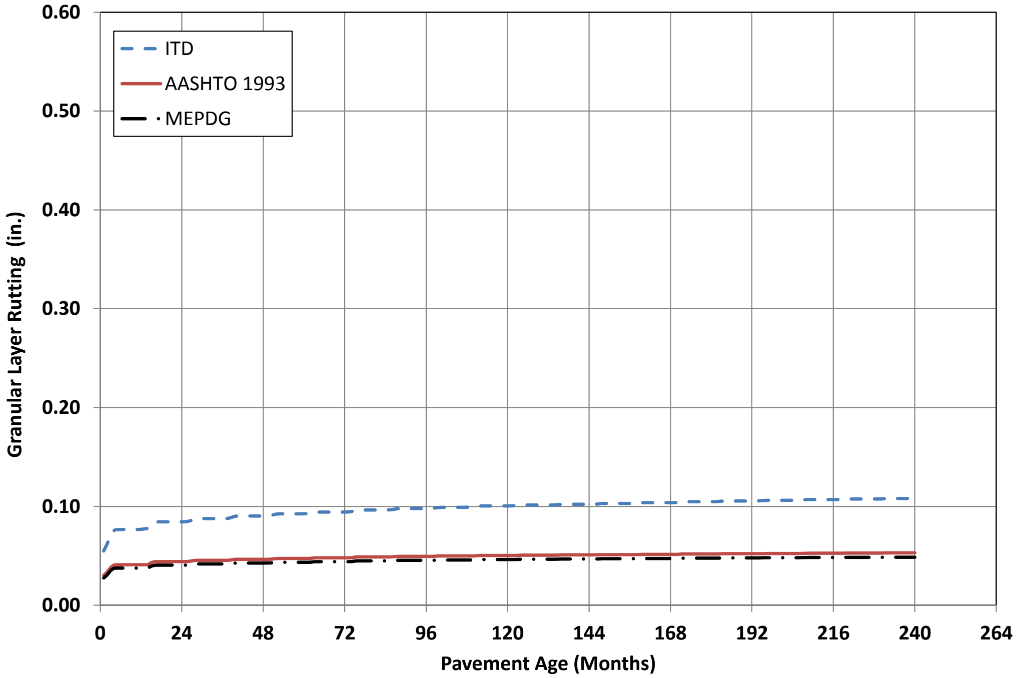


Figure 149. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-2(b) Project

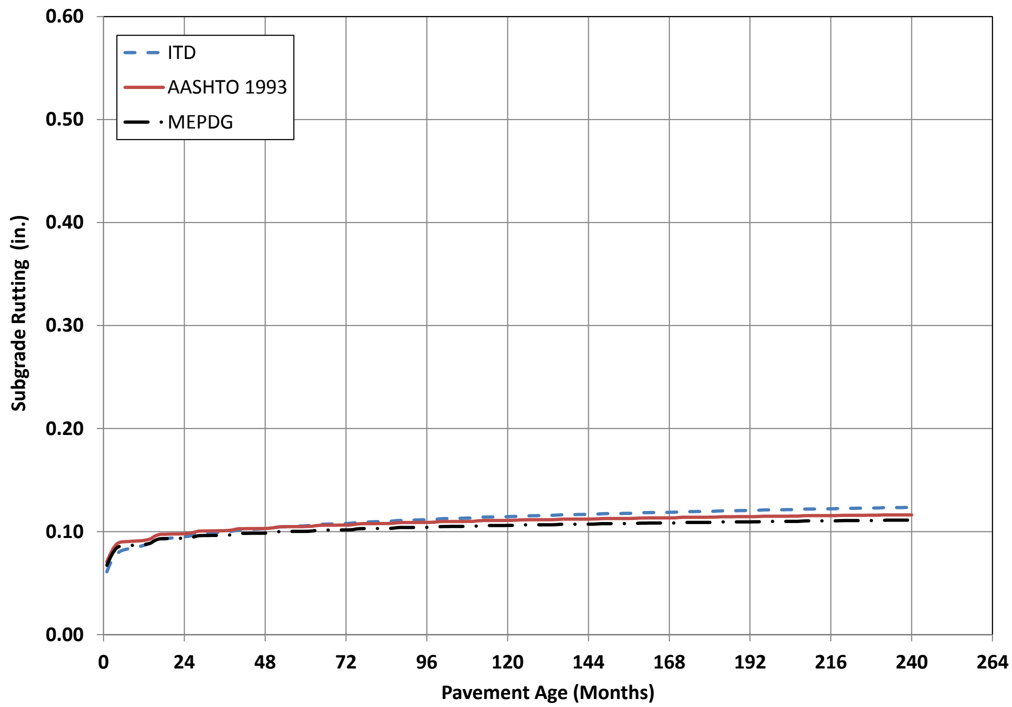


Figure 150. Comparison of the Predicted Subgrade Layer Rutting, US-2(b) Project

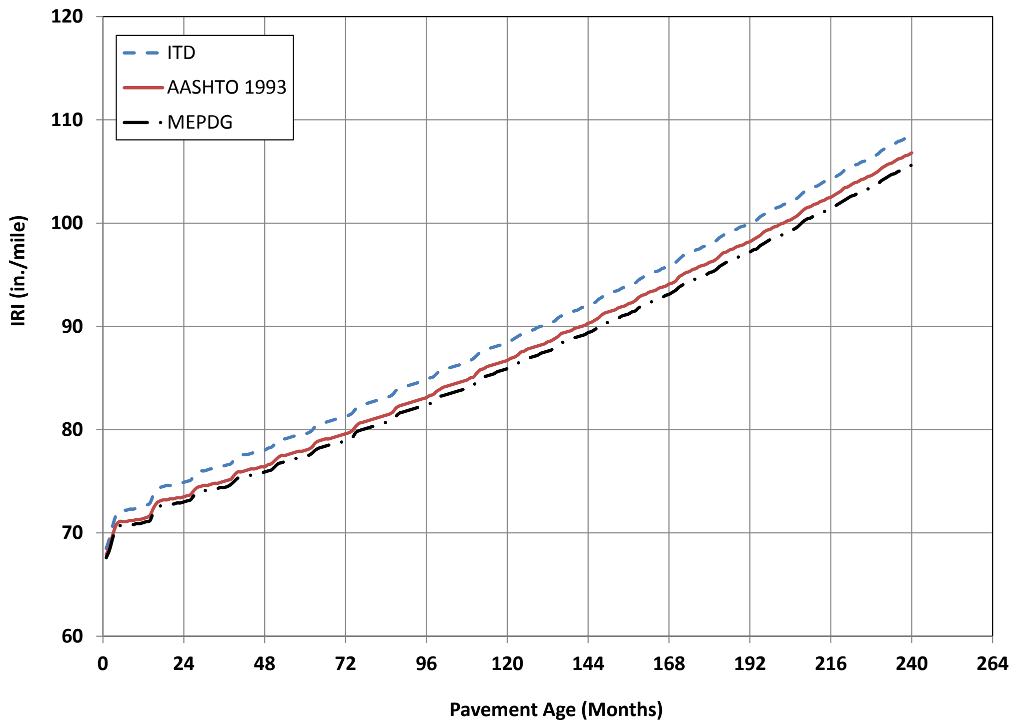


Figure 151. Comparison of the Predicted IRI, US-2(b) Project

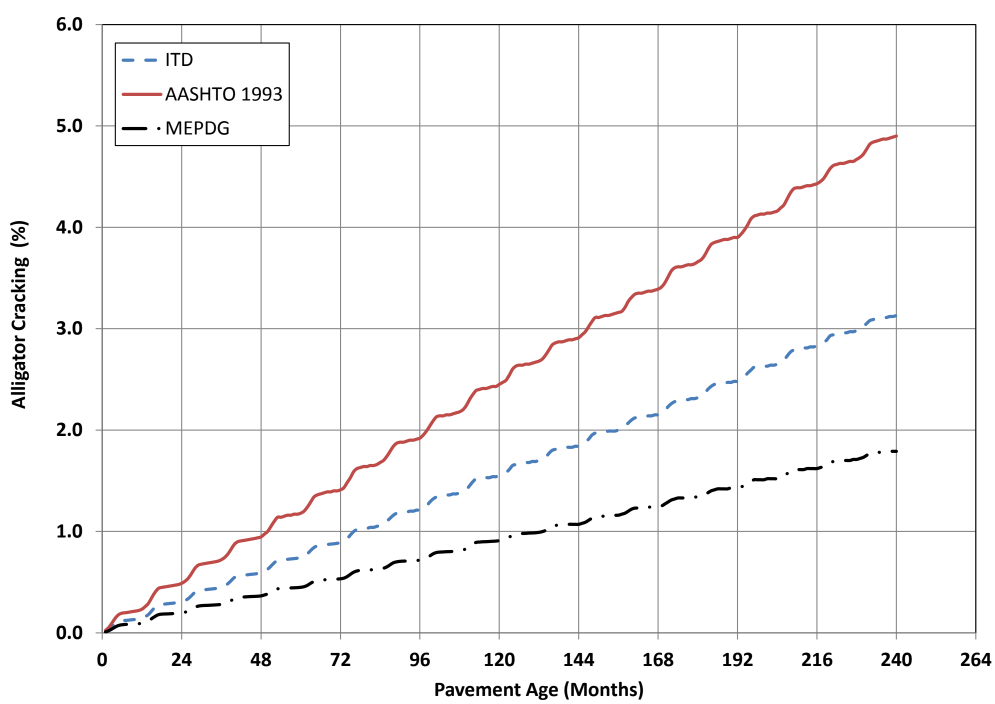


Figure 152. Comparison of the Predicted Alligator Cracking, US-2(c) Project

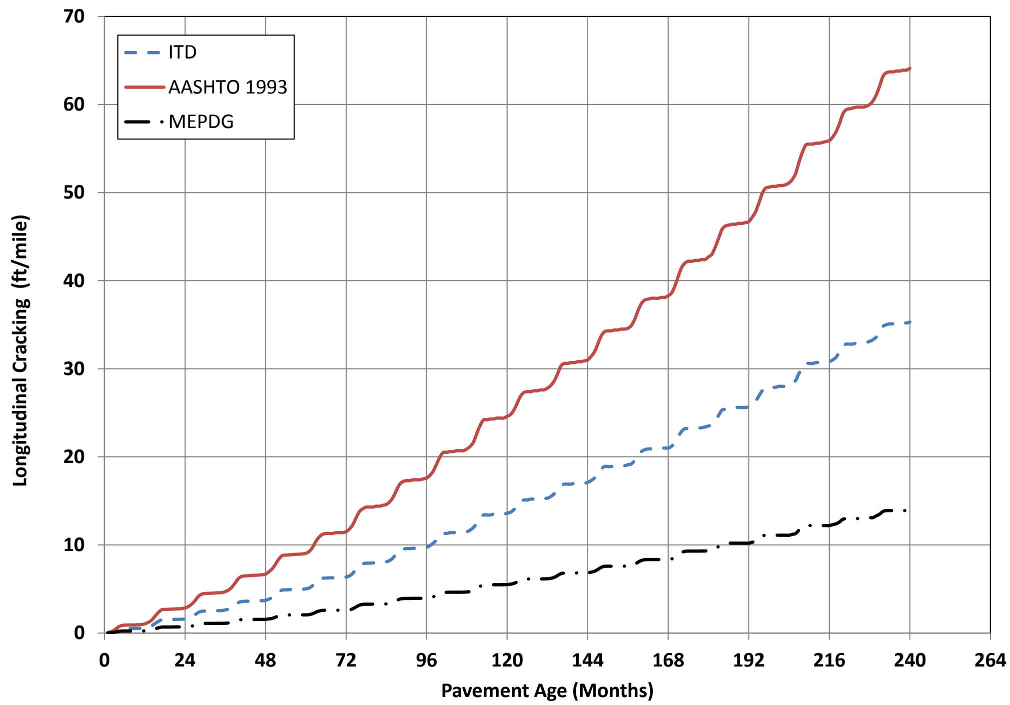


Figure 153. Comparison of the Predicted Longitudinal Cracking, US-2(c) Project

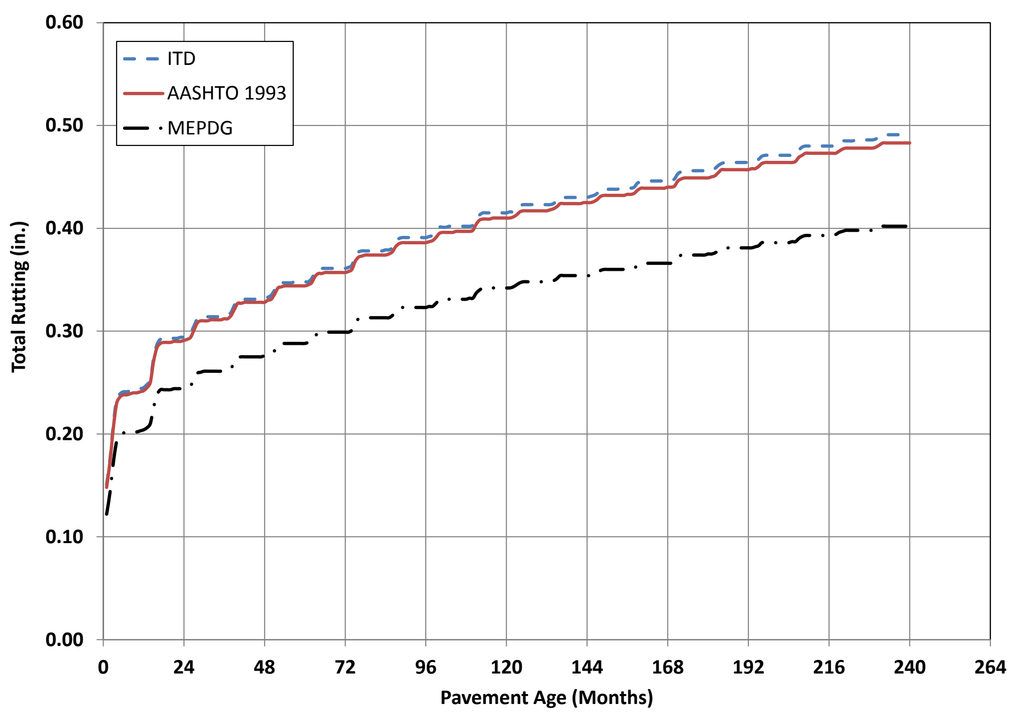


Figure 154. Comparison of the Predicted Total Rutting, US-2(c) Project

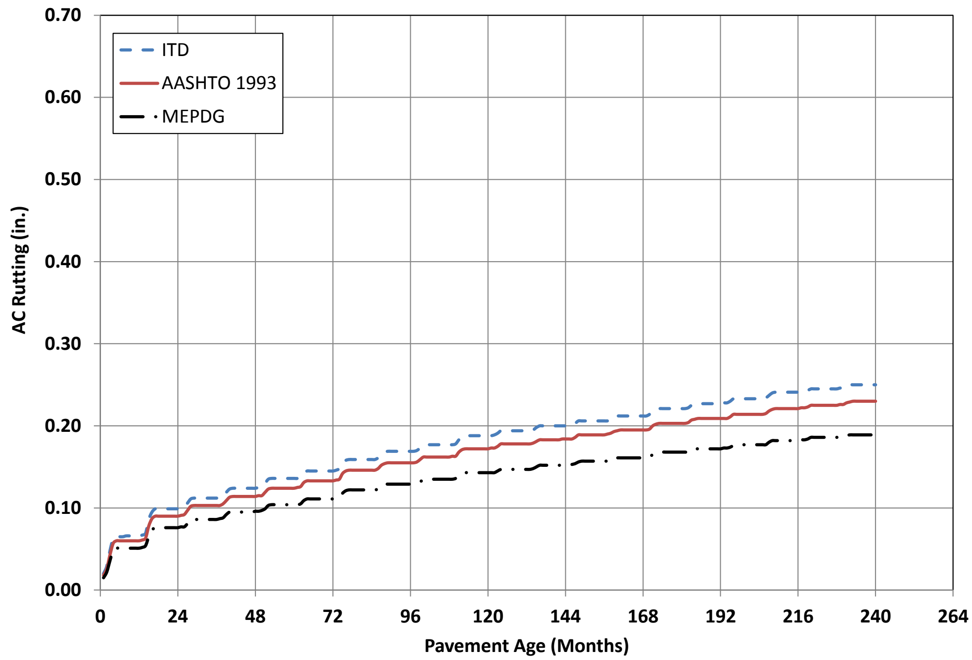


Figure 155. Comparison of the Predicted AC Layer Rutting, US-2(c) Project

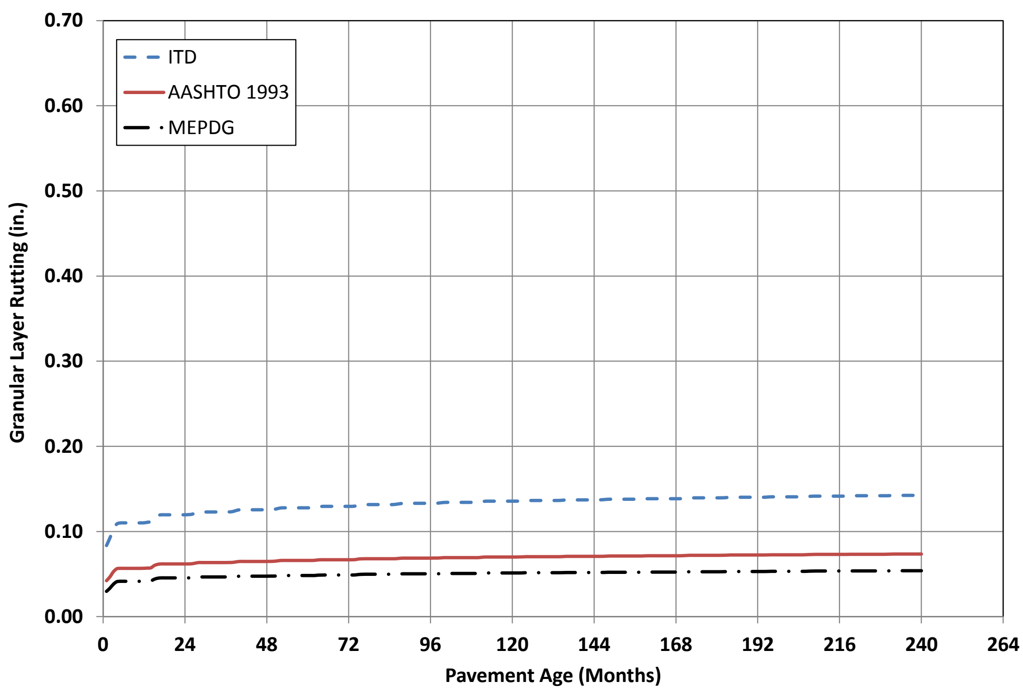


Figure 156. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-2(c) Project

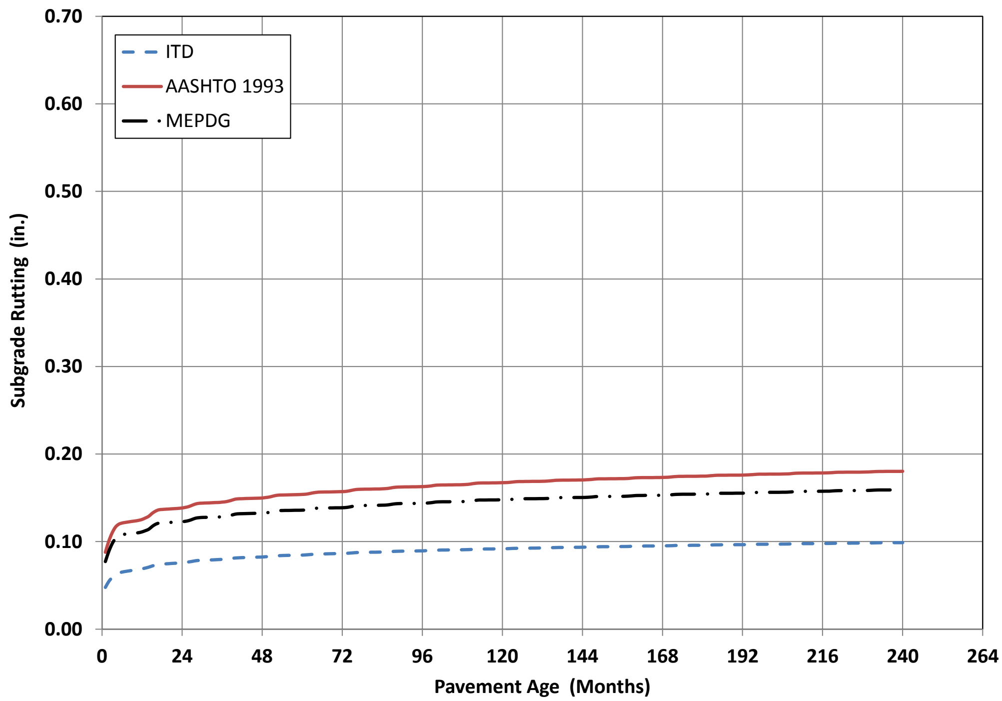


Figure 157. Comparison of the Predicted Subgrade Layer Rutting, US-2(c) Project

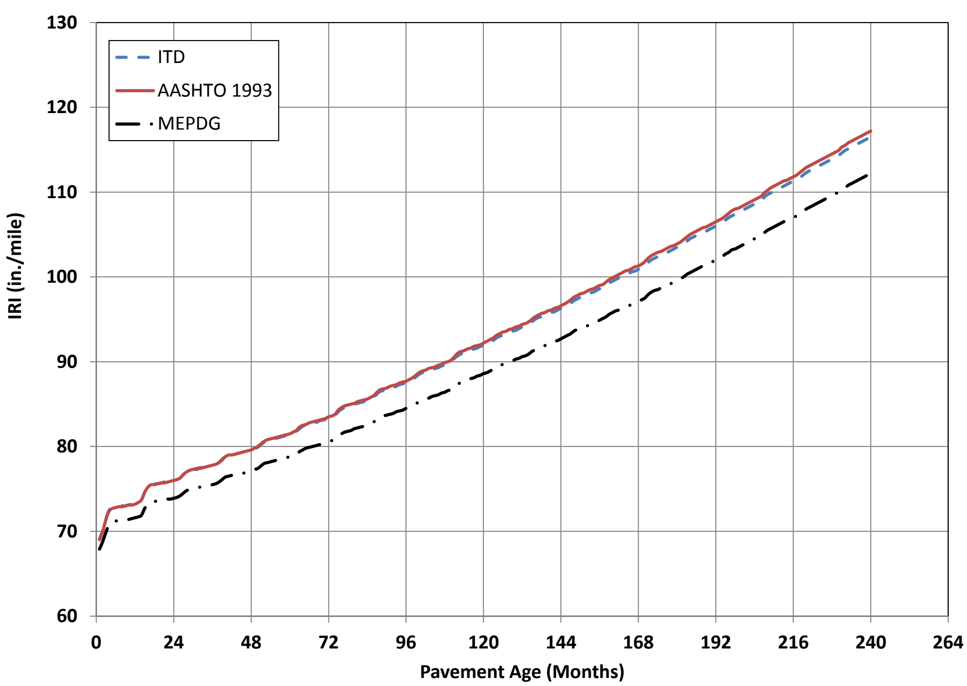


Figure 158. Comparison of the Predicted IRI, US-2(c) Project

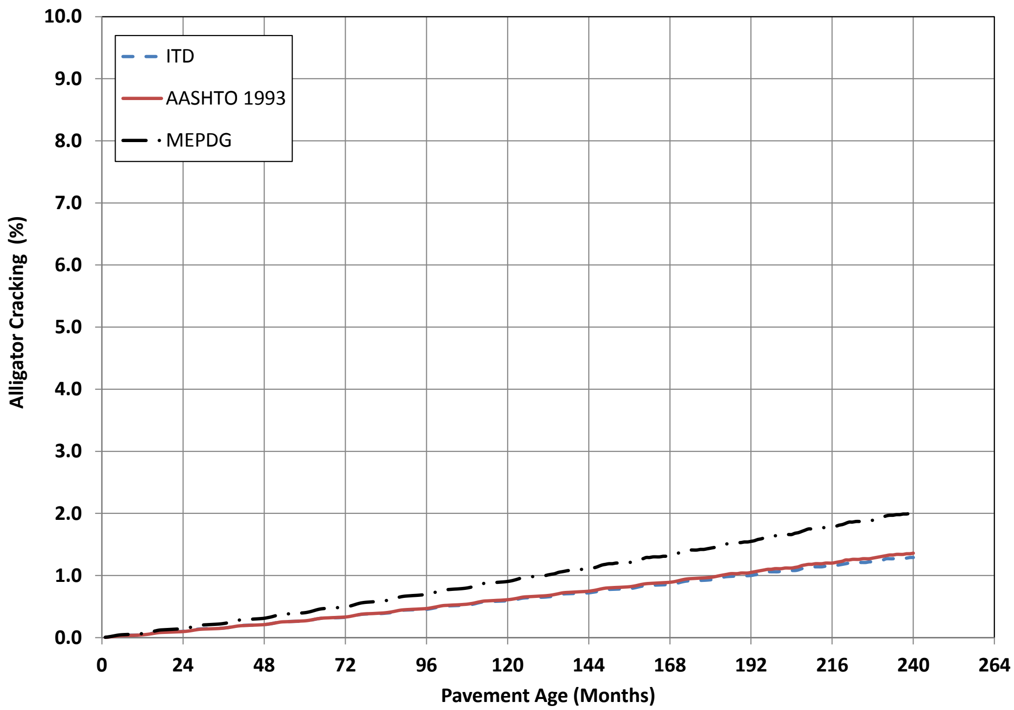


Figure 159. Comparison of the Predicted Alligator Cracking, SH-62 Project

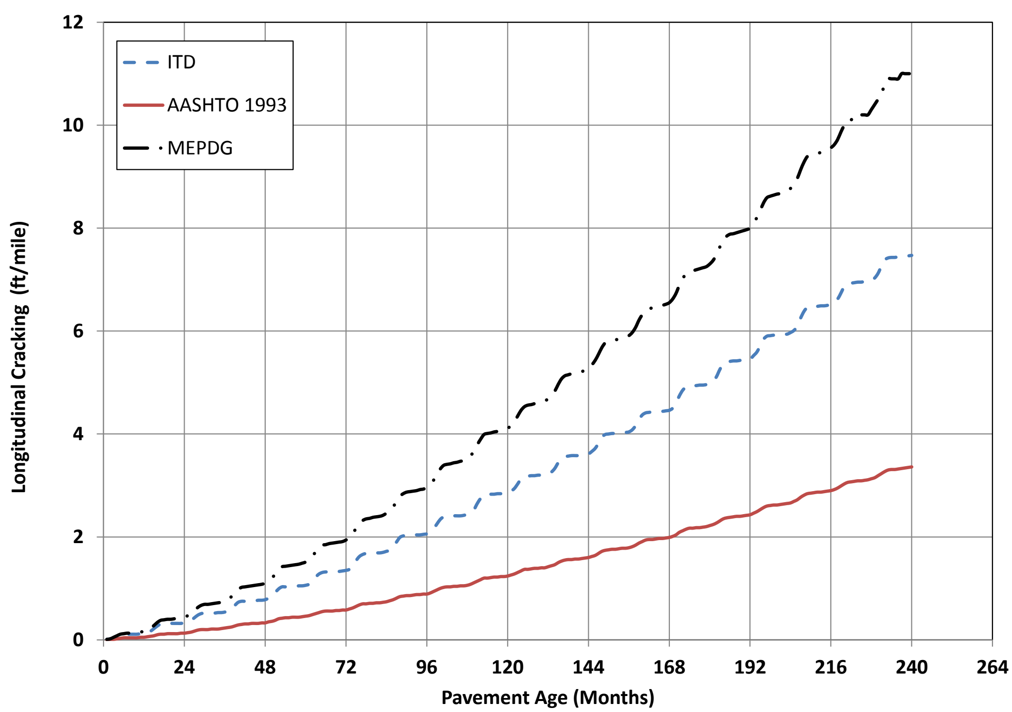


Figure 160. Comparison of the Predicted Longitudinal Cracking, SH-62 Project

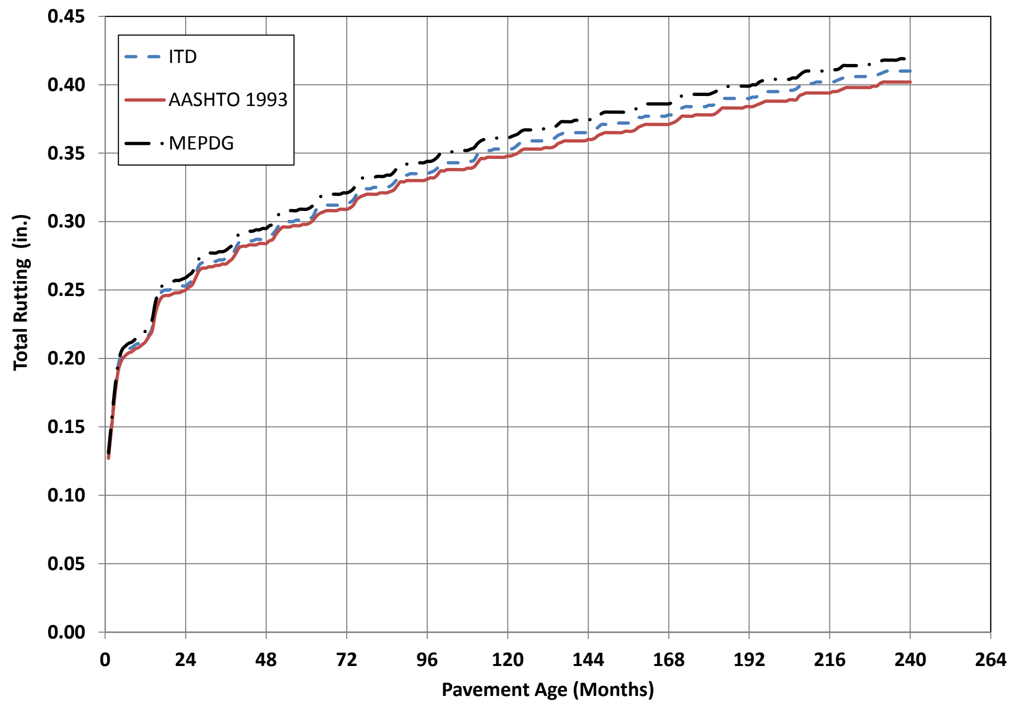


Figure 161. Comparison of the Predicted Total Rutting, SH-62 Project

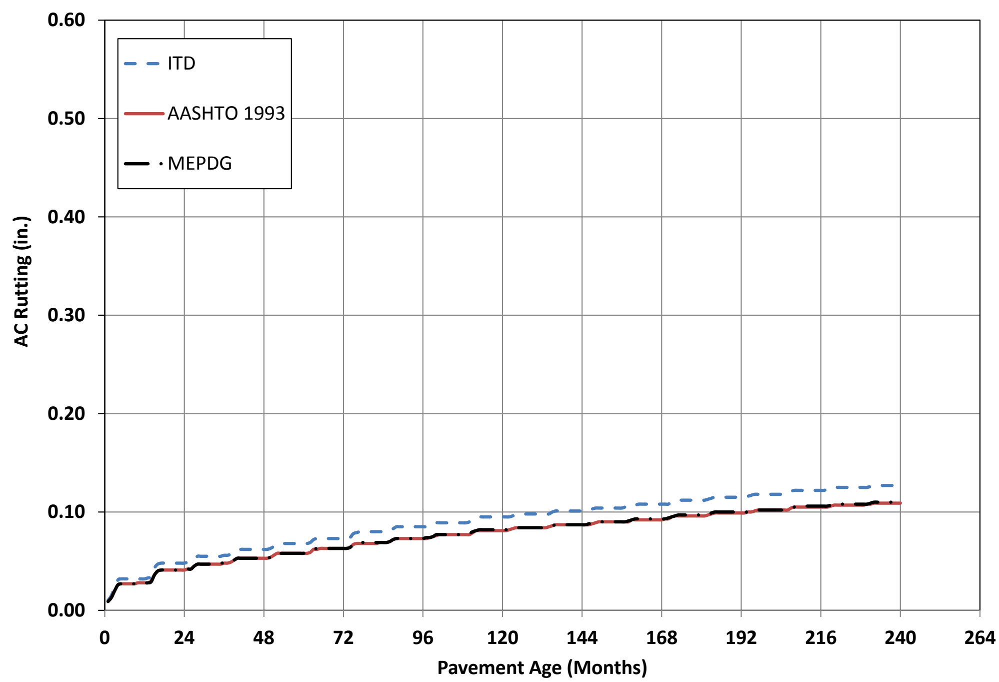


Figure 162. Comparison of the Predicted AC Layer Rutting, SH-62 Project

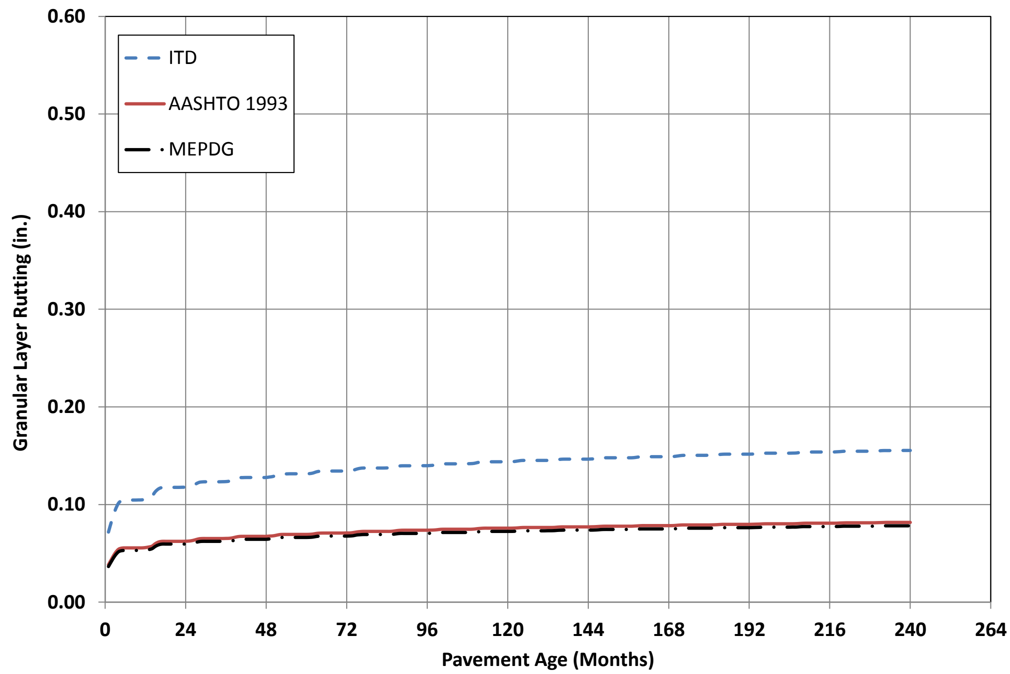


Figure 163. Comparison of the Predicted Unbound Granular Layer(s) Rutting, SH-62 Project

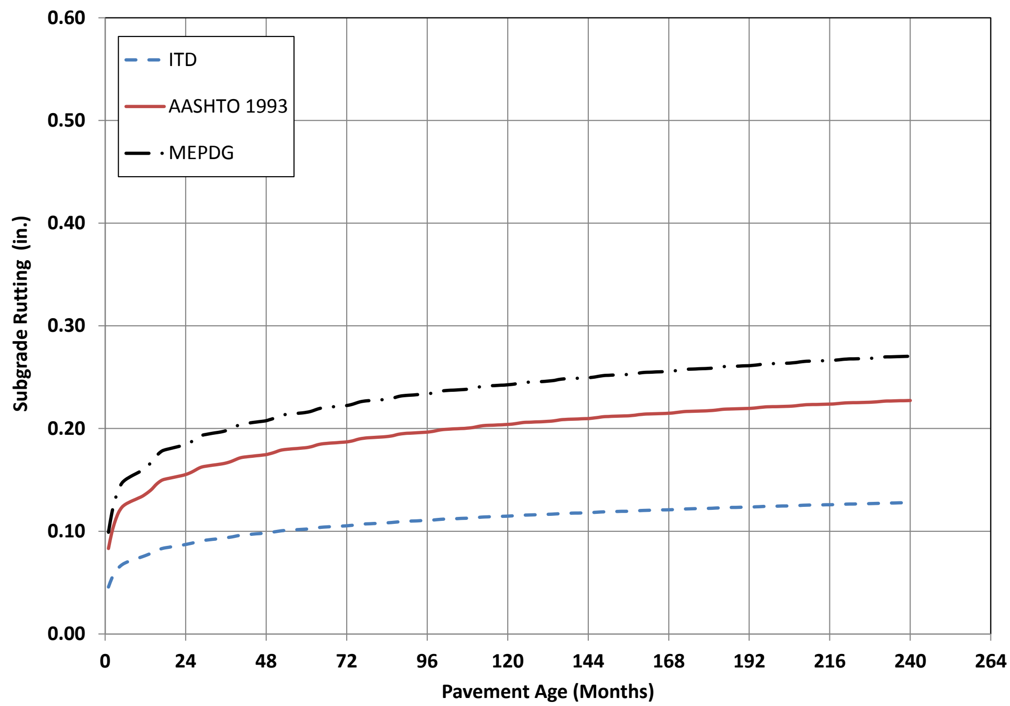


Figure 164. Comparison of the Predicted Subgrade Layer Rutting, SH-62 Project

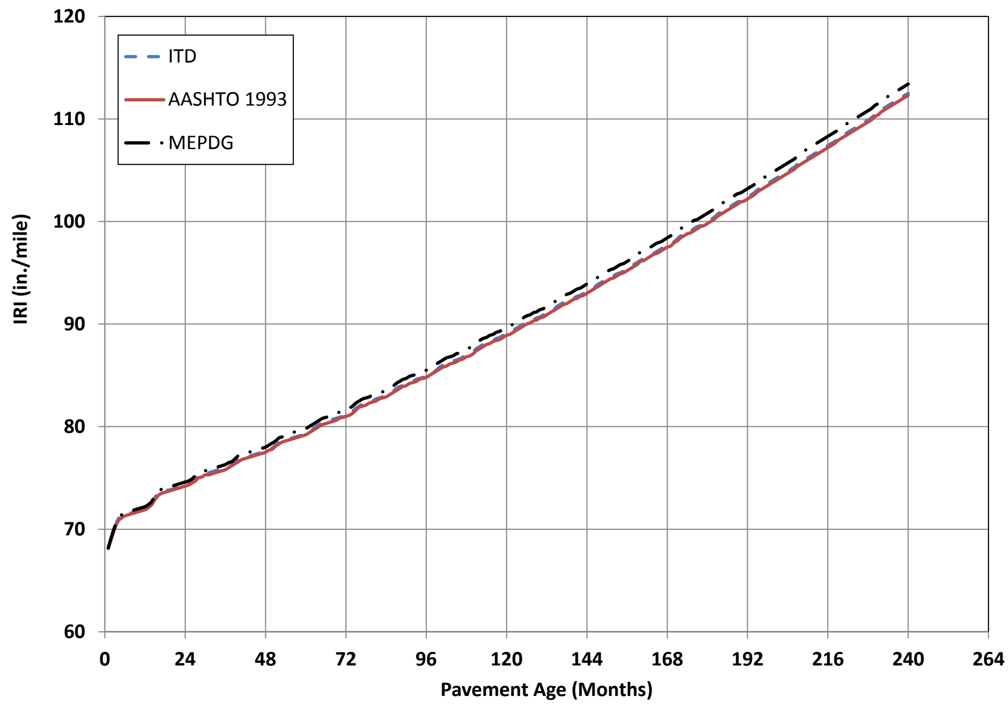


Figure 165. Comparison of the Predicted IRI, SH-62 Project

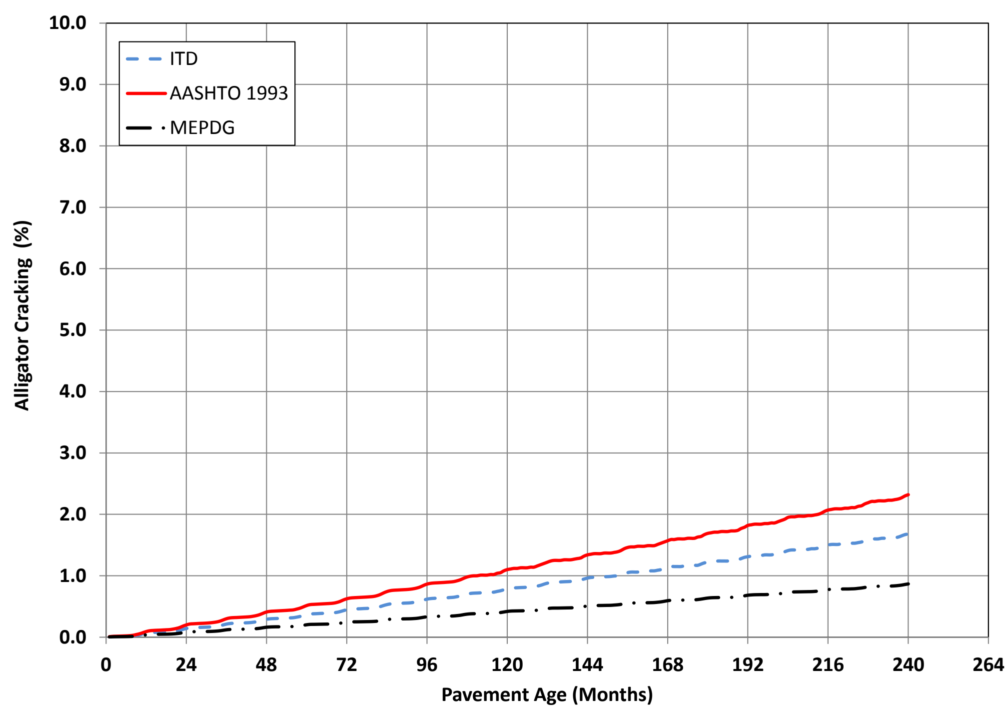


Figure 166. Comparison of the Predicted Alligator Cracking, SH-3 Project

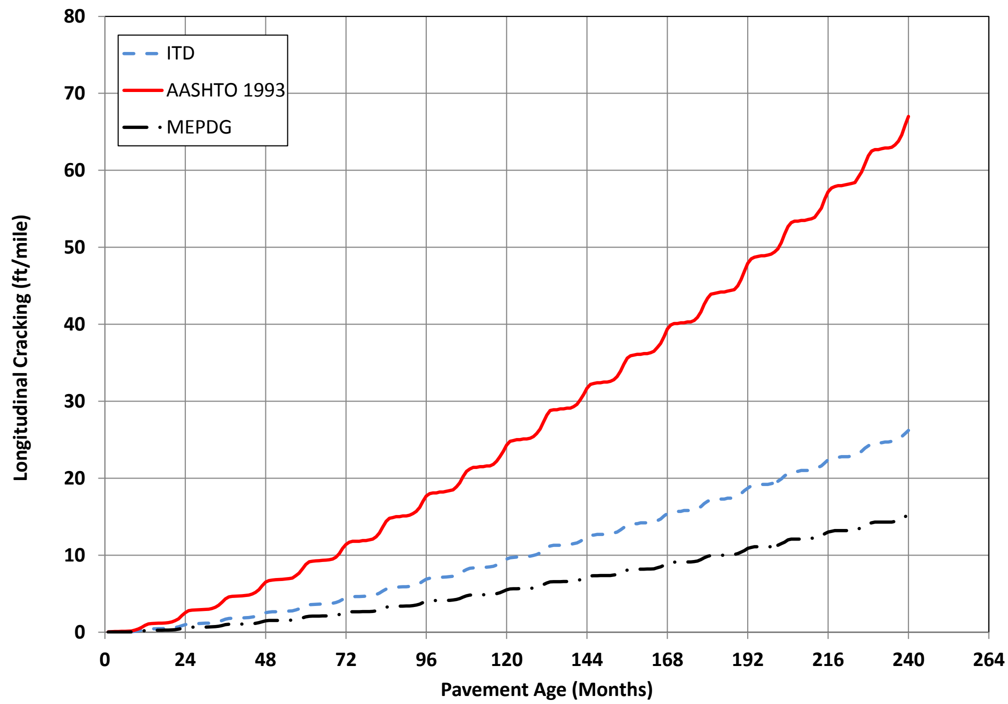


Figure 167. Comparison of the Predicted Longitudinal Cracking, SH-3 Project

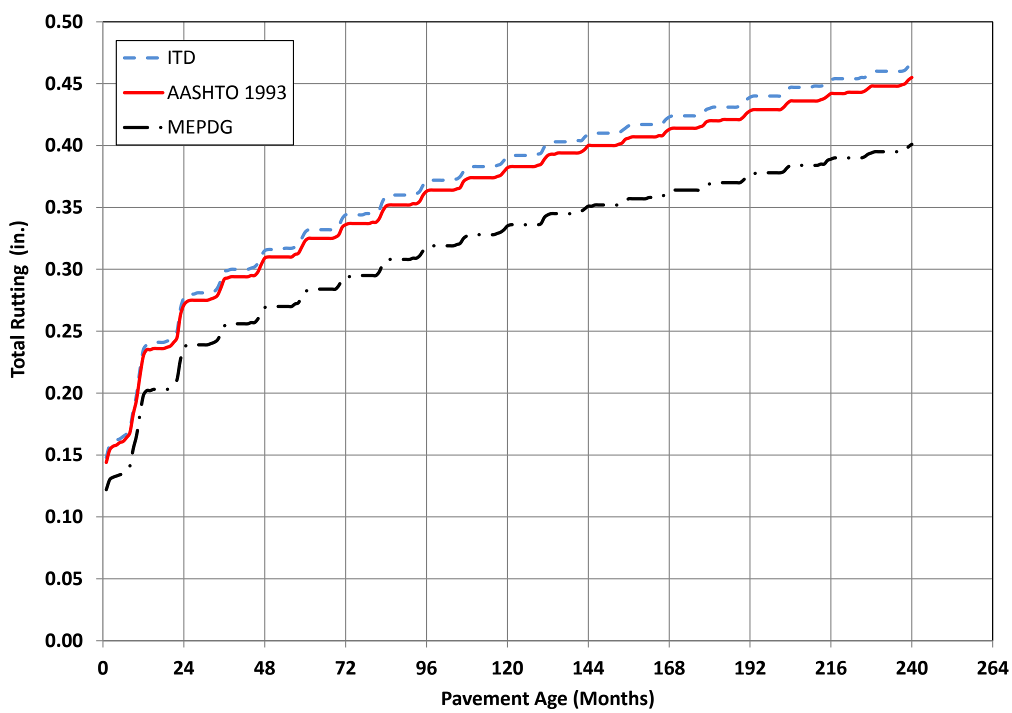


Figure 168. Comparison of the Predicted Total Rutting, SH-3 Project

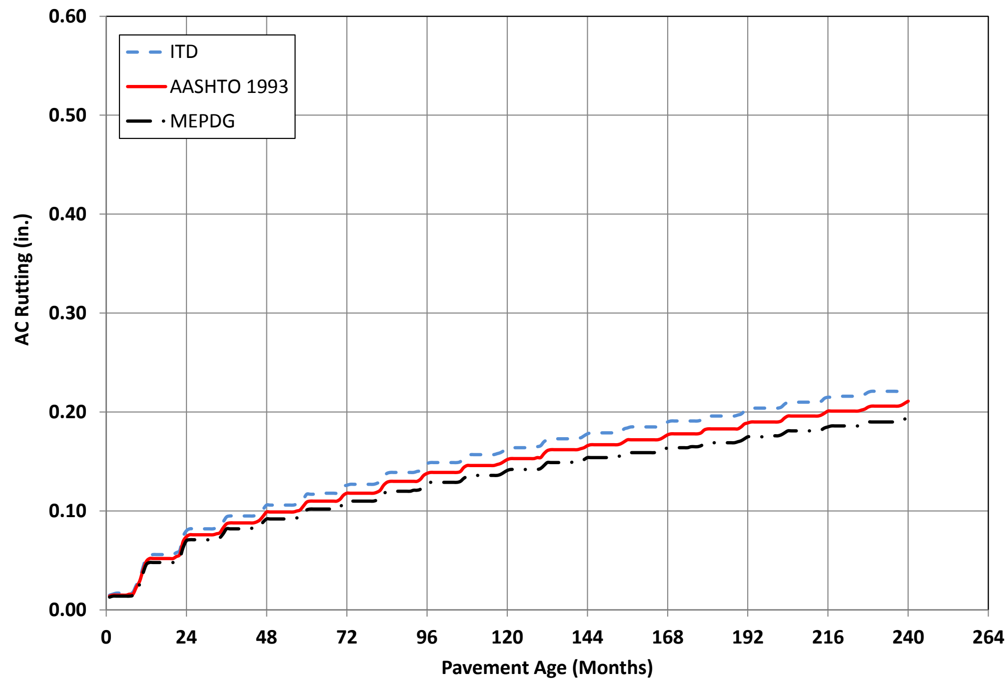


Figure 169. Comparison of the Predicted AC Layer Rutting, SH-3 Project

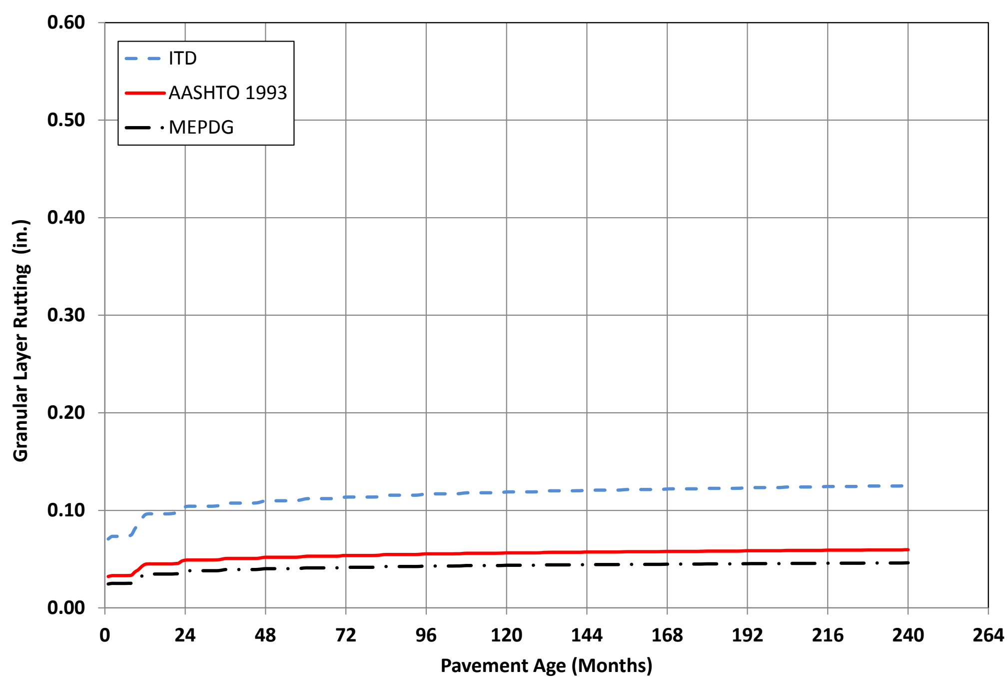


Figure 170. Comparison of the Predicted Unbound Granular Layer(s) Rutting, SH-3 Project

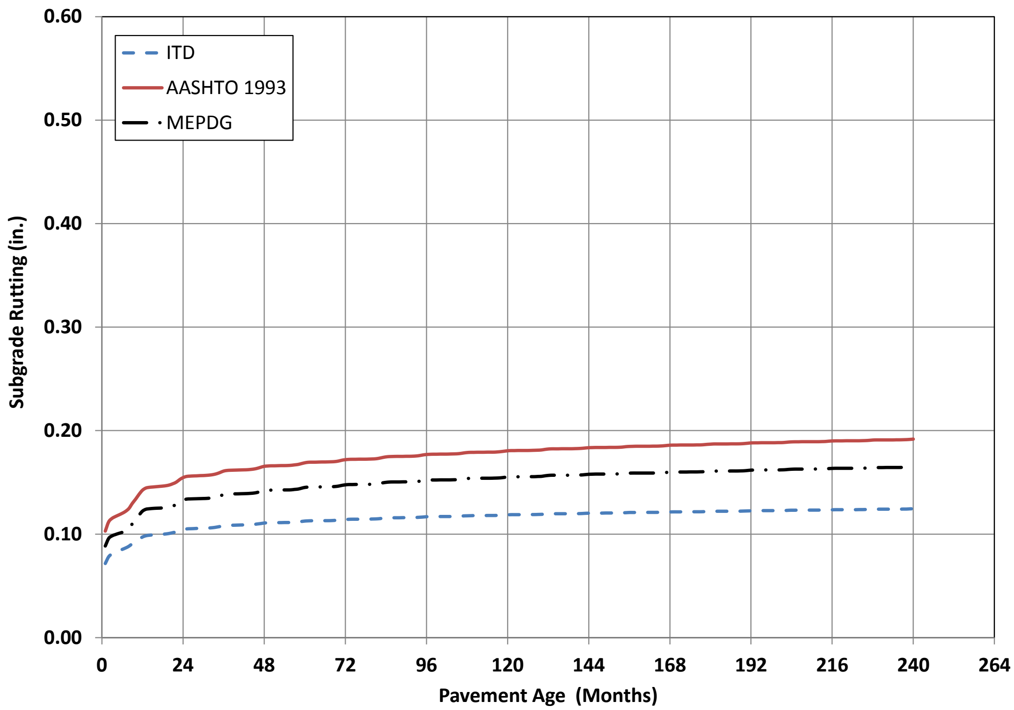


Figure 171. Comparison of the Predicted Subgrade Layer Rutting, SH-3 Project

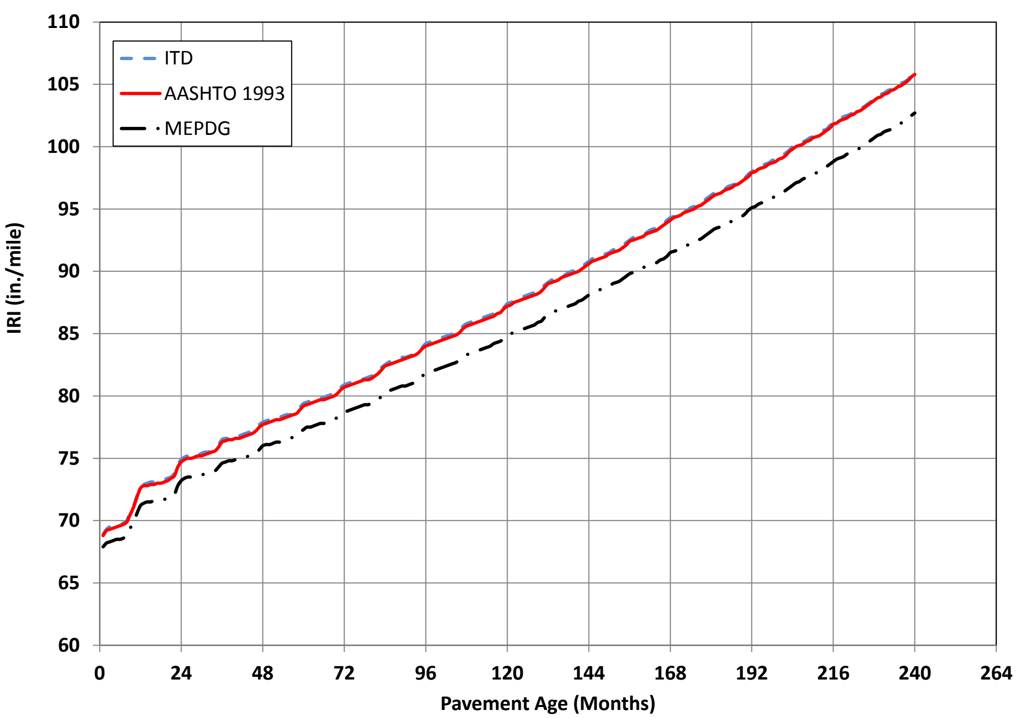


Figure 172. Comparison of the Predicted IRI, SH-3 Project

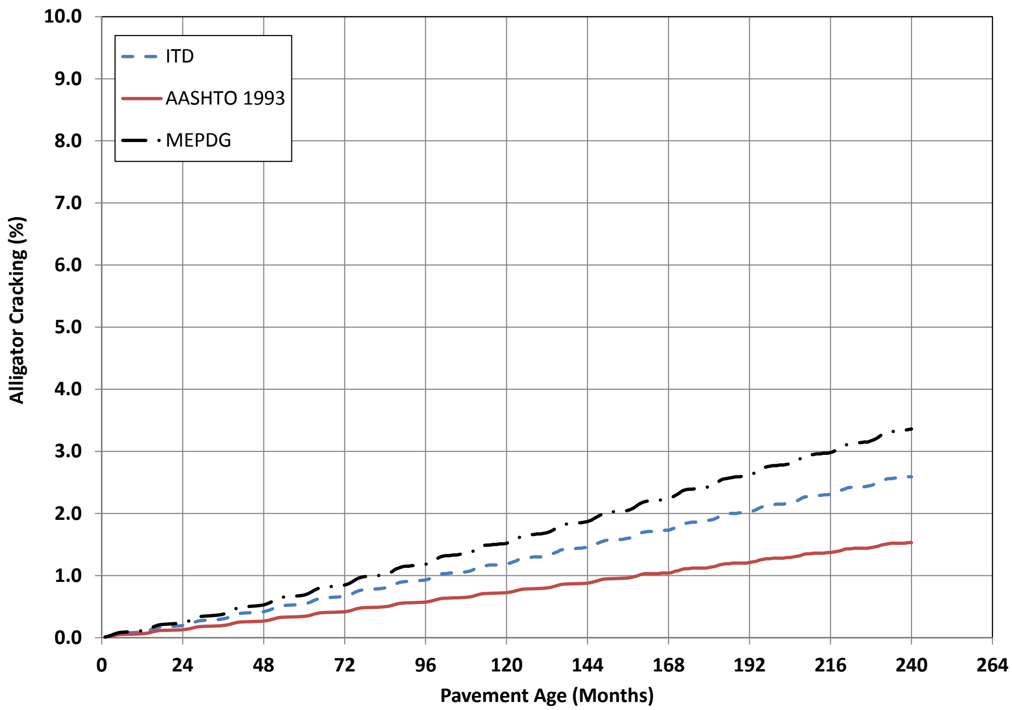


Figure 173. Comparison of the Predicted Alligator Cracking, SH-19 Project

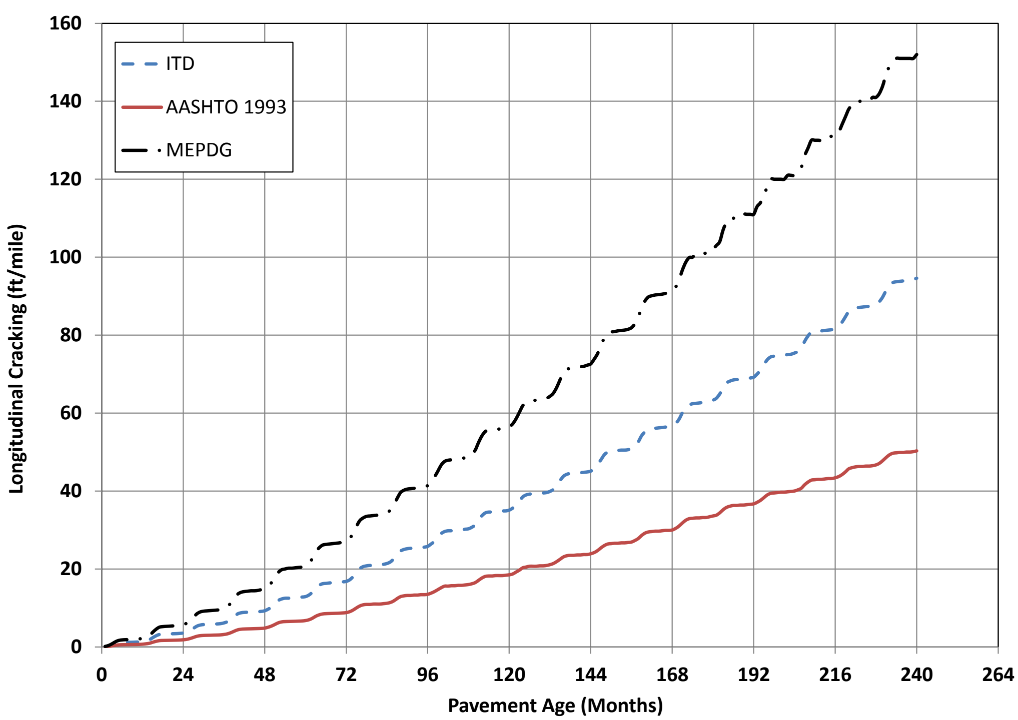


Figure 174. Comparison of the Predicted Longitudinal Cracking, SH-19 Project

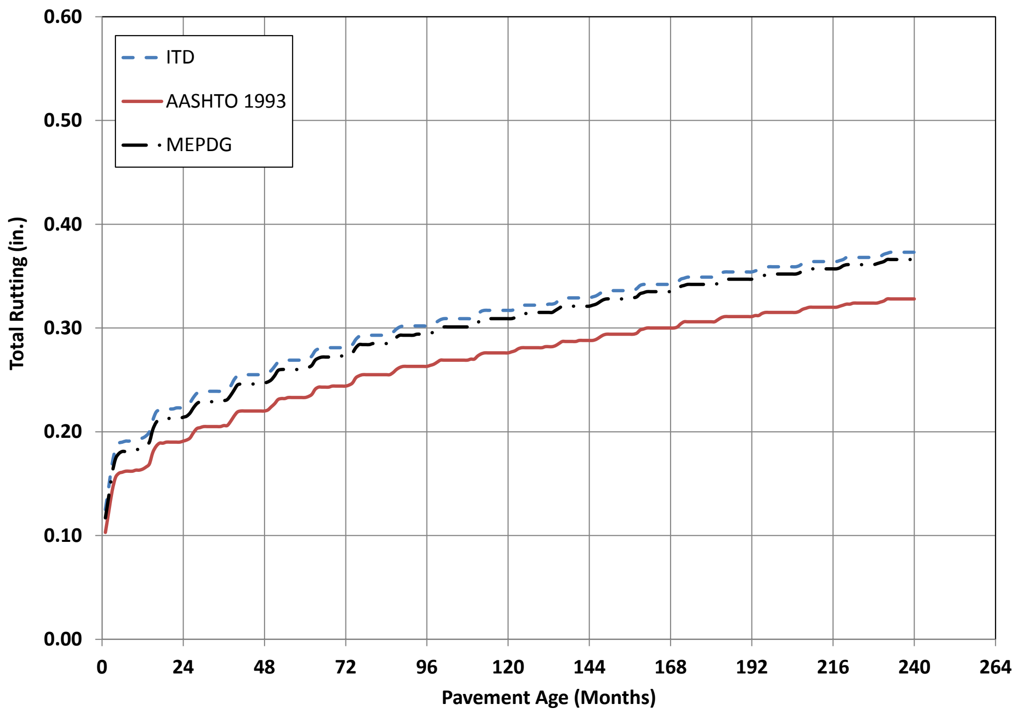


Figure 175. Comparison of the Predicted Total Rutting, SH-19 Project

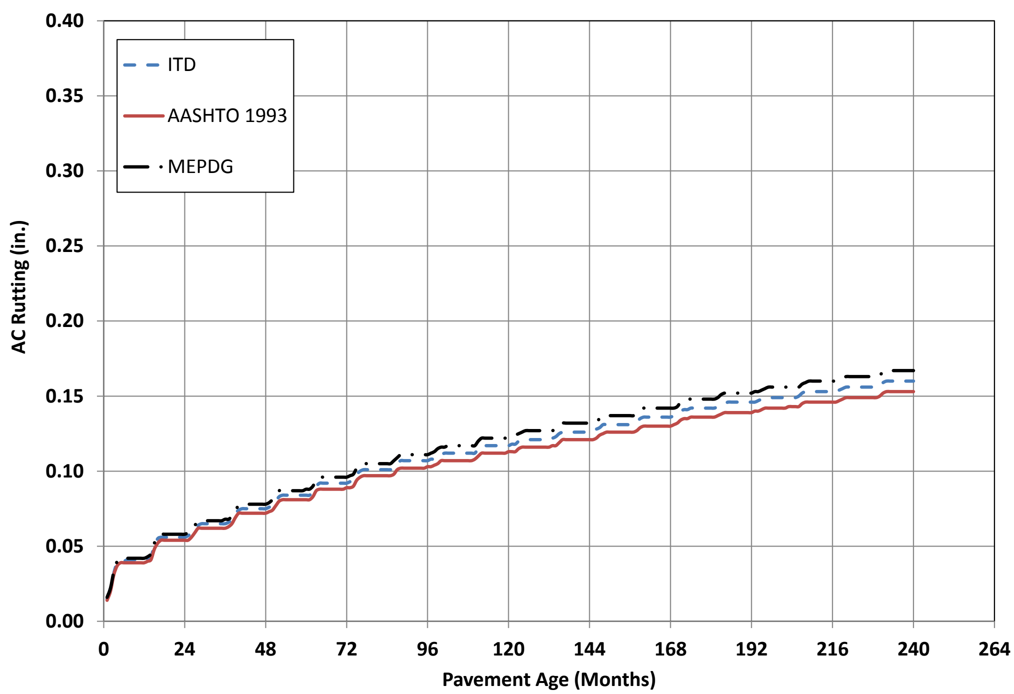


Figure 176. Comparison of the Predicted AC Layer Rutting, SH-19 Project

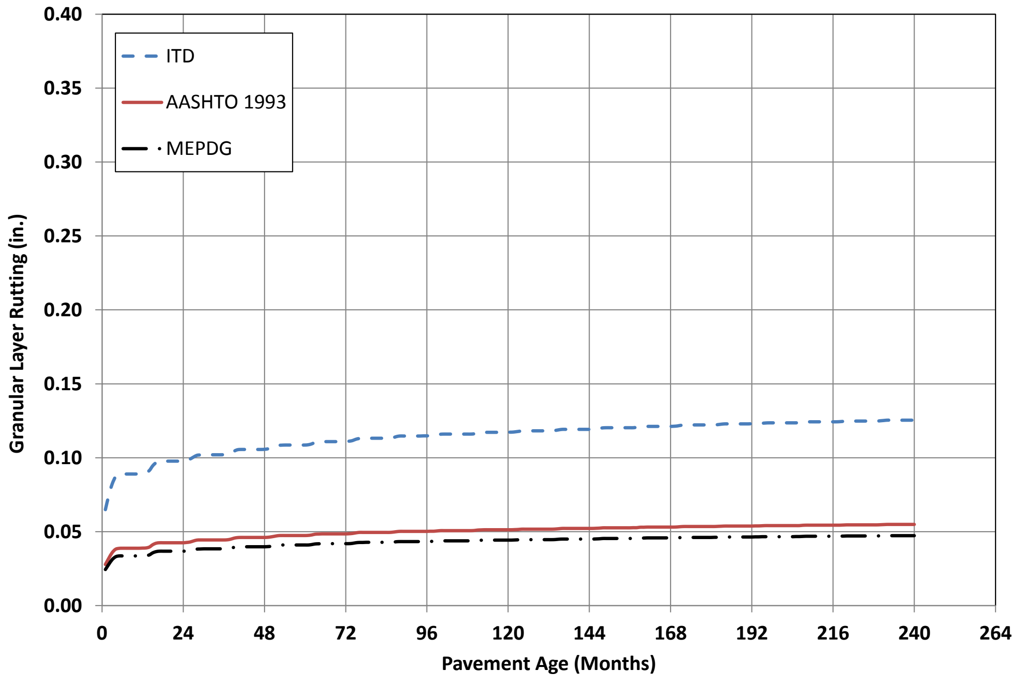


Figure 177. Comparison of the Predicted Unbound Granular Layer(s) Rutting, SH-19 Project

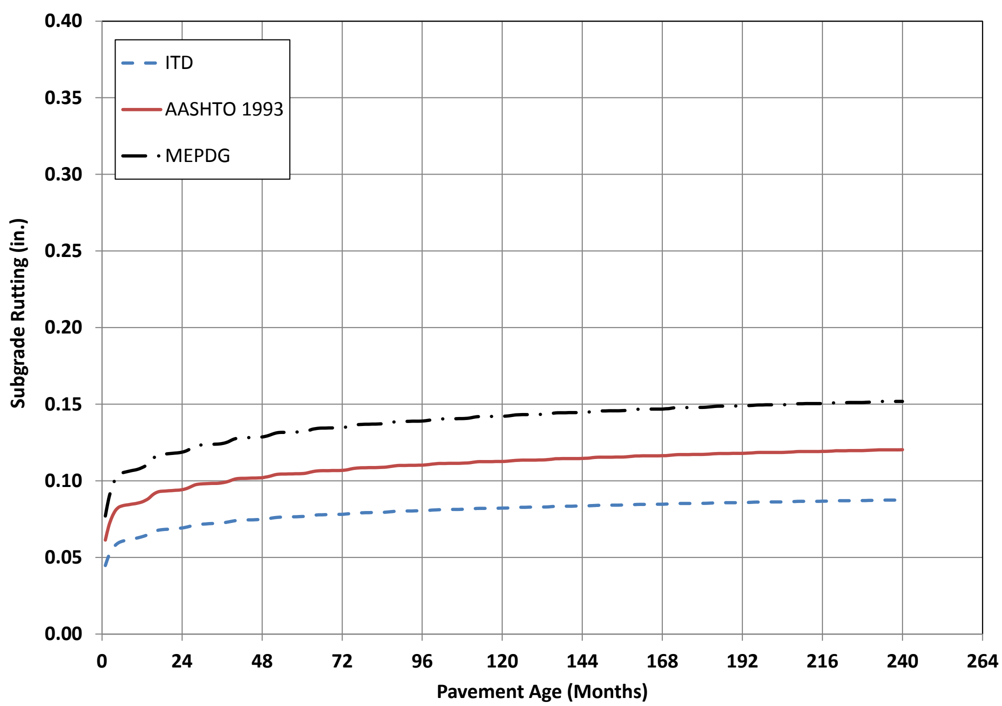


Figure 178. Comparison of the Predicted Subgrade Layer Rutting, SH-19 Project

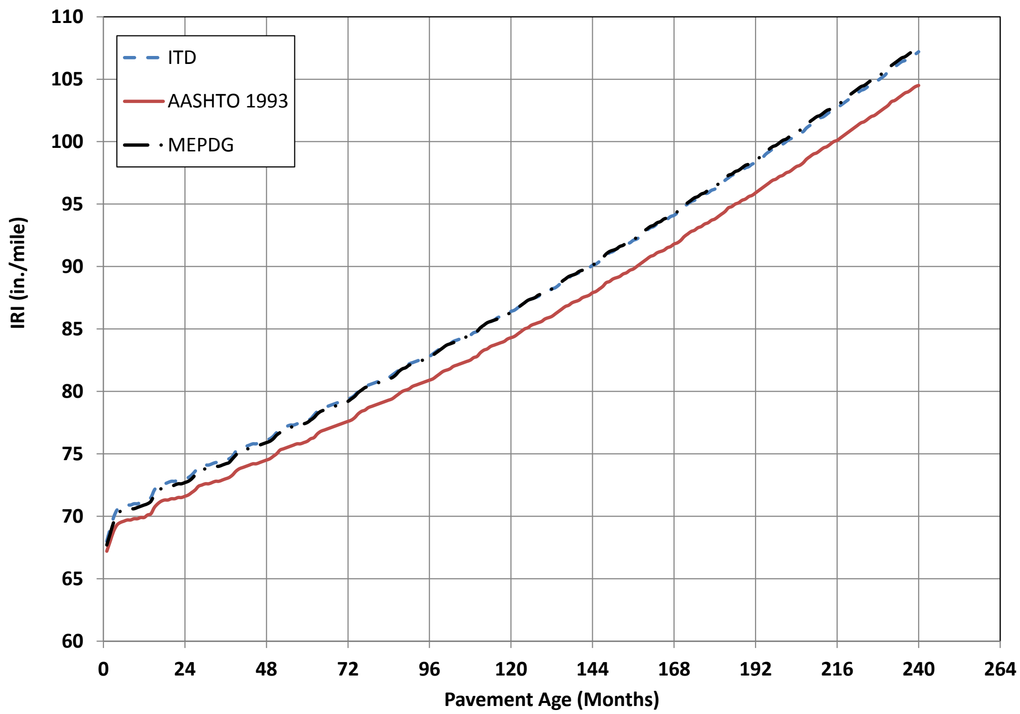


Figure 179. Comparison of the Predicted IRI, SH-19 Project

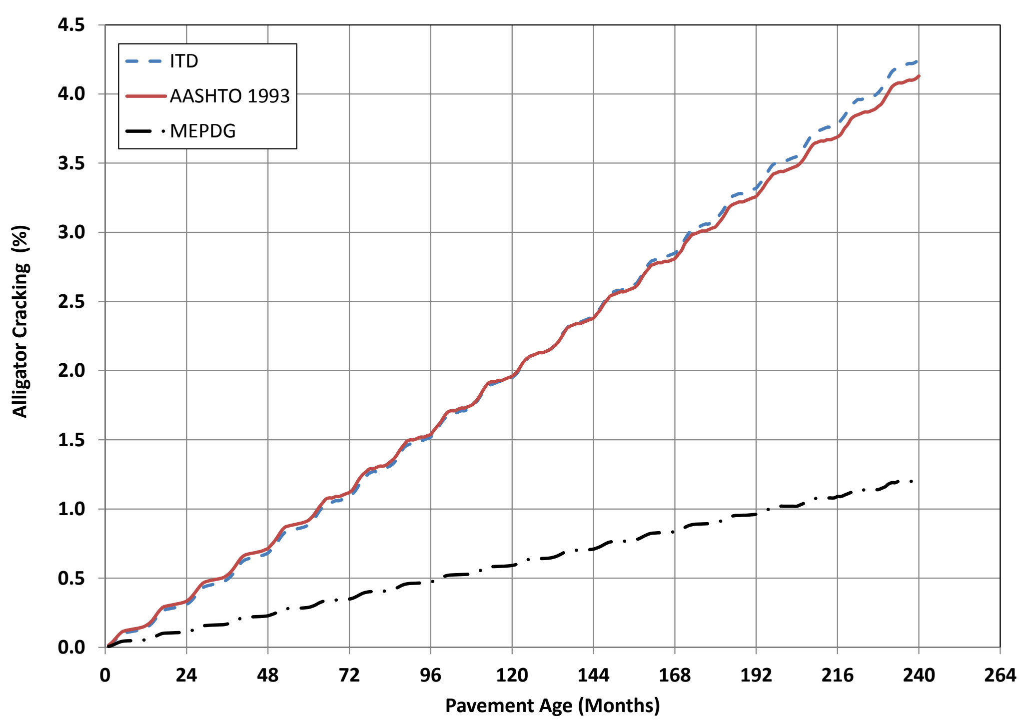


Figure 180. Comparison of the Predicted Alligator Cracking, US-95 Project

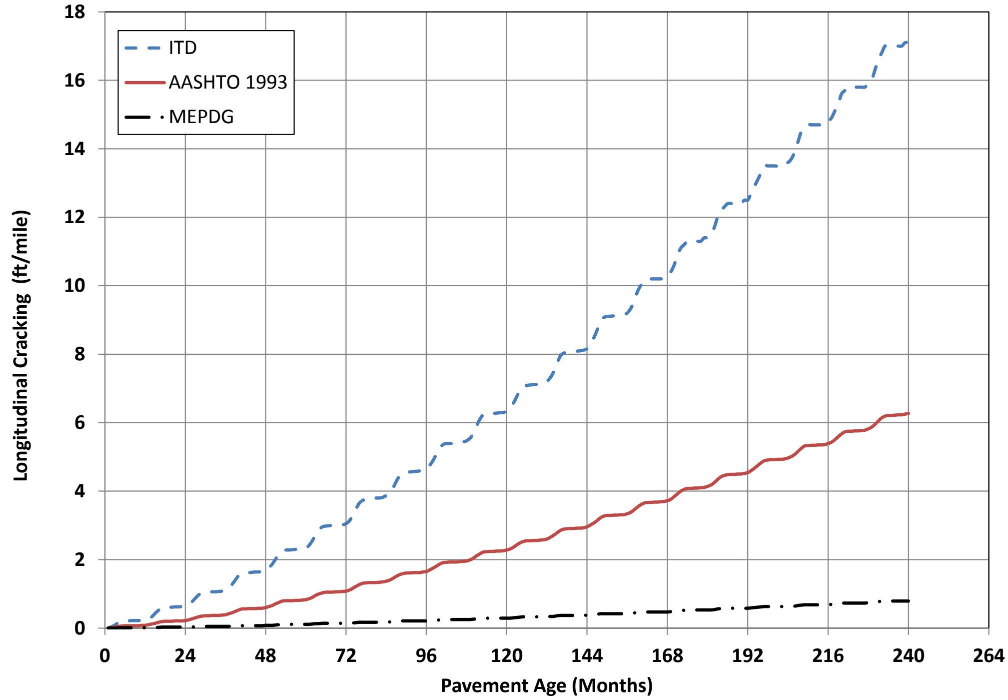


Figure 181. Comparison of the Predicted Longitudinal Cracking, US-95 Project

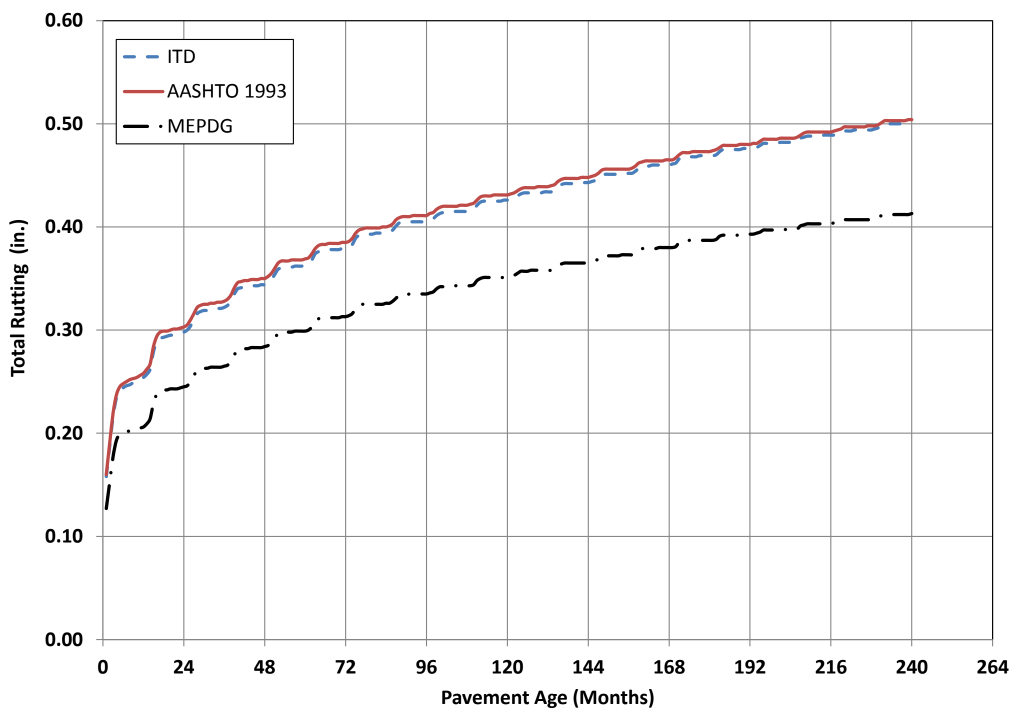


Figure 182. Comparison of the Predicted Total Rutting, US-95 Project

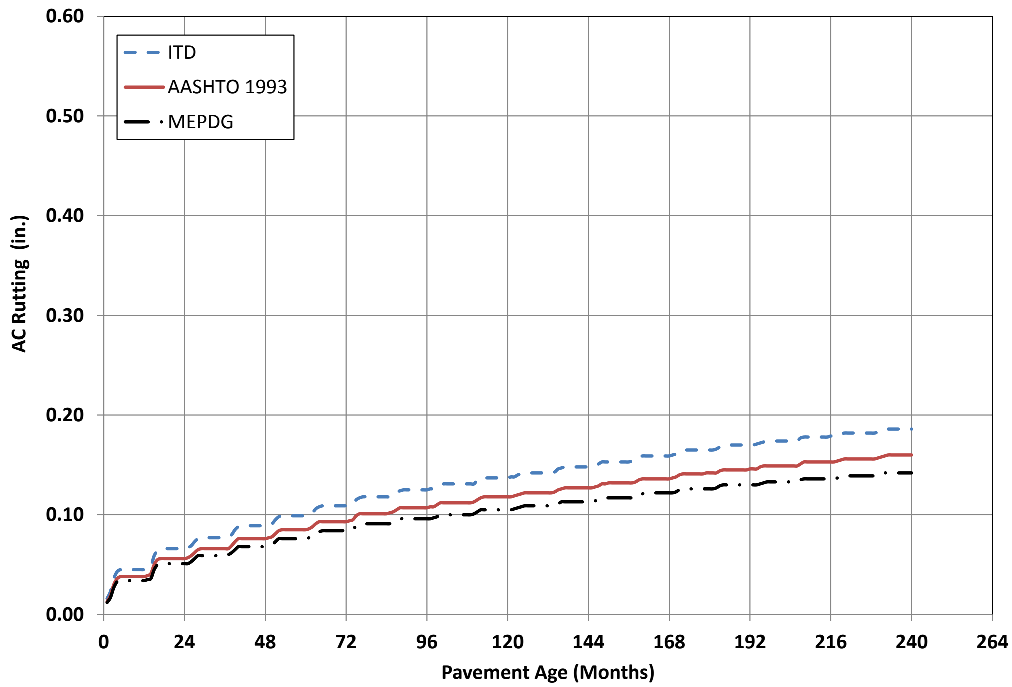


Figure 183. Comparison of the Predicted AC Layer Rutting, US-95 Project

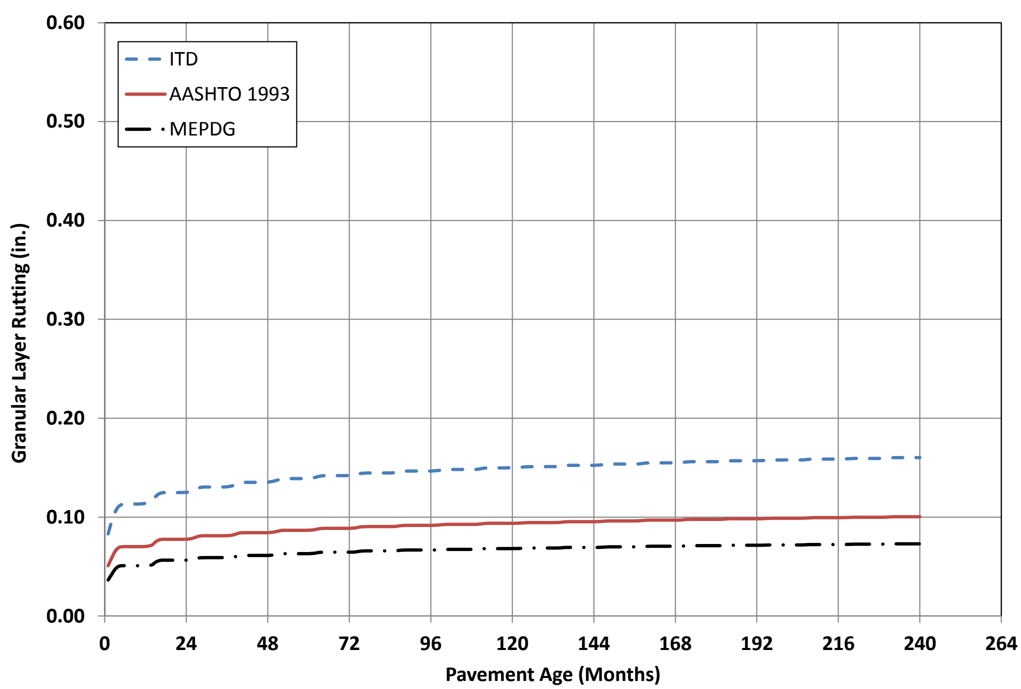


Figure 184. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-95 Project

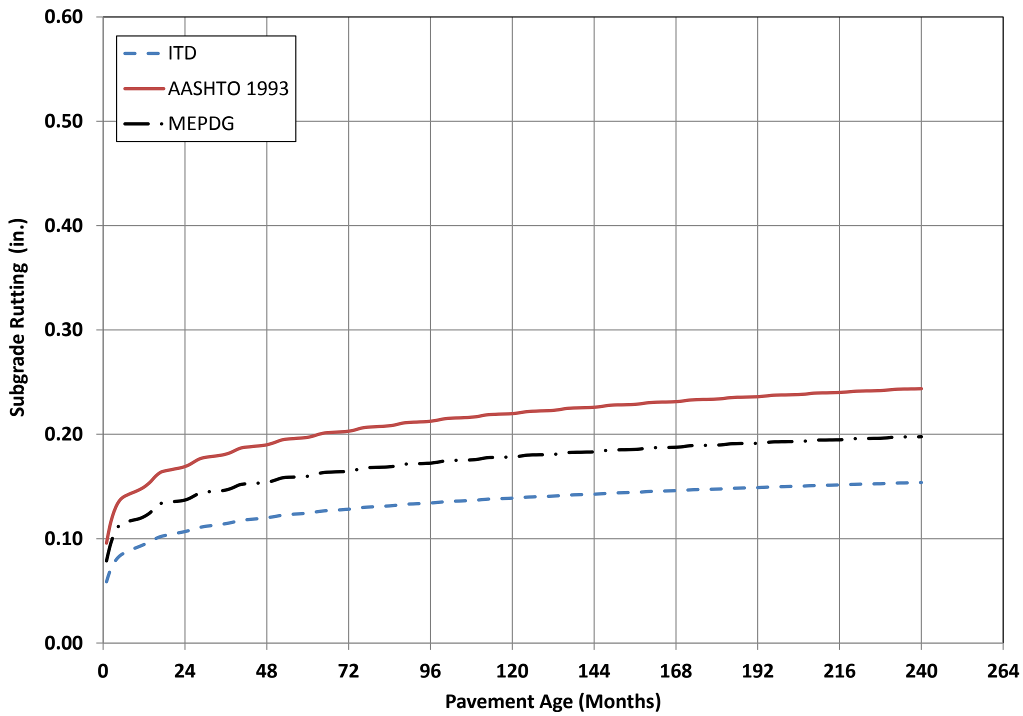


Figure 185. Comparison of the Predicted Subgrade Layer Rutting, US-95 Project

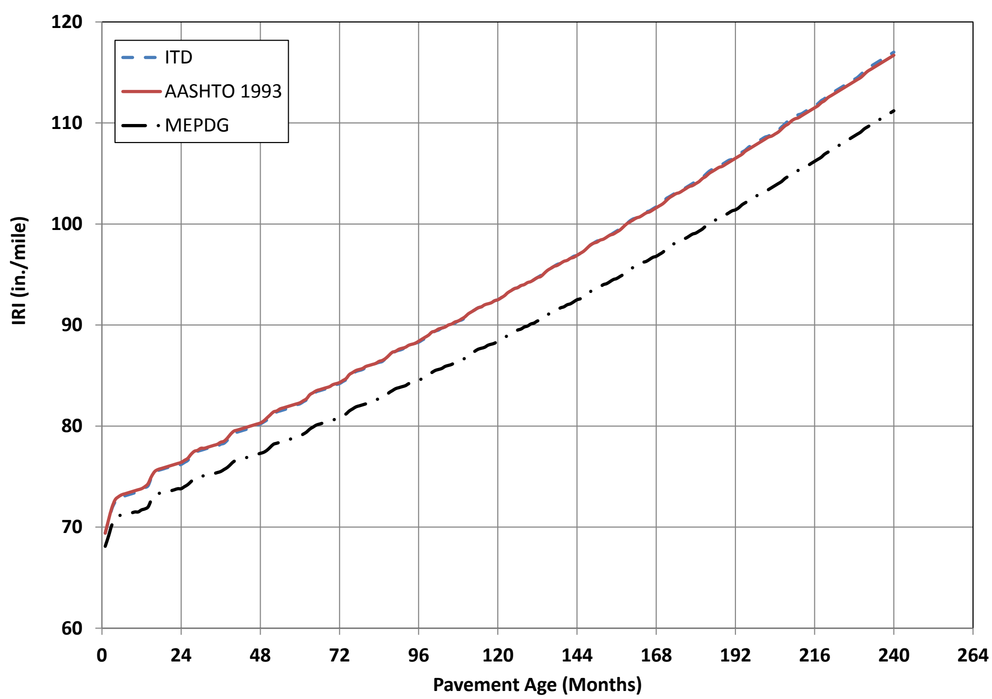


Figure 186. Comparison of the Predicted IRI, US-95 Project

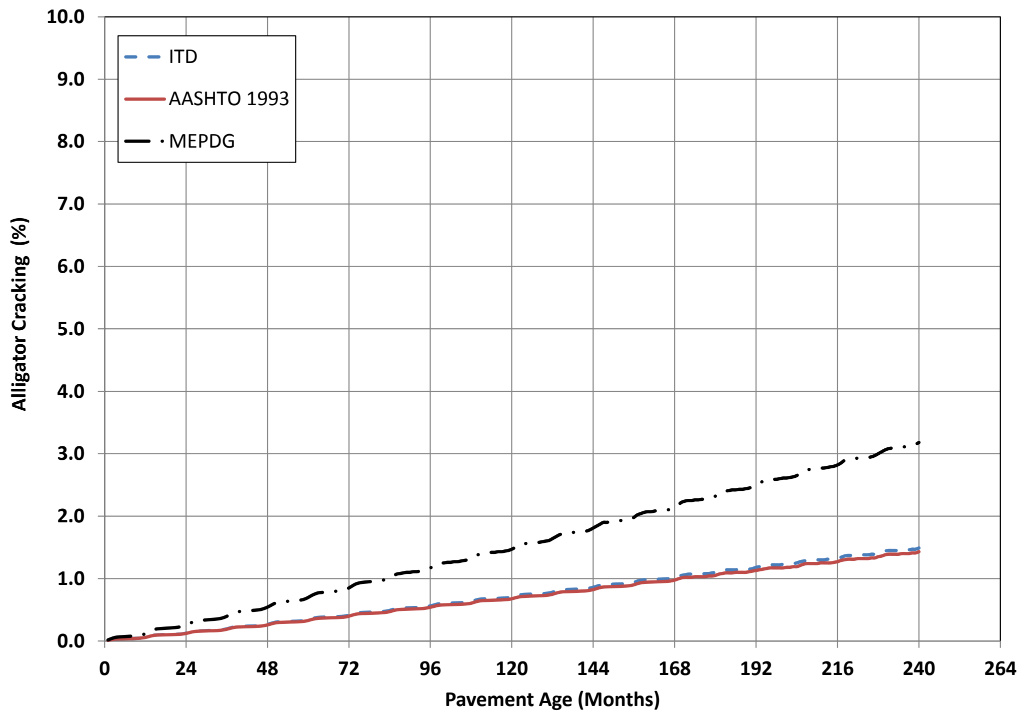


Figure 187. Comparison of the Predicted Alligator Cracking, US-93 Project

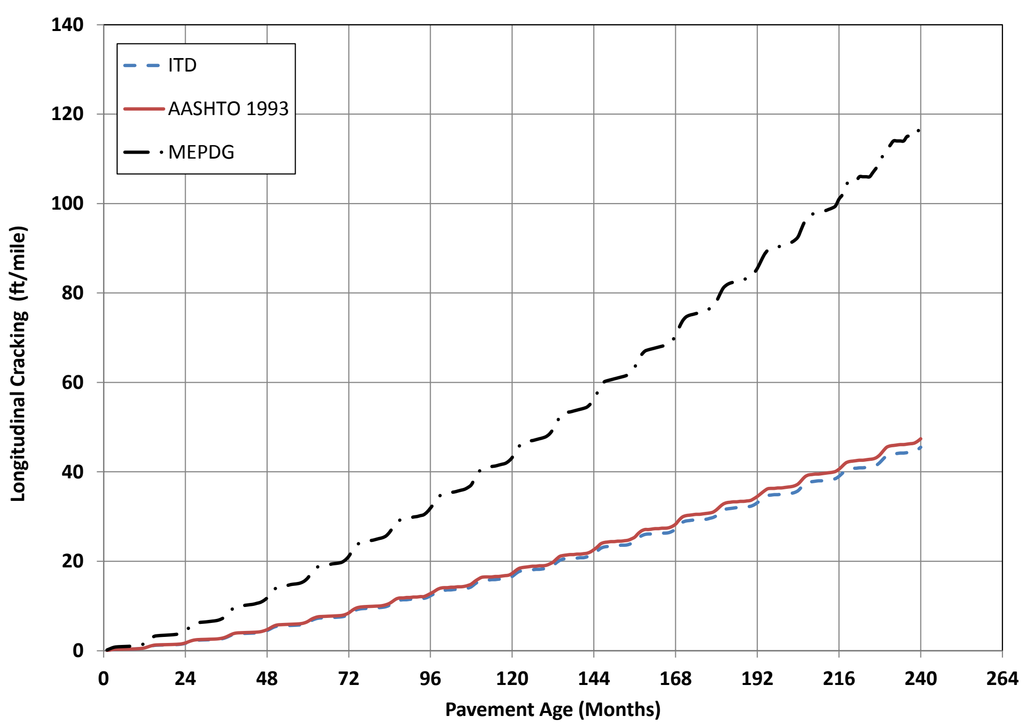


Figure 188. Comparison of the Predicted Longitudinal Cracking, US-93 Project

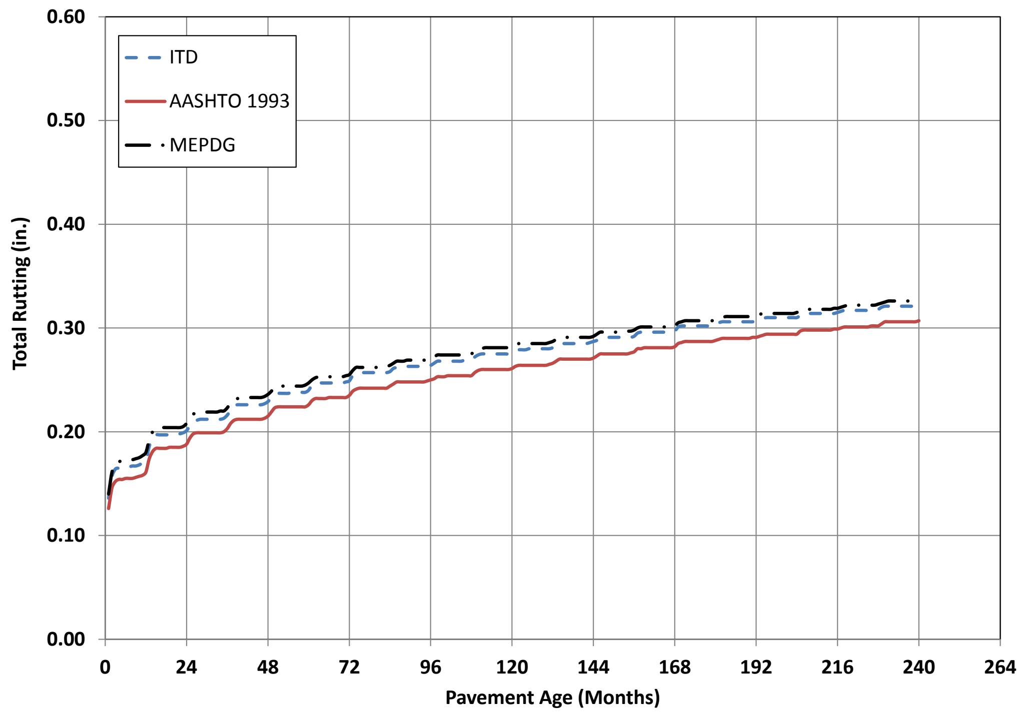


Figure 189. Comparison of the Predicted Total Rutting, US-93 Project

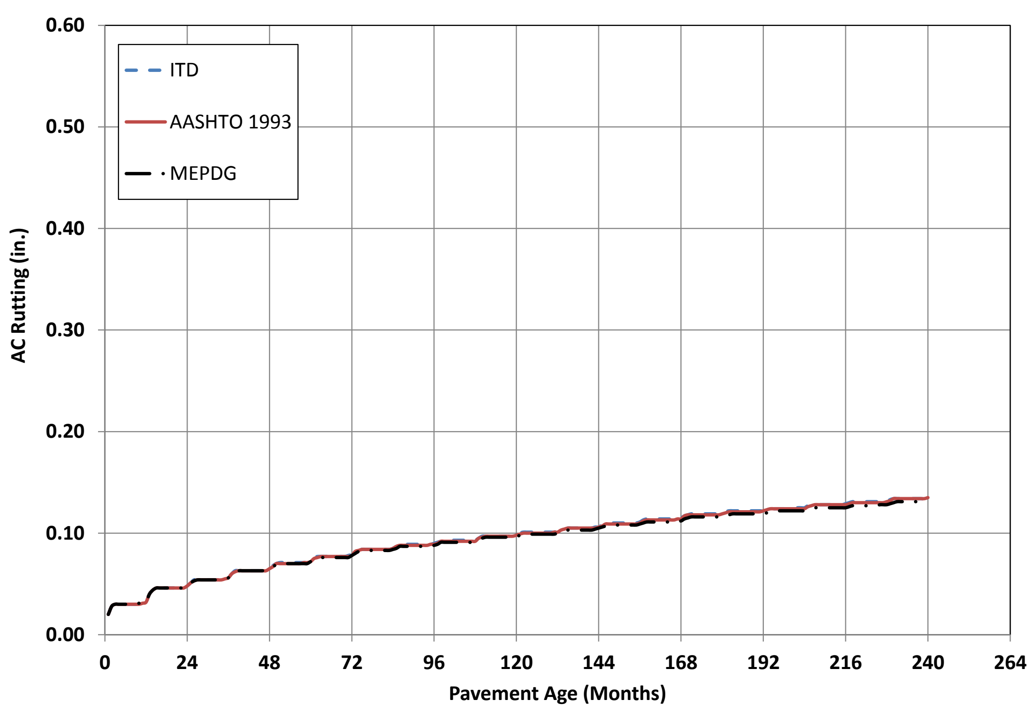


Figure 190. Comparison of the Predicted AC Layer Rutting, US-93 Project

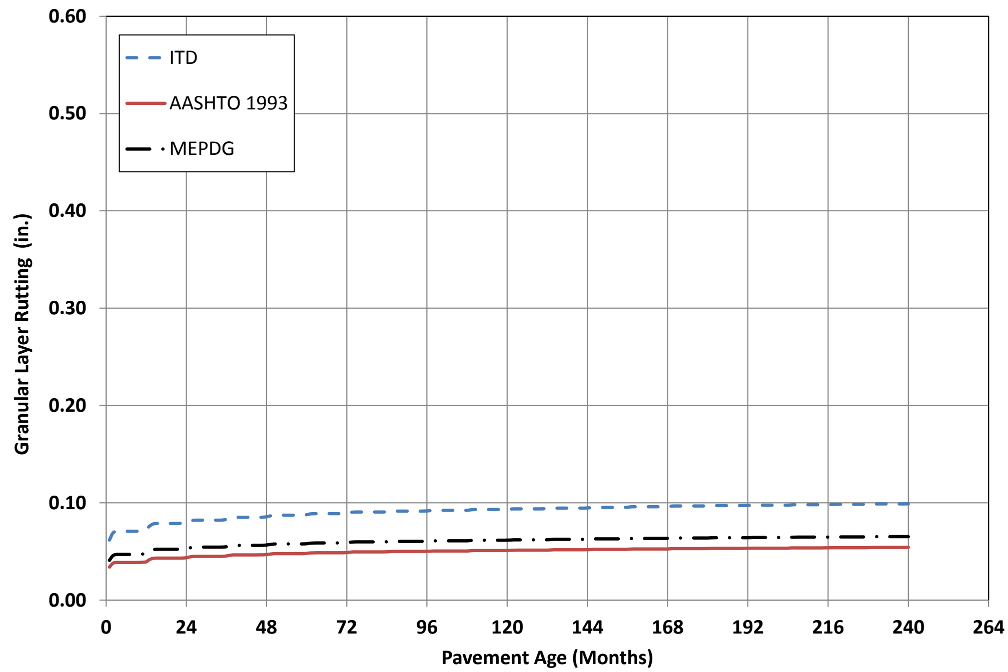


Figure 191. Comparison of the Predicted Unbound Granular Layer(s) Rutting, US-93 Project

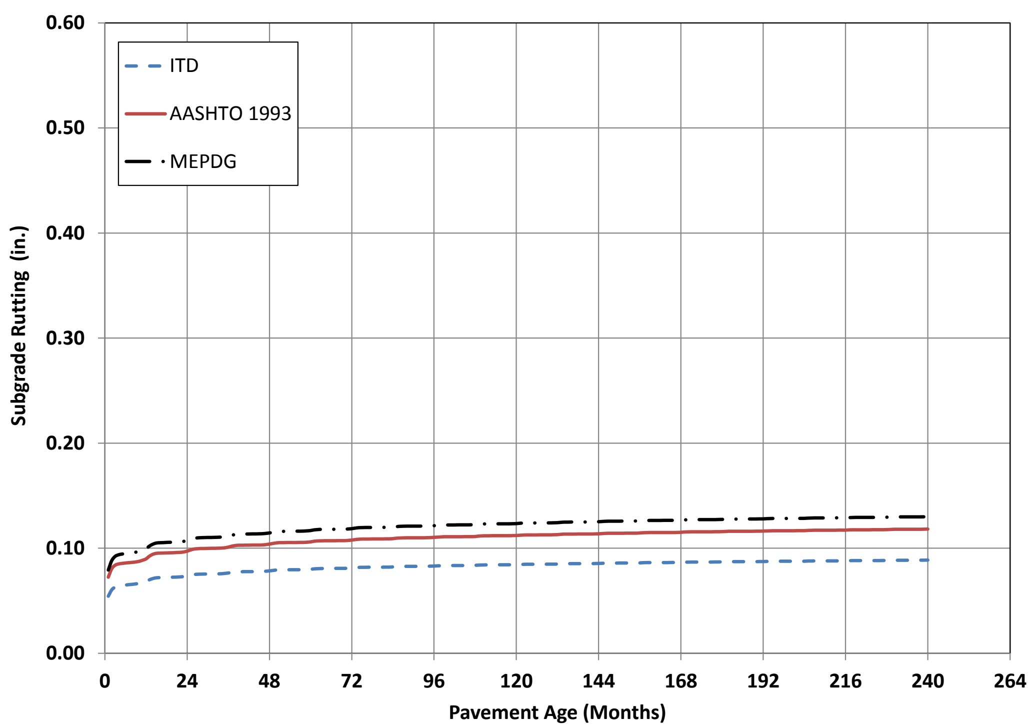


Figure 192. Comparison of the Predicted Subgrade Layer Rutting, US-93 Project

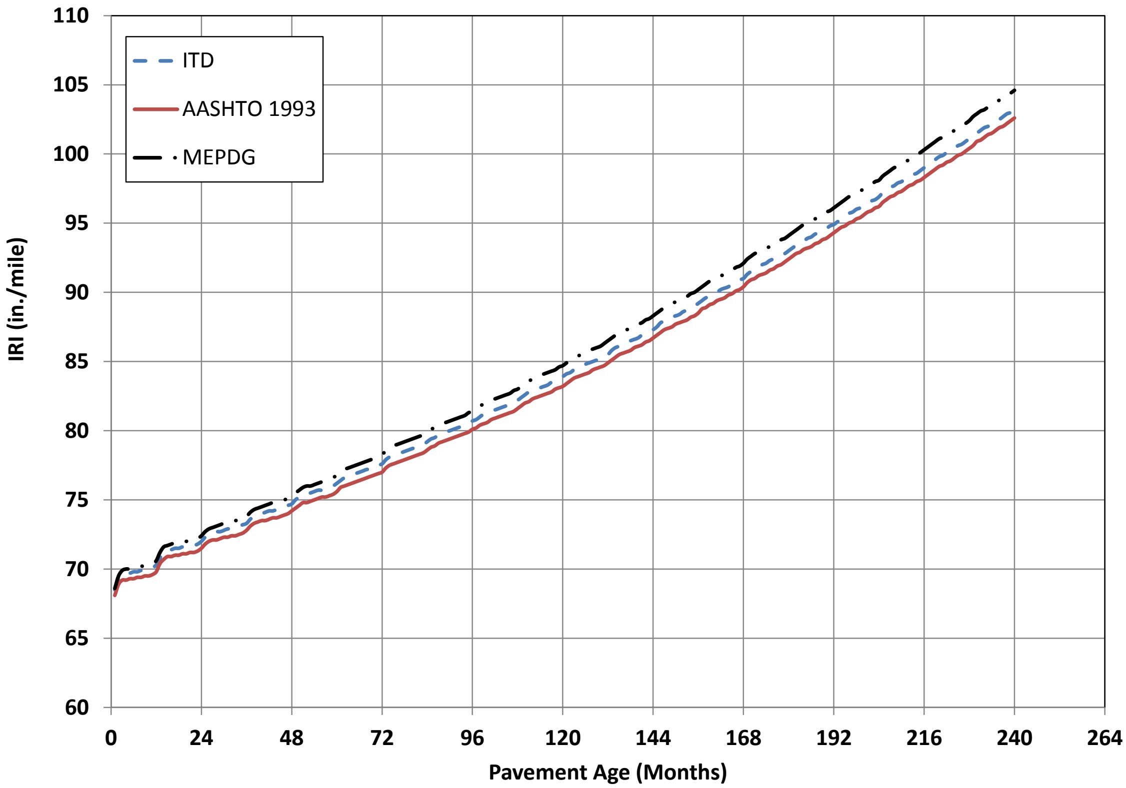


Figure 193. Comparison of the Predicted IRI, US-93 Project

Appendix D
ATR Stations AADT Database

ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT
1	1990	28475	3	1990	7733	5	1990	3623	8	1990	8134	10	1990	4077
1	1991	31213	3	1991	7801	5	1991	3700	8	1991	8596	10	1991	4403
1	1992	33224	3	1992	7956	5	1992	3846	8	1992	9306	10	1992	4951
1	1993	36983	3	1993	8164	5	1993	3871	8	1993	9789	10	1993	5237
1	1994	36922	3	1994	7677	5	1994	3993	8	1994	10199	10	1994	5539
1	1995		3	1995	7890	5	1995	3984	8	1995		10	1995	5701
1	1996		3	1996	8400	5	1996	3968	8	1996	10262	10	1996	5704
1	1997	40812	3	1997	8677	5	1997	4059	8	1997	10358	10	1997	
1	1998	42326	3	1998	9018	5	1998	4065	8	1998	10980	10	1998	6053
1	1999	42474	3	1999	9215	5	1999	4364	8	1999	11090	10	1999	6189
1	2000		3	2000	9090	5	2000		8	2000	11050	10	2000	6297
1	2001	43008	3	2001	9025	5	2001	4289	8	2001	11199	10	2001	6536
1	2002	44645	3	2002	9055	5	2002	4422	8	2002	11341	10	2002	6767
1	2003	45531	3	2003	9080	5	2003	4328	8	2003	11430	10	2003	7002
1	2004	47195	3	2004		5	2004	4325	8	2004	11724	10	2004	7190
1	2005	48877	3	2005	8871	5	2005	4415	8	2005		10	2005	7440
1	2006	50615	3	2006	9047	5	2006	4141	8	2006	11406	10	2006	7484
1	2007	50785	3	2007		5	2007	4197	8	2007		10	2007	7679
1	2008	47946	3	2008	9284	5	2008		8	2008	10489	10	2008	
1	2009	48603	3	2009	9439	5	2009	4396	8	2009	10976	10	2009	6802
2	1990	13006	4	1990	10303	6	1990	8151	9	1990	5311	11	1990	1426
2	1991	13745	4	1991	10862	6	1991	8356	9	1991	5346	11	1991	1472
2	1992	14962	4	1992	11562	6	1992	8705	9	1992	5476	11	1992	1566
2	1993	15597	4	1993	11974	6	1993	9230	9	1993	6034	11	1993	1568
2	1994	16042	4	1994	12720	6	1994	9312	9	1994	5990	11	1994	1653
2	1995	16776	4	1995	13395	6	1995		9	1995		11	1995	1687
2	1996		4	1996	13591	6	1996	9664	9	1996		11	1996	
2	1997		4	1997	14158	6	1997	10149	9	1997	7251	11	1997	1610
2	1998	18779	4	1998		6	1998	10284	9	1998	7345	11	1998	1642
2	1999	19687	4	1999	15310	6	1999	10643	9	1999	7526	11	1999	1680
2	2000	19787	4	2000	15150	6	2000	10342	9	2000	7672	11	2000	1700
2	2001	20163	4	2001	15127	6	2001	10399	9	2001	7823	11	2001	1688
2	2002	20937	4	2002	15659	6	2002	10677	9	2002	8058	11	2002	1634
2	2003	20598	4	2003	15731	6	2003	10541	9	2003	7929	11	2003	1612
2	2004		4	2004	15623	6	2004	10460	9	2004	7813	11	2004	1588
2	2005	20606	4	2005	15529	6	2005	10380	9	2005	8056	11	2005	1522
2	2006	21340	4	2006	15796	6	2006	10294	9	2006	8629	11	2006	1556
2	2007		4	2007	16041	6	2007	10158	9	2007	8527	11	2007	1644
2	2008		4	2008		6	2008		9	2008	7902	11	2008	1457
2	2009	20658	4	2009	15768	6	2009	10391	9	2009	7583	11	2009	1538

ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT
12	1990	2292	14	1990	2376	17	1990	1870	19	1990	1824	22	1990	5037
12	1991	2384	14	1991	2352	17	1991	1900	19	1991	1914	22	1991	5367
12	1992	2534	14	1992	2390	17	1992	1951	19	1992	1978	22	1992	5878
12	1993	2662	14	1993	2487	17	1993	1943	19	1993	2062	22	1993	6211
12	1994	2712	14	1994	2640	17	1994	2037	19	1994	2143	22	1994	6537
12	1995	2790	14	1995		17	1995	2073	19	1995	2091	22	1995	6870
12	1996	2716	14	1996	2897	17	1996	1985	19	1996	1969	22	1996	7081
12	1997	2819	14	1997		17	1997	1974	19	1997	2064	22	1997	7348
12	1998	2990	14	1998	3180	17	1998	1942	19	1998	2167	22	1998	7881
12	1999	3208	14	1999	3308	17	1999	2025	19	1999	2200	22	1999	8248
12	2000	3287	14	2000	3450	17	2000	2028	19	2000	2121	22	2000	8223
12	2001	3359	14	2001		17	2001	2021	19	2001	2120	22	2001	
12	2002	3428	14	2002	3613	17	2002	2139	19	2002		22	2002	
12	2003	3458	14	2003	3668	17	2003	2087	19	2003	2101	22	2003	8716
12	2004	3484	14	2004	3778	17	2004	2057	19	2004	2169	22	2004	
12	2005	3546	14	2005	3807	17	2005	1980	19	2005	2064	22	2005	8858
12	2006	3660	14	2006	3911	17	2006	1877	19	2006	2003	22	2006	9017
12	2007	3981	14	2007	3904	17	2007	1986	19	2007	2050	22	2007	9244
12	2008	3740	14	2008	3526	17	2008	1826	19	2008		22	2008	8770
12	2009	3729	14	2009	3400	17	2009	1954	19	2009	1970	22	2009	9391
13	1990	2156	15	1990	1981	18	1990	4501	21	1990	8045	24	1990	13708
13	1991	2099	15	1991	2063	18	1991	4729	21	1991	8653	24	1991	14470
13	1992	2250	15	1992	2246	18	1992	4860	21	1992	9641	24	1992	15105
13	1993	2319	15	1993	2342	18	1993	5207	21	1993	10421	24	1993	15679
13	1994	2656	15	1994	2447	18	1994	5321	21	1994	10850	24	1994	16206
13	1995	2934	15	1995		18	1995	5279	21	1995	11413	24	1995	16940
13	1996	2787	15	1996	2421	18	1996	5473	21	1996	11542	24	1996	
13	1997	2832	15	1997	2541	18	1997	5788	21	1997	11750	24	1997	18205
13	1998	2733	15	1998	2585	18	1998	6101	21	1998	12333	24	1998	19257
13	1999	2650	15	1999	2754	18	1999	6252	21	1999	13101	24	1999	20060
13	2000	2742	15	2000		18	2000	6193	21	2000		24	2000	20402
13	2001	2619	15	2001	2598	18	2001	6228	21	2001	13078	24	2001	20250
13	2002	2621	15	2002	2620	18	2002	6439	21	2002	13377	24	2002	
13	2003	2707	15	2003	2688	18	2003		21	2003	13657	24	2003	21343
13	2004	2756	15	2004	2768	18	2004	6262	21	2004	14206	24	2004	21095
13	2005	2735	15	2005	2638	18	2005	6299	21	2005	14315	24	2005	20703
13	2006	2688	15	2006	2734	18	2006	6464	21	2006	14804	24	2006	21197
13	2007	2700	15	2007	2763	18	2007	6631	21	2007	15136	24	2007	21837
13	2008	2523	15	2008	2638	18	2008	6201	21	2008		24	2008	20633
13	2009	2545	15	2009		18	2009	6376	21	2009	14074	24	2009	21945

ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT
25	1990	11362	27	1990	1080	29	1990	3211	31	1990	912	34	1990	613
25	1991	11863	27	1991	1127	29	1991	3359	31	1991	932	34	1991	621
25	1992	12788	27	1992	1225	29	1992	3574	31	1992	980	34	1992	665
25	1993	13226	27	1993	1324	29	1993	3588	31	1993	975	34	1993	708
25	1994	13332	27	1994	1344	29	1994	3750	31	1994	1081	34	1994	719
25	1995	14064	27	1995	1423	29	1995		31	1995	1128	34	1995	707
25	1996	14230	27	1996	1434	29	1996	3644	31	1996		34	1996	699
25	1997	14502	27	1997	1452	29	1997	3846	31	1997	1318	34	1997	681
25	1998	15499	27	1998	1461	29	1998	3900	31	1998	1309	34	1998	677
25	1999	16902	27	1999	1456	29	1999	3906	31	1999	1551	34	1999	
25	2000	17069	27	2000	1503	29	2000	3803	31	2000	1530	34	2000	646
25	2001	17343	27	2001	1603	29	2001		31	2001	1566	34	2001	644
25	2002	18023	27	2002	1587	29	2002	3872	31	2002	1689	34	2002	
25	2003	17976	27	2003	1617	29	2003	3801	31	2003	1711	34	2003	
25	2004	17754	27	2004	1621	29	2004	3738	31	2004		34	2004	621
25	2005	17951	27	2005	1599	29	2005	3656	31	2005	1693	34	2005	605
25	2006	18621	27	2006	1607	29	2006	3683	31	2006	1770	34	2006	609
25	2007		27	2007	1593	29	2007	3747	31	2007		34	2007	568
25	2008	17629	27	2008	1363	29	2008	3430	31	2008		34	2008	
25	2009	17985	27	2009	1375	29	2009	3455	31	2009	1747	34	2009	594
26	1990	2679	28	1990	1108	30	1990	4279	32	1991	2354	35	1990	1004
26	1991	2848	28	1991	1173	30	1991	4373	32	1992	2478	35	1991	1011
26	1992	3019	28	1992	1238	30	1992	4600	32	1993	2533	35	1992	1096
26	1993	3124	28	1993	1176	30	1993	4638	32	1994	2841	35	1993	1071
26	1994	3362	28	1994	1248	30	1994	5011	32	1995		35	1994	1143
26	1995	3507	28	1995	1246	30	1995	5192	32	1996	2951	35	1995	1132
26	1996	3453	28	1996	1235	30	1996	5258	32	1997	2953	35	1996	1152
26	1997	3359	28	1997	1202	30	1997	5558	32	1998	2921	35	1997	1216
26	1998	3659	28	1998	1224	30	1998	5787	32	1999		35	1998	1264
26	1999	3747	28	1999	1267	30	1999	6137	32	2000	3069	35	1999	1282
26	2000	3633	28	2000	1278	30	2000	6077	32	2001		35	2000	1261
26	2001	3672	28	2001	1348	30	2001	6230	32	2002		35	2001	1318
26	2002	4052	28	2002	1396	30	2002		32	2003	3161	35	2002	1322
26	2003	4048	28	2003	1284	30	2003		32	2004	3232	35	2003	1357
26	2004	4111	28	2004	1247	30	2004	6575	32	2005	3120	35	2004	1276
26	2005	4080	28	2005	1218	30	2005	6786	32	2006	3180	35	2005	1211
26	2006	4103	28	2006	1184	30	2006	6987	32	2007	3359	35	2006	1228
26	2007	4279	28	2007	1170	30	2007	6941	32	2008	3164	35	2007	1280
26	2008	3798	28	2008	1056	30	2008	6582	32	2009		35	2008	1243
26	2009	3711	28	2009	1077	30	2009	6871				35	2009	1299

ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT
25	1990	11362	27	1990	1080	29	1990	3211	31	1990	912	34	1990	613
25	1991	11863	27	1991	1127	29	1991	3359	31	1991	932	34	1991	621
25	1992	12788	27	1992	1225	29	1992	3574	31	1992	980	34	1992	665
25	1993	13226	27	1993	1324	29	1993	3588	31	1993	975	34	1993	708
25	1994	13332	27	1994	1344	29	1994	3750	31	1994	1081	34	1994	719
25	1995	14064	27	1995	1423	29	1995		31	1995	1128	34	1995	707
25	1996	14230	27	1996	1434	29	1996	3644	31	1996		34	1996	699
25	1997	14502	27	1997	1452	29	1997	3846	31	1997	1318	34	1997	681
25	1998	15499	27	1998	1461	29	1998	3900	31	1998	1309	34	1998	677
25	1999	16902	27	1999	1456	29	1999	3906	31	1999	1551	34	1999	
25	2000	17069	27	2000	1503	29	2000	3803	31	2000	1530	34	2000	646
25	2001	17343	27	2001	1603	29	2001		31	2001	1566	34	2001	644
25	2002	18023	27	2002	1587	29	2002	3872	31	2002	1689	34	2002	
25	2003	17976	27	2003	1617	29	2003	3801	31	2003	1711	34	2003	
25	2004	17754	27	2004	1621	29	2004	3738	31	2004		34	2004	621
25	2005	17951	27	2005	1599	29	2005	3656	31	2005	1693	34	2005	605
25	2006	18621	27	2006	1607	29	2006	3683	31	2006	1770	34	2006	609
25	2007		27	2007	1593	29	2007	3747	31	2007		34	2007	568
25	2008	17629	27	2008	1363	29	2008	3430	31	2008		34	2008	
25	2009	17985	27	2009	1375	29	2009	3455	31	2009	1747	34	2009	594
26	1990	2679	28	1990	1108	30	1990	4279	32	1991	2354	35	1990	1004
26	1991	2848	28	1991	1173	30	1991	4373	32	1992	2478	35	1991	1011
26	1992	3019	28	1992	1238	30	1992	4600	32	1993	2533	35	1992	1096
26	1993	3124	28	1993	1176	30	1993	4638	32	1994	2841	35	1993	1071
26	1994	3362	28	1994	1248	30	1994	5011	32	1995		35	1994	1143
26	1995	3507	28	1995	1246	30	1995	5192	32	1996	2951	35	1995	1132
26	1996	3453	28	1996	1235	30	1996	5258	32	1997	2953	35	1996	1152
26	1997	3359	28	1997	1202	30	1997	5558	32	1998	2921	35	1997	1216
26	1998	3659	28	1998	1224	30	1998	5787	32	1999		35	1998	1264
26	1999	3747	28	1999	1267	30	1999	6137	32	2000	3069	35	1999	1282
26	2000	3633	28	2000	1278	30	2000	6077	32	2001		35	2000	1261
26	2001	3672	28	2001	1348	30	2001	6230	32	2002		35	2001	1318
26	2002	4052	28	2002	1396	30	2002		32	2003	3161	35	2002	1322
26	2003	4048	28	2003	1284	30	2003		32	2004	3232	35	2003	1357
26	2004	4111	28	2004	1247	30	2004	6575	32	2005	3120	35	2004	1276
26	2005	4080	28	2005	1218	30	2005	6786	32	2006	3180	35	2005	1211
26	2006	4103	28	2006	1184	30	2006	6987	32	2007	3359	35	2006	1228
26	2007	4279	28	2007	1170	30	2007	6941	32	2008	3164	35	2007	1280
26	2008	3798	28	2008	1056	30	2008	6582	32	2009		35	2008	1243
26	2009	3711	28	2009	1077	30	2009	6871				35	2009	1299

ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT
36	1990	1246	39	1990	2194	41	1990	5261	43	1990	2484	45	1990	568
36	1991	1277	39	1991	2266	41	1991	5665	43	1991	2683	45	1991	556
36	1992	1362	39	1992	2425	41	1992	6239	43	1992	2928	45	1992	558
36	1993	1383	39	1993	2507	41	1993	6527	43	1993	2898	45	1993	525
36	1994	1345	39	1994	2607	41	1994	7359	43	1994	2858	45	1994	580
36	1995	1363	39	1995		41	1995	7712	43	1995	3095	45	1995	552
36	1996	1366	39	1996	2560	41	1996		43	1996	2996	45	1996	515
36	1997	1402	39	1997	2610	41	1997	7533	43	1997	2909	45	1997	513
36	1998	1462	39	1998	2723	41	1998	7865	43	1998	3023	45	1998	510
36	1999	1487	39	1999	2724	41	1999	7966	43	1999	3005	45	1999	501
36	2000	1507	39	2000	2714	41	2000	8034	43	2000	3079	45	2000	503
36	2001	1518	39	2001	2788	41	2001	8149	43	2001	3077	45	2001	517
36	2002	1915	39	2002	2908	41	2002	8445	43	2002	3208	45	2002	525
36	2003	1883	39	2003	2943	41	2003	8574	43	2003	3393	45	2003	566
36	2004	1428	39	2004	2969	41	2004	8772	43	2004	3780	45	2004	566
36	2005	1420	39	2005	2941	41	2005	8966	43	2005	4138	45	2005	571
36	2006	1391	39	2006	2960	41	2006	9284	43	2006	4324	45	2006	558
36	2007	1486	39	2007	2972	41	2007	9498	43	2007	4325	45	2007	522
36	2008	1359	39	2008	2790	41	2008		43	2008	3579	45	2008	482
36	2009	1248	39	2009	2927	41	2009		43	2009	3495	45	2009	489
38	1990	1364	40	1990	3896	42	1990	1682	44	1990	3616	47	1990	6235
38	1991	1386	40	1991	4133	42	1991	1792	44	1991	3882	47	1991	6051
38	1992	1503	40	1992	4520	42	1992	1964	44	1992		47	1992	6613
38	1993	1442	40	1993	4760	42	1993	2087	44	1993		47	1993	6609
38	1994	1478	40	1994	5214	42	1994	2177	44	1994	4337	47	1994	7062
38	1995		40	1995		42	1995		44	1995	4743	47	1995	7011
38	1996	1357	40	1996	5429	42	1996	2124	44	1996	4924	47	1996	
38	1997	1417	40	1997	5768	42	1997	2100	44	1997	4916	47	1997	7418
38	1998	1385	40	1998		42	1998	2248	44	1998	5217	47	1998	7512
38	1999	1422	40	1999	6135	42	1999	2308	44	1999	5435	47	1999	7607
38	2000	1400	40	2000	6386	42	2000	2203	44	2000	5294	47	2000	7201
38	2001	1432	40	2001	6218	42	2001	2227	44	2001	5414	47	2001	7131
38	2002	1524	40	2002	6260	42	2002	2322	44	2002	5676	47	2002	7454
38	2003	1547	40	2003	6433	42	2003	2338	44	2003	5758	47	2003	7560
38	2004	1576	40	2004	6640	42	2004	2388	44	2004	5905	47	2004	7724
38	2005	1651	40	2005	6771	42	2005	2567	44	2005	5876	47	2005	7744
38	2006	1650	40	2006	6787	42	2006	2801	44	2006	5991	47	2006	7719
38	2007	1615	40	2007	6950	42	2007	2840	44	2007	6010	47	2007	7807
38	2008	1457	40	2008	6662	42	2008	2653	44	2008	5590	47	2008	7021
38	2009	1513	40	2009	6795	42	2009		44	2009	5912	47	2009	6882

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48	1990		50	1990	1018	53	1990	2238	55	1990	445	57	1990	2234
48	1991		50	1991	1043	53	1991	2121	55	1991	452	57	1991	2428
48	1992	24080	50	1992	1067	53	1992	2253	55	1992	483	57	1992	2404
48	1993	25480	50	1993	1023	53	1993	2529	55	1993	467	57	1993	2391
48	1994	27789	50	1994	1091	53	1994	2543	55	1994	534	57	1994	2470
48	1995		50	1995	1094	53	1995	2883	55	1995	541	57	1995	2179
48	1996	26267	50	1996	1061	53	1996	2862	55	1996	530	57	1996	1963
48	1997	26981	50	1997	1098	53	1997	3128	55	1997	547	57	1997	2011
48	1998		50	1998		53	1998	3097	55	1998	547	57	1998	1957
48	1999	28978	50	1999		53	1999	3055	55	1999	550	57	1999	2085
48	2000	29702	50	2000	1113	53	2000	3106	55	2000	525	57	2000	
48	2001	30815	50	2001	1121	53	2001	3257	55	2001	540	57	2001	
48	2002	29798	50	2002		53	2002	3196	55	2002	534	57	2002	2139
48	2003	30809	50	2003	1144	53	2003	3143	55	2003	556	57	2003	2227
48	2004	32823	50	2004	1125	53	2004	3157	55	2004	546	57	2004	2269
48	2005	34293	50	2005	1083	53	2005	3153	55	2005	540	57	2005	2307
48	2006	32992	50	2006		53	2006	3266	55	2006	534	57	2006	2207
48	2007	30847	50	2007	1018	53	2007	3181	55	2007	564	57	2007	2359
48	2008	29405	50	2008	952	53	2008	2845	55	2008		57	2008	2111
48	2009		50	2009	966	53	2009	2853	55	2009	555	57	2009	
49	1990	1614	51	1990	9775	54	1990	1462	56	1990	432	58	1990	434
49	1991	1604	51	1991	10040	54	1991	1459	56	1991	442	58	1991	444
49	1992	1780	51	1992	11043	54	1992	1513	56	1992	462	58	1992	476
49	1993	1742	51	1993	11356	54	1993	1521	56	1993	448	58	1993	458
49	1994	1850	51	1994	11888	54	1994	1719	56	1994	466	58	1994	502
49	1995	1875	51	1995	12321	54	1995		56	1995	489	58	1995	
49	1996	1762	51	1996	12398	54	1996	1759	56	1996	475	58	1996	479
49	1997	1740	51	1997	13416	54	1997	1813	56	1997	511	58	1997	499
49	1998	1866	51	1998	14413	54	1998	1823	56	1998	499	58	1998	519
49	1999		51	1999	14607	54	1999	1871	56	1999	524	58	1999	538
49	2000	1783	51	2000	14535	54	2000		56	2000	540	58	2000	540
49	2001	1879	51	2001	15035	54	2001	1892	56	2001	522	58	2001	551
49	2002		51	2002	16381	54	2002	1864	56	2002	529	58	2002	
49	2003	1967	51	2003	17536	54	2003	1907	56	2003	555	58	2003	
49	2004	1979	51	2004	18349	54	2004	1880	56	2004	544	58	2004	542
49	2005	1942	51	2005	18546	54	2005	1850	56	2005	508	58	2005	524
49	2006	2008	51	2006	19259	54	2006	1827	56	2006	466	58	2006	506
49	2007	2038	51	2007	19998	54	2007	1903	56	2007	495	58	2007	506
49	2008	1845	51	2008	18549	54	2008	1674	56	2008	463	58	2008	466
49	2009	1931	51	2009	19350	54	2009	1768	56	2009	475	58	2009	502

ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT
59	1990	1324	61	1990	3419	68	1990	8931	80	1990	4142	82	1990	620
59	1991	1352	61	1991	3549	68	1991	8675	80	1991	4413	82	1991	623
59	1992	1455	61	1992	3685	68	1992	8830	80	1992	4653	82	1992	629
59	1993	1444	61	1993	3813	68	1993	7607	80	1993	4906	82	1993	615
59	1994	1575	61	1994	3839	68	1994	9034	80	1994	5266	82	1994	814
59	1995	1746	61	1995	4013	68	1995	10911	80	1995	5471	82	1995	
59	1996	1606	61	1996	3998	68	1996	11313	80	1996	5622	82	1996	789
59	1997	1676	61	1997	4295	68	1997	11605	80	1997	3798	82	1997	748
59	1998	1688	61	1998	4397	68	1998	11894	80	1998	3722	82	1998	743
59	1999	1794	61	1999	4509	68	1999	12336	80	1999		82	1999	692
59	2000	1761	61	2000		68	2000	12687	80	2000		82	2000	671
59	2001	1890	61	2001	4572	68	2001	12756	80	2001	3993	82	2001	651
59	2002	1951	61	2002	4753	68	2002	13008	80	2002	3964	82	2002	659
59	2003	1949	61	2003	4916	68	2003	13405	80	2003	4101	82	2003	695
59	2004	1976	61	2004	4875	68	2004		80	2004	4115	82	2004	689
59	2005	2041	61	2005	4770	68	2005	13457	80	2005	4147	82	2005	691
59	2006	2202	61	2006	4834	68	2006	13091	80	2006	4187	82	2006	719
59	2007	2597	61	2007	4923	68	2007	13258	80	2007	4253	82	2007	770
59	2008	2164	61	2008	4695	68	2008	12514	80	2008	4379	82	2008	732
59	2009	2053	61	2009	4886	68	2009		80	2009	4351	82	2009	694
60	1990	3772	67	1990	9026	72	1990	4471	81	1990	350	83	1990	354
60	1991	3909	67	1991	9492	72	1991	4671	81	1991	343	83	1991	467
60	1992	4200	67	1992	9895	72	1992	5035	81	1992	441	83	1992	568
60	1993	4175	67	1993	10176	72	1993	5058	81	1993	405	83	1993	593
60	1994	4099	67	1994	10684	72	1994	5546	81	1994	385	83	1994	689
60	1995		67	1995		72	1995		81	1995		83	1995	746
60	1996	4439	67	1996		72	1996		81	1996	413	83	1996	692
60	1997	4571	67	1997	11492	72	1997	5651	81	1997		83	1997	557
60	1998	4647	67	1998	12079	72	1998		81	1998	373	83	1998	639
60	1999	4812	67	1999	12297	72	1999	5915	81	1999		83	1999	686
60	2000	4889	67	2000	12154	72	2000	6070	81	2000	416	83	2000	685
60	2001		67	2001	12063	72	2001	6077	81	2001	463	83	2001	720
60	2002	4678	67	2002	12300	72	2002		81	2002	435	83	2002	726
60	2003	4663	67	2003		72	2003	6370	81	2003		83	2003	720
60	2004	4519	67	2004	11966	72	2004	6364	81	2004	444	83	2004	709
60	2005	4450	67	2005	11974	72	2005	6392	81	2005	427	83	2005	691
60	2006	4421	67	2006	12070	72	2006	6503	81	2006	414	83	2006	658
60	2007	4580	67	2007	12215	72	2007	6657	81	2007	434	83	2007	677
60	2008		67	2008	11461	72	2008	6101	81	2008		83	2008	623
60	2009	4453	67	2009	11763	72	2009	6320	81	2009	436	83	2009	663

ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT
84	1990	599	123	1990	11145	148	1990	1426	193	1990	1426	202	1990	17930
84	1991	685	123	1991	11343	148	1991	1472	193	1991	1472	202	1991	20068
84	1992	742	123	1992	12626	148	1992	1566	193	1992	1566	202	1992	19441
84	1993	751	123	1993	12841	148	1993	1568	193	1993	1568	202	1993	16029
84	1994	769	123	1994	13562	148	1994	1653	193	1994	1653	202	1994	14893
84	1995		123	1995	14169	148	1995	1687	193	1995	1687	202	1995	14880
84	1996		123	1996	14649	148	1996		193	1996		202	1996	
84	1997	688	123	1997	15248	148	1997	1610	193	1997	1610	202	1997	14862
84	1998	679	123	1998		148	1998	1642	193	1998	1642	202	1998	15146
84	1999	697	123	1999		148	1999	1680	193	1999	1680	202	1999	15166
84	2000	689	123	2000	16545	148	2000	1700	193	2000	1700	202	2000	15539
84	2001	716	123	2001	16944	148	2001	1688	193	2001	1688	202	2001	14862
84	2002	719	123	2002	17759	148	2002	1634	193	2002	1634	202	2002	14251
84	2003	702	123	2003		148	2003	1612	193	2003	1612	202	2003	14202
84	2004	683	123	2004	18002	148	2004	1588	193	2004	1588	202	2004	13095
84	2005	633	123	2005	18214	148	2005	1522	193	2005	1522	202	2005	12718
84	2006	608	123	2006	18963	148	2006	1556	193	2006	1556	202	2006	12596
84	2007	594	123	2007	19108	148	2007	1644	193	2007	1644	202	2007	
84	2008	527	123	2008	17499	148	2008	1457	193	2008	1457	202	2008	12249
84	2009	562	123	2009	18310	148	2009	1538	193	2009	1538	202	2009	11932
85	1990	613	138	1990	1426	169	1990	1426	200	1990	25592	205	1990	15144
85	1991	651	138	1991	1472	169	1991	1472	200	1991	23714	205	1991	16009
85	1992	687	138	1992	1566	169	1992	1566	200	1992	23519	205	1992	15740
85	1993	687	138	1993	1568	169	1993	1568	200	1993	20539	205	1993	15282
85	1994	701	138	1994	1653	169	1994	1653	200	1994	21642	205	1994	16016
85	1995		138	1995	1687	169	1995	1687	200	1995		205	1995	
85	1996		138	1996		169	1996		200	1996	24627	205	1996	15869
85	1997	659	138	1997	1610	169	1997	1610	200	1997	24584	205	1997	15779
85	1998	669	138	1998	1642	169	1998	1642	200	1998	24860	205	1998	15864
85	1999	678	138	1999	1680	169	1999	1680	200	1999	24932	205	1999	16102
85	2000	659	138	2000	1700	169	2000	1700	200	2000	24673	205	2000	16352
85	2001	677	138	2001	1688	169	2001	1688	200	2001	24715	205	2001	
85	2002	667	138	2002	1634	169	2002	1634	200	2002	23781	205	2002	
85	2003	668	138	2003	1612	169	2003	1612	200	2003	23762	205	2003	16261
85	2004	645	138	2004	1588	169	2004	1588	200	2004	23681	205	2004	16450
85	2005	599	138	2005	1522	169	2005	1522	200	2005	23397	205	2005	16707
85	2006	570	138	2006	1556	169	2006	1556	200	2006		205	2006	17739
85	2007	561	138	2007	1644	169	2007	1644	200	2007	23843	205	2007	17329
85	2008	493	138	2008	1457	169	2008	1457	200	2008	22847	205	2008	16128
85	2009		138	2009	1538	169	2009	1538	200	2009	21913	205	2009	16271

ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT
209	1990	32404	212	1990	17795	217	1990	24757	221	1990	13199	224	1990	1116
209	1991	31135	212	1991		217	1991	25488	221	1991	13260	224	1991	1361
209	1992	16983	212	1992		217	1992	28675	221	1992	15632	224	1992	
209	1993	15298	212	1993	19731	217	1993	28352	221	1993	15359	224	1993	
209	1994	15905	212	1994	20820	217	1994	31480	221	1994	15932	224	1994	2292
209	1995	14817	212	1995	21614	217	1995		221	1995	16939	224	1995	2407
209	1996		212	1996	20991	217	1996		221	1996		224	1996	2477
209	1997	13759	212	1997	20811	217	1997	32955	221	1997	18570	224	1997	2428
209	1998	12453	212	1998	20298	217	1998	35856	221	1998	19051	224	1998	2600
209	1999	13858	212	1999	20909	217	1999	35119	221	1999	20542	224	1999	2742
209	2000	13719	212	2000	20895	217	2000	33938	221	2000	21335	224	2000	3100
209	2001	11960	212	2001	20027	217	2001	34064	221	2001	21665	224	2001	3492
209	2002	11356	212	2002	19176	217	2002	34654	221	2002	27233	224	2002	3862
209	2003	10788	212	2003	19641	217	2003	36968	221	2003	21520	224	2003	4457
209	2004	10686	212	2004	19598	217	2004	37244	221	2004	21891	224	2004	5017
209	2005		212	2005	19541	217	2005	37698	221	2005	23528	224	2005	5708
209	2006	12592	212	2006		217	2006	38586	221	2006	27455	224	2006	7101
209	2007	12534	212	2007	19550	217	2007		221	2007		224	2007	
209	2008	12139	212	2008	18901	217	2008	36430	221	2008		224	2008	6106
209	2009	11986	212	2009	18591	217	2009	34415	221	2009	19127	224	2009	6110
210	1990	29972	213	1990	31652	218	1990	18978	223	1990	3359	225	1990	1294
210	1991	14621	213	1991	35425	218	1991	20302	223	1991	3086	225	1991	1785
210	1992	26804	213	1992	37769	218	1992	20327	223	1992	4333	225	1992	1613
210	1993	19105	213	1993	41097	218	1993	20719	223	1993	4011	225	1993	1641
210	1994	19852	213	1994	42607	218	1994	22359	223	1994	4228	225	1994	1721
210	1995		213	1995	43309	218	1995	25125	223	1995	4468	225	1995	2095
210	1996	19790	213	1996		218	1996	22717	223	1996	4782	225	1996	2247
210	1997	19626	213	1997	44856	218	1997	20890	223	1997	4701	225	1997	2990
210	1998	19552	213	1998		218	1998	20545	223	1998	5066	225	1998	2793
210	1999	18833	213	1999	44690	218	1999	21541	223	1999		225	1999	
210	2000	17845	213	2000	42987	218	2000	25981	223	2000	5598	225	2000	3757
210	2001	17410	213	2001	31802	218	2001	25883	223	2001	5838	225	2001	4141
210	2002	16728	213	2002	27657	218	2002	27113	223	2002	5533	225	2002	4567
210	2003	13968	213	2003	27861	218	2003	26971	223	2003	6384	225	2003	4887
210	2004	15272	213	2004	26243	218	2004		223	2004	7134	225	2004	4967
210	2005	15614	213	2005		218	2005	26480	223	2005	7972	225	2005	5106
210	2006	15665	213	2006	26471	218	2006	26396	223	2006	10265	225	2006	5927
210	2007	15572	213	2007	26004	218	2007	26307	223	2007	7117	225	2007	6332
210	2008	14891	213	2008	24867	218	2008	24988	223	2008	6823	225	2008	5943
210	2009	14717	213	2009	24324	218	2009	25173	223	2009	6101	225	2009	6758

ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT	ATR	Year	AADT
226	1990	7155	248	1990	7966	252	1990	5270	254	1990	17145	256	1990	13942
226	1991		248	1991	8351	252	1991	5341	254	1991	17275	256	1991	13289
226	1992		248	1992	8725	252	1992	5618	254	1992	17990	256	1992	13952
226	1993	8252	248	1993	9055	252	1993	5605	254	1993	18029	256	1993	14145
226	1994	9006	248	1994	9375	252	1994	6030	254	1994	18383	256	1994	14305
226	1995	9194	248	1995	9675	252	1995	6050	254	1995		256	1995	15233
226	1996	9569	248	1996	9826	252	1996	5957	254	1996		256	1996	14611
226	1997	10139	248	1997	10103	252	1997	5949	254	1997	18217	256	1997	
226	1998	11272	248	1998	10777	252	1998	5795	254	1998	17942	256	1998	14135
226	1999	12171	248	1999	10885	252	1999	5862	254	1999	18162	256	1999	
226	2000	12414	248	2000	11051	252	2000	5805	254	2000		256	2000	
226	2001	12315	248	2001	11233	252	2001	5285	254	2001	17660	256	2001	
226	2002	12474	248	2002	10880	252	2002	5191	254	2002		256	2002	
226	2003	11651	248	2003	10675	252	2003	5230	254	2003	17423	256	2003	14390
226	2004	11667	248	2004	10965	252	2004	5218	254	2004	16947	256	2004	14163
226	2005	12345	248	2005	11007	252	2005	5320	254	2005	17852	256	2005	
226	2006	13274	248	2006	10967	252	2006	5409	254	2006		256	2006	13749
226	2007	10435	248	2007	11120	252	2007	5309	254	2007	17214	256	2007	13651
226	2008	9600	248	2008	10543	252	2008	5376	254	2008		256	2008	13399
226	2009	9428	248	2009	10711	252	2009	5335	254	2009	16777	256	2009	13444
227	1990	9704	250	1990	15841	253	1990	3253	255	1990	7335	257	1990	2235
227	1991	9913	250	1991	15191	253	1991	3279	255	1991	8336	257	1991	2300
227	1992	10432	250	1992	16107	253	1992	3508	255	1992	8737	257	1992	2516
227	1993	11010	250	1993	16970	253	1993	3402	255	1993	9024	257	1993	2620
227	1994	11219	250	1994	17993	253	1994		255	1994	9301	257	1994	2589
227	1995	11700	250	1995		253	1995		255	1995		257	1995	
227	1996	11768	250	1996	18390	253	1996		255	1996		257	1996	
227	1997	12080	250	1997	18295	253	1997	4206	255	1997		257	1997	2516
227	1998	12194	250	1998	19112	253	1998	4235	255	1998	10308	257	1998	2534
227	1999	12259	250	1999	19692	253	1999	4188	255	1999	10234	257	1999	2616
227	2000		250	2000	20017	253	2000	4086	255	2000	10044	257	2000	2599
227	2001	12158	250	2001	19071	253	2001	4159	255	2001	10055	257	2001	2591
227	2002	12706	250	2002	18623	253	2002	4243	255	2002		257	2002	2487
227	2003		250	2003	17938	253	2003	4079	255	2003		257	2003	
227	2004		250	2004	18797	253	2004	4057	255	2004		257	2004	2470
227	2005	12869	250	2005	18432	253	2005	4090	255	2005	9666	257	2005	2345
227	2006	13658	250	2006	19107	253	2006	4055	255	2006		257	2006	2452
227	2007	12968	250	2007	20418	253	2007	3922	255	2007	9350	257	2007	
227	2008	12375	250	2008	21087	253	2008	3843	255	2008		257	2008	2188
227	2009	11920	250	2009	21979	253	2009	3744	255	2009	8943	257	2009	2283

ATR	Year	AADT
264	1990	46263
264	1991	50538
264	1992	53057
264	1993	52405
264	1994	52325
264	1995	
264	1996	
264	1997	62617
264	1998	66817
264	1999	68958
264	2000	71662
264	2001	72205
264	2002	71836
264	2003	71632
264	2004	72735
264	2005	73992
264	2006	
264	2007	
264	2008	66672
264	2009	
272	1990	3538
272	1991	3624
272	1992	3548
272	1993	3779
272	1994	4197
272	1995	4407
272	1996	4672
272	1997	4851
272	1998	5485
272	1999	5538
272	2000	5661
272	2001	5867
272	2002	6316
272	2003	6406
272	2004	6937
272	2005	8187
272	2006	9136
272	2007	9549
272	2008	8547
272	2009	8567