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Development of DARWin-ME Design Guideline for Louisiana Pavement Design

by

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16. Abstract

The AASHTOWare Pavement ME^{TM} Design is the next generation of AASHTO pavement design software, which builds upon the newly developed NCHRP Mechanistic-Empirical Pavement Design Guide (MEPDG). Pavement ME^{TM} reflects a major change in the methods and procedures engineers use to design pavement structure and represents the most current advancements in pavement design. In preparation for DOTD to adopt the new design guide, there is an urgent need to evaluate the MEPDG pavement design software based on typical Louisiana pavement structures and local conditions.

This study selected a total of 162 projects (pavement sections) from the existing DOTD highway network for the evaluation of MEPDG pavement design, local calibration, and validation of Pavement ME in Louisiana. The selected projects consisted of flexible pavements with five types of base (asphalt concrete base, rubblized PCC base, crushed stone or recycled PCC base, soil cement base, and stabilized base with a stone interlayer), rigid pavements with three types of base (unbound granular base, stabilized base, and asphalt mixture blanket), and HMA overlay on top of existing flexible pavements. Pavement design information including structure, materials, and traffic were retrieved from multiple network-level data sources at DOTD. A Louisiana default input strategy of Pavement ME that reflects Louisiana's condition and practice was developed from results of sensitivity analysis. In addition, based on a consensus distress survey and pavement management system (PMS) distress triggers, the design reliability and performance criteria were established for different highway classes in Louisiana. The predicted performance from the Pavement ME was then compared with the corresponding measured performance retrieved from PMS.

The analysis results indicate that the Pavement ME's nationally-calibrated distress models generally under-predict alligator cracking, but over-predict rutting for DOTD's flexible pavement types. For rigid pavements, Pavement ME over-predicts slab cracking but under-predicts joint faulting. For those nationally-calibrated distress models that showed constant bias and large variation, local calibration was carried out against the performance data retrieved from PMS. After the local calibration, the Pavement ME designs were verified by additional projects outside of the evaluation projects' pool.

Based on the results of this study, an implementation guideline document was prepared. The document contains all necessary design input information and calibration coefficients for DOTD to use the latest MEPDG software on a day-to-day basis for design and analysis of new and rehabilitated pavement structures in Louisiana.

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ABSTRACT

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This study selected a total of 162 projects (pavement sections) from the existing DOTD highway network for the evaluation of MEPDG pavement design, local calibration and validation of Pavement ME in Louisiana. The selected projects consisted of flexible pavements with five types of base (asphalt concrete base, rubblized Portland cement concrete base, crushed stone or recycled PCC base, soil cement base, and stabilized base with a stone interlayer), rigid pavements with three types of base (unbound granular base, stabilized base, and asphalt mixture blanket), and HMA overlay on top of existing flexible pavements. Pavement design information including structure, materials, and traffic were retrieved from multiple network-level data sources at DOTD. A Louisiana default input strategy of Pavement ME that reflects Louisiana's condition and practice was developed from results of sensitivity analysis. In addition, based on a consensus distress survey and pavement management system (PMS) distress triggers, the design reliability and performance criteria were established for different highway classes in Louisiana. The predicted performance from the Pavement ME was then compared with the corresponding measured performance retrieved from PMS.

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Based on the research results of this study, an implementation guideline document was prepared. The document contains all necessary design input information and calibration coefficients for DOTD to use the latest MEPDG software (Pavement ME) on a day-to-day basis for design and analysis of new and rehabilitated pavement structures in Louisiana.

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IMPLEMENTATION STATEMENT

The primary result from this study is the *Implementation Guidelines of AASHTOWare® Pavement ME™ for Louisiana* (Appendix A). Conclusions on model evaluation, calibration, and validation are summarized in the document, as well as the recommended Louisiana default input strategy and design criteria. It is expected that pavement design engineers at DOTD will be able to follow the guidelines and conduct parallel design using the current design method and Pavement ME on real projects. This period of in-house practice will (1) provide engineers more experience to the new procedure, (2) identify and troubleshoot any design issues, and (3) revise and modify any default design inputs or calibration coefficients to advance the implementation process.

Engineers should be aware that the *Implementation Guidelines* is not a comprehensive pavement design manual but rather an up-to-date implementation guide that moves Pavement ME from research to practice in Louisiana. If a large discrepancy is found between the two design methods, careful engineering judgment should be applied to develop a reasonable design. Such cases should also be sent to LTRC for future update, modification and improvement of the *Implementation Guidelines*.

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INTRODUCTION

Problem Statement

The Louisiana Department of Transportation and Development (DOTD) has the responsibility of administrating and maintaining more than 17,000 miles of state, U.S., and interstate highway pavement structures. Currently, DOTD is using the 1993 AASHTO Pavement Design Guide and its associated design software, DARWin 3.1 in all pavement design activities *[1]*. The basis of the 1993 AASHTO Pavement Design Guide is the empirical equations developed from the AASHO Road Test. Due to the empirical characteristics and other limitations, this design guide cannot accurately predict the performance of designed pavement structures.

Pavement ME™ is the next generation of AASHTOWare® pavement design software, which builds upon the newly developed NCHRP Mechanistic-Empirical Pavement Design Guide *[2]*. Pavement ME™ reflects a major change in the methods and procedures engineers use to design pavement structure and represents the most current advancements in pavement design. This design approach, by taking advantage of the advances in material mechanics, axle-load spectra and climate data for predicting pavement performance, can result in smoother, longer-lasting and more cost-effective pavements.

According to AASHTO, the 1993 AASHTO Pavement Design Guide was sunset and all technical support on DARWin 3.1 ceased on July 1, 2012. To follow the national trend, DOTD plans to adopt the new pavement design package – Pavement ME in the next few years. However, Pavement ME is fundamentally different in many aspects from DARWin 3.1 and requires a large number of design inputs, most of which are never required in the 1993 Design Guide. Therefore, to support the DOTD implementation of Pavement ME and provide related pavement design guidelines for design engineers, there is an urgent need to evaluate the M-E pavement design software based on typical Louisiana pavement structures and local conditions.

Literature Review

From AASHO Road Test to Pavement ME™

NCHRP Synthesis 457 conducted a survey among U.S., Puerto Rico, and Canadian state highway and provincial transportation agencies on their current pavement design practices *[3]*. The survey results found that AASHTO empirical methods are by far the most utilized, with 48 of the responding agencies using the AASHTO Interim Guide for Design of Pavement Structures (AASHTO 1972) through the AASHTO Guide for Design of Pavement Structures, with 1998 Supplement (AASHTO 1998) *[3]*. All those versions are based on empirical performance equations developed using 1950's AASHO Road Test data. The 1986 and 1993 AASHTO Guides contain some refinements in materials input parameters, design reliability, and empirical procedures for rehabilitation design, but the empirical nature of these design guides have not been changed. Many serious limitations exist in these design guides such as *[2]*:

- Heavy truck traffic design volume levels have increased tremendously since the design of pavements used in the AASHO Road Test in the 1960s. Truck configurations have also greatly changed from the 1960s.
- Because the AASHO Road Test was conducted at one specific geographic location, it is impossible to address the effects of different climatic conditions on pavement performance.
- Only one type of subgrade was used for all test sections at the Road Test. Only one hot mix asphalt (HMA) mixture and one PCC (Portland cement concrete) mixture were used at the Road Test.

In recognition of the limitations of the AASHTO Guide, the Joint Task Force on Pavements (JTFP) initiated an effort in 1996 to develop a design guide based as fully as possible on mechanistic principles. The Mechanistic Empirical Pavement Design Guide (MEPDG) is the end result of that goal.

The original research version of the MEPDG software (version 0.7) was first released in July 2004. The software was updated under other National Cooperative Highway Research Program (NCHRP) projects several times (version 0.8 released in November 2005, version 0.9 released in July 2006, version 1.0 released in April 2007, and version 1.1 released in September 2009). Version 1.0 of the MEPDG was adopted as an interim AASHTO pavement design procedure in 2007. A *Manual of Practice* was published in 2008 to assist state highway agencies to implement the M-E design method *[4]*. In April 2011, AASHTO released the commercial software under the name of DARWin-ME to be consistent with the original AASHTO 1993 Design software (DARWin). DARWin is the acronym of Pavement Design, Analysis and Rehabilitation for Windows. In 2013, AASHTO underwent another rebranding of AASHTOWare and released the software under a new name Pavement ME™ for mechanistic empirical pavement design. As of the publication of this final report, the latest version of M-E design software is AASHTO Pavement ME version 2.0.

The new mechanistic empirical pavement design procedure takes advantage of advances in material characterization, axle load spectra, and climatic models to predict pavement performance; hence it provides multiple benefits *[2]*. For example,

- The new Design Guide directly considers many material properties and design factors that were not considered in the 1993 Guide. For instance, MEPDG uses dynamic modulus to characterize hot mix asphalt instead of using layer coefficient "a."
- Models in the new Design Guide were developed and validated using data from the Long Term Pavement Performance (LTPP) program. The 1993 Guide, by contrast, was based on the 2-year-long AASHO Road Test. The more current LTPP data are expected to provide more accurate predictions, resulting in economic benefits to highway agencies.
- The application of axle load spectra and detailed parameters to characterize traffic in MEPDG offers plenty of freedom to model different traffic conditions and future changes such as increased loads, high tire pressures and multiple axles.
- The M-E design procedure provides tools to evaluate the effect of material variations on pavement performance.

Difference between the 1993 Design and the New M-E Design

The 1993 AASHTO Pavement Design Guide primarily relies on two empirical equations developed from the AASHO Road Test. The traffic input is the 18 kip equivalent single axle load (ESAL). When performing a flexible pavement design, another empirical parameter- the structural layer coefficient must be pre-determined or assigned for each material used in the design. Environmental effects on pavement design are taken into consideration through materials inputs, such as changing the resilient modulus of subgrade and applying a drainage coefficient. In terms of pavement performance, an empirical pavement index value - the pavement serviceability index (PSI) is considered as the fundamental design criterion in the 1993 Design Guide *[1]*. The general design inputs required by the 1993 AASHTO Pavement Design Guide are summarized in Figure 1. Pavement design using this approach is to determine a set of required pavement layer thicknesses using empirical equations so that the loss of pavement serviceability index (ΔPSI) within the design period is lower than a pre-determined design ΔPSI value. Due to its empirical nature and many other limitations, the 1993 Design Guide does not provide any capability of predicting traffic-induced pavement distresses.

Flexible pavement	Rigid pavement
• Traffic (to be converted into a design ESAL) o AADT o Percent truck in the design direction o Percent truck in the design lane o Vehicle class distribution o Growth factor • Structure o Layer thickness • Material o Structural layer coefficient o Drainage coefficient o Subgrade resilient modulus	• Traffic (to be converted into a design ESAL) o AADT o Percent truck in the design direction o Percent truck in the design lane o Vehicle class distribution o Growth factor • Structure o Layer thickness • Material o Modulus of rupture for the PCC o Elastic Modulus of the PCC o Load transfer coefficient o Drainage coefficient o Modulus of the subgrade reaction

Figure 1

Design inputs required by the 1993 AASHTO Guide

Different from the 1993 Guide, the new M-E method and its accompanying software are based on comprehensive mechanistic models and empirical transfer functions which were calibrated using comprehensive data from the Long Term Pavement Performance program. Pavement ME provides the ability to design three categories of pavement structures, namely new flexible pavement, new rigid pavement, and pavement rehabilitation. Detailed pavement structure types under each category are shown in Figure 2.

New Flexible Pavement	New Rigid Pavement	Pavement Rehabilitation
• Conventional flexible pavement • Deep strength flexible pavement • Full-depth AC pavement • Semi-rigid pavement	• Jointed plain concrete pavement (JPCP) • Continuously reinforced concrete pavement (CRCP)	• AC over AC • AC over JPCP or CRCP • AC over fractured JPCP or CRCP • JPCP restoration • Bounded PCC over JPCP

Figure 2

Pavement structure types supported by Pavement ME

The design criteria used in Pavement ME are pavement performance indicators at the end of the service life. Two sets of different pavement performance indicators as shown in Table 1 are used in Pavement ME depending on the type of pavement surface (asphalt concrete [AC] or PCC).

Table 1

Performance indicators considered in Pavement ME

AC-surfaced pavements		PCC-surfaced pavements	
Performance indicator	**Unit**	**Performance indicator**	**Unit**
Alligator cracking	% [1]	Transverse cracking (for JPCP)	% [2]
Longitudinal cracking	ft./mi.	Mean transverse faulting (for JPCP)	in.
Transverse (thermal) cracking	ft./mi.	CRCP Punchouts	#/mi.
Reflective cracking[3]	%	IRI	in./mi.
Rutting	in.		
IRI	in./mi.		

Note: [1]percent of total lane area; [2]percent of slabs cracked; [3]only for AC over stabilized base and AC overlays

Compared to the 1993 AASHTO Pavement Design Guide, Pavement ME is a complex pavement design and analysis system. The effects of climate, traffic, and materials on the pavement performance are considered in a more rational way. In Pavement ME, pavement performance is predicted via mechanistic response models and empirical damage models. Specifically, mechanistic response models calculate structural responses (stress, strain, or deflection) under monotonic loads, and damage models predict pavement performance throughout the designated service life based on the calculated mechanistic response.

Different types of mechanistic response models are used in Pavement ME. For a new flexible pavement design, structural responses are calculated based on either an elastic layer theory program, JULEA, or a finite element program, DCS2D. DCS2D is used only when the Level-1 unbound materials inputs are provided. The mechanistic model for new rigid pavement design in Pavement ME is a neural-network (NN) model developed based on a finite element program, ISLAB2000. For pavement rehabilitation designs, JULEA or DCS2D response model is used for AC overlay of existing AC and AC overlay of fractured PCC pavements, and the NN response model is used for other types of rehabilitated pavements.

Empirical damage models in Pavement ME software are also called transfer functions. Each of these transfer functions is used to predict a certain type of pavement distress based on the structural response at certain critical locations. For example, bottom-up fatigue cracking of AC in a flexible pavement is calculated based on the horizontal tensile resilient strain at the bottom of the AC layers. These transfer functions have been calibrated based on observed pavement performance from a large number of test sites in North America (so-called nationally calibrated models). Details of these mechanistic and empirical models that are used in Pavement ME are presented in Appendix B.

Another big difference between the 1993 Design Guide and the new M-E procedure is the required inputs. As shown in Figure 1, the 1993 Guide requires a handful of inputs. But the Pavement ME requires hundreds of inputs to conduct a comprehensive analysis. The following presents an introduction of the required input parameters by the Pavement ME. Details can be referred to MEPDG documents and the AASHTO *Manual of Practice [2],[4]*.

Traffic Inputs. Instead of using ESALs as in the 1993 AASHTO Guide, full axle load spectra are used to characterize traffic loading in Pavement ME. Ideally, axle load data should be collected from Weigh-In-Motion (WIM) stations installed in the same segment of the designed highway. For agencies that do not have the resources to collect accurate WIM data, a set of default traffic inputs are provided in Pavement ME based on axle load data from nearly 200 WIM stations included in the LTPP program. The detailed traffic inputs are presented in Figure 3.

Roadway Specific Inputs	From WIM Data	Not From WIM Data
• Initial two-way average annual daily truck traffic (AADTT) • Number of lanes • Percent trucks in the design direction • Percent trucks in the design lane • Operational speed • Growth of truck traffic	• Axle load distribution for each axle type • Normalized truck volume distribution • Axle configurations (axle spacing and wheelbase) • Monthly distribution factors • Hourly distribution factors	• Dual tire spacing • Tire pressure • Axles per truck • Lateral wander of axle loads

Figure 3

Traffic inputs required by Pavement ME

Climate Inputs. The analysis of pavement performance in Pavement ME requires detailed climate data including hourly temperature, precipitation, wind speed, relative humidity, percentage sunshine, etc. These data are used to predict the temperature and the moisture content in each of the pavement layers as well as to provide some of the inputs for smoothness prediction models *[5]*.

All climate information is available from weather station data included in the Pavement ME software. Users can simply input the longitude, latitude and altitude of the project and select data from one or more nearby weather stations. The depth of the groundwater table is also a climate input required by Pavement ME. This information, if not available from field tests, can be obtained from the National Water Information System (http://wdr.water.usgs.gov/nwisgmap/) provided by the United States Geology Survey (USGS).

Structure Inputs. Structure inputs for new pavement designs include thickness and material type of each layer. For rehabilitation of existing pavements, structure inputs also include the condition of the existing pavement such as rutting, cracking and backcalculated modulus from deflection testing.

Material Inputs. A summary of major material inputs for Pavement ME is shown in Table 2. Pavement ME requires that the input material properties should reflect in-situ conditions right after construction. In the design stage, however, most of these parameters have to be estimated from laboratory/field tests or experience. The MEPDG adopts a hierarchical approach for materials inputs, which is based on the philosophy that the level of engineering effort exerted for characterizing the paving materials and the foundation should be consistent with the relative importance, size, and cost of the design project.

The hierarchical approach includes three levels of input. Level 1 inputs provide the highest level of accuracy and would have the lowest level of uncertainty or error. It would typically be used for designing heavily trafficked pavements or wherever there is dire safety or economic consequences of early failure. Level 1 material inputs require laboratory or field testing, site-specific axle load spectra data collection, or nondestructive deflection testing. Level 2 inputs provide an intermediate level of accuracy and would be close to the typical procedure used with earlier editions of the AASHTO Guide. This level could be used when resources or testing equipment are not available for tests required for Level 1. Level 3 inputs provide the lowest level of accuracy. This level might be used for designs where there are minimal consequences of early failure (e.g., lower volume roads). Inputs would be user-selected values or typical average for the region *[2]*.

Table 2
Major material input considerations in Pavement ME *[2]*

Materials Category	Materials Inputs Required		
	For critical response computations	**For distress/transfer functions**	**For climatic modeling**
Hot-mix asphalt materials	• Dynamic modulus (E*) • Poisson's ratio	• Tensile strength, creep compliance, coefficient of thermal expansion	• Surface shortwave absorptivity, thermal conductivity, heat capacity of HMA • Asphalt binder viscosity
PCC materials	• Static modulus of elasticity • Poisson's ratio • Unit weight • Coefficient of thermal expansion	• Modulus of rupture, split tensile strength, compressive strength, cement type, cement content, water/cement ratio, ultimate shrinkage, amount of reversible shrinkage	• Surface shortwave absorptivity, thermal conductivity, heat capacity of PCC
Chemically stabilized materials	• Elastic modulus, resilient modulus for lime stabilized soil • Poisson's ratio • Unit weight	• Minimum resilient modulus, modulus of rupture, erodibility	• Thermal conductivity and heat capacity of PCC
Unbound base/subbase and subgrade materials	• Seasonally adjusted resilient modulus • Poisson's ratio • Unit weight • Coefficient of lateral pressure	• Gradation parameters and base erodibility	• Plasticity index, gradation parameters, effective grain sizes, specific gravity, saturated hydraulic conductivity, optimum moisture contents, parameters to define the soil water characteristic curve
Recycled concrete materials	• Resilient modulus • Poisson's ratio	• Base erodibility	• Thermal conductivity and heat capacity
Recycled hot asphalt mix	Treated same as hot-mix asphalt surface course		
Recycled cold asphalt mix	Treated same as hot-mix asphalt base course		
Cold recycled asphalt pavement	Treated same as granular materials with no moisture sensitivity		
Bedrock	• Elastic modulus • Poisson's ratio • Unit weight	None	None

Since the first release of MEPDG in 2004, a large number of studies have been conducted by state highway agencies and universities to help in acclimating the M-E design procedure to local conditions. Commonly addressed topics include sensitivity analysis, characterizing materials per the M-E requirements, developing statewide default load spectra, development of local input strategy or input catalog, evaluating the M-E design software to local conditions, local calibration, and parallel design using both the M-E method and the current in-use design method. A comprehensive literature review on topics that are related to this study was conducted. Summary and findings from this effort follow.

Sensitivity Analyses of Input Parameters

Due to the vast amount of input parameters required by the M-E design procedure, it is of interest to know which parameters are more influential to the predicted pavement performance. Such information is beneficial to design engineers in (1) adjusting design items to meet the design criteria, (2) planning data collection activities, and (3) developing default input strategy for state and local highway agencies. The influence of individual input parameters to the predicted pavement performance is often investigated through sensitivity analyses.

Many researchers have investigated the sensitivity of predicted pavement performance to design inputs using the research version of the M-E design software (MEPDG) *[6-9]*. Due to the large amount of inputs involved, most researchers considered only the local sensitivity where input parameters were varied one at a time. In a local sensitivity analysis, the interactions between input parameters are ignored and conclusions are largely influenced by the variation range applied in the analysis. Buck et al. performed a full-factorial sensitivity analysis (involved 3072 runs of the design software) to investigate the influence of 10 design variables *[6]*. Orobio and Zaniewski performed a sampling based sensitivity analysis on flexible pavement materials inputs. In their study, the Latin Hypercube Sampling technique was utilized to reduce the number of software runs; standardized regression coefficients and Gaussian stochastic process were used to categorize relative importance of the materials inputs *[9]*. Effective binder content, as-built air voids, Poisson's ratio, surface shortwave absorption of asphalt layers, and resilient modulus of subgrade were found to have a significant effect on the performance of two typical flexible pavement structures.

NCHRP supported a comprehensive study in which global sensitivity analyses were performed for five pavement types under five climate conditions and three traffic levels using MEPDG software *[10]*. Depending on the base case, approximately 25 to 35 design inputs were evaluated. Correlations among design inputs (e.g., between PCC elastic modulus and modulus of rupture) were also considered in the study. In total over 41,000 MEPDG runs were performed for the global sensitivity analysis. The study confirmed many findings that were identified in one-at-a-time sensitivity analyses. For example, (1) HMA properties (e.g. dynamic modulus and thickness) are very sensitive to flexible pavement performance; (2) little or no thermal cracking was predicted when using the correct binder grade for the climate; (3) jointed portland cement concrete pavements (JPCP) are sensitive to slab width, PCC strength and stiffness, PCC thickness, coefficient of thermal expansion and joint spacing. Surprisingly the study also found that pavement performance were quite sensitive to some parameters that had not been recognized before such as (1) flexible pavement performance were sensitive to Poisson's ratio and HMA unit weight, and (2) rigid pavement performance were sensitive to PCC unit weight.

Michigan conducted a sensitivity analysis on pavement rehabilitation models in 2013 *[11]*. The study used both the one-at-a-time method and a global sensitivity analysis. In general, the study found that overlay thickness and HMA volumetrics are the most significant inputs for HMA overlays. The interaction between overlay air voids and existing pavement thickness significantly impacts all performance measures among HMA rehabilitation options.

It should be noted that the result of any sensitivity analysis is associated with the pavement structure investigated. For example, pavement rutting may be more sensitive to the subgrade modulus in a thinner pavement structure than in a pavement with thicker AC and base course layers. Therefore, sensitivity analysis should be conducted based on typical pavement structures and materials in each state agency.

Local Evaluation and Calibration of the New M-E Design Guide

Since the first release of MEPDG in 2004, many state highway agencies have sponsored or conducted research studies to locally evaluate/calibrate the new M-E Design Guide *[8, 12-24]*. Since local conditions differ significantly among each state, the conclusions drawn from local evaluation/calibration studies are not always the same.

Flexible Pavements. For the rutting model, it is commonly found that MEPDG over-predicts total rutting *[6, 13, 16, 19]*. It is difficult to tell in which layer(s) the permanent deformation is over-estimated because the permanent deformation in an individual layer is rarely measured. Many states attributed this phenomenon to an over-estimation of deformation in unbound layers. A study in Minnesota further pointed out that the predicted deformation in the unbound layers developed in the first month is unreasonably high *[19]*. In fact, a research project sponsored by NCHRP is underway to enhance models of unbound base and subgrade in Pavement ME *[25]*. Regarding the possible influence from HMA input levels, data from FHWA's accelerated loading facility (ALF) found that the rutting predicted by Level 3 analysis with the MEPDG software were generally higher than the rutting measured on the ALF lanes but were in the same ballpark. The rutting predicted by Level 1 analysis, however, was significantly higher than the rutting measured on the ALF lanes *[26]*. The study explained that the main reason might be because the MEPDG permanent deformation model was calibrated with stiffness predicted by use of the MEPDG stiffness equation rather than with values of E* from the simple performance tester.

For fatigue alligator cracking, studies generally found that MEPDG under-predicts alligator cracking *[17, 27, 28]*. Researchers also attributed this discrepancy to the difference of cracking definition between MEPDG and local pavement management system (PMS) and the difficulty of measuring alligator cracking in the field *[29]*. For example, North Carolina did not directly measure the distress area but rates each section between 0 and 10 based on the severity of cracking *[13]*. Arizona only measures the cracking area of the first 1,000 ft^2 on the right travel lane forward from each mile-post and uses it to represent the whole mile; however, the ADOT PMS database does not distinguish between different types of cracks such as alligator, longitudinal or transverse *[29]*.

In terms of the IRI model, studies predominately showed that predicted IRI matched relatively well with measured IRI *[15, 17, 30]*. However, it is noted that the IRI model may need to be calibrated after the calibration of cracking and rutting model because the IRI model is a function of cracking, rutting and site factor. Li et al. found that the IRI model cannot be calibrated due to a software bug in version 1.0 *[15]*. No similar case was reported on any later versions of the software in the literature.

Some studies found a poor prediction power and a high standard error inherent to the longitudinal cracking model *[12, 18]*. It is expected that the longitudinal cracking model will be revised in later versions of the M-E design software *[12]*. In recognition of this, the longitudinal model was not recommended for use in making design decisions in Montana until new models were developed. Run-time issues with the design software were also identified. A number of issues of the software in analyzing semi-rigid pavements were pointed out by a study in Minnesota *[19]*. For example, the degradation model for cement treated layer and the reflective cracking model in AC overlays are not implemented by the software properly.

A previous study (07-6P) conducted at LTRC evaluated the MEPDG software for a number of typical flexible pavements in Louisiana and provided a recommended input strategy (including traffic, climate, and material inputs) for flexible pavement design in Louisiana *[31]*.

Rigid Pavements. The rigid pavement design in the MEPDG has also been investigated by many researchers, but the results are mixed without any unanimous conclusions.

For the transverse cracking model, Mallela et al. conducted a study for Ohio DOT using LTPP data *[8]*. It was found that the MEPDG prediction generally agreed well with the field measured transverse cracking, faulting, and IRI. This conclusion was confirmed by Missouri *[21]*. However, Washington found that the default 1-37A software over-predicted the percent of cracked slabs and calibration coefficients had to be developed *[32]*. A recent study in Iowa using PMS data also found that the nationally calibrated DARWin-ME over-predicted transverse cracking to a significant amount *[33]*. Bustos et al. also reported an over-prediction of slab cracking for Argentina *[34]*. Nevertheless, one common finding widely reported was that measured cracking in the field was very low (many data points were zero) *[21, 28]*.

Regarding the joint faulting model, a good match between predictions and measurements were observed in Ohio *[8]* and Colorado *[28]*. Missouri found that the MEPDG over-predicted faulting for JPCPs that are either not doweled or have long joint spacings (> 20 ft.) *[21]*. On the contrary, the Iowa study found that the nationally calibrated DARWin-ME under-predicted joint faulting *[33]*.

For the IRI model, almost all studies found it to be adequate for local agencies without further calibration *[28]*. Iowa reported an overestimation of IRI from the DARWin-ME software *[33]*.

In terms of data preparation for evaluating rigid pavements, Missouri used a combined database of LTPP and PMS *[21]*. Field transverse cracking (in percent of cracked slabs) was obtained from the distress maps or videos collected during the pavement condition survey in order to get an accurate estimate. Montana and Colorado made use of both LTPP and PMS data *[12], [28]*. Manual distress surveys were conducted for cracking and faulting. The recent Iowa study only applied PMS data *[33]*. In addition, the study introduced a method on how to adjust calibration coefficients based on sensitivity index (S_{ijk}). The method could reduce the number of trial-and-error runs of the M-E software.

Pavement Rehabilitation. Compared to the amount of studies on new flexible and rigid pavements, not many efforts were devoted to evaluate pavement rehabilitation using the new M-E method. Iowa conducted a preliminary analysis based on six projects which were resurfaced between 1991 and 1994 *[16]*. It was found that there were differences between MEPDG model predictions and actual longitudinal cracking values observed in HMA overlaid pavement sections. The MEPDG model under-estimated rutting in HMA over JPCP, while it over-estimated rutting in HMA over HMA sections. The MEPDG model provided good predictions compared with actual IRI data in HMA overlaid pavement sections. Later Iowa conducted another evaluation using more (60) projects *[35]*. Both the nationally and locally calibrated rutting models provided good estimation to field measurements. After local calibration, the accuracy of rutting predictions was improved a little, but this improvement was not considered significant. The IRI model provided good estimation to field measurements. Iowa DOT pavement management information system does not differentiate thermal cracking from reflection cracking measurements. Thus, thermal cracking and reflection cracking were not evaluated. In general, previous studies found that field distresses of rigid pavement sections were low. A longer service period was needed for rigid pavements to show sufficient amount of distresses in order to fully validate the MEPDG model.

In a study conducted by Oregon, it was found that DARWin ME over predicted total rutting compared to the measured total rutting *[36]*. Further, it was observed that most of the rutting predicted by DARWin ME occurred in the subgrade. For alligator cracking, the software under predicted the amount of cracking considerably. After calibration, both rutting and alligator cracking models provided reasonable predictions. However, the study did not evaluate the reflective cracking model.

It should also be pointed out that a close look at the national calibration of MEPDG revealed that no projects in Louisiana were included in the national calibration process, neither for flexible pavement models nor for rigid pavement models *[2]*. Therefore, local evaluation and maybe calibration for Louisiana are highly recommended to assure the nationally calibrated models are applicable to Louisiana conditions.

Comparison of the New M-E Design with Current Design Guide

An immediate question on switching to a new design method is whether the new method produces consistent designs with the current design method. Many efforts have been made to compare the new design guide with current versions since the MEPDG was released.

Carvalho and Schwartz pointed out that there are two approaches to compare the 1993 AASHTO Guide and the MEPDG *[37]*. The most direct approach is using both design methods to design a pavement structure for the same project. However, three issues are associated with the direct comparison: (1) the input parameters required by the two design methods are quite different, which makes it difficult to specify equivalent design scenarios; (2) multiple designs may satisfy the performance requirements; and (3) the required pavement thickness depends on the design criteria specified in each design method. An alternative approach is to evaluate whether both design methods predict performance in a consistent way across a range of design conditions. Carvalho and Schwartz used the second approach to compare the 1993 AASHTO Guide and the MEPDG in designing a simple three-layer flexible pavement structure. A number of pavement sections were first designed using the 1993 AASHTO Guide based on typical materials and climate conditions in five states with three traffic volumes. Then the same pavement structures were analyzed using the MEPDG for the same design period. It was found that the predicted pavement distresses (fatigue cracking and rutting) by the MEPDG increased significantly with the traffic volume although the same level of performance (ΔPSI = 1.7) was used in the 1993 AASHTO Guide. Since the MEPDG was calibrated with a larger set of pavement sections across the U.S., Carvalho and Schwartz concluded that 1993 AASHTO Guide underestimates the pavement distress for higher traffic levels *[37]*.

Li et al. used locally calibrated MEPDG to evaluate the pavement design catalog (a table of suggested pavement thicknesses for different design scenarios) for both flexible and rigid pavements in Washington *[38]*. They assumed that the locally calibrated MEPDG reflected the historical pavement performance in Washington. The pavement thicknesses developed from the 1993 AASHTO Guide were input into the MEPDG. Based on the design criteria specified by WSDOT, they found that the 1993 AASHTO Guide over-designs both flexible and the rigid pavements for WSDOT at all ESAL levels.

El-Badawy et al. compared the Idaho pavement design procedure with the 1993 AASHTO Guide and the MEPDG *[39]*. Flexible pavements of six existing roads in Idaho were re-designed using the 1993 AASHTO Guide and the MEPDG. In the MEPDG design, a special set of traffic inputs were used so that only one type of axle load (equal to ESAL) was applied. The Enhanced Integrated Climate Model (EICM) was deactivated in all the MEPDG analyses. MEPDG default design criteria were used. The comparison indicated that the 1993 AASHTO Guide and MEPDG generated similar pavement structures for all locations, whereas the Idaho design procedure was found overly-conservative.

Gedafa et al. compared the design AC and PCC thicknesses based on 1993 AASHTO Guide and the MEPDG for five existing roads in Kansas *[40]*. Two versions of MEPDG design software were used (1.0 and 1.1). Both the MEPDG default and the Kansas modified design criteria were used. Overall, comparisons showed that the required AC and PCC thicknesses by MEPDG were much thinner than those required by the 1993 AASHTO Guide, except for one JPCP pavement.

McCracken et al. compared the 1993 AASHTO Guide and the MEPDG by designing a JPCP pavement using both methods *[41]*. Both Level-1 and Level-3 material inputs were used. It was shown that different levels of inputs can change the design thickness by about 2 in.

Elfino et al. compared the two design methods in a case study of a CRCP pavement project in Virginia *[42]*. Level-1 inputs were used in the design. For this project, the MEPDG (software version 1.0) predicted a 65-year service life using the pavement structure designed by the 1993 Guide for a 30-year design life. A re-design using the MEPDG showed that 10 in. CRCP is adequate for a design life of 30 years as oppose to 13 in. required by the 1993 Design Guide.

Vandenbossche et al. analyzed test cells with JPCP pavements on MnROAD using both the 1993 Design Guide and the MEPDG *[43]*. In most of cases, the MEPDG predicted longer service lives than did the 1993 Guide. This result indicated that the MEPDG generally results in thinner concrete pavement sections than the 1993 AASHTO Guide.

Boone conducted a comparative analysis of Ontario pavements using the AASHTO 1993 Guide and the MEPDG *[44]*. The MEPDG was found to over-predict pavement distresses in new flexible pavement. The primary modes of failure were permanent deformation and roughness. The asphalt layer thicknesses produced using the MEPDG method were consistently higher than thicknesses from the 1993 method. For asphalt overlays, the MEPDG was found to over-predict pavement distresses, primarily asphalt layer permanent deformation and roughness. In terms of rigid pavement, it was found that the AASHTO 1993 method generally under-predicted pavement performance. If used for design, the MEPDG would predict thinner concrete layer thicknesses than the 1993 method. It should be pointed out, however, models were not locally calibrated before being used for design in this study. It is very likely that the result would be different if the design comparison was performed after the models were locally calibrated.

OBJECTIVE

The objectives of this research were:

(1) to evaluate the mechanistic-empirical pavement design guide using the latest software Pavement ME^{TM} based on typical Louisiana traffic, materials and environmental information;

(2) to assess the short and long-term performance of typical Louisiana pavement structures using Pavement ME's nationally calibrated performance models; and

(3) to develop implementation guidelines (including a recommended input strategy) for future assessment and adoption of Pavement ME^{TM} in Louisiana.

SCOPE

A total of 162 pavement projects were selected from the existing DOTD highway network and evaluated in this study using Pavement ME^{TM} version 2.0. The selected projects consisted of flexible pavements with five types of base (asphalt concrete base, rubblized PCC base, crushed stone or recycled PCC base, soil cement base, and stabilized base with a stone interlayer), rigid pavements with three types of base (unbound granular base, stabilized base, and stabilized base with a HMA or stone blanket layer), and an HMA overlay on top of existing flexible pavement. A Louisiana default input strategy for Pavement ME that reflects Louisiana conditions and practice was developed. The design reliability and performance criteria were established. Local calibration was conducted on fatigue cracking and rutting models for new flexible pavements, fatigue cracking and joint faulting models for new rigid pavements, and reflective cracking model for AC overlays. Finally, an implementation guide that contains all necessary design input and calibration coefficients was prepared to assist DOTD in implementing the M-E design method.

METHODOLOGY

Louisiana does not have any LTPP sections that have the necessary data for use in validating and calibrating the MEPDG distress/IRI prediction models for local implementation. Therefore, this study was solely based on extracting the required design inputs and pavement performance data from a group of selected existing pavement projects of DOTD and to perform a comprehensive pavement design evaluation for different new and rehabilitated pavement types in Louisiana using the AASHTOWare Pavement ME Design (hereafter called Pavement ME). In general, the following steps were taken in this research study:

- Step 1 – Project Selection
- Step 2 – Develop Louisiana Pavement ME Design Criteria
- Step 3 – Sensitivity Analysis of Pavement ME's Distress/IRI Models
- Step 4 – Determination of Pavement ME Design Inputs
- Step 5 – Interpretation and Validation of LA-PMS Performance Data
- Step 6 – Evaluation of Pavement ME Distress/IRI Models
- Step 7 – Local Calibration
- Step 8 – Design Examples and Implementation Guidelines

Project Selection

The selection of existing pavement projects involved querying and reviewing project plans and extracting historical measured distress/IRI data from multiple DOTD databases such as Content Manager, Tracking of Projects (TOPS), Mainframe, Visiweb, and LA-PMS. The following guidelines were used during the project selection:

- Overall, the selected projects should cover different geographic and climatic regions (i.e., north and south, west and east) of Louisiana under a variety of traffic conditions (i.e., high-, medium-, and low-volume).
- Rural highway projects were preferred due to having a consistent pattern of traffic. Special attention (using right-of-way images from Visiweb) should be given to projects inside a city to ensure excluding major intersections and bridges from project segment consideration.
- For each selected project, the history of construction, maintenance, and rehabilitation within a control section was reviewed. To be able to obtain the necessary distress/IRI performance data, only projects that have at least three historical PMS performance

data records (that means pavement in-service time greater than 6 years) were considered as candidates.

- Since plan changes during construction may alter the initial plan, only projects with as-built plan records were considered.
- Preference was given to projects with the original 1993 pavement design document as stored in the Content Manager database.
- For pavement rehabilitation, only projects with an overlay thickness of HMA greater than 3.5 in. were included since thin overlays less than 3.5 in. are most likely functional overlays rather than structural overlays.

In total, 162 existing pavement projects were selected for this research. Figure 4 shows the location of these projects.

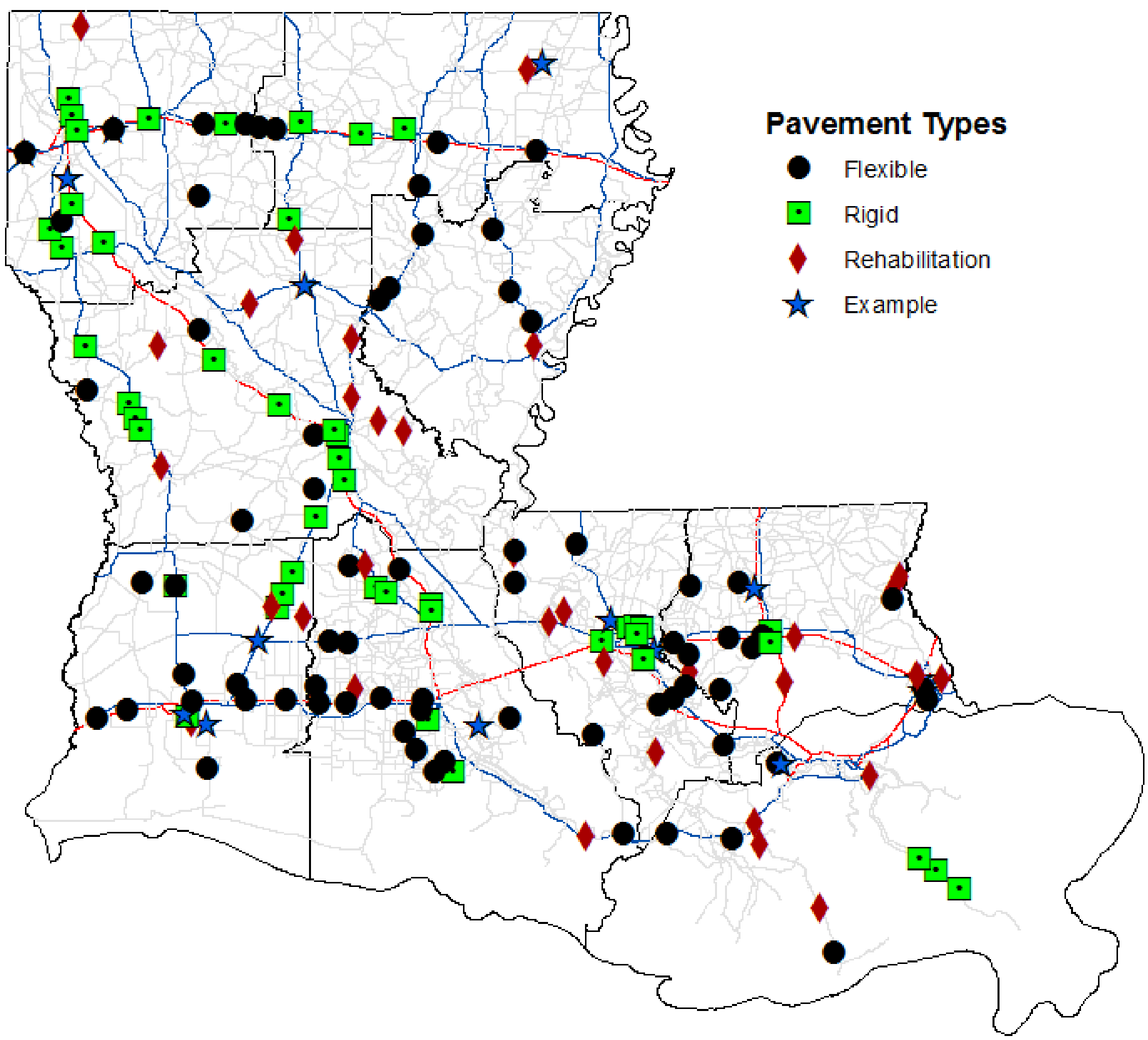


Figure 4

Pavement sections selected to evaluate and calibrate Pavement ME for Louisiana

Flexible Pavement Projects

Five typical flexible pavement types were identified in Louisiana, as shown in Figure 5. AC over AC base is usually used for medium- and high-volume roads in Louisiana. AC over RPCC (rubblized PCC) is a pavement type resulting from interstate reconstruction, in which the existing concrete layer is rubblized and used as a base layer. Both AC over crushed stone and AC over soil cement are currently used for medium- and low-volume state highways in Louisiana. A special type of flexible pavement which uses a stone interlayer between AC and soil cement to mitigate reflective cracking was also evaluated. This alternative was adopted by DOTD after successful pavement testing and field performance at LTRC in 2000 *[45]*.

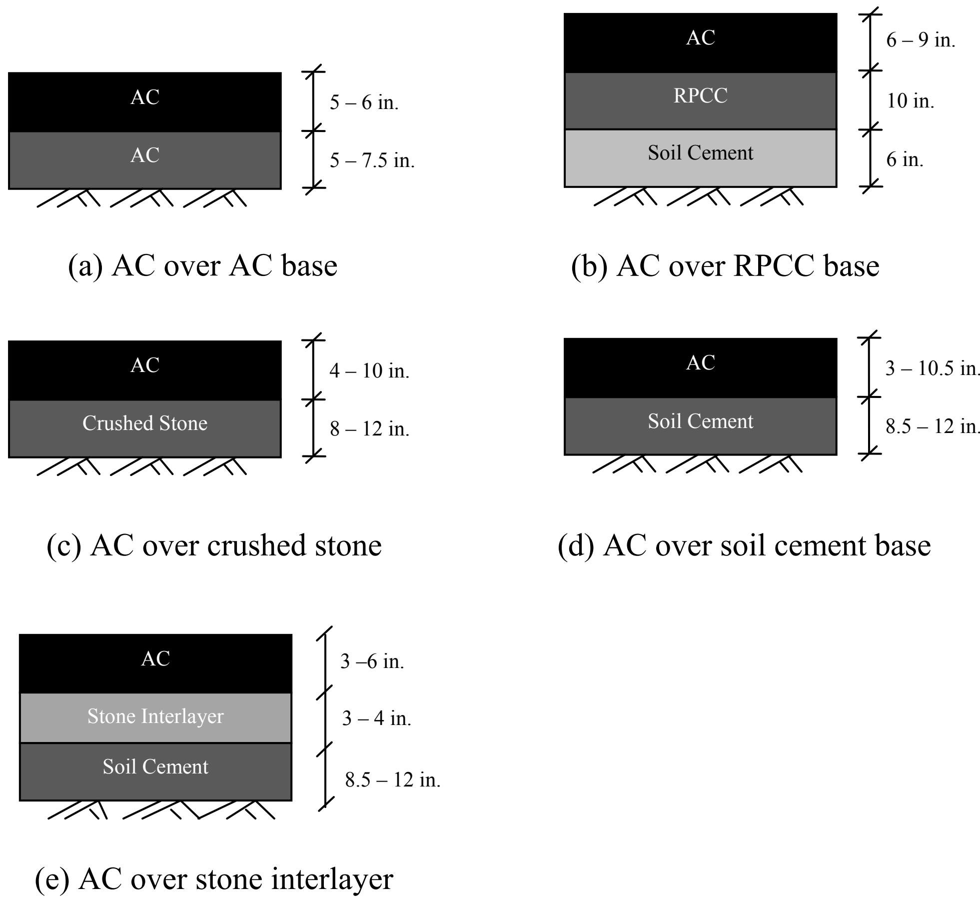


Figure 5
Typical flexible pavement structures in Louisiana (not to scale)

In total, 71 projects were selected to evaluate Pavement ME for designing flexible pavements. Tables 3 and 4 list project ID, route number, age as of the end of 2014, initial AADT, and the surface and base layer for the selected projects. Table 3 shows that 20 projects are on interstate highways, 11 projects are on US highways and 40 projects are on state highways. In terms of age, 9 of them are more than 20 years old, 47 are between 10 and 20 years old, and 15 of them are less than 10 years old. As of the end of 2014, these projects are, on average, at 14.3 years life with a standard deviation of 4.7 years (Figure 6). Regarding initial traffic, AADT ranges from 500 to 68,800 vehicles per day. In terms of pavement structure, the asphalt layer ranges from 3 in. to 13 in., averaging at 6.4 in.

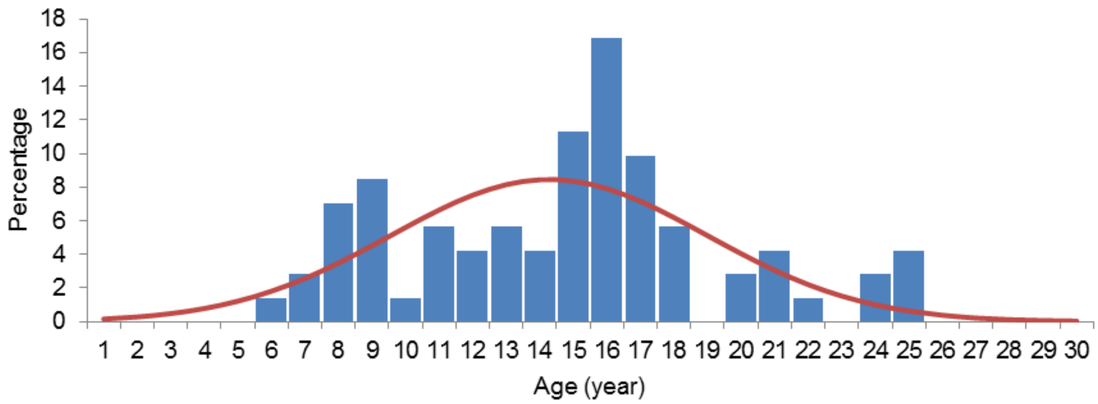


Figure 6
Ages of selected flexible pavement projects

Table 3
Selected projects to evaluate flexible pavement models

Base Type	PROJECT	DIS	PAR	ROUTE	# Lanes	Open To Traffic	Age (yr)	$AADT_0$	$AADTT_0$	Design ESAL (million)
AC_AC	015-05-0038	58	30	US 165	4	5/24/2002	13	5,400	324	1.3
	019-05-0025	61	63	US 61	4	9/2/2003	11	7,200	1,080	4.7
	026-04-0024	58	13	LA 15	4	8/23/2006	8	4,800	912	3.0
	055-06-0049	03	57	LA 14	4	4/19/2001	14	14,800	1,628	7.3
	267-02-0022	61	3	LA 431	2	9/14/2004	10	9,625	770	2.8
	424-07-0007	02	55	LA 3052	4	7/2/1998	16	16,200	2,430	9.9
AC_Interlayer	015-05-0035	58	30	US 165	4	8/30/2007	7	6,200	620	2.2
	015-07-0043	58	11	US 165	4	1/14/2008	7	6,300	756	3.5
	015-08-0028	05	37	US 165	4	6/20/2006	9	7,900	1,580	6.4
	201-02-0012	03	1	LA 97	2	4/15/1991	24	2,000	160	0.7
	203-03-0016	03	20	LA 29	2	5/31/2006	9	2,100	168	0.7
	219-30-0012	61	39	LA 10	2	1/26/1999	16	700	70	0.3
	237-05-0001	03	50	LA 1255	2	2/22/2006	9	1,000	50	0.2
	393-02-0005	03	57	LA 343	2	8/22/2007	7	2,400	192	0.8
	414-03-0024	61	3	LA 30	2	5/19/2006	9	10,900	872	3.7
AC_RPCC	450-03-0037	07	27	I-10	4	5/10/2002	13	31,077	7,614	31.4
	450-03-0064	07	27	I-10	4	6/7/2004	11	34,500	9,936	37.7
	450-04-0065	03	1	I-10	4	9/14/2001	13	41,200	7,416	29.0
	450-04-0069	03	1	I-10	4	10/5/2004	10	38,700	8,901	33.9
	450-04-0084	03	1	I-10	4	6/30/2004	10	34,500	9,660	31.5
	450-05-0046	03	28	I-10	4	8/24/2000	14	50,600	9,108	28.8
	450-18-0088	62	52	I-10	6	12/15/2006	8	68,800	11,696	41.1
	450-91-0076	07	10	I-10	4	6/12/2003	12	42,600	8,946	34.9
	450-91-0139	07	10	I-10	6	8/20/2007	7	39,700	9,131	38.9
	451-01-0083	04	9	I-20	4	11/12/1999	15	57,400	13,202	57.5
	451-02-0048	04	8	I-20	4	9/15/2008	6	36,669	7,700	27.0
	451-04-0030	04	7	I-20	4	2/11/1998	17	23,500	8,225	36.5
	451-04-0032	04	7	I-20	4	2/5/1997	18	17,600	6,160	27.4
	451-05-0062	05	31	I-20	4	2/5/1997	18	17,600	6,160	27.4
	451-05-0075	05	31	I-20	4	9/30/1998	16	24,200	5,808	22.1
	451-06-0092	05	37	I-20	4	9/14/1999	15	38,800	5,820	19.9
	451-07-0063	05	42	I-20	4	12/11/2006	8	22,900	4,580	18.4
	454-02-0026	62	32	I-12	4	6/5/2001	14	39,000	8,190	36.5
	454-02-0043	62	32	I-12	4	4/6/2000	15	48,700	10,714	45.9
	454-03-0028	62	53	I-12	4	12/10/1999	15	33,200	9,296	36.7

Table 3

Selected projects to evaluate flexible pavement models (cont.)

Base Type	PROJECT	DIS	PAR	ROUTE	# Lanes	Open To Traffic	Age (yr)	$AADT_0$	$AADTT_0$	Design ESAL (million)
AC_SC	012-10-0011	03	20	US 190	4	4/23/1990	25	6,200	1,240	4.6
	018-30-0018	62	52	LA 433	2	12/22/1999	15	5,300	265	1.4
	024-02-0014	07	10	US 171	4	11/25/1997	17	11,400	2,622	11.3
	029-07-0055	08	40	LA 496	2	9/21/2000	14	2,100	168	1.7
	031-09-0027	07	6	LA 27	2	9/18/1990	24	2,500	450	1.9
	036-03-0016	58	21	LA 4	2	2/21/1997	18	6,200	496	1.3
	057-06-0020	03	20	LA 13	2	11/1/1995	19	2,800	784	3.5
	064-02-0021	02	29	LA 1	2	8/9/1993	21	6,400	448	2.0
	067-03-0009	04	7	LA 4	2	12/13/1996	18	1,200	156	0.7
	073-02-0008	08	40	LA 112	2	4/22/1994	21	500	50	0.2
	080-01-0017	03	57	US 167	4	7/9/1991	23	20,700	1,656	6.5
	139-06-0011	08	58	LA 463	2	4/27/1999	16	900	99	0.6
	211-04-0009	03	1	LA 755	2	7/30/1999	15	4,700	329	4.0
	230-03-0022	61	24	LA 75	2	9/26/2003	11	2,000	180	0.6
	260-03-0010	62	32	LA 22	2	3/20/2000	15	4,300	344	1.7
	261-02-0020	62	32	LA 42	2	3/26/1999	16	4,100	328	3.3
	262-04-0005	62	46	LA 16	2	11/19/1999	15	3,290	461	2.0
	268-01-0014	62	32	LA 447	2	7/25/2000	14	11,485	919	4.1
	397-04-0004	03	57	LA 89	2	7/2/1999	15	2,700	216	2.5
	432-01-0018	08	43	LA 191	2	7/11/2000	14	3,500	945	2.5
	803-32-0001	61	3	LA 938	2	2/25/1999	16	2,400	192	2.3
	810-07-0014	07	10	LA 3020	2	10/6/1998	16	3,400	238	3.2
	828-15-0012	03	28	LA 93	2	11/18/1998	16	11,400	912	4.8
	839-02-0016	61	39	LA 419	2	3/19/1999	15	1,400	112	0.7
AC_UB	005-06-0033	02	29	US 90	4	5/11/1998	17	11,400	1,368	6.4
	014-02-0022	07	27	US 165	4	12/9/2008	6	9,000	1,350	5.3
	024-04-0013	07	6	US 171	4	9/17/2007	7	4,000	640	2.3
	026-05-0017	58	13	LA 15	2	8/1/2002	12	3,400	340	2.8
	034-05-0025	08	35	LA 6	4	12/7/1994	20	9,750	1,268	6.2
	058-02-0009	62	52	LA 41	4	5/17/2005	10	10,850	3,038	13.1
	098-03-0010	04	16	LA 5	2	11/4/1994	20	1,300	312	1.3
	193-02-0039	07	12	LA 27	2	8/19/2002	12	3,700	777	2.6
	262-06-0009	62	46	LA 16	2	1/3/1994	21	2,600	390	1.7
	424-05-0068	03	51	LA 3052	4	5/25/1999	17	24,600	3,444	11.7
	428-03-0010	02	45	LA 3127	4	11/19/1990	24	5,800	928	3.8
	847-02-0019	61	47	LA 641	2	9/5/2000	14	9,100	546	2.3

Table 4

Pavement structure of the selected projects to evaluate flexible pavement models

Base Type	PROJECT	ASPHALT	HMA (in.)	BASE	Base (in.)
AC_AC	015-05-0038	2 in. SMA WC + 4 in. Level 2 BC + 6.5 in. Type 5A BS	12.5		
	019-05-0025	1.5 in. Type 8F WC + 4 in. Type 8 BC + 7.5 in. Type 5A BS	13.0		
	026-04-0024	1.5 in. Type 8 WC + 4 in. Type 8 BC + 6 in. Type 5 BS	11.5		
	055-06-0049	1.5 in. Type 8F WC + 4 in. Type 8 BC + 6.5 in. Type 5A BS	12.0		
	267-02-0022	2 in. TYPE 8F WC + 3 in. TYPE 8 BC + 5 in. TYPE 5 BS	10.0		
	424-07-0007	1.5 in. Type 8F WC+4 in. Type 8 BC + 7 in. Type 5A BS	12.5		
AC_Interlayer	015-05-0035	2 in. Level 1 WC + 4 in. Level 1 BC	6.0	Class II stone base	4
	015-07-0043	2 in. Level 1 WC + 4 in. Level 1 BC	6.0	Class II base	4
	015-08-0028	2 in. Level 1 WC + 4 in. Level 1 BC	6.0	Class II base	4
	201-02-0012	1.5 in. Level 1 WC + 2 in. Level 1 BC	3.5	Class II stone base	4
	203-03-0016	1.5 in. Level 1 WC + 2 in. Level 1 BC	3.5	Class II stone base	4
	219-30-0012	1.5 in. Type 3 WC + 2 in. Type 3 BC	3.5	Stone	4
	237-05-0001	1.5 in. Level 1 WC+1.5 in. Level 1 BC	3.0	Class II stone base	3
	393-02-0005	1.5 in. Level 1 WC + 2 in. Level 1 BC	3.5	Class II stone base	4
	414-03-0024	2 in. Level 1 WC + 2.5 in. Level 1 BC	4.5	Class II stone	4
AC_RPCC	450-03-0037	2 in. Level 3 WC + 5.5 in. Level 3 BC	7.5	Rubblized PCC	10
	450-03-0064	2 in. Level 3 WC + 6 in. Level 3 BC	8.0	Rubblized PCC	10
	450-04-0065	2 in. SMA WC + 5.5 in. Type 8 BC	7.5	Rubblized PCC	10
	450-04-0069	2 in. Level 3 WC + 6 in. Level 3 BC	8.0	Rubblized PCC	10
	450-04-0084	2 in. Level 3 WC + 6 in. Level 3 BC	8.0	Rubblized PCC	10
	450-05-0046	2 in. SMA WC + 4 in. Type 8 BC	6.0	Rubblized PCC	10
	450-18-0088	2 in. SMA WC + 6 in. Level 3 BC	8.0	Rubblized PCC	10
	450-91-0076	2 in. SMA WC + 5.5 in. Level 3 BC	7.5	Rubblized PCC	10
	450-91-0139	2 in. Level 3F WC + 5.5 in. Level 3 BC	7.5	Rubblized PCC	10
	451-01-0083	2 in. SMA WC + 1.5 in. Type 8 WC + 4.5 in. Type 8 BC	8.0	Rubblized PCC	10
	451-02-0048	2 in. Level 2F WC + 5 in. Level 2 BC	7.0	Rubblized PCC	10
	451-04-0030	1.5 in. Type 8F WC+1.5 in. Type 8 WC +4 in. Type 8 BC	7.0	Rubblized PCC	10
	451-04-0032	1.5 in. WC +1.5 in. Type 8 WC + 4 in. Type 8 BC	7.0	Rubblized PCC	10
	451-05-0062	1.5 in. WC +1.5 in. Type 8 WC + 4 in. Type 8 BC	7.0	Rubblized PCC	10
	451-05-0075	2 in. SMA WC + 4 in. Level 3 BC	6.0	Rubblized PCC	10
	451-06-0092	2 in. SMA WC + 4.5 in. Type 8 BC	6.5	Rubblized PCC	10
	451-07-0063	2 in. Level 2F WC + 4 in. Level 2 BC	6.0	Rubblized PCC	10
	454-02-0026	4 in. Level 3 WC + 4 in. Level 3 BC	8.0	Rubblized PCC	10
	454-02-0043	2 in. Level 3 WC + 4 in. Level 3 BC	6.0	Rubblized PCC	10
	454-03-0028	2 in. SMA WC + 4 in. Type 8 BC + 3 in. Type 5A BS	9.0	Rubblized PCC	10

*Note: WC = Wearing Course, BC = Binder Course, BS = Base Course

Table 4

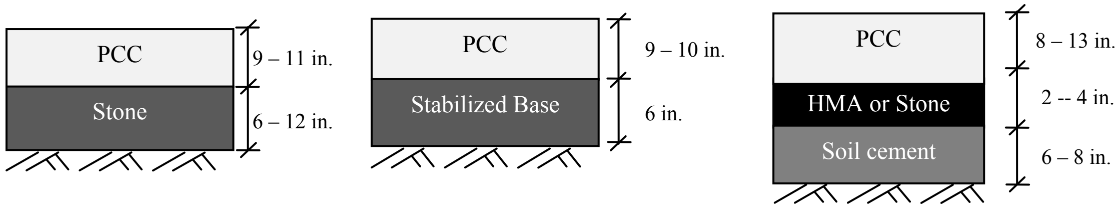
Pavement structure of the selected projects to evaluate flexible pavement models (cont.)

Base Type	PROJECT	ASPHALT	HMA (in.)	BASE	Base (in.)
AC_SC	012-10-0011	1.5 in. Type 3 WC + 4 in. Type 3 BC + 2.5 in. Type 5A BS	8.0	Class I base (in-place cement stabilized)	8.5
	018-30-0018	1.5 in. Type 8 WC + 2 in. Type 8 BC	3.5	Cement stabilized	8.5
	024-02-0014	1.5 in. Type 8F WC + 4 in. Type 8 BC + 5 in. Type 5A BS	10.5	Class II base	8.5
	029-07-0055	1.5 in. Type 3 WC + 2 in. Type 3 BC	3.5	Cement stabilized	8.5
	031-09-0027	1.5 in. Type 3 WC + 1.5 in. Type 3 BC	3.0	In-place cement stabilized	8.5
	036-03-0016	1.5 in. Type 8 WC + 2 in. Type 8 BC	3.5	Cement stabilized	8.5
	057-06-0020	1.5 in. Type 1 WC + 2 in. Type 1 BC + 4.5 in. Type 5A BS	8.0	Class I base, Item 301(01)	8.5
	064-02-0021	1.5 in. Type 8 WC + 2 in. Type 8 BC	3.5	In place cement stab base	8.5
	067-03-0009	1.5 in. Type 3 WC + 2 in. Type 3 BC	3.5	Lime fly ash stabilized	10
	073-02-0008	1.5 in. Type 1 WC + 2 in. Type 1 BC	3.5	In-place cement treatment	8.5
	080-01-0017	2 in. Type 8 WC + 3.5 in. Type 8 BC	5.5	Cement stabilized	8.5
	139-06-0011	1.5 in. Type 8 WC + 2 in. Type 8 BC	3.5	Cement treated	12
	211-04-0009	1.5 in. Type 8 WC + 2 in. Type 8 BC	3.5	Cement stabilized	8.5
	230-03-0022	1.5 in. Type 3 WC + 3.0 in. Type 3 BC + 4.5 in. BS	9.0	In-place cement treated (6%)	12
	260-03-0010	1.5 in. Type 8 WC + 2 in. Type 8 BC	3.5	Cement stabilized	8.5
	261-02-0020	1.5 in. Type 8F WC + 2 in. Type 8 BC	3.5	Cement stabilized	8.5
	262-04-0005	1.5 in. Type 8 WC + 4.5 in. Type 8 BC	6.0	Class II base	8.5
	268-01-0014	2 in. Type 8F WC + 2.5 in. Type 8 BC	4.5	Cement stabilized	8.5
	397-04-0004	1.5 in. Type E 8 WC + 2 in. Type 8 BC	3.5	Cement stabilized	8.5
	432-01-0018	2 in. Type 8F WC + 2.5 in. Type 8 BC	4.5	Cement stabilized	8.5
	803-32-0001	1.5 in. Type 3 WC + 2 in. Type 3 BC	3.5	Cement treated	12
	810-07-0014	1.5 Type 8 WC + 2 in. Type 8 BC	3.5	Cement stabilized	8.5
	828-15-0012	1.5 Type 8 WC + 2 in. Type 8 BC	3.5	Cement stabilized	8.5
	839-02-0016	1.5 in. Type 3 WC + 2 in. Type 3 BC	3.5	Cement treated	12
AC_UB	005-06-0033	1.5 in. Type 8F WC + 4 in. Type 8 BC + 3.5 in. Type 5A BS	9.0	Class II stone base	8
	014-02-0022	2 in. Level 2F WC + 5 in. Level 2 BC	7.0	Class II base	9
	024-04-0013	2 in. Level 2F WC + 3 in. Level 2 BC + 4 in. Level 2 BS	9.0	Class II base	10
	026-05-0017	1.5 in. Type 8 WC + 2.5 in. Type 8 BC	4.0	Stone or recycled PCC	12
	034-05-0025	1.5 in. Type 3 WC + 5 in. Type 3 BC + 3 in. Type 5A BS	9.5	Class I base	8.5
	058-02-0009	2 in. Type 8F WC + 3 in. Type 8 BC + 5 in. Type 5 BS	10.0	Stone	10
	098-03-0010	1.5 in. Type 8 WC + 5.5 in. Type 8 BC	7.0	Class I base	8.5
	193-02-0039	1.5 in. Type 8F WC + 3 in. Type 8 BC	4.5	Stone	12
	262-06-0009	1.5 in. Type 3 WC + 4.5 in. Type 3 BC	6.0	Crushed stone base	8.5
	424-05-0068	1.5 in. Type 8F WC + 4 in. Type 8 BC + 3 in. Type 5A BS	8.5	sand-shell or stone base	8.5
	428-03-0010	1.5 in. Type 3 WC + 4 in. Type 3 BC + 2 in. Type 5A BS	7.5	8 in. crushed stone + 4 in. selected material	8
	847-02-0019	1.5 in. Type 8F WC + 4.5 in. Type 8 BC	6.0	Stone	8.5

*Note: WC = Wearing Course, BC = Binder Course, BS = Base Course

Rigid Pavement Projects

A review of plan files of rigid pavements revealed that there are three typical rigid pavement structures: PCC over unbound base, PCC over stabilized base, and PCC over a HMA or stone blanket layer over soil cement base, as shown in Figure 7.



(a) PCC + Unbound base (b) PCC + Stabilized Base (c) PCC + HMA/Stone blanket

Figure 7
Typical rigid pavement structures in Louisiana (not to scale)

In total, 43 projects were determined as suitable evaluation sections (Figure 4), including 13 Interstate sections, 14 US highway sections, and 16 LA state highway sections. Although more than 200 projects were initially identified as candidates, most of them were eliminated due to several reasons including (1) being a city street with many intersections and possibly complicated traffic patterns; (2) although the project database indicated that a project was a rigid pavement, right-of-way images show it to be asphalt pavement. As a matter of fact, it was found during project selection that the primary application of rigid pavement on state highways are for passing through cities and towns where intersections and traffic signals are usually present.

In addition, it was found that there are primarily three cases of slab width and shoulder type: widened slab with tied PCC shoulder, tied curb and gutter in urban areas, and un-widened slab with HMA shoulder.

Overall, Table 5 shows the experimental design matrix. Slab thickness ranges from 8 to 13 in. with 10 and 11 in. as the majority (65%). Section length ranges between 0.2 mile and 3 miles with an average of 1.0 mile. A total of 67.5 miles of concrete pavements were surveyed in this study.

Table 5
Experiment design matrix of rigid pavement sections

Base Type	Shoulder Type	Slab Thickness (in.) 8	9	10	11	12	13	Total number of projects
Interlayer	Curb	2		3				5
	HMA			9	1	1	2	13
	PCC	1		1			2	4
Stabilized base	Curb		1					1
	HMA		1	1				2
	PCC							0
Unbound base	Curb		3	2	2			7
	HMA		2	1	1			4
	PCC			1	6			7
Total number of projects		3	7	18	10	1	4	43

Tables 6 and 7 list some primary parameters for these projects such as route number, pavement structure, slab width, shoulder type, and initial AADT. These projects were constructed between 1974 and 2010, with an average age of 20.3 years at the end of 2014 (Figure 8). Initial AADT ranges between 3,000 and 30,000 vehicles per day, and initial AADTT ranges between 400 and 4,500 vehicles per day. Truck percentages are between 5% and 32%. In terms of lane width and shoulder types, rigid pavements in urban areas are normally 13 ft. wide with monolithic curbs. Others are mainly widened to 15 ft., especially those constructed recently.

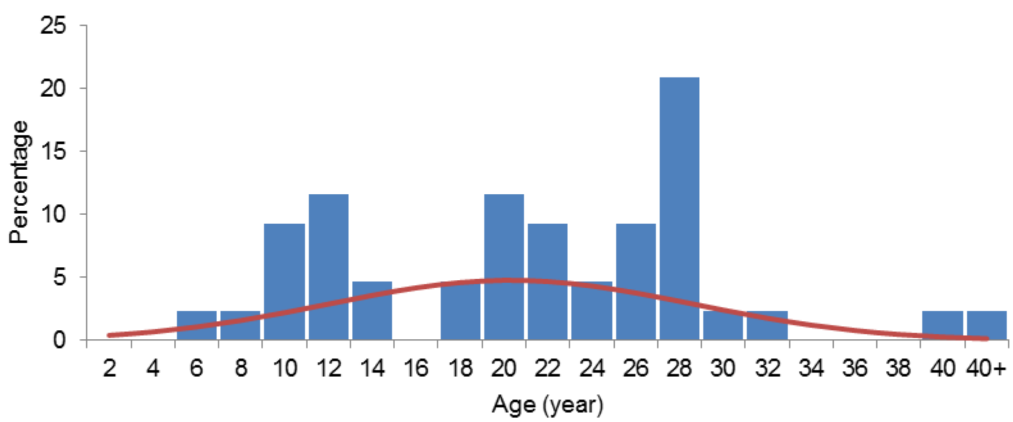


Figure 8
Ages of selected rigid pavement projects

Table 6
Selected projects to evaluate rigid pavement models

PROJECT	DIS	PAR	ROUTE	NUM LANES	OPEN TO TRAFFIC	AGE (yr)	$AADT_0$	TRUCK PCT	$AADTT_0$	GR	20-yrs Traffic (million)
008-30-0037	08	40	US 71	4	9/18/1997	17	25,179	6.9	1,737	0.6	6.7
013-01-0017	61	61	LA 415	4	8/16/1993	21	15,339	13.2	2,025	2	8.9
013-08-0015	62	53	US 190	4	3/3/1975	40	21,330	7.1	1,514	0.2	5.6
014-03-0028	07	2	US 165	4	5/30/2003	11	6,544	14.6	955	1.1	3.8
014-05-0020	08	40	US 165	4	1/18/2005	10	5,717	17.5	1,000	0.6	3.8
023-06-0035	05	25	US 167	4	3/18/2003	12	10,808	15	1,621	0.1	5.9
023-10-0034	05	31	US 167	4	4/28/2008	6	8,332	16.1	1,341	0.9	5.3
025-01-0027	08	58	US 171	4	4/6/2005	9	3,257	12.5	407	0.04	1.5
025-02-0026	08	43	US 171	4	4/6/2005	9	3,206	12.5	401	1.5	1.7
025-02-0033	08	43	US 171	4	6/2/2004	10	3,152	12.5	394	1.5	1.6
025-03-0025	08	43	US 171	4	9/12/2003	11	3,211	14.9	478	0.7	1.9
025-06-0027	04	16	US 171	4	8/12/2002	12	4,421	27.5	1,216	1.5	5.1
025-06-0031	04	16	US 171	4	1/31/2003	12	5,042	27.5	1,387	0.7	5.4
044-01-0013	04	8	LA 3	4	8/23/1985	29	8,696	10.8	939	4.5	5.3
044-01-0022	04	8	LA 3	4	5/20/1988	26	9,322	10.8	1,007	2.6	4.7
055-07-0032	03	23	LA 14	4	3/3/1989	26	8,192	6.7	549	1.8	2.4
062-03-0019	02	38	LA 23	4	3/9/2005	10	7,124	32.1	2,287	0.4	8.6
062-04-0018	02	38	LA 23	4	5/6/2002	12	8,754	13.5	1,182	0.03	4.3
062-05-0018	02	38	LA 23	4	8/1/1974	40	6,622	14.2	940	0.4	3.5
066-07-0027	03	20	US 167	4	5/5/1995	19	13,311	11	1,464	1.6	6.2
066-07-0030	03	20	US 167	4	7/27/1995	19	7,946	11	874	0.5	3.3
193-06-0025	07	10	LA 14	4	1/5/1994	21	12,176	13.5	1,644	1.3	6.7
255-02-0014	61	17	LA 408	4	10/28/1988	26	18,640	7.1	1,323	2.6	6.2
255-02-0022	61	17	LA 408	4	10/28/1988	26	12,115	7.1	860	2.3	3.9
315-02-0037	05	37	LA 143	4	5/10/1996	18	11,207	9.9	1,109	1.4	4.6
451-03-0037	04	60	I-20	4	7/1/1987	27	20,465	21.9	4,482	2.8	21.3
451-04-0029	04	7	I-20	4	8/1/1996	18	19,904	21.9	4,359	2	19.1
451-06-0080	05	37	I-10	4	2/25/1988	27	17,140	15.8	2,708	3	13.2
452-90-0039	62	53	I-55	4	8/14/1990	24	14,007	15.4	2,157	3.6	11.1
455-02-0003	03	49	I-49	4	11/1/1983	31	6,936	21.4	1,484	4.9	8.8
455-02-0004	03	49	I-49	4	9/29/1987	27	6,210	21.4	1,329	4.9	7.9
455-05-0017	08	40	I-49	4	2/12/1988	27	5,371	20.8	1,117	5.2	6.8
455-05-0021	08	40	I-49	4	11/12/1991	23	8,890	20.8	1,849	3.6	9.6
455-05-0022	08	40	I-49	4	4/15/1991	23	10,358	20.8	2,154	2.6	10
455-05-0026	08	40	I-49	4	8/19/1992	22	10,180	26.3	2,677	4.1	14.6
455-06-0008	08	35	I-49	4	2/2/1988	27	6,172	26.8	1,654	4	8.9
455-07-0009	04	16	I-49	4	5/14/1987	27	4,914	25.9	1,273	5.1	7.7
455-07-0012	04	16	I-49	4	9/25/1986	28	7,435	25.9	1,926	4.4	10.8
808-07-0029	04	8	LA 3105	4	5/4/1993	21	27,443	5.4	1,482	1.1	6
817-08-0021	61	17	LA 946	4	9/16/1994	20	20,184	7.1	1,433	2.2	6.4
817-08-0023	61	17	LA 946	4	1/6/2010	5	14,162	7.1	1,004	1.3	4.1
817-40-0004	61	17	LA 3246	6	3/5/1997	18	28,158	4.8	1,352	2.4	4.1
828-39-0018	03	28	LA 3073	4	11/17/1986	28	25,484	7.1	1,809	2.9	8.7

Table 7

Pavement structure of the selected projects to evaluate rigid pavement models

PROJECT	GROUP	SLAB (in.)	Dowel Bar (in.)	BASE	SUBBASE and SUBGRADE	Slab Width (ft)	Shoulder (ft)
008-30-0037	PCC_Blanket	10.0	1.500	Asphalt concrete base Type 5B 2 in.	Soil cement 8.5 in. + Subgrade treatment working table 6 in.	12	HMA 10
013-01-0017	PCC_Blanket	11.0	1.375	HMA Type 5B 2 in.	6 in. subgrade treatment working table	15	HMA 7
013-08-0015	PCC_UB	9.0	1.250	granular base 6 in.	unknown	12	Curb 0
014-03-0028	PCC_UB	11.0	1.500	crushed stone or recycled PCC 8.5 in.	9% lime treatment 11.8 in thick	13	Curb 0
014-05-0020	PCC_UB	10.0	1.500	stone or recycled PCC 8 in.	Lime treatment (Type D), 12 in	13	Curb 0
023-06-0035	PCC_Blanket	10.0	1.500	Asphalt base course (Type 5B) 2 in.	Crushed stone or crushed concrete 6 in. + 12 in. Type D lime (9%) as directed	13	Curb 0
023-10-0034	PCC_Blanket	8.0	1.250	stone or recycled PCC 4 in.	Soil cement 8 in.	13	Curb 0
025-01-0027	PCC_UB	11.0	1.500	stone or recycled PCC 10 in.	Lime treatment 12 in. (9%)	15	PCC 7
025-02-0026	PCC_UB	11.0	1.500	Class I stone or recycled PCC 10 in.	12 in. lime treated subgrade	15	PCC 7
025-02-0033	PCC_UB	11.0	1.500	Class I base (stone or recycled PCC) 10 in.	12 in. lime treatment 9% by volume	13	Curb 0
025-03-0025	PCC_UB	11.0	1.500	stone or recycled PCC 10 in.		15	PCC 7
025-06-0027	PCC_UB	11.0	1.500	stone Class I base 2 in. + stone or recycled PCC 6 in.	12 in. Type D lime treated	15	PCC 7
025-06-0031	PCC_UB	11.0	1.500	Stone or Recycled PCC 8 in.	12 in. Type D lime treated, 9% by volume	15	PCC 7
044-01-0013	PCC_SC	9.0	1.250	Soil cement or cement treated sand clay gravel 6 in.	unknown	12	HMA 10
044-01-0022	PCC_SC	10.0	1.250	Soil cement or cement treated sand clay gravel 6 in.	unknown	12	HMA 10
055-07-0032	PCC_UB	9.0	1.250	crushed stone base 6 in.	6 in. Type D lime treatment 12% by volume	12	HMA 10
062-03-0019	PCC_UB	10.0	1.500	stone or recycled PCC 8 in.	unknown	12	HMA 10
062-04-0018	PCC_UB	11.0	1.500	stone or recycled PCC 8 in.	12 in. Type D lime treated, 9% by volume	15	PCC 7
062-05-0018	PCC_UB	11.0	1.250	stone or recycled PCC 8 in.	12 in. Type D lime treated, 9% by volume	12	HMA 10
066-07-0027	PCC_Blanket	10.0	1.250	Type 5B asphalt base 2 in.	6 in. subgrade treatment working table	13	Curb 0
066-07-0030	PCC_UB	10.0	1.250	(stone or recycled PCC or shell or sand-shell 6 in.	unknown	13	Curb 0
193-06-0025	PCC_SC	9.0	1.250	Soil cement 6 in.	12 in. lime treatment	13	Curb 0

Table 7

Pavement structure of the selected projects to evaluate rigid pavement models (cont.)

PROJECT	GROUP	SLAB (in.)	Dowel Bar (in.)	BASE	SUBBASE and SUBGRADE	Slab Width (ft)	Shoulder (ft)
255-02-0014	PCC_UB	9.0	1.250	Base course 6 in.	Unknown	13	Curb 0
255-02-0022	PCC_UB	9.0	1.250	Base course 6 in.	Unknown	12	HMA 10
315-02-0037	PCC_Blanket	10.0	1.250	Asphalt base course 2 in.	6.5 in. subgrade treatment	12	HMA 8
451-03-0037	PCC_Blanket	13.0	1.500	Asphalt base course (Type 5B) 2 in.	Subbase treatment 6 in.	15	HMA 7
451-04-0029	PCC_Blanket	13.0	1.500	HMA Type 5B 2 in.	cement treatment 6 in. working table	15	PCC 7
451-06-0080	PCC_Blanket	13.0	1.500	Asphalt base course (Type 5B) 2 in.	8.5 in. subgrade treatment working table	15	HMA 7
452-90-0039	PCC_Blanket	12.0	1.375	HMA Type 5B 2 in.	crushed stone 6 in. or cement or lime treated soil 8.5 in.	15	HMA 7
455-02-0003	PCC_Blanket	10.0	1.250	HMA Type 5B 2 in.	Selected material, top 6 in. treated w/ lime or cement	15	HMA 7
455-02-0004	PCC_Blanket	10.0	1.250	HMA Type 5B 2 in.	cement treatment 6 in. + 4 in. selected soil subgrade	15	PCC 7
455-05-0017	PCC_Blanket	10.0	1.250	HMA Type 5B Item 501(2) 2 in.	11 in. embankment (top 6 in. Cement treated @8% item 305(1)) + selected material 5 in.	15	HMA 7
455-05-0021	PCC_Blanket	10.0	1.250	HMA Type 5B 2 in.	8 in. embankment (top 6 in. treated as working table)	15	HMA 7
455-05-0022	PCC_Blanket	10.0	1.250	HMA Type 5B Item 501(2) 2 in.	11 in. embankment (top 6 in. treated item 305(1))	15	HMA 7
455-05-0026	PCC_Blanket	13.0	1.500	HMA base Type 5B 2 in.	6 in. subgrade treatment working table	15	PCC 7
455-06-0008	PCC_Blanket	10.0	1.250	HMA Type 5B 2 in.	cement treatment 8% by volume 6 in. + selected material 5 in.	15	HMA 7
455-07-0009	PCC_Blanket	10.0	1.250	HMA Type 5B 2 in.	11 in. embankment (top 6 in. cement treated item 305(1)) + selected material 5 in.	15	HMA 7
455-07-0012	PCC_Blanket	10.0	1.250	HMA Type 5B 2 in.	cement treatment 8% by volume 6 in. + selected material 5 in.	15	HMA 7
808-07-0029	PCC_Blanket	8.0	1.125	HMA Type 5B 2 in.	unknown subgrade treatment 8.5 in.	13	Curb 0
817-08-0021	PCC_Blanket	8.0	1.125	Asphalt base course 2 in.	subgrade treatment 6 in.	15	PCC 7
817-08-0023	PCC_UB	10.0	1.500	Crushed stone 8 in.	12 in. Type D lime treated	15	PCC 7
817-40-0004	PCC_Blanket	10.0	1.500	Asphalt concrete base (Type 5B) 2 in.	12 in. subgrade layer	13	Curb 0
828-39-0018	PCC_UB	9.0	1.250	Crushed stone or shell 6 in.	6 in. lime treatment Type D, 10% by volume	13	Curb 0

Rehabilitation Projects

After reviewing the design files, it was found that AC overlays on top of existing asphalt layers is the predominate type for pavement rehabilitation in Louisiana. Therefore, this study only analyzed this type of pavement rehabilitation since PCC overlays are rarely used in Louisiana. In Pavement ME, AC over rubblized PCC (RPCC) could be designed as either new HMA pavement or AC overlays. This study evaluated AC over RPCC as new flexible pavement design so it was not included in the rehabilitation analysis.

In total, 33 projects were selected to evaluate Pavement ME for AC overlay design. According to the existing base type, these projects were assigned into three groups: AC over AC over soil cement, AC over AC over unbound base and AC over AC over PCC, as shown in Figure 9. These projects all have an overlay layer with 3.5 in. or thicker, so as to be considered as a structural overlay rather than a functional overlay. Overall, the overlay thickness is between 3.5 in. and 9 in. with an average of 4.6 in. The existing AC before and after milling averages 8.4 in. and 5.7 in., respectively. Initial AADT is 14,300 vehicles per day and the initial AADTT averages 2,050 vehicles per day.

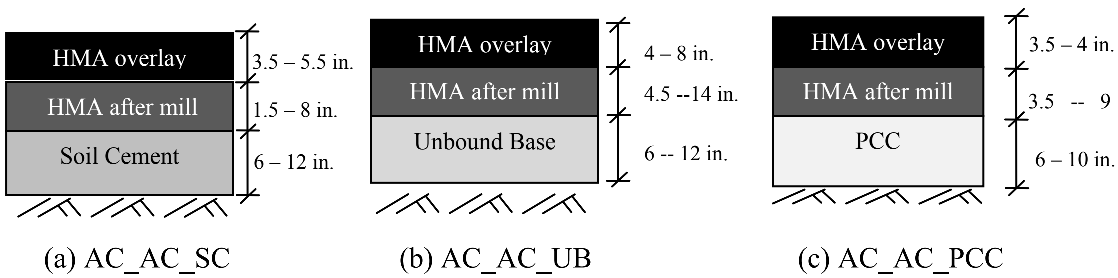


Figure 9
Typical pavement overlay structures in Louisiana (not to scale)

Table 8 and 9 show the primary properties of these projects: three are on Interstates, seven on U.S. highways, and the rest on state highways. All projects were opened to traffic after 1999. As of the end of 2014, the age of these projects ranges from 6 years to 15 years, averaging 9.5 years (Figure 10).

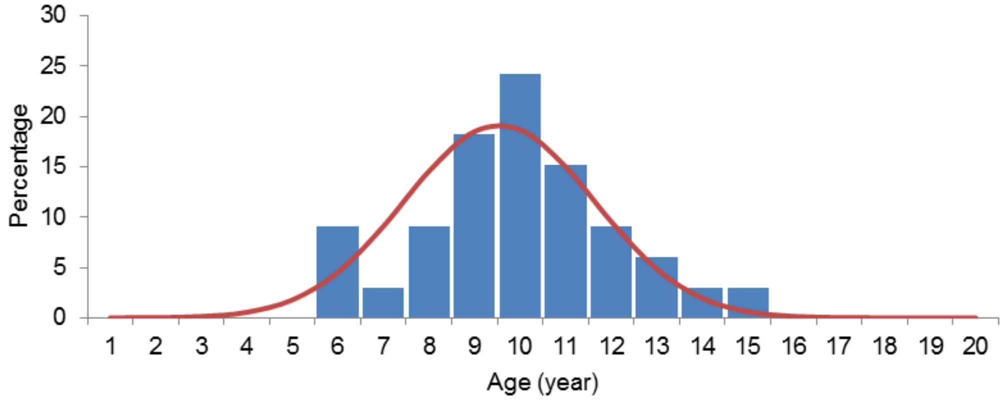


Figure 10
Ages of selected rigid pavement projects

Table 8

Selected projects to evaluation pavement rehabilitation models

GROUP	PROJECT	DIS	ROUTE	NUM LANE	OPEN DATE	Age	$AADT_0$	$AADTT_0$	10 Years ESAL (million)
AC_AC_SC	015-03-0023	08	US 165	4	9/7/2004	9	9,600	1,162	2.40
AC_AC_SC	025-01-0036	08	US 171	2	4/2/2004	10	16,500	2,062	4.37
AC_AC_SC	026-03-0036	58	US 65	4	10/20/2003	10	8,200	911	1.88
AC_AC_SC	028-05-0038	07	LA 26	2	4/19/2004	10	6,500	1,254	3.12
AC_AC_SC	030-02-0028	62	LA 21	4	12/21/2007	6	7,400	851	1.93
AC_AC_SC	030-03-0018	62	LA 21	4	12/21/2007	6	7,400	851	1.93
AC_AC_SC	033-04-0016	08	LA 115	2	8/11/2004	9	3,700	296	0.55
AC_AC_SC	034-04-0010	08	LA 6	2	7/17/2003	10	5,900	1,015	2.22
AC_AC_SC	052-03-0026	61	LA 1	2	10/17/2002	11	6,500	891	2.06
AC_AC_SC	057-03-0045	03	LA 13	2	4/20/2004	10	8,500	1,163	2.58
AC_AC_SC	074-02-0022	08	LA 28	2	7/1/2002	11	8,100	1,013	2.28
AC_AC_SC	200-01-0007	07	LA 104	2	4/10/2008	6	1,200	97	0.16
AC_AC_SC	206-01-0011	03	US 167	2	2/23/2001	13	3,000	600	0.95
AC_AC_SC	262-01-0032	62	LA 16	2	1/25/2005	9	11,200	896	1.68
AC_AC_SC	410-01-0030	02	LA 428	4	10/29/2003	10	55,200	3,975	6.50
AC_AC_SC	424-08-0030	02	US 90	4	5/30/2006	8	26,300	3,287	6.96
AC_AC_SC	454-03-0056	62	I-12	4	5/20/1999	15	36,700	7,670	16.62
AC_AC_UB	037-02-0037	05	LA 2	2	11/22/2004	9	4,600	630	1.33
AC_AC_UB	193-06-0033	07	LA 14	2	8/8/2007	6	8,500	1,148	2.64
AC_AC_UB	232-30-0004	61	LA 70	2	11/3/2003	10	7,800	1,170	2.60
AC_AC_UB	424-05-0106	03	US 90	4	7/21/2005	8	23,800	3,284	4.77
AC_AC_UB	450-18-0089	62	I-10	4	10/27/2003	9	31,700	6,689	13.93
AC_AC_UB	452-90-0137	62	I-55	4	3/14/2006	8	18,600	3,330	6.48
AC_AC_UB	454-04-0067	62	I-12	4	12/17/2003	10	56,600	8,603	21.46
AC_AC_UB	829-14-0026	02	LA 655	2	5/5/2005	8	3,900	237	0.40
AC_AC_UB	829-26-0011	02	LA 3235	4	7/19/2005	8	8,400	630	1.04
AC_AC_PCC	008-02-0029	61	US 190	4	11/14/2003	10	11,000	2,200	4.76
AC_AC_PCC	015-04-0045	08	US 165	2	6/22/2001	12	3,700	462	1.00
AC_AC_PCC	022-02-0033	08	US 84	2	8/15/2000	13	3,200	522	1.12
AC_AC_PCC	023-05-0033	08	US 167	4	2/25/2002	12	4,900	607	1.28
AC_AC_PCC	044-03-0009	04	LA 3	2	9/11/2003	10	16,061	1,733	5.25
AC_AC_PCC	050-07-0067	61	LA 1	4	5/27/2005	9	38,500	6,776	13.76
AC_AC_PCC	052-01-0017	61	LA 1	2	11/9/2004	9	9,100	1,511	2.54

Table 9

Pavement structure of the selected projects to evaluation pavement rehabilitation models

PROJECT	OVERLAY	h_OL (in.)	h_AC (in.)	h_mill (in.)	BASE	h_base (in.)
015-03-0023	2 in. BC + 1.5 in. WC	3.5	10.0	2	Cement stabilized	7.5
025-01-0036	2 in. BC + 2 in. WC	4	10.0	3	Soil cement	6.0
026-03-0036	2 in. BC + 2 in. WC	4	9.0	4	Soil cement	9.0
028-05-0038	3 in. BC + 2 in. WC	5	4.0	2	Cement stabilized	8.5
030-02-0028	2 in. BC + 2 in. WC	4	6.0	2	Cement stabilized	8.5
030-03-0018	2 in. BC + 2 in. WC	4	6.0	2	Cement stabilized	8.5
033-04-0016	4 in. BC + 1.5 in. WC	5.5	4.0	2	Soil cement	7.5
034-04-0010	2 in. BC + 2 in. WC	4	8.0	2.5	Soil cement	7.0
052-03-0026	2.5 in. BC + 2 in. WC	4.5	4.0	2	Soil cement	11.0
057-03-0045	3 in. BC + 2 in. WC	5	6.0	2	Soil cement	6.5
074-02-0022	2.5 in. BC + 1.5 in. WC	4	8.5	3.5	Cement treated	8.5
200-01-0007	2 in. Level 1 BC + 2 in. Level 1 WC	4	5.5	1	Soil cement	8.5
206-01-0011	3 in. Type 8 BC +1.5 in. Type 8 WC	4.5	3.5	2	Soil cement	8.0
262-01-0032	2.5 in. BC + 2 in. WC	4.5	4.0	2	Soil cement	8.0
410-01-0030	2 in. BC + 2 in. WC	4	9.0	4	Cement stabilized	7.0
424-08-0030	3.5 in. BC + 2 in. WC	5.5	7.0	2	Cement treated	9.0
454-03-0056	4 in. permeable BS + 3 in. Type 8 BC + 2 in. SMA	9	15.5	8	Soil cement	8.5
037-02-0037	3 in. Level 1 BC + 2 in. Level 1 WC	5	9.5	4	Sand clay gravel	12
193-06-0033	4.5 in. Level 2 BC + 2 in. Level 2F WC	6.5	6.0	1.5	Gravelly sand loam	6.0
232-30-0004	3 in. Type 8 BC + 2 in. Type 8F WC	5	8.5	2	Gravel sand	6.0
424-05-0106	2.5 in. BC + 2 in. WC	4.5	12.5	2	Shelly sandy loam (A-1-B)	12.0
450-18-0089	3 in. ATPB + 3 in. BC + 2 in. WC	8	20.0	6	Compacted sand/shell	10.0
452-90-0137	2.5 in. Level 2 BC + 2 in. Level 2 WC	4.5	10.5	2	Sand/Shell base	12.0
454-04-0067	2 in. Level 3 WC + 3 in. Level 3 BC	5	15.0	2	existing aggregate base	9.0
829-14-0026	2 in. BC + 2 in. WC	4	5.0	0	Gravelly sandy loam	12
829-26-0011	2 in. BC + 2 in. WC	4	8.0	2	Sandy loam	12
008-02-0029	2 in. Type 8 BC + 1.5 in. Type 8F WC	3.5	8.5	3.5	PCC	8.0
015-04-0045	2 in. Type 8 BC + 2 in. SMA	4	8.0	4	PCC	6.0
022-02-0033	2 in. Type 8 BC + 2 in. Type 8 WC	4	12.0	3	PCC (8 in. at edge, 6 in. at center)	6.0
023-05-0033	2 in. Type 8 BC + 2 in. SMA	4	9.0	1.5	PCC	8.0
044-03-0009	2 in. Level 2 BC + 1.5 in. Level 2 WC	3.5	9.0	2	PCC	8.0
050-07-0067	2 in. Level 2 BC + 1.5 in. Level 2 WC	3.5	7.5	2	PCC	10.0
052-01-0017	2 in. Level 2 BC + 1.5 in. Level 2F WC	3.5	6.5	3	PCC	8.0

*Note: WC = Wearing Course, BC = Binder Course, BS = Base Course

Develop Louisiana M-E Pavement Design Criteria

Different from the AASHTO 1993 Design Guide, which uses present serviceability index (PSI) as the sole design criterion, the new Pavement ME requires a suite of design criteria based on individual performance indicators such as smoothness (IRI), cracking, rutting and faulting. The default design criteria and design reliability levels as recommended by the *Manual of Practice [4]* are presented in Table 10. Since Louisiana has very limited LTPP data sites that were used in the NCHRP study which developed the Pavement ME, there is a need to evaluate Table 10 and develop a set of criteria that match with the state-of-the-practice in pavement design for Louisiana.

Table 10

Recommended design criteria of ME pavement design *[4]*

Pavement Type	Distress	Interstate	Primary	Secondary
	Reliability (rural)	95	85	75
HMA pavement and overlays	Alligator cracking, %	10	20	35
	Rutting, in.	0.40	0.50	0.65
	Transverse cracking, ft/mi	500	700	700
	IRI, in./mi.	160	200	200
JPCP new, CRCP, and overlays	Faulting, in.	0.15	0.20	0.25
	Transverse cracking, %	10	15	20
	IRI, in./mi.	160	200	200

This task was accomplished through a survey of DOTD engineers designed to capture their experience and the state-of-practice. First, 32 (18 HMA-surfaced and 14 PCC-surfaced) representative distressed pavement images were selected from Visidata and LA-PMS. These sections were carefully selected to represent different highway classifications (interstate, arterial and collector), different traffic volumes (AADT ranging from 320 to 47,500), and different geographical locations in Louisiana. Furthermore, they were intentionally selected to include different distress types and severities such as fatigue cracking, longitudinal cracking, transverse cracking, and rutting. Details of these selected sections are included in Appendix C.

An online survey of these 32 images was developed and sent out through email to a wide variety of DOTD engineers, such as district engineers, design engineers, research engineers, management engineers, and maintenance engineers. In the survey, participants were asked to select one of the following choices: (1) Do nothing, (2) Overlay, and (3) Major rehabilitation for each pavement section based on their experience and the condition of each section. The survey results (Table 11) were then statistically analyzed against the measured pavement distress for the corresponding 0.1-mile section retrieved from LA-PMS. Besides regression analysis, the current DOTD PMS distress triggers, such as the ones shown in Figure 11, were considered. Finally, a suite of design criteria was proposed in Table 12 for use in the implementation of Pavement ME in Louisiana.

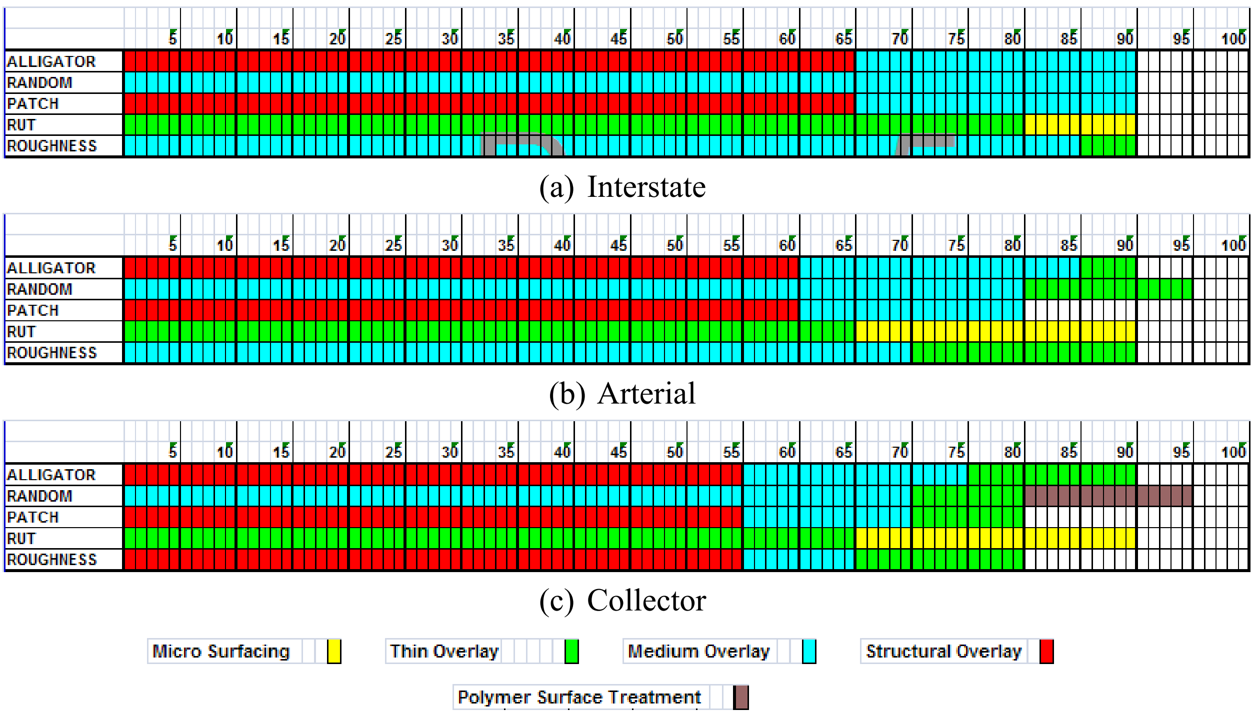


Figure 11

Distress triggers for flexible pavements currently used by DOTD

Table 11
Results of design criteria consensus survey

Pvmt Type	Survey ID	Control Section	Route	AADT	Survey Results		
					Routine Maintenance	Overlay	Reconstruction
ASP	1	454-02	I-0012	47,500	14	10	0
ASP	2	454-03	I-0012	42,800	6	13	5
ASP	3	455-05	I-0049	14,300	7	12	4
ASP	4	455-07	I-0049	12,300	12	10	2
ASP	5	024-06	US0171	12,000	8	12	3
ASP	6	014-06	US0165	16,300	1	12	10
ASP	7	080-01	US0167	15,700	0	14	9
ASP	8	050-06	LA0001	12,500	0	7	16
ASP	9	009-02	US0071	2,800	15	7	1
ASP	10	060-03	LA0067	8,400	4	15	3
ASP	11	196-04	LA0014	2,100	12	9	1
ASP	12	810-25	LA3063	2,700	8	14	1
ASP	13	188-01	LA0112	2,700	12	10	1
ASP	14	029-05	LA0121	3,000	4	12	7
ASP	15	155-02	LA0143	1,600	4	13	6
ASP	16	227-04	LA0413	2,700	2	6	15
ASP	17	852-13	LA1077	1,370	14	7	2
ASP	18	116-04	LA0478	320	8	7	8
JCP	21	455-07	I-0049	14,600	20	3	0
JCP	22	427-01	LA3132	38,400	14	8	1
JCP	23	453-01	I-0059	29,000	11	7	5
JCP	24	452-90	I-0055	25,500	7	9	7
JCP	25	025-07	US0171	6,500	9	8	6
JCP	26	809-08	LA0526	11,600	13	8	2
JCP	27	254-02	LA0037	31,600	9	8	6
JCP	28	016-01	US0165	28,200	4	4	15
JCP	29	809-10	LA3194	10,000	19	3	0
JCP	30	817-08	LA0946	32,300	3	10	10
JCP	31	239-02	LA0083	1,010	19	4	0
JCP	32	239-02	LA0083	1,010	14	8	1
JCP	33	245-02	LA0315	2,600	1	10	13
JCP	34	006-07	US0090	1,340	8	8	8

Table 12
Recommended design criteria of ME pavement design for Louisiana

Pavement Type	Distress	Interstate	Primary	Secondary
	[a]Reliability Level, %	95	90	80
New AC and AC overlay	Alligator cracking, %	15	25	35
	Total rutting, in.	0.40	0.50	0.65
	[b]AC rutting, in.	0.40	0.50	0.65
	Transverse cracking, ft/mi	500	700	700
	Reflective cracking, %	15	25	35
	IRI, in./mi.	160	200	200
New PCC	Faulting, in.	0.15	0.20	0.25
	Transverse cracking, %	10	15	20
	IRI, in./mi.	160	200	200
Note: a. Reliability level is not applicable to reflective cracking. b. AC rutting uses the same criteria as total rutting.				

Sensitivity Analysis of Pavement ME's Distress/IRI Models

A sensitivity analysis was conducted in this study on Pavement ME's distress/IRI models in order to (1) identify sensitive inputs so that special attention would be given while developing the input strategy for Louisiana pavement design; (2) have hands-on experience of the Pavement ME software using typical structure, materials, traffic and climate in Louisiana; and (3) have a pilot view of the reasonableness and sensitivity of the distress models.

Major findings are listed below.

- Sensitive factors for flexible pavements include HMA properties (binder grade, effective binder content, and air voids), HMA thickness, base modulus, and subgrade modulus.
- Pavement ME tends to over-predict rutting more significantly for pavements on softer subgrade and for pavements in south Louisiana. No significant difference was found for fatigue cracking between softer and stiffer subgrade.

- Overall the major factors that influence JPCP performance are coefficient of thermal expansion (CTE), PCC slab thickness, joint spacing, climate location, and PCC strength. As expected, factors that help JPCP perform better (less distress) are shorter joint spacing, thicker PCC slab, and stronger PCC materials. In addition, widened slabs also greatly improves the performance.
- Surprisingly, the following factors are found insignificant for rigid pavement: base thickness, base modulus, and subgrade modulus. Hence, using default inputs for these parameters such as base material type and strength, subgrade type and strength in the evaluation process will not produce a significant influence on the results.
- Water table depth makes a difference only when the depth is less than 3 ft. In other words, 5 ft. produces the same result as 20 ft. does. Considering the general geographic condition of Louisiana, this study assumed 5 ft. as the water table depth for all projects instead of the national recommended default 20 ft.
- For overlay design, total cracking is found to be sensitive only to the existing pavement condition. The influence on total cracking by changing the existing condition from very poor to excellent was from 5% to 9.5% for the selected case.
- The rutting model for AC overlay is sensitive to overlay thickness, existing rutting, subgrade modulus, and overlay HMA properties (binder grade, effective binder content, and air voids).
- Factors that influence the IRI model for AC overlay are similar to those for the rutting model, including overlay thickness, existing rutting, subgrade modulus, and overlay HMA properties. This is probably because the predicted cracking is at a low level and the IRI is highly related to rutting.

Determination of Pavement ME Design Inputs

Pavement ME requires hundreds of design inputs, many of which are either unavailable from DOTD databases or never tested before in Louisiana. Considering this study was planned to use the existing materials and LA-PMS's stored pavement performance, the following input strategies were considered:

- Starting from Level 3 national defaults;
- Investigating important parameters based on sensitivity analysis, interviewing DOTD engineers and reviewing project-level data in DOTD databases; and

- Localize those parameters where Louisiana's condition is different from the national default.

Pavement Materials and Design Inputs

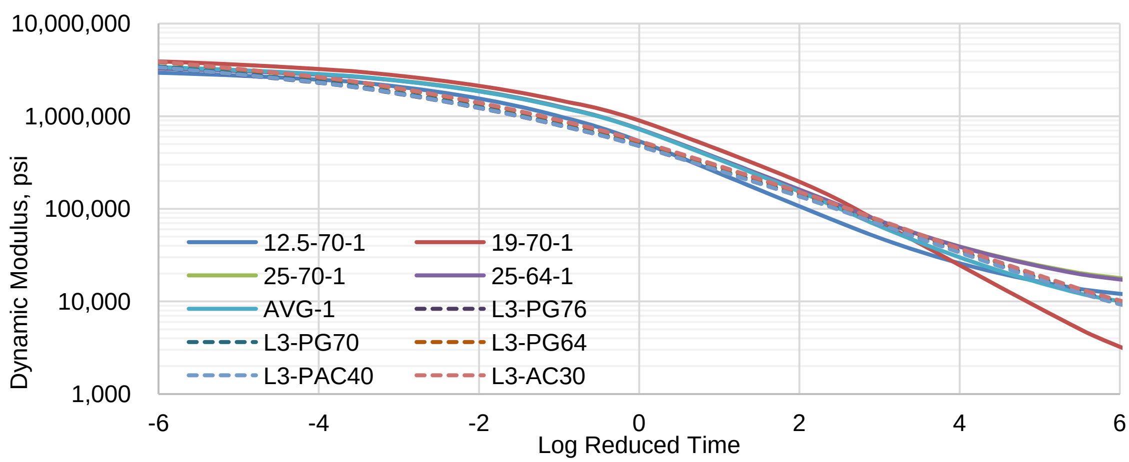
Asphalt Concrete. Louisiana has a database named MATT that records the job mix formulas (e.g., aggregate gradation, binder type, and volumetric properties) of asphalt mixtures approved by DOTD. However, the database was not designed to accommodate Superpave properties. Hence, data for old Marshall mixtures are well represented whereas Superpave mixtures are not. A previous study conducted at LTRC *[31]* found that within the practical range of variation, most parameters do not have a significant influence on the shape of the predicted master curve. Asphalt binder type is the primary influential factor in the model. The study also developed representative master curves based on asphalt binder type, as shown in Table 13.

Table 13

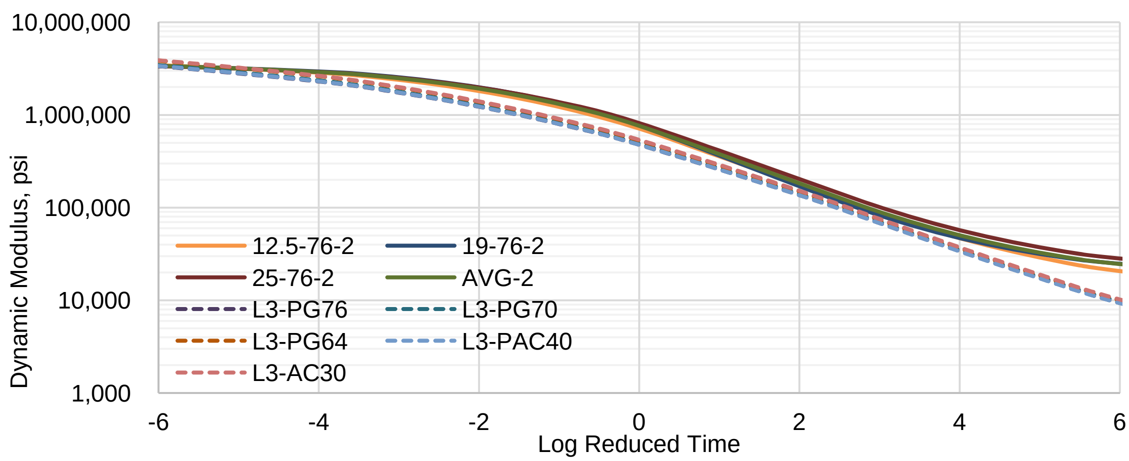
Default AC material input parameters for typical AC mixtures in Louisiana *[31]*

Design Input	Superpave	Superpave	Superpave	Conventional	Conventional
Asphalt Binder	PG 76-22	PG 70-22	PG 64-22	PAC-40	PAC-30, AC-30
Use (WC=wearing course, BC=binder course, BS= base course)	Level 2 WC Level 2 BC	Level 1 WC Level 1 BC	Level 1 BS	Type 8 WC Type 8 BC	Type 5 BS
Cumulative % passing 3/4 inch sieve	95	96	89	95	89
Cumulative % passing 3/8 inch sieve	69	72	72	70	74
Cumulative % passing #4 sieve	48	52	54	51	56
% passing #200 sieve	5.1	5.6	5.3	5.2	5.5
Effective binder content (%)	9.49	9.46	9.17	10.04	9.42
In-place air voids (%)	6.95	6.90	6.94	6.92	6.86
Total unit weight (pcf)	144	144	144	144	144

During this study, another project sponsored by LTRC to characterize common Louisiana asphalt mixtures using simple performance test protocols was completed *[46]*. A catalog of dynamic modulus values was developed based on laboratory testing of 28 asphalt mixtures. Since it was expected that laboratory tested data would provide a higher level of accuracy than using the Witzack prediction equation, a task was conducted to compare the two datasets. Dynamic modulus curves were plotted together in one graph for this purpose, as shown in Figure 12. Dashed lines are data generated from the five mixtures in Table 13 and denoted as L3. It was found that the two datasets were similar to each other with slight difference at the right side (represents high temperature and low loading frequency). In other words, keeping all other inputs the same, a project would expect similar performance using the two datasets except for possibly rutting.



(a) Mixtures for traffic level 1



(a) Mixtures for traffic level 2

Figure 12

Comparison of dynamic modulus based on mixture volumetric properties *[31]* and laboratory testing *[46]*

Table 13 was adopted as the HMA input for this research for three reasons: (1) it generated similar dynamic modulus curves with laboratory tested data; (2) neither Table 13 nor laboratory data were project-level data for the selected projects; and (3) a large portion of the selected projects were Marshall mixtures constructed before 2000.

Default values were accepted for other required inputs such as reference temperature, Poisson's ratio, thermal conductivity, and heat capacity.

Table 13 was also used for HMA blanket layers in rigid pavements and existing asphalt layers for pavement rehabilitation projects.

PCC. A sensitivity analysis of a typical Louisiana rigid pavement structure revealed that Pavement ME is very sensitive to the coefficient of thermal expansion (CTE) and modulus of rupture. In addition, results showed that the Level 3 input combination of modulus of rupture and elastic modulus could predict a better match with Level 1 input than using Level 3 compressive strength *[47]*. Although it is common to see modulus of rupture values over 600 psi in laboratory tests, this research used 600 psi as the 28-day modulus of rupture for PCC slab in order to be consistent with the current design practice of AASHTO 1993. Note that this value assumes substitution of 20% fly ash for cement and the use of gravel aggregate, and is therefore conservative when alternative materials are used *[48]*.

A study found that the CTE for Louisiana PCC mixtures using Kentucky limestone, Mexican limestone, and gravel was 4.96, 4.90, and 7.14 $\times 10^{-6}$/°F, respectively *[49]*. The ME default CTE for limestone is 5.5 $\times 10^{-6}$/°F. Field data of 43 selected projects did not show a distinctive difference between projects with stone and with gravel. This research used the national default CTE value as a conservative input.

In summary, national default values were used for PCC mixtures except these three parameters:

- Aggregate type for PCC slab is limestone.
- 28-day modulus of rupture for PCC slab is 600 psi.
- 28-day elastic modulus for PCC slab is 4,200,000 psi.

RPCC. The Pavement ME default resilient modulus for RPCC is 150 ksi. A study in New Jersey found that the average modulus of RPCC is between 160 and 200 ksi *[50]*. In general the modulus of rubblized PCC is higher than the modulus of a typical granular base and lower than the modulus of stabilized base materials. In 2005 a series of FWD tests conducted at I-10 in Louisiana suggested that the resilient modulus of rubblized PCC ranged from 124 ksi to 1,656 ksi with an average of 847 ksi *[51]*. It was theorized that the high values were probably because the rubblization process only rubblized the upper part and introduced diagonal cracks in the lower part of the slab. In this study, the resilient modulus of the RPCC was taken as 200 ksi, a value that agrees with most studies. In addition, this value also agrees with the material coefficient (0.25) used in the current pavement design process, in which HMA is assigned 0.44 and crushed stone 0.14. Pavement ME default values were adopted for other material properties: unit weight (= 150 pcf), Poisson's ratio (= 0.3), thermal conductivity (= 1.25 BTU/hr-ft-°F), and heat capacity (= 0.28 BTU/lb-°F).

Stabilized Base Material. Base materials could be stabilized with different types of "binder" such as asphalt, cement, lime, fly ash, or a combination of them. Asphalt-treated base will be considered as "asphalt mixture" with properties listed in Table 13. Cement treated base will be modeled as "chemically stabilized layer," including cement stabilized and soil cement. The default resilient modulus for 8.5 in. cement stabilized base and 12 in. cement treated base is recommended as 100,000 psi and 80,000 psi, respectively. National defaults are used for other parameters.

This recommendation is also applicable for other stabilized base materials such as lime cement fly ash and lime fly ash.

For rehabilitation and RPCC projects, the existing soil cement layer has most likely deteriorated under traffic loading. Therefore, the deteriorated modulus of 25,000 psi should be applied as recommended by the *Manual of Practice [4]*. In addition, due to the software bug which does not allow a modulus lower than 100,000 psi for soil cement, it is modeled as a crushed stone layer and assigned a modulus of 25,000 psi for design purposes.

Unbound Granular Base Material. Limestone and recycled PCC are the most widely used unbound materials in Louisiana. The preliminary result from a study (LTRC 10-3GT) on base materials in Louisiana found that the typical resilient modulus for Kentucky limestone, Mexican limestone, and recycled PCC is 30,500psi, 23,500psi, and 27,000psi, respectively. For consistency, this research applied 27,000 psi for all crushed stone and recycled PCC base.

Specially, if the unbound stone is placed between two stabilized layers such as the case for a stone interlayer, the modulus should be increased to 50,000 psi as a consideration of the increased confinement.

Subgrade. The first resource used in this research was the Arizona State University Soil Unit Map Application (http://nchrp923b.lab.asu.edu/), a product of NCHRP 9-23 (Environmental Effects in Pavement Mix and Structural Design Systems). This application provides soil data specifically for civil engineering purposes, such as AASHTO classification, CBR, resilient modulus, gradation, liquid limit, plastic index, and hydraulic parameters.

There are also coring/sampling log records for some projects in the Mainframe/MATT database. However, Louisiana does not have resilient modulus test data for subgrade soil. Instead, each parish in Louisiana uses a default resilient modulus value taken from Parish Maps. These default moduli were used as the input for subgrade in this study.

Type D lime treatment, also called a working table, is commonly used in Louisiana to prepare the subgrade for pavement construction. Because lime treated subgrade is not a uniform layer and has not been assigned a structural contribution in current design practice, this study did not assign additional strength to the lime treated layer. Instead, it is considered as untreated subgrade and assigned with the corresponding modulus according to the Parish Map.

Rigid Pavement Design Features. Rigid pavement also needs to specify joint spacing, joint design, and slab/base interface condition. As opposed to many other states in which joint spacing is 15 ft., Louisiana has been using 20 ft. for many years without any issue with transverse cracking. During the process of validating PMS data, manual measurement of joint spacing in 43 projects verified that concrete slabs are mainly 20 ft. long. A widened slab is also a common practice in Louisiana. After interviewing engineers at DOTD, the following parameters were applied in this research:

- Joint spacing of 20 ft. has been used in Louisiana for many years.
- Dowel bars are required for all rigid pavements in Louisiana. The required dowel bar size changes slightly according to historical practices or specifications. The required size corresponding to each project's construction year was adopted in this study.
- Erodibility index was assumed to be level 3 (erosion resistant) for stabilized base and level 4 (fairly erodible) for unbound base.
- As recommended by the *Manual of Practice*, PCC slab and base was assumed to have full contact over the design life *[4]*.

Existing Pavement Condition. Pavement ME requires layer-specific rutting and fatigue cracking for Level 1 overlay design. For Level 3 design, the software requires total rutting at the surface and the overall condition rating from very poor to very good. The *Manual of Practice* (Tables 8, 9, 10) provides a condition rating based on the quantity of distresses such fatigue cracking, longitudinal cracking, rutting and IRI. This table was used as the guidance to assess the condition before overlay *[4]*. The average rutting of existing pavement was retrieved from PMS for each project. In case PMS data were not available, the distress rating was assigned as poor and the existing rutting was assumed to be 0.25 in. for design purposes.

Climate

The location (longitude, latitude, and elevation) of a project was obtained from LA-PMS at the mid-point of the project or Google Earth (version 7.1.2.2041) if the GPS data was not present in the plan file. The climate station closest to the project or a virtual station generated from multiple nearby climate stations was utilized based upon the GPS coordinates of each project. Water table depth was assumed to be 5 ft. as a conservative input because sensitivity analysis showed that Pavement ME predicted similar results for 5 ft. and 20 ft. water table depth.

Traffic

Traffic data were retrieved from the TATV database in the Mainframe system at DOTD. Traffic volume (AADT) data were collected approximately every three years at each station. Data from the nearest station inside or close to the project were used. Directional distribution, lane distribution, and vehicle class distribution were based on the original traffic assignment, which was stored in an electronic document management system called Content Manager. It should be noted that the design AADT in the plan files for a few projects were found to be quite different from the measured AADT in the field. Since the main task of this study was to compare the predicted pavement performance with the measured performance, the measured AADT was used in modeling to simulate the actual situation in the field. Traffic growth rate was also calculated based on the measured traffic volume in the field rather than the plan files.

DOTD supported a research project to develop truck axle load spectra from existing data to support the implementation of the mechanistic empirical pavement design procedure *[52]*. Data collected from portable WIM sites were used as the data source. For single axle, the developed load spectra were very similar to the national default values; however, the load spectra for tandem and tridem axles were found to be quite different from the defaults. The project also noticed the limitations of portable WIM data and hence recommended developing a strategic plan for installing permanent WIM sites so that more reliable load spectra could be determined in the future.

Initially the load spectra developed from portable WIM data were adopted for this study, but it was found that the developed spectra only generated about 20% of ESALs compared to the original AASHTO 1993 design. ESALs from the national defaults matched quite well with the original design. Considering the limitations of portable WIM data, the national default load spectra were used in this study for flexible, rigid, and rehabilitation projects. Other traffic data such as hourly adjustment, monthly adjustment, axle per truck, and axle configuration used national defaults.

Interpretation and Validation of LA-PMS Performance Data

DOTD began collecting pavement distress data by windshield surveys in the early 1970s. Since 1995, DOTD has used the Automatic Road Analyzer (ARAN) to conduct network-level pavement condition surveys *[53]*. Pavement distress data collected for flexible and composite pavements are alligator cracking, random cracking, rutting, IRI and patching. For rigid pavements, IRI, faulting, longitudinal cracking, transverse cracking, and patching are collected. Punchouts for continuous reinforced concrete pavement (CRCP) have been collected since 2009. Louisiana network-level pavement condition surveys are conducted once every two years, and the data are stored in LA-PMS. The mean and standard deviation of IRI and rutting are calculated and reported for each 0.1-mi. subsection. The length of transverse and longitudinal cracking is summed up and reported every 0.1 mi. for three severity levels: low, medium and high.

Interpretation of PMS Data

Because some of the distress parameters (e.g., cracking) considered in the Pavement ME use different units of measurement, a unit conversion analysis of distress parameters is presented below:

Fatigue (Alligator) Cracking. In LA-PMS, fatigue cracking includes longitudinal cracks in the wheelpath and interconnected transverse and longitudinal cracks (which is defined as alligator cracking by LTPP) in the wheelpath. Alligator cracking is reported in square feet units with high, medium and low severity ratings. The following method is used to convert alligator cracking from square feet to percentage for each 0.1 mile section.

$$PercentFatigueCracking = \frac{ALGCRK_H + ALGCRK_M + ALGCRK_L}{LaneWidth * 528} * 100 \qquad (1)$$

Transverse Cracking. The total length of transverse cracking is reported in feet for high, medium, and low severities for every 0.1 mile section *[54, 55]*. In this study, transverse cracking of the three severities were added together without any weight factor for flexible pavement. For rigid pavement transverse cracking was used to estimate the percentage of cracked slabs. For pavement rehabilitation, transverse cracking was considered as reflective cracking and used to calculate the total cracking (alligator cracking + reflective cracking) as a percentage.

$$TotalCRK = FatigueCRK + \frac{(TRNCRK_H + TRNCRK_M + TRNCRK_L) * 1}{LaneWidth * 528} * 100 \qquad (2)$$

Rutting and IRI. The mean and standard deviation of total rutting in inches and IRI in in./mi. are reported for every 0.1-mi. subsection. The definition and unit in LA-PMS are the same as they are in Pavement ME. Hence, no unit conversion was needed.

Faulting. DOTD collects faulting for concrete pavements in the outside traffic lane. ARAN reports faulting for concrete pavements at both joints and transverse cracking, wherever an elevation difference is detectable. The minimum faulting was set at 0.2 in. during data collection; therefore, only faulting over 0.2 in. were reported and stored in the pavement management system. In other words, a zero value in PMS could mean either a perfect joint or a small fault that was less than 0.2 in.

For every 0.1 mile interval, the pavement management system includes five faulting-related data points: the average faulting, the maximum positive faulting, the maximum negative faulting, the number of positive faulting, and the number of negative faulting. Different from IRI and rutting, the variation (standard deviation) of faulting is not reported to PMS.

During this research, efforts were made to estimate an appropriate average value for faulting between 0 and 0.2 in. by comparing faulting from PMS and from profile measurement on ten selected sections, a method used by Utah DOT *[56]*. However, no significant improvement was found by assuming such a value. Therefore, this study calculated the average faulting of each project by averaging the available data in PMS without assuming any arbitrary value.

Percentage of Cracked Slabs. The LTPP database includes both length and numbers of transverse cracking in low, moderate, and high severities. In the national calibration, percent slabs cracked was computed by summing all transverse cracks observed (all severities) for a given test section and dividing it by the number of slabs within the test section *[2]*. The results were multiplied by 100 to transform it into a percentage. The computed percent slabs cracked might exceed 100% in situations where there were multiple cracks per slab. The cap used for all calculations was 100%.

$$PercentCrackedSlab = \frac{No.TransL + No.TransM + No.TransH}{TotalNo.Slabs} * 100 \qquad (3)$$

Louisiana cracking and patching protocol only identifies two types of cracking on jointed concrete pavement and continuously reinforced concrete surfaces: transverse cracking and longitudinal cracking *[55]*. They are defined as

- Transverse Cracking — A transverse crack is any visible crack that projects within 45° of perpendicular to the longitudinal centerline.
- Longitudinal Cracking — A longitudinal crack is any visible crack that projects within 45° of parallel to the longitudinal center line.

Cracks are surveyed in the main travel lane and rated with three severity levels: low, medium and high. For each level of severity, the total linear feet of cracking is recorded and reported at 0.1 mile intervals. To match the definition of Pavement ME, the percentage of cracked slab in a 0.1 mile section is calculated using equation (4):

$$PercentCrackedSlab = \frac{(TransL + TransM + TransH)/l}{528/JointSpacing} * 100 \quad (4)$$

where l = average length of each transverse crack.

The assumption of equation (4) is that only one transverse crack occurs in each slab before all slabs in a section cracked. By manually evaluating pavement images of the selected 43 sections, it was determined reasonable to assume 12 ft. as the average length of each transverse crack. In case the calculated percentage was over 100%, the value was capped at 100%.

Validation of PMS Data

The pavement management system was designed for monitoring the condition of the existing pavement network, and network-level optimization of resources. It was not designed for project-level research. However, the evaluation and calibration of Pavement ME requires project-level data, including traffic, material, and historical performance. In addition, the definition of distresses may not be the same in PMS as it is in Pavement ME (defined in LTPP). Therefore, a task to validate the PMS data was conducted in this study.

Pavement images and longitudinal profiles in the main traffic lane of the selected 43 rigid pavement sections (Table 6) were collected using a digital highway data vehicle (Figure 13) operated by the Louisiana Transportation Research Center (LTRC). To meet the specification of AASHTO R36*, Standard Practice for Evaluating Faulting of Concrete Pavements*, the profiler was operated at the maximum sampling rate (1 in. per sample) *[57]*. No digital filtering was applied during data collection. Profiles of both the left and right wheelpaths were collected.

The percentage of cracked slabs was determined through a manual distress survey of pavement images on a workstation as presented in Figure 13. During the manual survey, mid-slab transverse cracking, longitudinal cracking and corner cracking were classified. The number and length of cracks were recorded. The percent of cracked slabs was calculated by dividing the number of slabs with mid-slab cracks to the total number of slabs identified in each 0.1 mile section.

Faulting and IRI were obtained by analyzing profile data using ProVAL software. The algorithm to estimate faulting from profile data has been verified with reliable and highly repeatable results *[58]*. These data from LTRC were considered as the basis upon which the distress from PMS were compared.

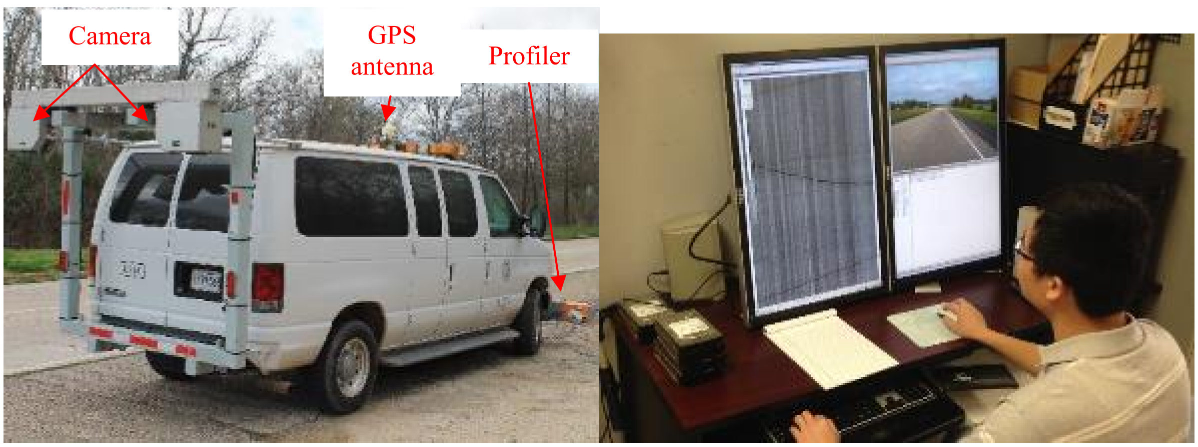
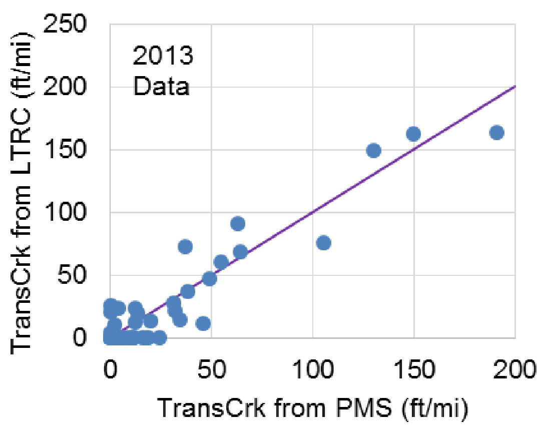


Figure 13

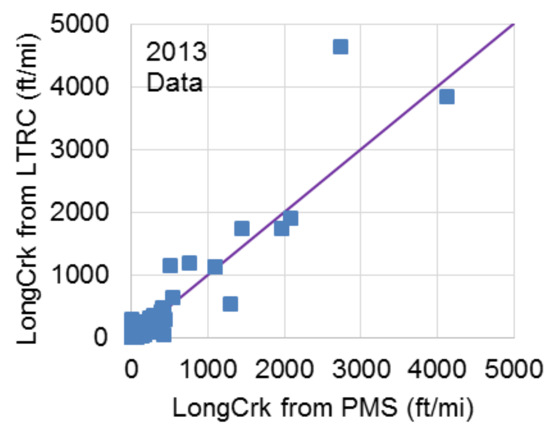
The digital highway data vehicle and workstation used in this study

Comparison of Cracking. The comparison of LTRC and 2013 PMS data is shown in Figure 14. It was found that the two datasets compared quite well. The correlation coefficient between the two for transverse cracking and longitudinal cracking was high, 0.80 and 0.94, respectively. Besides visual examination, the analysis of variance (ANOVA) was conducted to test the effect of a single factor – data source (PMS vs. LTRC) – on the collected cracking data. The advantage of ANOVA is that it not only compares the difference of mean values but also takes variation into consideration by comparing the variance between groups and within groups. The factor under study would be claimed significant only when the mean between the two datasets is different to a level that causes the variance between the groups being significantly larger than the variance within the groups. Results are listed in Table 14 and Table 15. At a significance level of $\alpha = 0.05$, the null hypothesis was accepted because both p-values were found larger than the significance level, indicating that there was no significant difference between the two datasets. Overall, both visual examination and statistical testing showed that transverse cracking and longitudinal cracking data from PMS and LTRC were comparable.

This result was beyond expectation initially because the challenge of automated cracking identification is well acknowledged. But it was later found reasonable because the cracking data for rigid pavement in the PMS database were obtained through a manual distress survey, not through automated software algorithms. The contractor first collected pavement image data using a multi-function pavement data collection vehicle. Then raters were assigned to identify cracks and other distresses for rigid pavements on a workstation. Automated software was only used to rate flexible pavements. Since rigid pavement only composes about 6% of the whole roadway system, manual distress surveys were manageable. Because the LTRC data were also based on manual distress surveys from pavement images, the two datasets should be comparable. There is about a one-year difference between the two datasets. The PMS data were collected from 9/15/2012 to 6/22/2013, while LTRC collected pavement images from 7/30/2013 to 6/30/2014.



(a)



(b)

Figure 14

Comparison of (a) transverse cracking and (b) longitudinal cracking

Table 14

Result of analysis of variance of transverse cracking

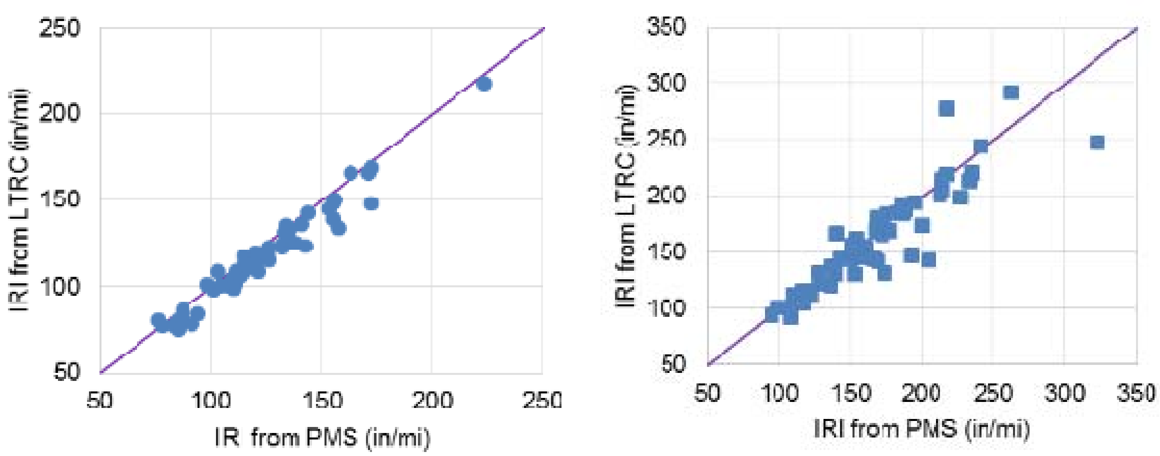
Source of Variation	Sum of squares	df	Mean square	F	P-value	F critical
Between Groups	1231.003	1	1231.003	0.490039	0.485388	3.927394
Within Groups	276325.6	110	2512.051			
Total	277556.6	111				

Table 15

Result of analysis of variance of longitudinal cracking

Source of Variation	Sum of squares	df	Mean square	F	P-value	F critical
Between Groups	31933.56847	1	31933.56847	0.047241	0.828339	3.927394
Within Groups	74357317.83	110	675975.6166			
Total	74389251.4	111				

Comparison of IRI. The average and standard deviation of IRI was calculated for each project based on LTRC collected data and PMS reported data. Figure 15 shows the comparison between LTRC and PMS. It is found that the average IRI from LTRC is slightly lower (on average, 6 in/mi) than the PMS data. If variation is considered, the comparison of average plus one standard deviation shows that LTRC and PMS compares well, with only very few outliers. In addition, one can see a slight trend that IRI matches better at low IRI values than at high IRI values. Similar to cracking data comparison, ANOVA was utilized to statistically compare the two datasets. Results of ANOVA are listed in Table 16 and Table 17. Since p-values are larger than 0.05, it could be concluded that the difference between the two datasets were intertwined with within-group variance, and the two datasets were not statistically different. Overall, as Figure 15 presents, the IRI data from PMS data were found comparable with LTRC data.



(a) average (b) average + standard deviation

Figure 15

Comparison of IRI

Table 16

Result of analysis of variance of IRI (average)

Source of Variation	Sum of squares	df	Mean square	F	P-value	F critical
Between Groups	1105.882	1	1105.882	1.397364	0.239625	3.92433
Within Groups	90220.27	114	791.4059			
Total	91326.15	115				

Table 17

Result of analysis of variance of IRI (average + std. dev.)

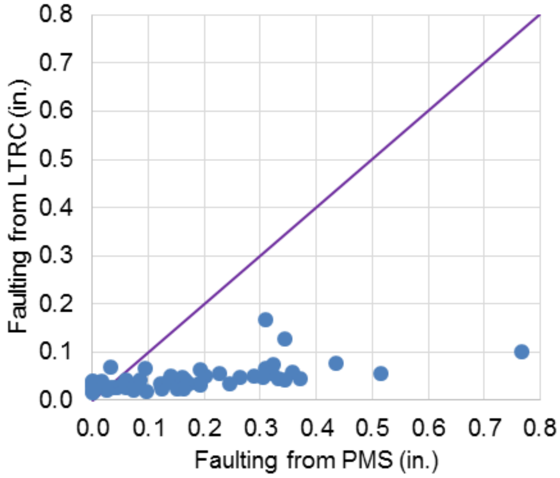
Source of Variation	Sum of squares	df	Mean square	F	P-value	F critical
Between Groups	1323.288	1	1323.288	0.688079	0.408552	3.92433
Within Groups	219240.5	114	1923.162			
Total	220563.8	115				

Comparison of Faulting. As mentioned before, PMS reports the average faulting for every 0.1 mile section with faulting less than 0.2 in. excluded. In this study, the average faulting for each project based on PMS data was calculated by summing the reported average faulting for each 0.1 mile and then dividing it by the length of the project. On the contrary, since faulting from LTRC measurement has no limitation of the 0.2 in. cutoff, this study calculated the average faulting based on LTRC data in two ways for the purpose of comparison:

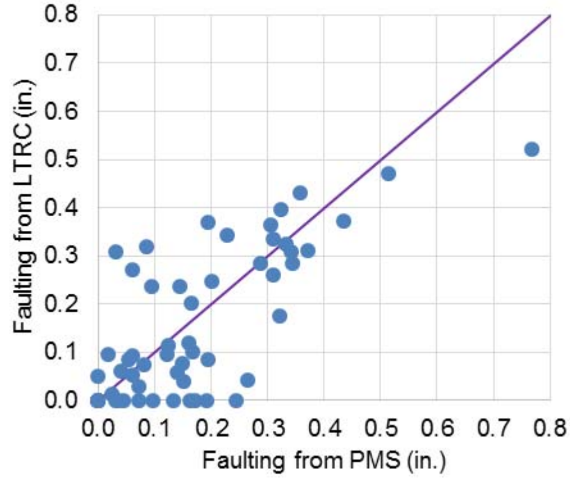
- Average faulting: calculate the average of all data in each project.
- Average of faulting over 0.2 in.: consider data less than 0.2 in. as zero, then calculate the average. This is similar to the PMS calculation.

Figure 16 compares the two averages from LTRC and PMS. It shows that faulting from LTRC measurement is very small, less than 0.1 in. except three projects. However, faulting from PMS is large, averaging at 0.17 in. with a maximum of 0.77 in. Based on engineering experience, faulting from PMS was definitely too large but faulting from LTRC seemed too small. Further site visit was needed to confirm that the LTRC method had captured the true faulting in the field. If the 0.2 in. cutoff was applied, LTRC did produce comparable data with PMS, as shown in Figure 16b. Results from ANOVA (Table 18 and Table 19) also confirm the visual observation – the average faulting from PMS and LTRC are different while the average of faulting over 0.2 in. from the two datasets are statistically not different.

Overall, results indicate that faulting data from PMS are highly questionable. It is unlikely, if not impossible, to provide reliable results using the PMS faulting data.



(a) average



(b) average of faulting over 0.2’’

Figure 16

Comparison of joint faulting

Table 18

Result of analysis of variance of joint faulting (average)

Source of Variation	Sum of squares	df	Mean square	F	P-value	F critical
Between Groups	0.424745	1	0.424745	35.83777	2.54E-08	3.92433
Within Groups	1.351114	114	0.011852			
Total	1.775858	115				

Table 19

Result of analysis of variance of joint faulting (average of faulting over 0.2’’)

Source of Variation	Sum of squares	df	Mean square	F	P-value	F critical
Between Groups	0.007219	1	0.007219	0.311223	0.578025	3.92433
Within Groups	2.644258	114	0.023195			
Total	2.651477	115				

To further verify the faulting estimation from ProVAL, site visits and manual measurements were conducted on four projects near Baton Rouge with varied ages of service. A ruler-like faultmeter (Figure 17) with an accuracy of 0.01 in. was used. The LTPP procedure of faulting measurement was followed *[59]*. Ten percent of the joints were randomly measured for each project. The results are shown in Table 20. It was obvious that the PMS data extremely over-estimated faulting comparing to either the ProVAL estimation or the manual measurement. Results from the ProVAL algorithm are much closer (in the same magnitude) to the manual measurements, while PMS data are in a different scale from the manual measurements, as shown in Figure 18. Results of ANOVA are listed in Table 21 through Table 23. Since all p-values are lower than 0.05, it indicated that the three datasets were statistically different, but it also showed that the sum of variance between LTRC and manual measurement was the smallest among the three datasets. Considering the limitation of PMS data and successful application of ProVAL *[58, 60]*, LTRC data were selected as the data source for this study.

Note: The zero line is at 2.00 in.

Figure 17

Manual measurement of joint faulting

Table 20

Comparison of faulting data from PMS, ProVAL and manual measurement (unit: inch)

Project	Road Name	Direction	PMS	LTRC and ProVAL	Manual Measurement
817-40-0004	Siegen Lane	Northbound	0.288	0.049	0.024
817-40-0004	Siegen Lane	Southbound	0.193	0.063	0.012
817-08-0021	Joor Rd	Northbound	0.767	0.100	0.029
255-02-0014	Hooper Rd	Eastbound	0.310	0.066	0.026
255-02-0022	Hooper Rd	Eastbound	0.193	0.032	0.015
Average			0.3502	0.0620	0.0212
Standard deviation			0.2391	0.0252	0.0073

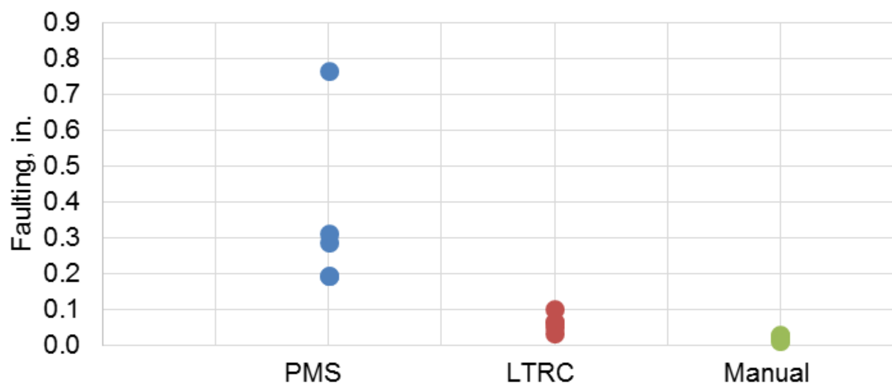


Figure 18

Comparison of faulting measurement

Table 21

Result of analysis of variance between PMS and LTRC

Source of Variation	Sum of squares	df	Mean square	F	P-value	F critical
Between Groups	0.2076481	1	0.2076481	7.186274	0.027895	5.317655
Within Groups	0.2311608	8	0.0288951			
Total	0.4388089	9				

Table 22

Result of analysis of variance between PMS and Manual

Source of Variation	Sum of squares	df	Mean square	F	P-value	F critical
Between Groups	0.270603	1	0.270603	9.459741	0.015217	5.317655
Within Groups	0.228846	8	0.028606			
Total	0.499448	9				

Table 23

Result of analysis of variance between LTRC and Manual

Source of Variation	Sum of squares	df	Mean square	F	P-value	F critical
Between Groups	0.004162	1	0.004162	12.12941	0.008286	5.317655
Within Groups	0.002745	8	0.000343			
Total	0.006906	9				

Evaluation of Pavement ME Distress/IRI Models

This study evaluated the Pavement ME national models from two perspectives:

1) Comparison of predicted performance from Pavement ME with measured performance from the field.

2) Comparison of the recommended pavement thickness from Pavement ME and the AASHTO 1993 Design (i.e., the original design).

Performance Comparison

At the project level, an Excel spreadsheet was used to compare the time series performance curve from both the Pavement ME and PMS. If the two curves matched well with each other, the model would be deemed as good. Otherwise, Pavement ME would over-predict if the predicted performance curve was on top of the measured performance curve; and vice versa. Figure 19 presents an example of this process. For this example, it was found that Pavement ME obviously under-predicted fatigue cracking, but over-predicted rutting. IRI from Pavement ME and PMS matched quite well. Because this process was a preliminary comparison, only a visual examination was conducted.

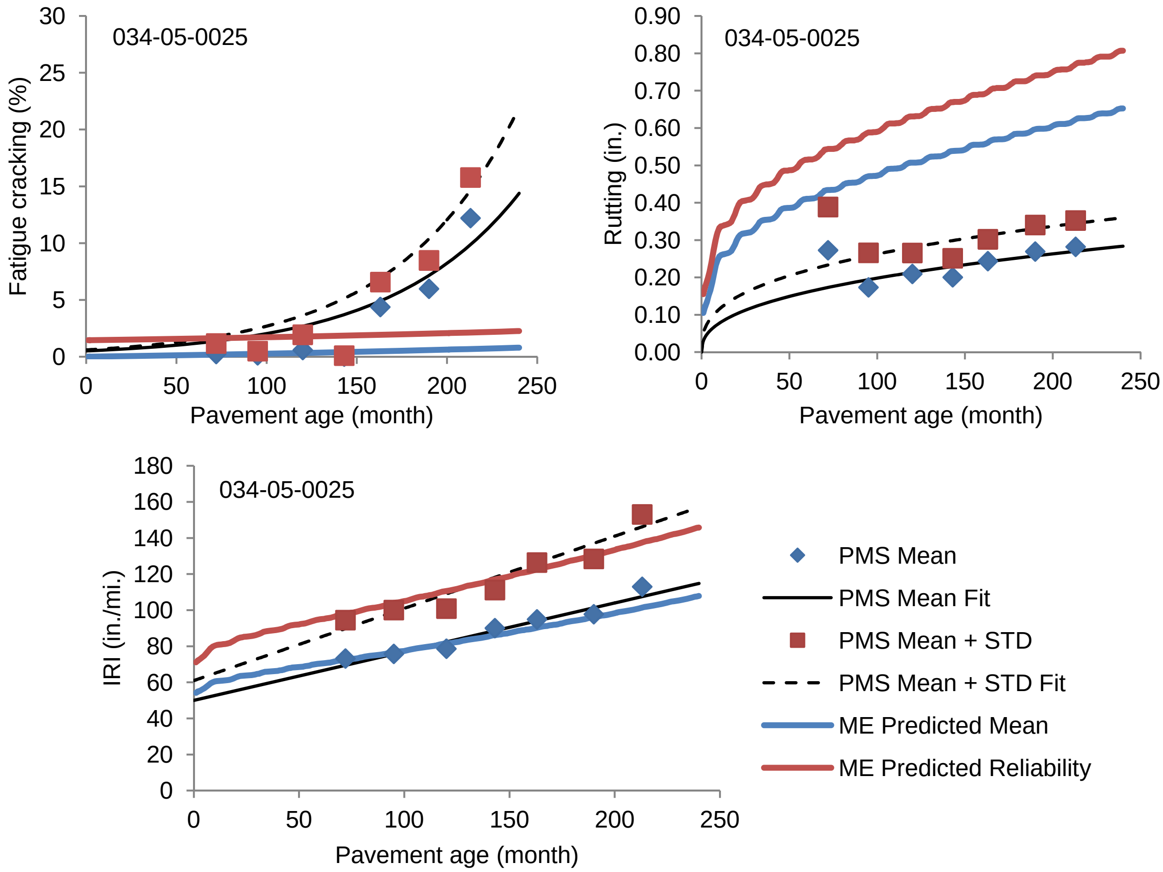


Figure 19
An example of project-level performance comparison

During this process, measured distress data were checked for anomalies and outliers. Observations that have irrational trends were excluded from further analysis. Figure 20 shows an example in which the second data point in the rutting graph and the third data point in the IRI graph were anomalies. By checking the raw data, it was found that the data records in the PMS database were null for these two points, meaning that no data were collected. Hence the calculated average performance are zero. Anomalies that could be explained were excluded from further analyses. It was also noticed in Figure 20 that the first data point in the rutting and the IRI graph were not on the trend line with other data points (above the fitted line). However, a closer check revealed that a rutting of 0.13 in. and IRI of 65 in/mi at month zero could not be stated as outliers because the open-to-traffic date was based on the final inspection date retrieved from the project management database and it was likely the actual open-to-traffic date was earlier than the record. In addition, it was also likely to observe rutting of 0.13 in. after a short time of trafficking depending on the quality of the asphalt mixture and the amount of heavy trucks. An initial IRI of 65 in/mi was in the normal range as well. Considering these reasons, the first data point was kept in the database and used for further analyses.

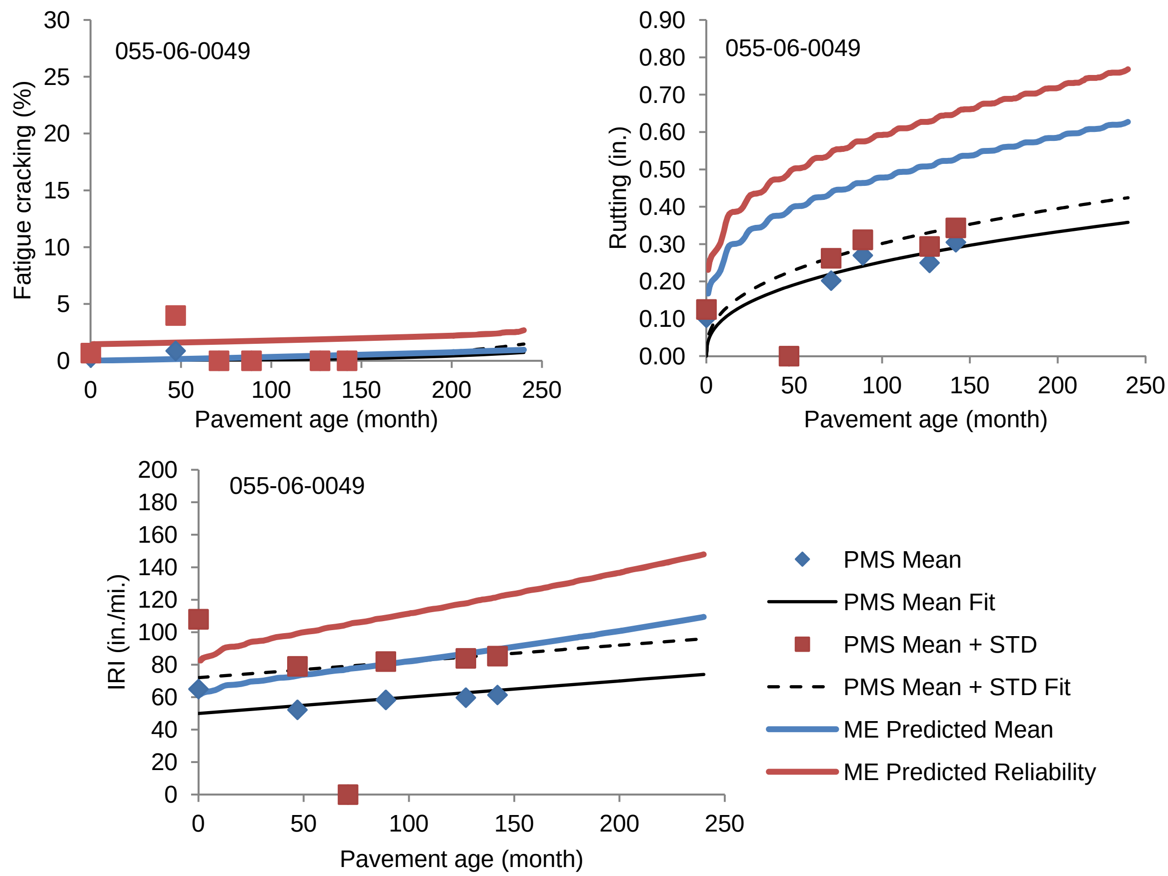


Figure 20

An example of identifying anomalies and outliers during the project-level performance comparison

After data from PMS and Pavement ME for each project were compared, data of interest such as predicted and measured fatigue cracking, predicted and measured rutting, and predicted and measured IRI from every project were compiled together for further analyses. At this level, predictions and measurements were compared as pairs. The ideal situation is that all data points would line up on the 45° line of equality if a model worked perfectly well. Otherwise, a model over-predicted the performance if predictions were larger than measurements; and vice versa.

Besides visual examination of the data, statistical analyses were conducted. The average bias and standard error were calculated according to the following equations *[33,61]*:

$$Bias = \frac{\sum_{i=1}^{n}\left(y_i^{pred} - y_i^{meas}\right)}{n} \tag{5}$$

$$S_e = \sqrt{\frac{\sum_{i=1}^{n}\left(y_i^{pred} - y_i^{meas}\right)^2}{n}} \tag{6}$$

$$S_y = \sqrt{\frac{\sum_{i=1}^{n}\left(y_i^{meas} - y^{mean}\right)^2}{n}} \tag{7}$$

where, *Se* = standard error of estimation

Sy = standard deviation of measured performance

y_i^{meas}= measured performance

y^{mean} = the average of measured performance

y_i^{pred}= predicted performance

n = total number of data points

The lower absolute value of the average bias and standard error means the better the model can predict a performance, and vice versa *[61]*. A positive value means that predicted performance is larger than measured performance; in other words, Pavement ME over-predicts. On the contrary, a negative value should be interpreted as Pavement ME under-predicting. The relative accuracy is usually described using standard error ratio S_e/S_y. The lower the ratio is, the better the model is. A perfect model would have zero bias, zero standard error of estimation, and zero standard error ratio.

Finally three hypothesis tests were conducted to statistically verify whether a bias exists *[21]*:

Test 1. Determine if the mean of residual error e_r (measured distress - predicted distress) is zero:

$$H_0: \sum (y_i^{meas} - y_i^{pred}) = 0 \tag{8}$$

Test 2. Determine if the linear regression relationship between measured and predicted distresses has an intercept of zero:

$$H_0: b_0 = 0 \tag{9}$$

Test 3. Determine if the linear regression relationship between measured and predicted distresses has a slope of 1.0:

$$H_0: m = 1 \quad (10)$$

where, b_0 and m are the intercept and slope of the regression model between predicted performance and measured performance. A rejection of any of the three null hypotheses indicates that bias exists between the predicted and measured distresses.

Thickness Comparison

While the pavement structure designed from AASHTO 1993 was used for performance comparison, each project was re-designed using Pavement ME for thickness comparison purposes. If the national model was suitable, the recommended thickness from Pavement ME would be close to the original design within a reasonable difference.

Similar to the performance comparison, visual examination and statistical analyses were conducted during thickness comparison.

Based on the results of the performance comparison and thickness comparison, Pavement ME design modules for designing each selected type of pavement structure were classified into the following three categories: (1) modules ready for Louisiana implementation without local calibration; (2) modules suitable for Louisiana implementation but need local calibration; and (3) modules that function improperly in the current version of Pavement ME. Local calibration was performed on modules in the second category if sufficient historical projects and PMS data are available.

Local Calibration

New Flexible Pavements

Based on Pavement ME analysis results, the bottom-up fatigue cracking model and rutting models were calibrated in this study. A summary of these models can be found in Appendix B of this report. Details can be referenced in the MEPDG documentation *[2]* and the AASHTO *Manual of Practice [4]*.

The calibration process included the following steps:

1. Evaluate the mechanism of the model. A sensitivity analysis of the N_f fatigue model and transfer function was conducted in an attempt to gauge the impact of calibration coefficients.

2. Learn from past experience of other states through a literature review.

3. Split the total project pool into two sets: one set of 80% of the projects was to be used for calibration, and the other set of 20% was used for validation. Projects were randomly selected within different subgroups (Interstates, US highways, and LA highways).

4. Based on the previous steps, it was decided to keep $b_{f2} = 1.0$ constant and to only change b_{f3} and b_{f1} to minimize the difference between predicted fatigue cracking and measured fatigue cracking. Since the model is more sensitive to b_{f3} than to b_{f1}, b_{f3} was first adjusted and then b_{f1}. The sum of the square error (SSE) between predictions and measurements was used as the objective parameter. This step needs iterative runs of Pavement ME software because strain is calculated monthly and N_f is an accumulation of monthly results.

$$SSE = \sum_{i=1}^{n} \left(y_i^{meas} - y_i^{pred}\right)^2 \tag{11}$$

5. After the values of b_{f3} and b_{f1} were determined, C_1 and C_2 in the transfer function were optimized to (1) minimize the sum of the square error between predicted fatigue cracking and measured fatigue cracking, and simultaneously to (2) meet the assumption of "an alligator cracking value of 50% cracking of the total area of the lane (6000 ft^2) occurs at a damage percentage of 100%," which was made during the national calibration *[62]*. A spreadsheet in Excel was created to calculate bottom-up cracking according to equation (18) as shown in Appendix B for every project. Damages for each month are available from Pavement ME output files; hence, no iterative run of the software was needed during this step. The optimization was executed using the Excel Solver tool.

6. Calibration coefficients were applied to the 20% of the validation projects. Comparisons of predictions and measurements were used to judge whether the coefficients were suitable for projects outside of the calibration pool.

In previous studies, most researchers chose to calibrate the rutting model for each pavement sublayer (i.e., AC, base and subgrade) separately. Without field trench test data, the field-measured total rutting on pavement surface had to be distributed to pavement sub-layers based on experience and/or certain assumptions. In this study, local calibration of rutting models was carried out using a special optimization procedure, where local calibration factors were adjusted together without the need to assume the sub-layer rutting. The calibration procedure is described as follows *[31]*.

In Pavement ME, the AC layer rutting model is given as:

$$\Delta_p = \beta_{r1} k_z \varepsilon_r H_{AC} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_3 \beta_{r3}} \tag{12}$$

where,

Δ_p	=	Accumulated permanent deformation in the AC layer
ε_r	=	Resilient strain at the mid depth of each sublayer
H_{AC}	=	Thickness of the AC layer
N	=	Number of axle-load repetitions
T	=	Pavement temperature
k_z	=	Depth confinement factor
k_1, k_2, k_3	=	Global field calibration factors
β_{r1}, β_{r2}, β_{r3}	=	Local field calibration factors

If only the two local calibration factors β_{r1} and β_{r3} are changed, while keeping $\beta_{r2} = 1$, the AC rutting damage model can be re-written as:

$$\Delta_p = \frac{\beta_{r1} \Delta_{AC}}{N^{k_3(1-\beta_{r3})}} \tag{13}$$

where, Δ_{AC} is the predicted AC rutting by the nationally calibrated model.

If the effect of the growth factor is neglected, the number of load repetitions N is approximately proportional to month by an unknown factor α (i.e., $N \approx \alpha \cdot Month$), where α is the approximate number of load repetitions per month. Then the predicted AC rutting can be expressed as:

$$\Delta_p = \frac{\beta_{r1} \Delta_{AC}}{(\alpha \cdot Month)^{k_3(1-\beta_{r3})}} \tag{14}$$

The value of α is hard to determine directly and is related to vehicle classification distribution, axles per truck, and axle load spectrum. However, it is a constant for each project and can be back-calculated if the predicted AC rutting with a β_{r3} other than one is known. For example, if all selected projects are analyzed with local calibration factors for AC rutting model are set as $\beta_{r1} = 1, \beta_{r2} = 1$, and $\beta_{r3} = 0.5$, the value of α can be back-calculated for each project.

The Pavement ME rutting damage model for unbound materials has only one local calibration factor β_{s1}. If the local calibration factor for unbound base rutting is denoted as β_{s1_BS}, and the local calibration factor for subgrade rutting is denoted as β_{s1_SG}, then the total rutting of a flexible pavement can be written as:

$$\Delta_t = \frac{\beta_{r1}\Delta_{AC}}{(\alpha \cdot Month)^{k_3(1-\beta_{r3})}} + \beta_{s1\_BS}\Delta_{BS} + \beta_{s1\_SG}\Delta_{SG} \tag{15}$$

Where, Δ_t is the predicted total rutting, Δ_{BS} and Δ_{SG} are the predicted rutting in the unbound base and subgrade respectively using nationally calibrated factors. With measured total rutting from a number of projects and the number of months when each rutting measurement was made, the local calibration factors β_{r1}, β_{r3}, β_{s1_BS}, and β_{s1_SG} can be calibrated together by minimizing the sum of squared errors of the total rutting.

New Rigid Pavements

Pavement ME has three distress models for rigid pavements: the transverse cracking model, the joint faulting model, and the IRI model. In the three models, there are respectively four, eight, and four calibration coefficients available for adjustments to match local experience *[4]*. For models with explicit equations, a nonlinear programming optimization technique through the MS Excel Solver has been commonly used to minimize the bias *[17], [33]*. Since this method can be conducted outside of the Pavement ME software, it is the easiest way to obtain the optimal calibration coefficients. However, this technique is not applicable if the model is implicit or some intermediate inputs are not available. For the latter case, a large number of iterations of the Pavement ME software is usually needed. To reduce the burden of software iteration, several statistical methods have been used to guide this procedure. The *Local Calibration Guide* provides recommendations for adjusting different coefficients to reduce bias and standard error *[21]*. Li et al. used elasticity to evaluate the relative impact of each factor to the model estimation *[32]*. Based on the relative impact, coefficients could be adjusted accordingly. Similar methods were also used by Bustosl et al. and Kim et al. *[34], [33]*.

Referring to the former studies, this research adjusted the calibration coefficients following a guided trial-and-error method. In detail, it involved the following steps.

1. Model all projects with the national models (no adjustment of any coefficients) and compare the predicted distress with the measured distress. This guides the direction of whether the predicted distress should be increased or decreased.

2. Split the total project pool into two sets: one set of 80% projects was to be used for calibration, and the other set of 20% was used for validation. Projects were randomly selected within different subgroups (Interstates, U.S. highways, and LA highways).

3. Identify two or three representative pavement structures and conduct a one-at-a-time sensitivity analysis of all calibration coefficients. This provided insight of how sensitive the distress model is to each coefficient. The range of change in this step was determined by referring to past studies or between 0.5 and 1.5 times of the national default value.

4. Interpret the mechanistic meaning of each coefficient to the distress model. For example, it was decided that C_1 and C_2 should be adjusted first for the cracking model since the predicted damage was not within the scale of the national calibration. The predicted stress of 20-ft.-long slabs severely escalated the damage from traffic loading. Hence, C_1 should be increased to increase the allowable number of load applications. To pinpoint the scale of adjustment, finite element analysis was conducted to compare the tensile stress for rigid pavement with 20-ft. and 15-ft. slabs. The average stress ratio was estimated to be 1.279, which was equivalent to increase of coefficient C_1 to 1.35 times, or from 2.0 to 2.70. This coefficient was further tuned to minimize the overall bias for all projects through iterative runs of the Pavement ME software. It was concluded that C_1=2.75 had the best result as shown in Figure 21. After the damage was regulated to a level comparable to the national calibration, then C_4 and C_5 were adjusted to reduce the bias between predictions and measurements.

5. For the faulting model (Appendix B), the *Local Calibration Guide [21]* recommends adjusting C_1 to reduce the bias and standard error. Sensitivity analysis showed that C_1 and C_6 are the most sensitive coefficients, followed by C_3. Mechanistically, C_1 is the linear multiplier to the initial maximum faulting; C_6 is the index of the power function that relates joint faulting with the erodibility factor, number of wet days and subgrade load; C_3 is the linear multiplier to the monthly mean faulting. Therefore, this study adjusted C_1, C_6, and C_3 following the guide of pilot projects and the trial and error method.

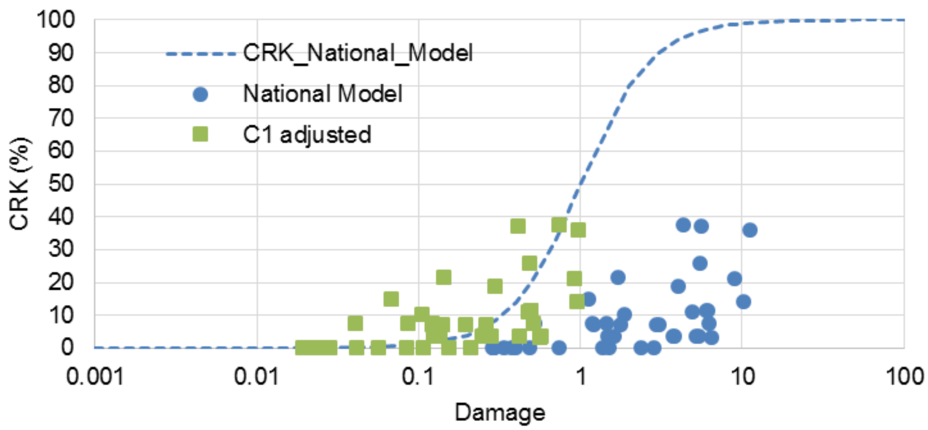


Figure 21

Adjust C_1 to reduce the predicted damage of rigid pavements

6. During the calibration process, the calibration coefficient was adjusted with the objective of minimizing the sum of standard error between predicted and measured faulting.

7. After both the cracking and faulting model were calibrated, the coefficients were applied to Pavement ME and another round of software execution was conducted. The predicted IRI was then compared with measured IRI. Since the two datasets were found statistically not different, it was decided to accept the national IRI model without further calibration.

8. Calibration coefficients were used on the 20% validation projects. Comparisons of predictions and measurements were used to judge whether the coefficients were suitable for projects outside of the calibration pool.

Pavement Rehabilitation

This study did not conduct a separate calibration effort on fatigue cracking and rutting models for AC overlays. Theoretically, rehabilitation models could be calibrated using the same method as new flexible pavements and new rigid pavements, with one difference —the reflective cracking model. However, this study found that national models performed similarly for AC overlay pavements and new flexible pavements—Pavement ME under-predicts fatigue cracking and slightly over-predicts rutting. This is not a surprise because new flexible pavement and AC overlays are mechanistically the same structure, composed of HMA layers, base layers and subgrade. The only difference is that all HMA layers are new materials in new flexible pavements while AC overlay pavements contain both new HMA mixtures and existing aged HMA mixtures. In addition, using only one set of calibration coefficients for asphalt materials is practically appealing because design engineers don't need to switch between different sets of coefficients. Therefore, the calibration coefficients obtained for new flexible pavements were applied to overlay designs in this study.

Reflective cracking, however, was found to be over-predicted. Hence, the model needed to be calibrated. The reflective cracking model as shown in Appendix B has two calibration coefficients *c* and *d*, which adjust the shape of the sigmoid curve in a similar way as the fatigue cracking as shown in Figure 21. Data of predicted and measured reflective cracking for each project were first assembled in an Excel spreadsheet. Then the Solver function was used to determine the optimum values of *c* and *d.* The objective was set to minimize the error between predictions and measurements.

Implementation Guidelines (Validation and Design Examples)

The last step of this research was to develop implementation guidelines for DOTD to adopt Pavement ME. Based on the results from model evaluation and local calibration, an implementation guideline which includes the recommended default inputs and calibration coefficients was proposed (Appendix A). Fifteen projects out of the calibration pool were selected as design examples. This effort was also completed to serve as an independent validation of the implementation guideline since these projects were not used in the evaluation and calibration process.

Table 24 through 26 list the primary information for the selected design examples for flexible pavements, rigid pavements, and rehabilitation pavements, respectively. Following similar guidelines as in the Project Selection, these projects were selected from different locations across the state and represented different levels of traffic. The original AASHTO1993 design report was required while PMS performance data were not, because the intention was comparing the recommended thickness instead of comparing predicted performance with measured performance.

These projects were simulated in Pavement ME software following the same procedure as was used for evaluating the national model. Material and traffic defaults for Louisiana as listed in the *Implementation Guidelines* (Appendix A) were used. The recommended design criteria as shown in Table 12 were applied. Starting with the recommended structure from the 1993 Design Guide, the thickness of HMA layer, PCC layer, or AC overlay layer was optimized to meet all design criteria. If the 1993 structure met all criteria, the layer thickness was decreased until any of the criteria failed. On the contrary, if the 1993 structure could not meet all design criteria, the layer thickness was increased until all criteria were satisfied. Layer thickness was adjusted in intervals of 0.5 in. The final recommended thickness was reported when all criteria were met. This process could also be completed using the optimization tool in the Pavement ME software. Both manual adjustment and the optimization tool were confirmed to provide the same result. This study used the manual adjustment method along with the batch run function in the software because this method was faster in approaching the recommended thickness than the optimization tool.

Finally, the recommended thicknesses from Pavement ME and from the 1993 Design were compared. Distress summaries and charts were also analyzed for their reasonableness.

Table 24

Traffic and structure information of design examples for new flexible pavements

Project	**852-03-0009**	**015-07-0044**	**014-03-0026**	**450-30-0085**	**452-90-0160**
Climate station	Baton Rouge	Monroe	Lake Charles	Lake Charles	Baton Rouge
Route	LA 1077	US 165	US 165	I-210	I-55
Initial AADT	5000	6300	9000	43700	27,500
$AADTT_0$	405	787	1359	6730	9515
Growth rate %	2.1	2.0	2.3	1.5	2.2
Design ESAL (millions)	1.48	3.53	3.91	16.88	31.09
Surface	2 in. Type 8F WC + 2 in. Type 8 BC	2 in. Level 1 WC + 4 in. Level 1 BC	2 in. Level 2F WC + 5 in. Level 2 BC	2 in. SMA wearing course + 5.5 in. Level 2 BC	2 in. Level 2F WC + 6 in. Level 2 BC
Base	Cement treated 12 in.	Class II base 4 in. + Cement stabilized 6 in.	Class II base 9 in.	RPCC 10 in. + Cement stabilized 6 in.	RPCC 10 in. + Cement stabilized 6 in.

Table 25

Traffic and structure information of design examples for new rigid pavements

Project	**451-01-0083**	**455-08-0061**	**014-03-0027**	**023-04-0021**	**817-41-0007**
Climate station	**Shreveport**	New Orleans	**Lake Charles**	**Alexandria**	**Baton Rouge**
Route	I-20	I-49	US 165	US 167	LA 3246
Initial AADT	28,700	14,400	12,000	18,000	31,350
$AADTT_0$ per lane	6,800	1,992	1,752	2,448	1,514
Growth rate %	1.7	2	2.5	2.0	1.7
Design ESAL (millions)	46.64	15.25	8.28	12.85	7.25
Lane width + Shoulder	15 + PCC	15	12 + Curb	15 + HMA	13 + Curb
Surface (inch)	13	10	11	10	9
Base	Class II stone 8 in.	Class II stone 8 in.	Class II stone 12 in.	Class II stone 8 in.	Class II stone 10 in.
Subgrade	Subgrade (A-6)	Subgrade (A-7-6)	Subgrade (A-6)	Subgrade (A-6)	Subgrade (A-7-6)

Table 26

Traffic and structure information of design examples for pavement rehabilitation

Project	**018-04-0040**	**037-02-0036**	**056-02-0021**	**193-05-0016**	**845-21-0003**
Climate station	New Orleans	Monroe	Lafayette	Lake Charles	New Orleans
Route	US-11	LA-2	LA-31	LA-14	LA 3160
Initial AADT	17,850	3,700	14,000	4,600	3,400
$AADTT_0$	1,942	507	1,106	621	207
Growth rate %	1.0	3.0	1.0	2.2	3
Design ESAL (millions)	2.38	1.17	1.73	1.38	0.39
Overlay	3.5 in. Level 2 BC + 2 in. Level 2F WC	3 in. Type 8 BC + 2 in. Type 8F WC	2 in. Level 2 BC+2 in. Level 2F WC	2 in. BC + 2 in. WC	3 in. Level 1 BC + 2 in. Level 1 WC
Existing surface	5 in. HMA	9.5 in. HMA	7 in. HMA	9.5 in. HMA	3.5 in. HMA
Milled thickness	2 in.	4 in.	4.5 in.	4 in.	2 in.
Existing AC rutting	0.34 in.	0.45 in.	0.39 in.	0.53 in.	0.21 in.
Base	Existing soil cement 8.5 in.	Existing soil cement 8.5 in.	Existing soil cement 7 in.	Existing soil cement 8.5 in.	Existing soil cement 6 in.
Subgrade	Subgrade (A-6)	Subgrade (A-6)	Subgrade (A-7-6)	Subgrade (A-6)	Subgrade (A-7-6)

DISCUSSION OF RESULTS

Evaluation of Flexible Pavement Models

Three distress models (load-related fatigue cracking, rutting, and IRI) were evaluated for flexible pavements.

Load-related Fatigue Cracking

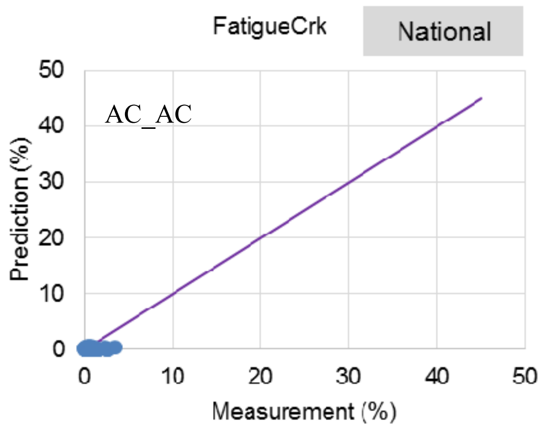
Figure 22 presents the comparison of predicted fatigue cracking from the Pavement ME and measured fatigue cracking from the PMS for the 71 selected projects. It was found that predictions from Pavement ME were very low (less than 5%) for all five base types. Measured fatigue cracking were also at a low level for AC over AC base and AC over RPCC base. For AC over crushed stone base, soil cement base, and stone interlayer, however, relatively high percentages of fatigue cracks were observed in the field according to the PMS data.

By referring to individual projects, the PMS measurements seemed reasonable.

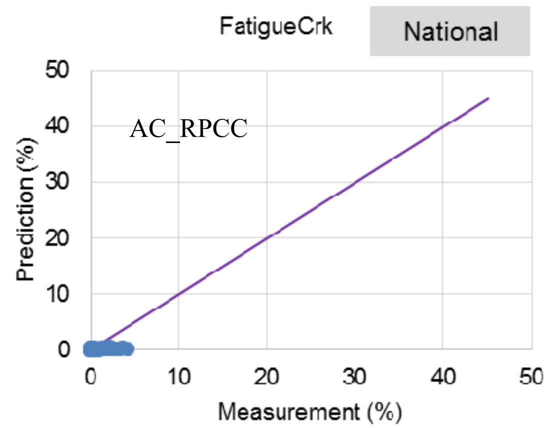
AC over AC Base. The six projects had a total thickness of 10 in. to 13 in. HMA on top of a lime treated subgrade. Between 8 years and 16 years of age, these projects didn't show alligator cracking. Only slight transverse and longitudinal cracking was recorded. Total rutting was less than 0.4 in. and IRI was less than 100 in/mi.

AC over Rubblized PCC. The twenty projects had 6 in. to 9 in. HMA over 10 in. of rubblized PCC. The base beneath the existing PCC was mainly 6 in. soil cement and the subgrade was lime treated. The overall structure was sound with very few longitudinal cracks, transverse cracks and alligator cracks. The total rutting was less than 0.3 in. and IRI was less than 120 in/mi.

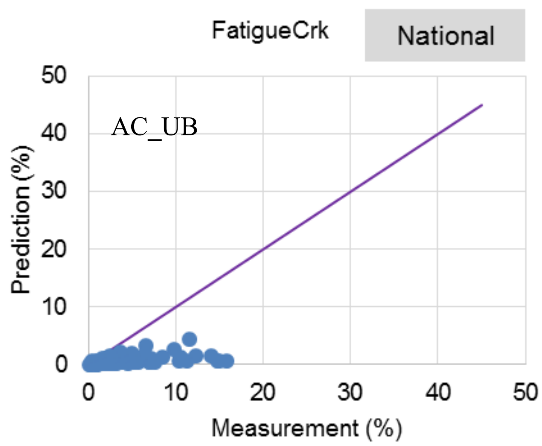
AC over Unbound Base. Alligator cracking as well as transverse and longitudinal cracking were observed in the field according to the PMS data. Meanwhile, Pavement ME predicted fatigue cracking up to 7.9% as well.



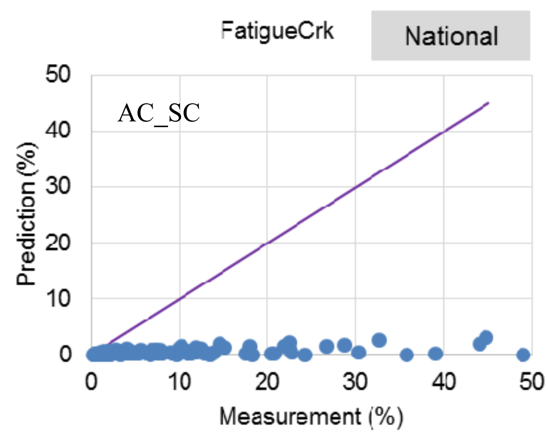
(a) AC over AC Base



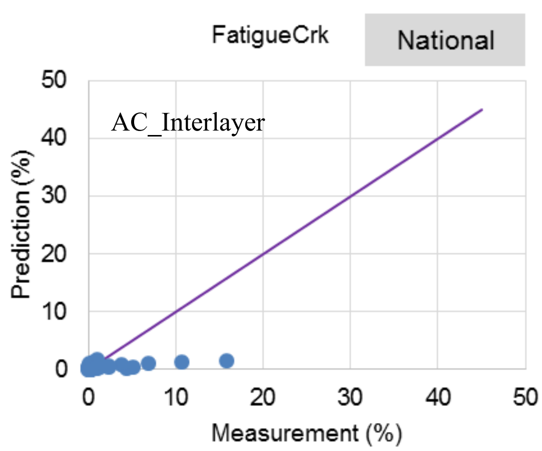
(b) AC over RPCC Base



(c) AC over Stone Base



(d) AC over Soil Cement Base



(e) AC over Stone Interlayer

Figure 22

Predicted vs. measured fatigue cracking of flexible pavements

AC over Soil Cement. Alligator cracking, extreme transverse and longitudinal cracking were observed in the field, especially for projects older than 10 years. This agrees with past research results. The shrinkage of soil cement reflects through the HMA surface and shows up as transverse cracking (note that PMS only recorded transverse cracking, not reflective cracking). For a one-mile long 11-ft. wide road, if reflective cracking occurs at 10 ft. intervals, the measured transverse cracking in PMS would be 5808 ft. However, Pavement ME did not predict fatigue cracking for the soil cement base. As suspected by Djakfar and Roberts, this was probably related to the fact that these cracks are indeed reflective cracking rather than fatigue cracking under load *[63]*.

The structural analysis tool in Pavement ME, JULEA, is mostly used to model unbound granular materials. Soil cement was assumed to have a minimum modulus of 100 ksi and its behavior under load would be different from that of an unbound granular material. Furthermore, the *Manual of Practice* noted that the damage and distress functions with cement treated layers were never calibrated under any NCHRP projects and hence were not recommended for use *[4]*.

Although Pavement ME includes a reflective cracking model, there are two critical issues. (1) The current version of software only considers reflective cracking for overlay projects. If it is a new project with AC over cement treated base, the reflective cracking model is not activated and no data is reported. Although this issue was reported by Saxena et al. *[64]* and Velasquez et al., it has not been solved in Pavement ME *[19]*. (2) The reflective cracking model in Pavement ME is an empirical equation. However, the model was not globally calibrated using an updated database under NCHRP Project 1-40D *[4]*.

For these reasons, soil cement projects were not included in the calibration process in this study. Users need to understand these limitations when designing pavement structures with soil cement layers using Pavement ME.

AC over Stone Interlayer. Field measured fatigue cracking was less than 10% with the exception of Project 201-02-0012 which was 24 years old. This project was the earliest application of stone interlayer structure constructed on state highway LA-97 near Jennings, Louisiana in 1991. Site visits of performance were conducted by researchers at LTRC until 2003 *[65]*. It was observed that, at the age of 10 years, the section with the stone interlayer was superior to the conventional soil cement base structure. In fact, the other nine stone interlayer projects which were less than 10 years old did perform well with very few transverse and longitudinal cracks. Therefore, it can be inferred that the stone interlayer was effective in retarding reflective cracking but did not stop it. In terms of Pavement ME prediction, however, it seems that the limitation on modeling soil cement layer also applies to the stone interlayer structure. Pavement ME predicted very few fatigue cracking.

Overall, it is obvious from Figure 22 that calibration was necessary to improve the prediction power of Pavement ME. Otherwise, no project would fail in fatigue cracking.

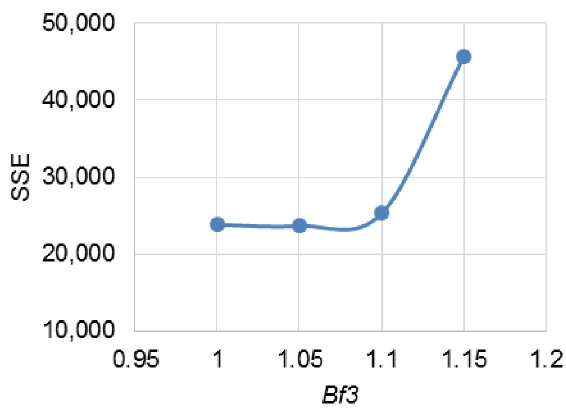
A hypothesis test was carried out to statistically evaluate the model performance (Table 27). As a supplement to visual evaluation of Figure 22, Table 27 indicates that the fatigue cracking model was biased for all flexible pavement types.

Table 27

Hypothesis analysis of fatigue cracking model for flexible pavements

Test	**Null Hypothesis (H_0)**	**National Model**		**Local Calibrated Model**	
		p-value	**Result**	**p-value**	**Result**
Test 1	H_0: mean (predicted-measured) = 0	7.45751E-15	Rejected	0.115751705	Accepted
Test 2	H_0: Slope = 1	9.7161E-243	Rejected	8.2151E-30	Rejected
Test 3	H_0: Intercept = 0	5.35995E-11	Rejected	1.65428E-05	Rejected

Local Calibration. Since Pavement ME under-predicted fatigue cracking, calibration coefficients had to be changed to improve the prediction. Referring to the transfer function equation (18) in Appendix B, the predicted damage *DI* has to be increased to increase the predicted cracking. On the contrary, the allowable axle load applications $\boldsymbol{N_f}$ has to be decreased to increase the predicted damage *DI* according to equation (17). Sensitivity analysis shows that $\boldsymbol{N_f}$ is extremely sensitive to $\boldsymbol{b_{f2}}$ and $\boldsymbol{b_{f3}}$. To reduce $\boldsymbol{N_f}$, $\boldsymbol{b_{f2}}$ has to be reduced, but $\boldsymbol{b_{f3}}$ has to be increased according to equation (16). Increasing $\boldsymbol{b_{f1}}$ to a large scale can also reduce $\boldsymbol{N_f}$. Literature shows that the coefficient on strain $\boldsymbol{b_{f2}}$ is usually hold constant *[3]*. Therefore, this study adjusted $\boldsymbol{b_{f3}}$ and $\boldsymbol{b_{f1}}$ to reduce the difference between predicted and measured fatigue cracking. In this iterative process, $\boldsymbol{b_{f3}}$ of 1.05, 1.1, and 1.15, $\boldsymbol{b_{f1}}$ of 0.5 and 5 were used as trial values. Figure 23 shows the result of this process. It was found that $\boldsymbol{b_{f3} = 1.05}$ and $\boldsymbol{b_{f1} = 1.0}$ achieved the minimum SSE and hence were selected for further analysis.



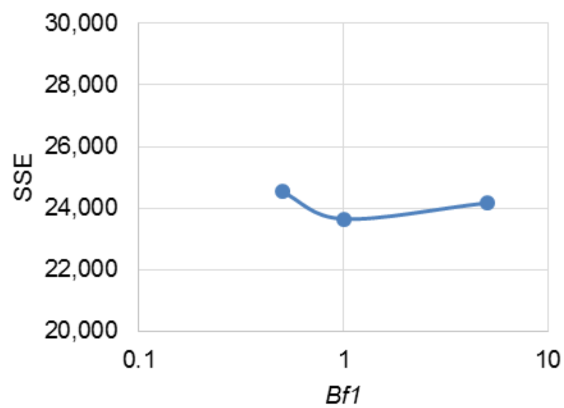


Figure 23

Adjust b_{f3} and b_{f1} to minimize the difference between predicted and measured fatigue cracking

After the allowable load repetition was adjusted, the transfer function was calibrated by adjusting C_1 and C_2. The calibrated S-curve as well as the national default model are shown in Figure 24. Predicted cracking was further increased to match the measured cracking in the field.

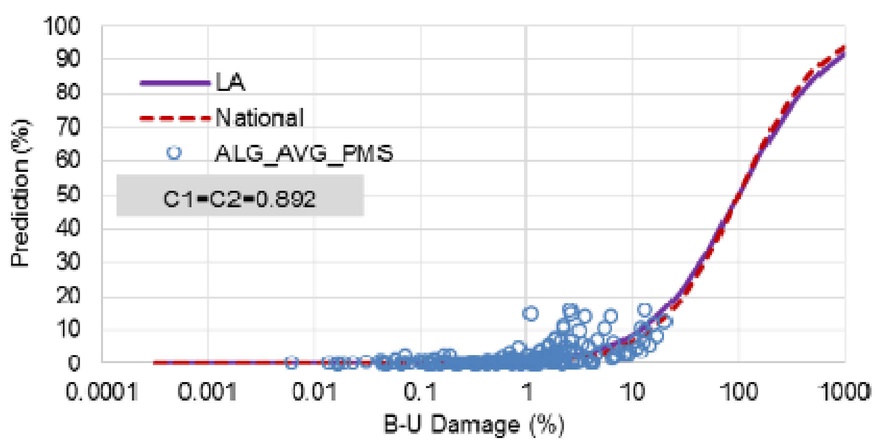


Figure 24

Calibration of the fatigue cracking transfer function

Overall, the national model and calibrated model for fatigue cracking are shown in Figure 25 for the calibration dataset and in Figure 26 for the validation dataset. Summary statistics were also calculated and presented. It is obvious that the calibrated model agrees better with field measurements once the adjustments were made. Bias, standard error, and Se/Sy were reduced. R-square was increased.

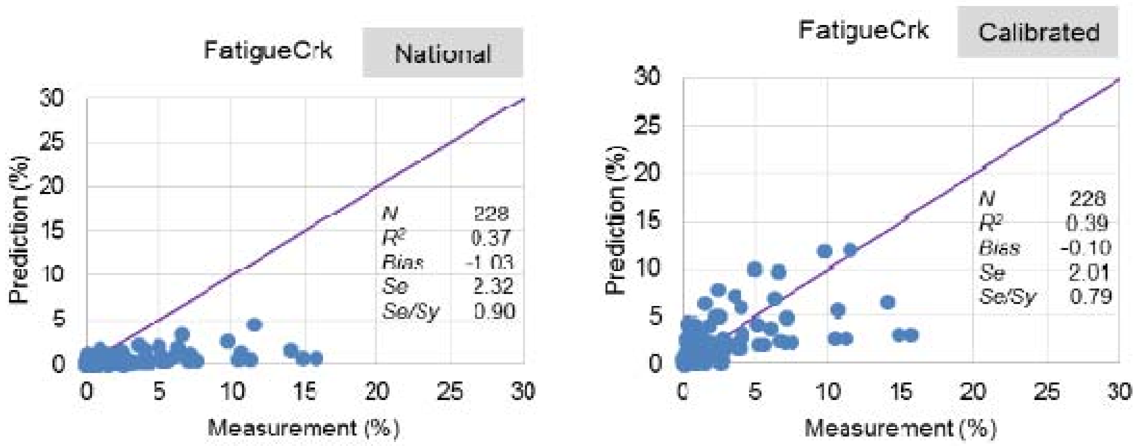


Figure 25

Predicted vs. measured fatigue cracking (a) national model (b) calibrated model (80% dataset)

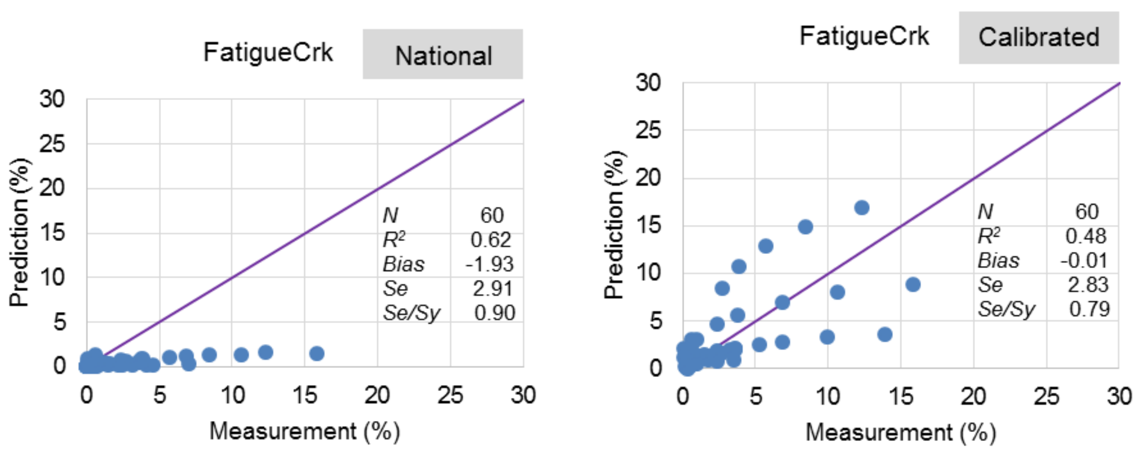


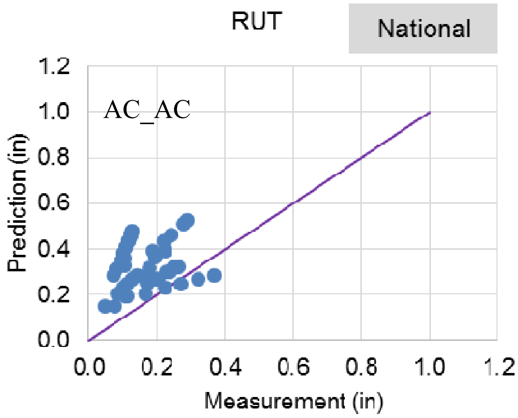
Figure 26

Predicted vs. measured fatigue cracking (a) national model (b) calibrated model (20% dataset)

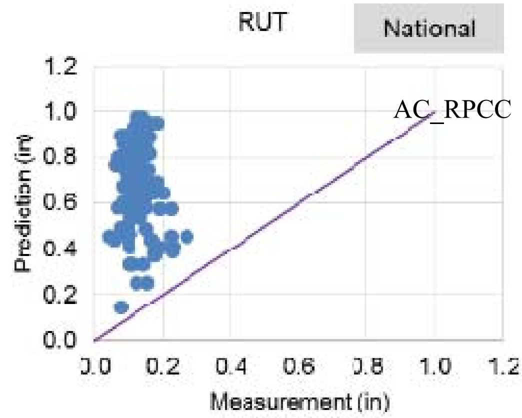
The hypothesis testing was evaluated and the results listed in Table 27. It was confirmed that the bias was reduced to a level that statistically insignificant (Test 1).

Rutting

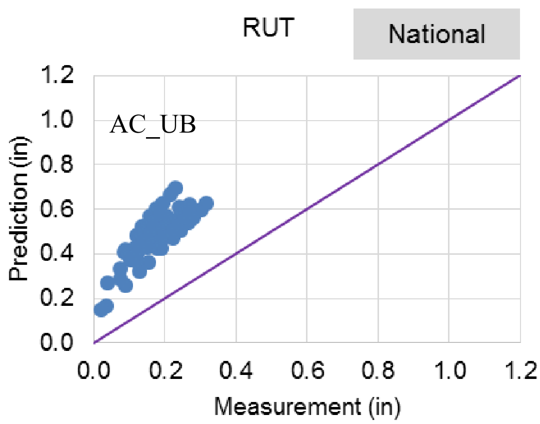
Figure 27 shows the comparison of predicted and measured total rutting. It was found that Pavement ME over-predicts rutting for almost all projects. In particular, the over-prediction was quite severe for AC over RPCC considering that measurements were about 0.2 in. but predictions were up to 1.0 in.



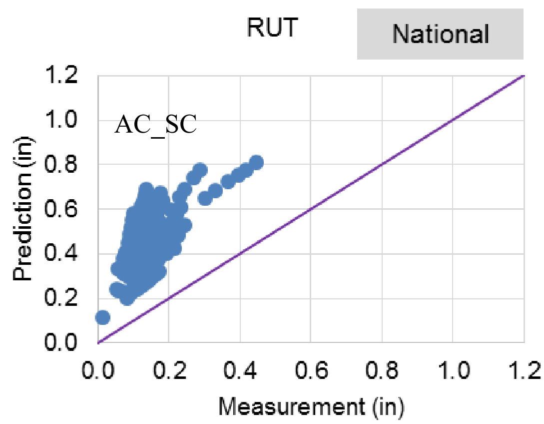
(a) AC over AC Base



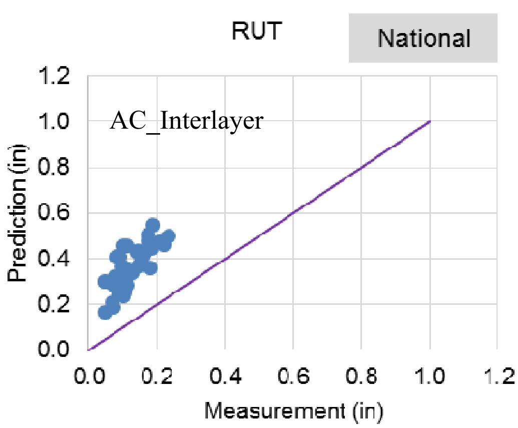
(b) AC over RPCC Base



(c) AC over Stone Base



(d) AC over Soil Cement Base



(e) AC over Stone Interlayer

Figure 27

Predicted vs. measured rutting of flexible pavements (national model)

For AC over AC base, measured rutting reached 0.4 in. and predicted rutting reached 0.6 in. Except for Project 015-05-0038, Pavement ME over-predicted rutting. According to the construction record, the six projects were built around 2000, with a total AC thickness of 10 in. to 13 in. The subgrade was lime treated. Being at 15 years of age that is close to the design life of 20 years, it seemed possible to observe rutting at this level.

For AC over RPCC, measured rutting were less than 0.3 in. This was probably attributed to two reasons. First, all RPCC projects evaluated were on Interstates. Hence, high quality asphalt binder and mixture were applied. Second, the pavement structure, which was composed of 6 in. to 9 in. HMA, 10 in. RPCC and 6 in. soil cement, was structurally sound *[51, 66]*. However, the prediction from Pavement ME was extremely high. A close check revealed that the extreme mainly came from the over-prediction of rutting in the AC layer (up to 0.7 in.) and subgrade layer (on average 0.25 in.). The rutting in RPCC and soil cement base was zero because Pavement ME assumes permanent deformation for stabilized base layers. Referring to equation (20) in Appendix B, the rutting model is a function of resilient strain in HMA, pavement temperature, HMA thickness and number of axle-load repetitions. In other words, the more the load repetition, the more rutting will occur. These RPCC projects were heavily loaded with 20-years design ESALs between 20 and 50 million. Hence, Pavement ME estimated severe rutting in the AC layer. All these suggest the AC rutting model needed to be calibrated.

For AC over unbound base, soil cement, and stone interlayer projects, Figure 27 shows that rutting was also over-predicted by Pavement ME.

It is helpful to understand the composition of total rutting since Pavement ME predicts rutting for individual layers (AC, unbound base, and subgrade). Since the pavement management system only provides the total rutting measurements at the surface and no trench testing was conducted during this study, it was decided to adopt the national default assumption of rutting composition. In other words, the same composition ratio as predicted by Pavement ME was used to decompose the measured total rutting to individual layers. For example, if the predicted total rutting was 0.60 in., consisted of 0.29 in. in AC, 0.05 in. in stone base and 0.26 in. in subgrade, then the measured total rutting 0.30 in. was decomposed proportionally to 0.15 in. in AC, 0.02 in. in base and 0.13 in. in subgrade. Following this procedure, rutting for all projects was analyzed and is shown in Figure 28. It indicates that rutting in the AC and subgrade were most likely being over-predicted. Very little rutting was predicted for the stone base layer.

Table 28 presents the results of hypothesis testing for the rutting model. Statistically, all null hypothesis were rejected. In summary, it was apparent that a local calibration of the rutting model was needed for Louisiana.

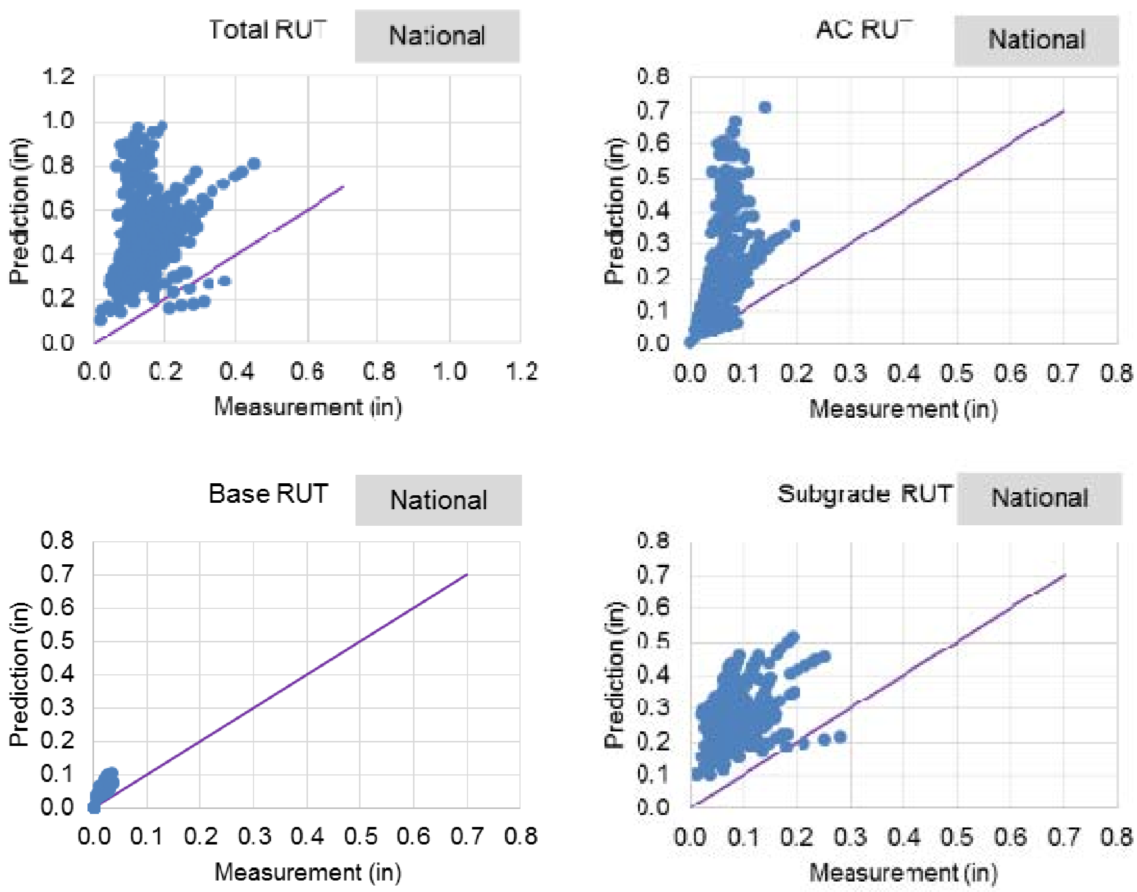


Figure 28

Predicted vs. measured rutting from AC, base, and subgrade layer (national model)

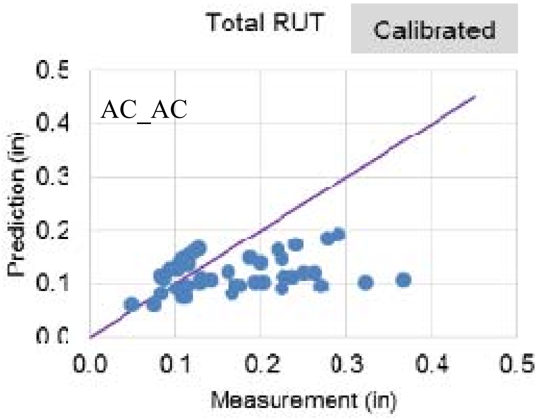
Table 28

Hypothesis analysis of rutting model for flexible pavements

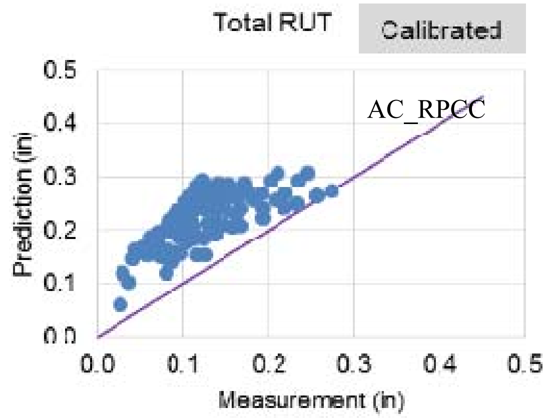
Test	Null Hypothesis (H_0)	National Model		Local Calibrated Model	
		p-value	Result	p-value	Result
Test 1	H_0: mean (predicted-measured) = 0	7.2511E-143	Rejected	8.91619E-31	Rejected
Test 2	H_0: Slope = 1	1.98921E-11	Rejected	1.25523E-58	Rejected
Test 3	H_0: Intercept = 0	1.99786E-85	Rejected	9.42201E-83	Rejected

Local Calibration. The rutting model was calibrated by adjusting calibration coefficients $\boldsymbol{\beta_{r1}}$, $\boldsymbol{\beta_{r3}}$ and $\boldsymbol{\beta_{s1\_SG}}$. As described in the Methodology, a method developed by Wu and Yang *[67]* was used to reduce the number of iterative executions of Pavement ME. Using this method, all projects with the default value and $\boldsymbol{\beta_{r3}}$ = 0.8 were analyzed. Then the parameter α was calculated as the bridge that connects different coefficients. With predicted rutting of individual layers (AC, base, subgrade), the three corresponding coefficients were optimized using Excel Solver. The objective was set to minimize the sum of the error between the predicted and the measured total rutting. The recommended coefficients were $\boldsymbol{\beta_{r1}}$= 0.8, $\boldsymbol{\beta_{r3}}$= 0.85, and $\boldsymbol{\beta_{s1\_SG}}$= 0.4. Figure 29 shows the comparison of total rutting after calibration. Compared to Figure 27, there is a significant improvement after calibration. The contribution from individual layers is presented in Figure 30, which also shows a good match with the estimated measurement.

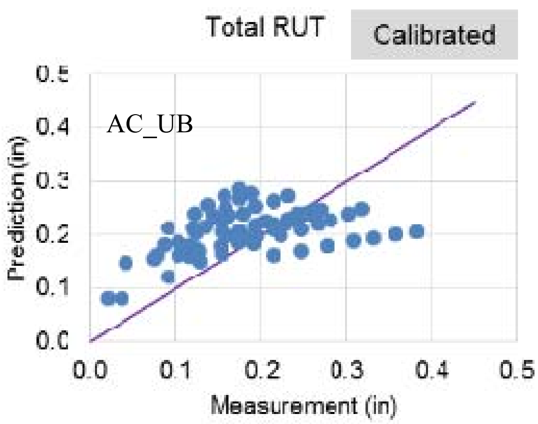
Table 29 lists the summary statistics of the rutting model before and after calibration. It shows that the bias was reduced from 0.35 in. to 0.04 in. The standard error of the estimate was reduced from 0.18 in. to 0.07 in. However, the R square was poor due to the influence of a few projects that had severe measured rutting and low level of prediction. Similarly, three null hypotheses were rejected as presented in Table 28. Nevertheless, the overall comparison for all projects shows a dramatic improvement after calibration. Moreover, design examples as will show later prove that the calibration coefficients can lead to reasonable designs.



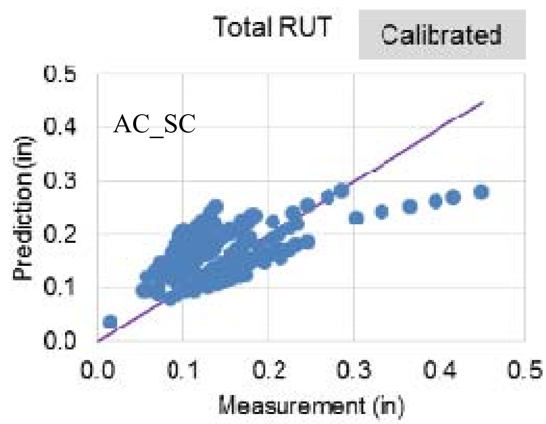
(a) AC over AC Base



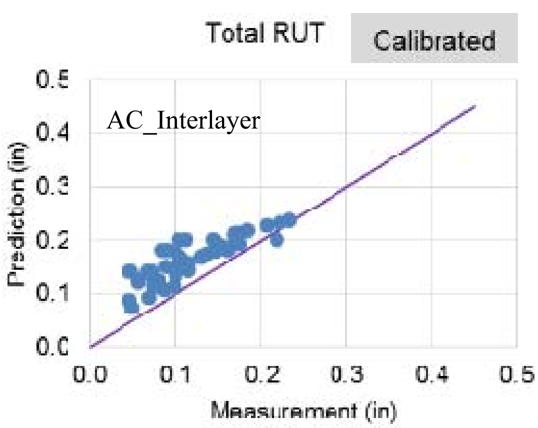
(b) AC over RPCC Base



(c) AC over Stone Base



(d) AC over Soil Cement Base



(e) AC over Stone Interlayer

Figure 29

Predicted vs. measured rutting of flexible pavements (calibrated model)

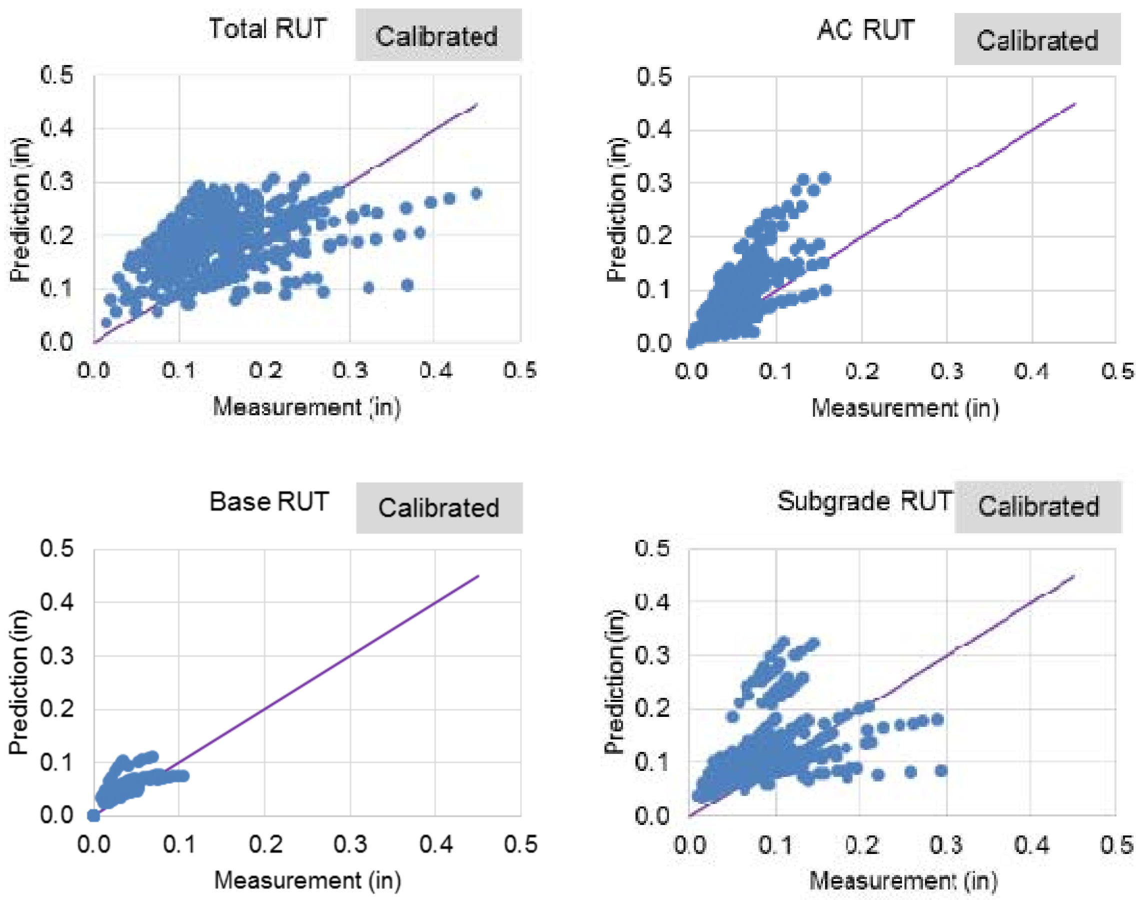


Figure 30

Predicted vs. measured rutting from AC, base and subgrade layer (calibrated model)

Table 29

Summary statistics of rutting model

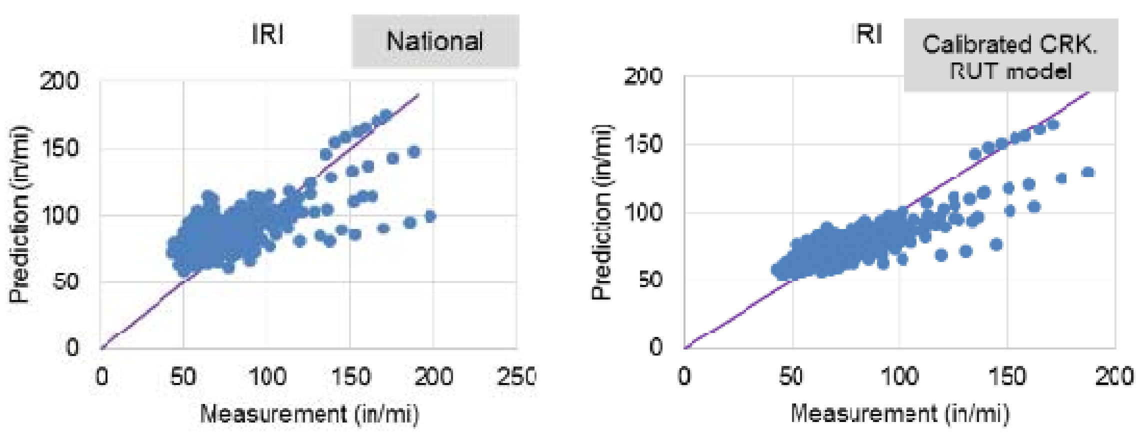
	National model	Calibrated model
N	426	426
R^2	0.04	0.17
Bias (in.)	0.35	0.04
Se (in.)	0.18	0.07
Se/Sy	2.97	1.00

IRI

Figure 31a shows the comparison of predicted IRI from the national model and measured IRI for the selected 71 projects. The first impression was that the IRI model was working much better than alligator cracking and rutting models, although Pavement ME slightly over-predicted IRI with a bias of 11.37 in/mi (Table 30).

The IRI model in Pavement ME is an empirical model of other distresses (fatigue cracking, transverse cracking, and rutting) and site factor. After the cracking and rutting model were calibrated, the IRI model was re-evaluated, as shown in Figure 31b. It shows that the predicted IRI was reduced after the calibration of fatigue cracking and rutting model. R square was increased to 0.69 and bias was negligible. A close look at individual projects revealed that the data points away from the equality line came from AC over soil cement projects, on which high levels of IRI were measured. This was very likely due to the fact that reflective cracking in the field increased the roughness; however, Pavement ME does not simulate reflective cracking from cement stabilized base at this time. Except for these few projects, other projects had a good match of predicted and measured IRI after the fatigue cracking and rutting model have been calibrated. Therefore, the IRI model was deemed suitable for use without a special calibration procedure.

Results of hypothesis testing as listed in Table 31 confirm the above observation.



(a) National model (b) Calibrated cracking and rutting model

Figure 31

Predicted vs. measured IRI of flexible pavements

Table 30

Summary statistics of IRI model

	National model	Calibrated model
N	426	426
R^2	0.53	0.69
Bias (in/mi)	11.37	-0.37
Se (in/mi)	15.20	13.15
Se/Sy	0.69	0.58

Table 31

Hypothesis analysis of IRI model for flexible pavements

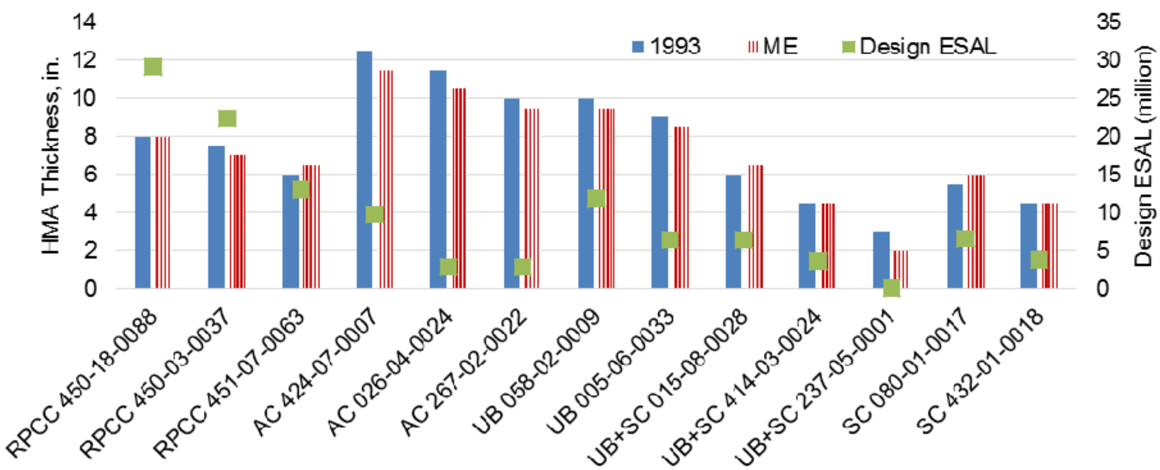
Test	**Null Hypothesis (H_0)**	**National Model**		**Local Calibrated Model**	
		p-value	**Result**	**p-value**	**Result**
Test 1	H_0: mean (predicted-measured) = 0	6.11901E-43	Rejected	0.553624944	Accepted
Test 2	H_0: Slope = 1	3.28298E-60	Rejected	4.21685E-91	Rejected
Test 3	H_0: Intercept = 0	6.14161E-83	Rejected	3.78794E-85	Rejected

Thickness Comparison

The calibrated Pavement ME was used to re-design the selected projects. Traffic, material and climate input remained the same. Design life and criteria as listed in the *Implementation Guidelines* (Appendix A) were followed. If the predicted performance (cracking, rutting and IRI) were higher than design criteria, the thickness of AC layer was increased at 0.5 in. intervals until the predicted performance met all criteria. Contrarily, if the predicted performance were lower than design criteria, the thickness of AC layer was reduced at 0.5 in. intervals until the predicted performance failed any of the criteria. In case the predicted performance did not fail even with a 2 in. thin AC layer, the design was terminated and 2 in. which was recommended as the minimum AC thickness. Finally, the recommended thicknesses from Pavement ME and from the 1993 Design Guide were compared, as shown in Figure 32.

Overall, it was found that the recommended thickness from Pavement ME was comparable to the 1993 Design Guide. The average difference was 0.3 in. with a range mainly between -1 in.

and 1 in. There is no discernible trend about the difference for different projects in Figure 32 but it was realized that the control distress was rutting for RPCC projects and fatigue cracking for other projects. Recall that the rutting model in Pavement ME is directly tied with the number of load repetitions: the more repetition, the more accumulated plastic deformation (rutting). In addition, the design criterion of rutting is 0.4 in. and 0.5 in. for interstate and other highways, respectively. Since the rutting model was calibrated to match field measured rutting, it would be unlikely to predict rutting to a level close to the design criterion for highways other than Interstates. Fortunately, the calibrated fatigue cracking worked well for pavements with different levels of traffic. It is expected that Pavement ME with local calibration coefficients is comparable and can be used interchangeably with the AASHTO 1993 method.



Note: RPCC = rubblized PCC base, AC = asphalt concrete base, UB = unbound granular base, UB+SC = stone interlayer and soil cement base, SC = soil cement base

Figure 32

Recommended thickness from the 1993 Design Guide and Pavement ME design for flexible pavements

Validation and Design Examples

Another five projects out of the evaluation pool were selected from the Track of Project (TOPS) database at DOTD. Starting with the designed structure from the AASHTO 1993 method, Pavement ME was applied to redesign these projects following the *Implementation Guidelines* (Appendix A). Figure 33 presents the recommended thickness. It shows that Pavement ME required a slightly thinner AC layer than the 1993 Design Guide with an average difference of 0.1 in. The design ESAL reported from Pavement ME is also included.

The design was controlled mainly by cracking except projects 452-90-0160 and 450-30-0085, which were controlled by rutting. As discussed before, Pavement ME predicted severe rutting as a result of large load repetitions. In addition, the default HMA input for interstates and other highways is not significantly different in this study. Hence, the current rutting model is more sensitive to traffic than to other factors.

It is also interesting to notice that a stone interlayer could reduce the requirement of HMA thickness. Comparing project 015-07-0044 with project 014-03-0026, they have similar traffic load but the interlayer structure requires 1.5 in. thinner HMA than the unbound base structure requires.

Comparing project 014-03-0026 with project 450-30-0085, the significant contribution of RPCC is obvious. The two projects require the same HMA thickness, but 450-30-0085 with RPCC could carry four times more traffic than the unbound base project does.

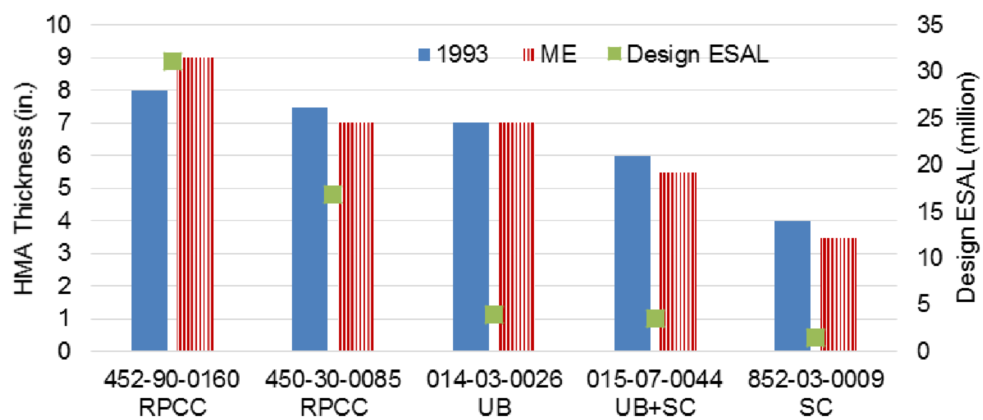


Figure 33

Recommended thickness for flexible pavement design examples

As an example, Figure 34 presents the distress summary for project 450-30-0085. This project reconstructed I-210 near Lake Charles, LA, in 2009. It shows that the 1993 Design Guide passed all criteria. When AC thickness was decreased to 7 in., all design criteria were still met. It failed on fatigue cracking if the thickness was further reduced to 6.5 in. Therefore, the recommended thickness was 7 in., 0.5 in. thinner than the 1993 Design Guide. The predicted IRI, cracking and rutting in the design life are shown in Figure 35.

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	136.25	95.00	99.45	Pass
Permanent deformation - total pavement (in.)	0.40	0.36	95.00	98.74	Pass
AC bottom-up fatigue cracking (percent)	15.00	3.26	95.00	100.00	Pass
AC thermal cracking (ft/mile)	500.00	34.59	95.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	5280.00	335.62	95.00	100.00	Pass
Permanent deformation - AC only (in.)	0.25	0.24	95.00	97.01	Pass
Chemically stabilized layer - fatigue fracture (percent)	25.00	12.20	-	-	-

(a) Distress summary for project 450-30-0085 (AC thickness = 7.5 in.)

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	137.03	95.00	99.40	Pass
Permanent deformation - total pavement (in.)	0.40	0.37	95.00	97.88	Pass
AC bottom-up fatigue cracking (percent)	15.00	6.76	95.00	100.00	Pass
AC thermal cracking (ft/mile)	500.00	34.59	95.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	5280.00	331.08	95.00	100.00	Pass
Permanent deformation - AC only (in.)	0.25	0.25	95.00	95.37	Pass
Chemically stabilized layer - fatigue fracture (percent)	25.00	12.20	-	-	-

(b) Distress summary for project 450-30-0085 (AC thickness = 7 in.)

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	138.05	95.00	99.30	Pass
Permanent deformation - total pavement (in.)	0.40	0.38	95.00	97.71	Pass
AC bottom-up fatigue cracking (percent)	15.00	22.57	95.00	85.11	Fail
AC thermal cracking (ft/mile)	500.00	34.59	95.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	5280.00	329.51	95.00	100.00	Pass
Permanent deformation - AC only (in.)	0.25	0.25	95.00	95.60	Pass
Chemically stabilized layer - fatigue fracture (percent)	25.00	12.20	-	-	-

(c) Distress summary for project 450-30-0085 (AC thickness = 6.5 in.)

Figure 34

Distress summary for project 450-30-0085 as reported by Pavement ME

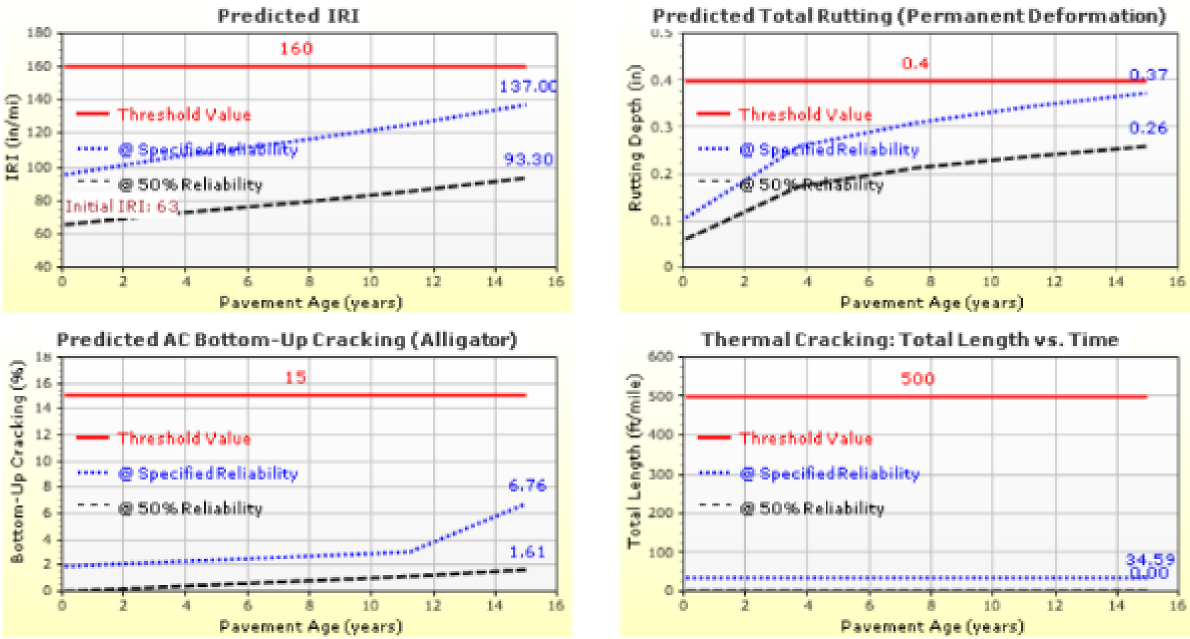


Figure 35

Pavement ME reported distress charts for project 450-30-0085 (at the recommended AC thickness of 7 in.)

Evaluate Rigid Pavement Models

Pavement ME predicts three distress types for rigid pavements: percentage of cracked slabs, joint faulting, and IRI. The three models are discussed separately in the following sections.

Transverse Cracking

The 43 projects with Louisiana specified inputs were modeled in Pavement ME version 2.0 to evaluate the nationally calibrated model. Figure 36 presents the comparison of cracked slabs from Pavement ME and LTRC measured cracking. It shows that Pavement ME over-predicts slab cracking. The bias is more than 60 and the standard error of estimation is 29.77. Summary statistics are also included in Figure 36. Although some studies such as Utah and Missouri found that the national cracking model predicted reasonable results as measured in the field, it was noticed that most of projects in these studies had a joint spacing of 15 ft. *[56] [21]*. However, 20 ft. joint spacing has been the common practice in Louisiana for many years. Sensitivity analysis and several case studies conducted during this research confirmed that Pavement ME is very sensitive to joint spacing. As shown in the MEPDG document Part 3, Chapter 4, Figure 3.4.15, if the joint spacing was increased from 15 ft. to 20 ft., the predicted slab cracking for a 10-in. thick slab would increase from 1% to 88% *[2]*.

The dramatic difference between 15 ft. and 20 ft. can also be explained by the fatigue model and truck configuration. According to the MEPDG document Part 3, Chapter 4, the transverse cracking model of rigid pavement includes bottom-up transverse cracking and top-down transverse cracking *[2]*. When the truck axles are near the longitudinal edge of the slab, midway between the transverse joints, a critical tensile bending stress occurs at the bottom of the slab. This stress increases greatly when there is a high positive temperature gradient through the slab (the top of the slab is warmer than the bottom of the slab). Repeated loadings under this condition result in fatigue damage along the bottom edge of the slab, which eventually results in a transverse crack that propagates to the surface of the pavement. On the contrary, repeated loading by heavy truck tractors with certain axle spacings when the pavement is exposed to high negative temperature gradients (the top of the slab is cooler than the bottom of the slab) result in fatigue damage at the top of the slab, which eventually results in a transverse or diagonal crack that is initiated on the surface of the pavement. Since the average axle spacing for short, medium and long are taken as 12, 15, and 18 ft. respectively in Pavement ME, a joint spacing of 15 ft. would not have any top-down fatigue cracking from trucks with long (18 ft.) axle spacing. In addition, the default composition of short, medium and long wheelbase trucks is 17%, 22%, and 61%, respectively. Hence, if the joint spacing is 20 ft., there will be a large amount of transverse cracking initiated from the top of the slab due to heavy trucks with long axle spacing. In summary, considering the mechanism of transverse cracking in rigid pavement and the default values of traffic, it is not a surprise that 20-ft. long slabs would have much more cracked slabs than 15-ft. long slabs do.

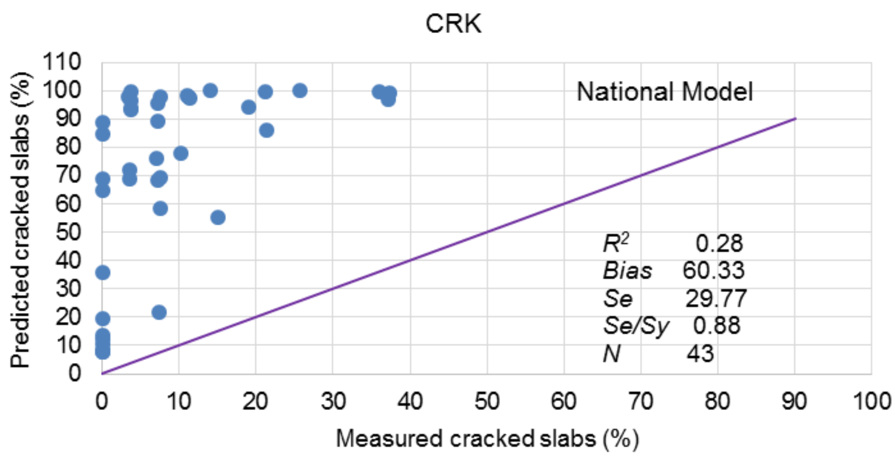


Figure 36

Predicted vs. measured slab cracking of rigid pavements (national model)

Hypothesis tests as recommended by the *Local Calibration Guide [21]* were conducted. Results in Table 32 confirmed that the fatigue cracking model needed to be calibrated for Louisiana.

Table 32

Hypothesis analysis of fatigue cracking model for rigid pavements

Test	**Null Hypothesis (H_0)**	**National Model**		**Local Calibrated Model**	
		p-value	**Result**	**p-value**	**Result**
Test 1	H_0: mean (predicted-measured) = 0	9.54E-16	Rejected	0.066104	Accepted
Test 2	H_0: Slope = 1	0.000381	Rejected	6.15E-07	Rejected
Test 3	H_0: Intercept = 0	3.92E-11	Rejected	0.323075	Accepted

Following the calibration procedure as elaborated in the Methodology, the transverse cracking model was first calibrated with 80% of the projects and then validated with the remaining 20% data. Figure 37 shows the comparison of predicted and measured cracking after calibration. The predicted cracking was clearly reduced and a better match with field measurement was achieved as compared to Figure 36. The bias was reduced from 60.33 to -0.26 (calibration) and -5.84 (validation). The standard error was reduced from 29.77 to 7.86 (calibration) and 11.15 (validation). The coefficient of determination was improved from 0.28 to 0.41 (calibration) and 0.47 (validation).

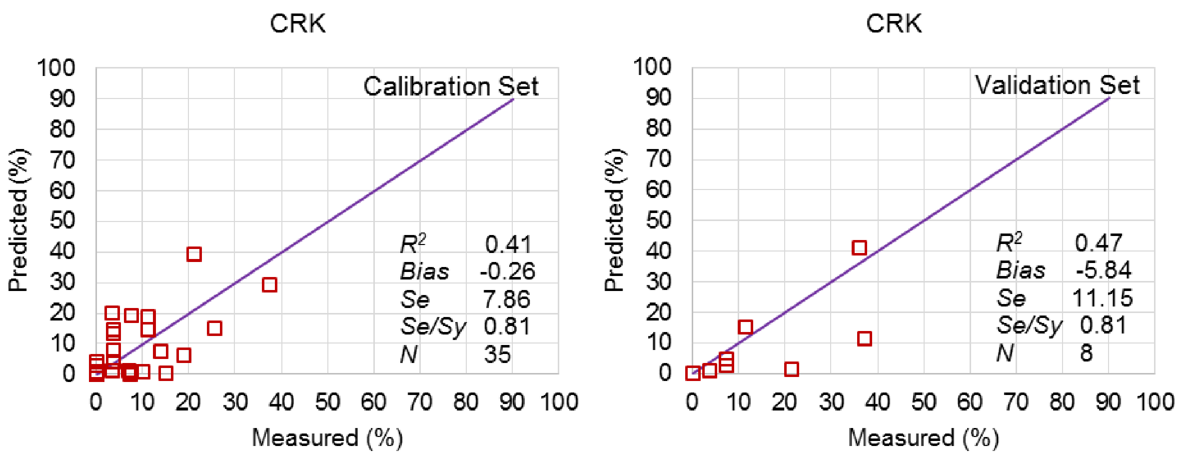


Figure 37

Predicted vs. measured slab cracking of rigid pavements (local calibration, 80% and 20% data)

All data were combined together to develop the calibration coefficients for Louisiana. The recommended coefficients are listed in Table 33. Referring to the transverse cracking model (Appendix B), C_1 was increased so that the allowable load repetition would be increased, which then reduces the predicted fatigue and corresponding transverse cracking. Figure 38 shows the comparison of predictions and measurements, which appear to scatter around the line of equality. Table 32 lists the result of hypothesis testing. The mean error and intercept were accepted as zero. The slope was found to be slightly over one.

Table 33

Local calibration coefficients for transverse cracking model

Model Types	Model Coefficients	National Model	Local Calibrated Model
Fatigue allowable load repetition N_f	C1	2	2.75
	C2	1.22	1.22
Transfer function	C4	1	1.16
	C5	-1.98	-1.73
Standard deviation		Pow(5.3116*CRACK,0.3903) + 2.99	National default

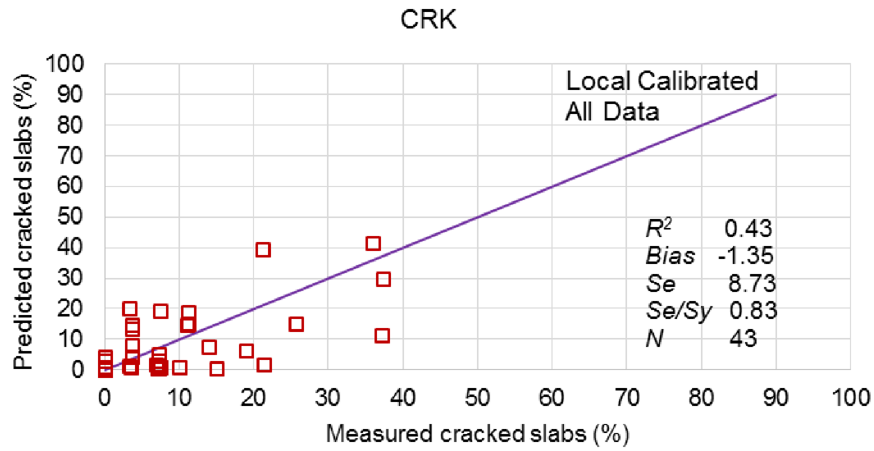


Figure 38

Predicted vs. measured slab cracking of rigid pavements (local calibration, all data)

Figure 39 shows the computed cumulated PCC fatigue damage and measured fatigue transverse cracking. The difference between the national model and the calibrated model is clearly exhibited. The calibrated transverse cracking model shows a good trend with the measured cracking.

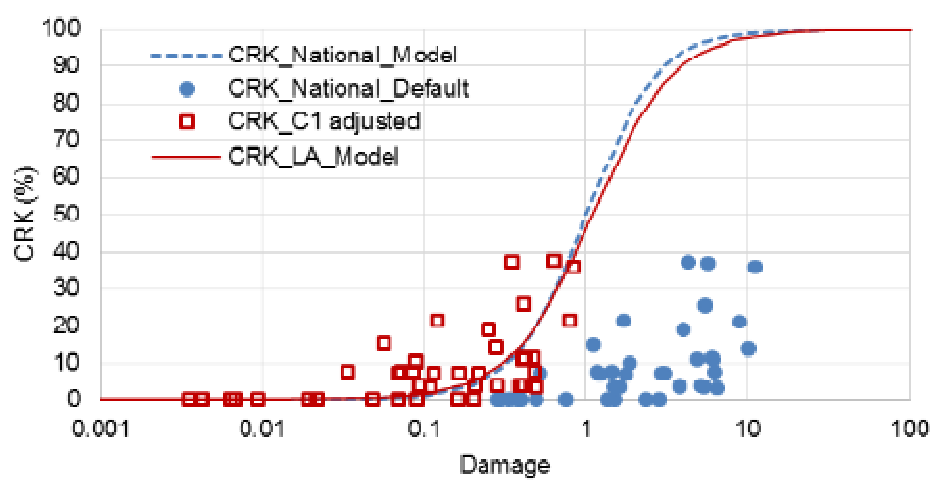


Figure 39

Transverse cracking vs. damage for rigid pavements

Figure 40 and Figure 41 show two examples of the predicted and measured cracking of project 066-07-0027 and 066-07-0030, located in Ville Platte, LA. The initial AADTT for the four-lane road is about 1,500 vehicles per day. The travel lane is 13 ft. wide with curb and gutter. The 1993 Design Guide recommended a 10 in. thick PCC slab. Field performance indicates that the pavement is still in good condition at its 19th year of service. If Pavement ME was used to design the two projects, it would recommend a 9 in. thick PCC slab, one inch thinner than the 1993 Design Guide.

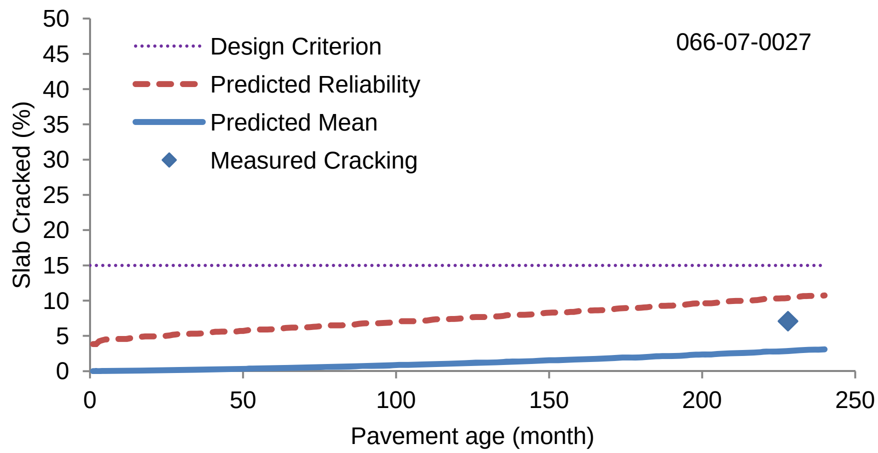


Figure 40

Predicted cracking with calibrated model for project 066-07-0027 (PCC over HMA blanket)

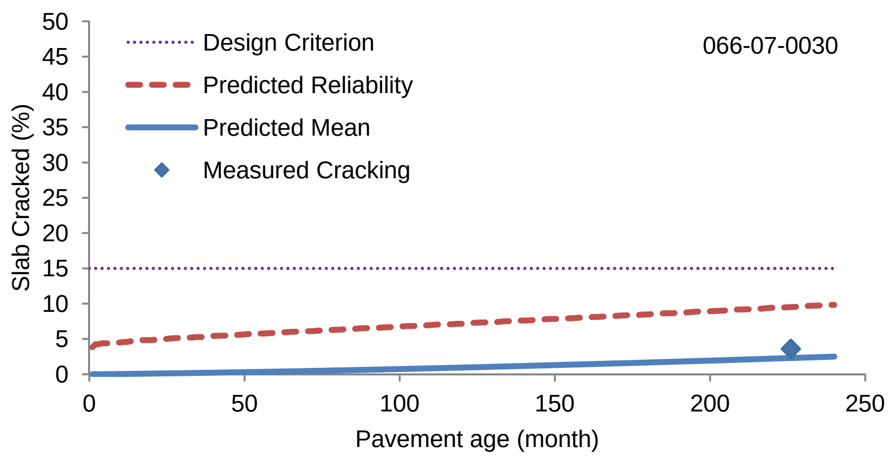


Figure 41

Predicted cracking with calibrated model for project 066-07-0030 (PCC over stone base)

Joint Faulting

Figure 42 presents the comparison of LTRC measured joint faulting and predicted faulting from the national model. It is found that, unlike slab cracking, all faulting data are close to the x-axis, which means that Pavement ME under-predicted joint faulting. Most of the measured faulting was less than 0.1 in., but predictions from Pavement ME were lower than 0.05 in. A close check with individual projects especially the five projects with measured faulting over 0.1 in. indicated that the five projects were all over 20 years old, and longitudinal joints appeared as not being saw cut. The problem with plastic inserts was found to cause random longitudinal cracking and hence not used after 1990s. As recommended by Smith et al. *[68]*, all joints in concrete pavement should be sawed. Although a few states have had success with plastic inserts, most states no longer allow their use. Improper placement of plastic inserts has been identified as a cause of random longitudinal cracking. It is also very difficult to seal the joint formed by plastic inserts. During this study, it was found that longitudinal cracking tends to occur in wheelpaths if plastic inserts were used. These random cracks severely reduced the integrity of the concrete slab and resulted in large faulting.

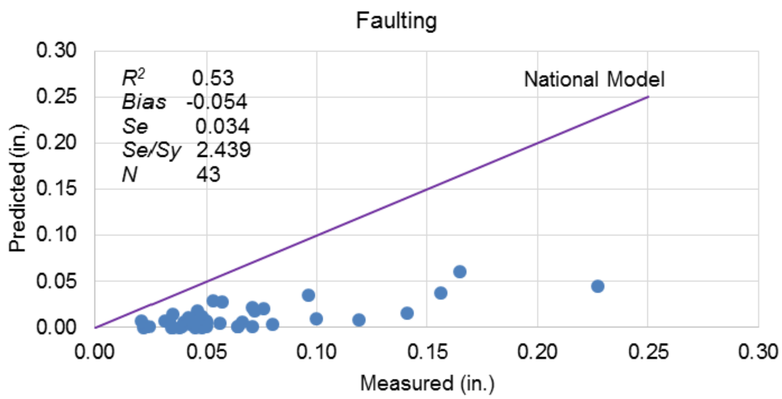


Figure 42

Predicted vs. measured joint faulting of rigid pavements (national model)

The results of hypothesis testing are listed in Table 34 where the intercept was accepted as zero and the other two hypothesis were rejected. Thus, the faulting model had to be calibrated for Louisiana: the predicted faulting needed to be increased.

Table 34

Hypothesis analysis of joint faulting model for rigid pavements

Test	**Null Hypothesis (H_0)**	**National Model**		**Local Calibrated Model**	
		p-value	**Result**	**p-value**	**Result**
Test 1	H_0: mean (predicted-measured) = 0	1.72E-12	Rejected	0.191778	Accepted
Test 2	H_0: Slope = 1	6.92E-08	Rejected	2.81E-07	Rejected
Test 3	H_0: Intercept = 0	0.117842	Accepted	0.388561	Accepted

Following the calibration procedure as elaborated in the Methodology, the faulting model was first calibrated with 80% of the projects and then validated with the remaining 20% data. Figure 43 shows the comparison of predicted and measured faulting after calibration. It shows that predictions were increased to better match the field performance. The bias was reduced from -0.054 to close to zero in the calibration dataset and -0.018 in the validation dataset.

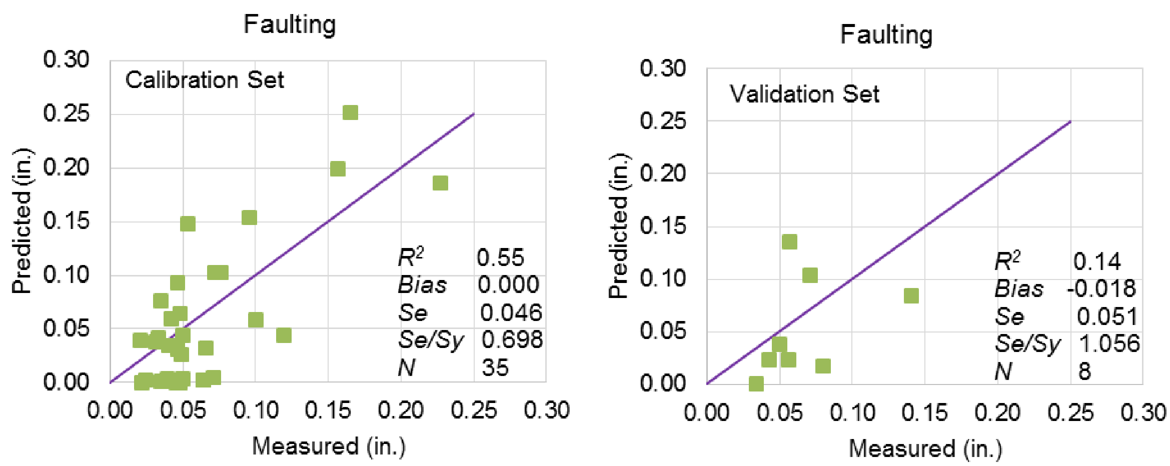


Figure 43

Predicted vs. measured joint faulting of rigid pavements (local calibration, calibration and validation set)

All data were used to develop the calibration coefficients for Louisiana as shown in Table 35. C_1, C_3, and C_6 were increased to increase the predicted faulting [equations (27) to (30) in Appendix B]. Figure 44 presents the comparison of measurements with predictions from the calibrated faulting model. Compared to the national model (Figure 42), a better match was achieved. The bias was reduced from -0.054 in. to -0.009 in. It should be explained that R^2 was reduced from 0.53 to 0.50 because the national model consistently under-predicted joint faulting (statistically a small variation). The calibrated model was capable of predicting joint faulting up to 0.25 in. with a slightly increased variation. The faulting model is very sensitive to slab width: widened slabs greatly reduce the potential of joint faulting, as projects with widened slabs are under the line of equality in Figure 44, while projects with un-widened slabs are above the line of equality. In addition, joint faulting in Pavement ME is the accumulation of monthly faulting; hence, pavements under heavy trafficking tend to have a large faulting.

Table 35

Local calibration coefficients for faulting model

Model Coefficients	National Model[1]	Local Calibrated Model
C1	1.0184	1.5276
C2	0.91656	0.91656
C3	0.0021848	0.00262
C4	0.000883739	0.000883739
C5	250	250
C6	0.4	0.55
C7	1.83312	1.83312
C8	400	400
Standard deviation	Pow(0.0097*FAULT,0.5178)+0.014	National default
Note: [1] These coefficients are according to Pavement ME software Version 2.0. They are different from the *Manual of Practice [4]* as shown in Appendix B.		

The results of hypothesis test are listed in Table 34. Statistically, it is verified that the bias and intercept are zero. The slope is 1.029, slightly over 1.0.

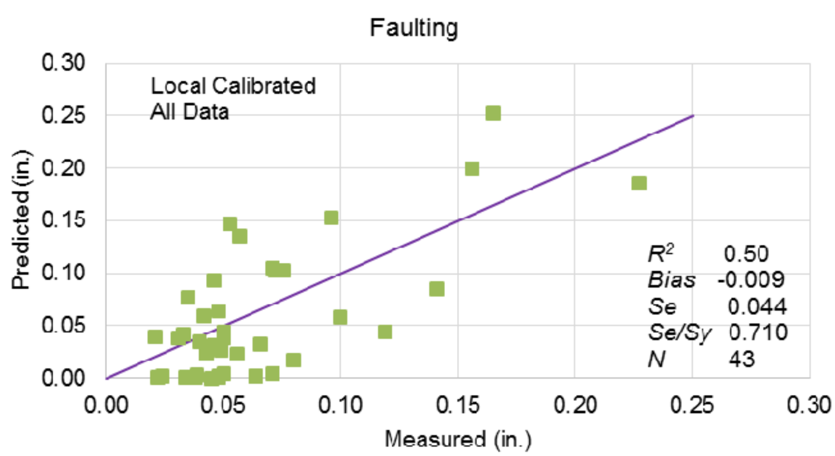


Figure 44

Predicted vs. measured joint faulting of rigid pavements (local calibration, all data)

Figure 45 shows an example of predicted faulting for project 451-04-0029, I-20 between Minden and Ruston, LA. The typical section for this four-lane road was 13 in. PCC surface, 2 in. HMA blanket base course, and 6 in. cement treated base. The outside lane was 15 ft. with the travel lane marked at 12 ft. The shoulder was 7 ft. PCC. Figure 45 shows that the predicted faulting from the calibrated model matches very well with the faulting in the field estimated from ProVAL based on longitudinal profile data.

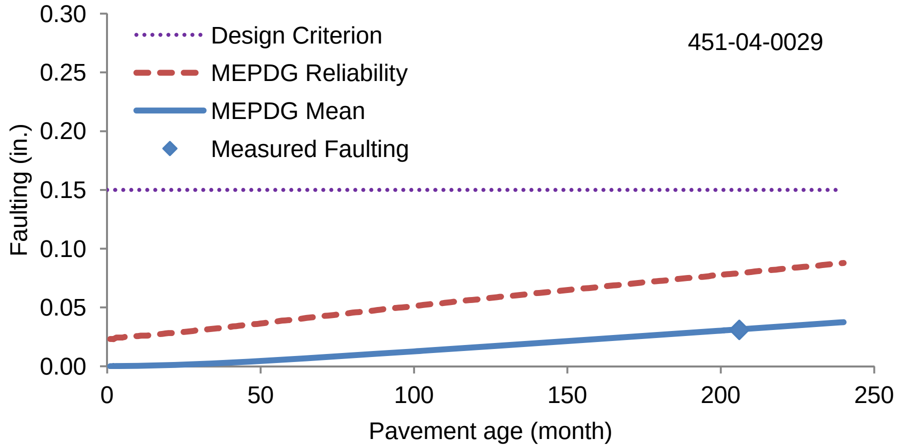


Figure 45

Predicted faulting with calibrated model for project 451-04-0029 (PCC over HMA blanket, slab widened to 15 ft.)

Figure 46 shows an example of predicted faulting for project 062-03-0019, LA 23 near West Point A La Hache, LA. The typical section for this four-lane road was 10 in. PCC surface and 8 in. stone base course. The travel lane was 12 ft. with a 10 ft. HMA shoulder. Figure 46 shows that the predicted faulting from the calibrated model matches well with the faulting in the field.

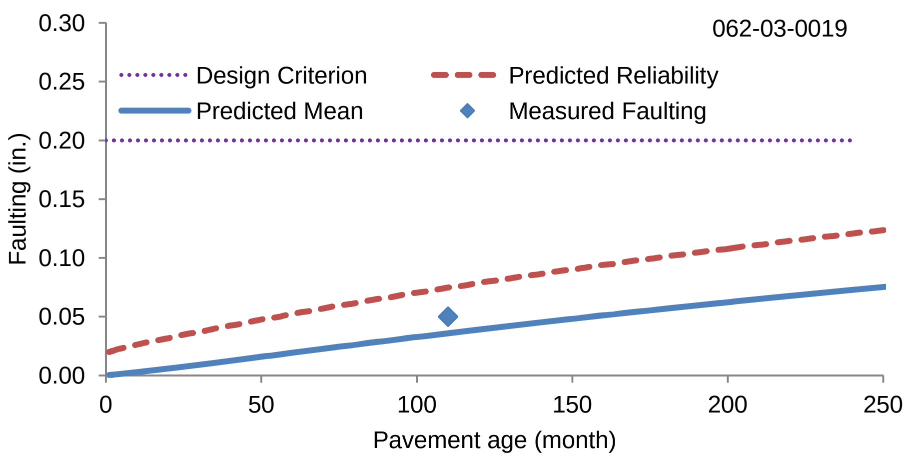


Figure 46

Predicted faulting with calibrated model for project 062-03-0019 (PCC over stone base, 12 ft. slab width)

IRI

The measured IRI versus predicted IRI before calibration is presented in Figure 47. It is found that Pavement ME over-predicted IRI. This is probably due to the over-prediction of cracking as shown in Figure 36. Summary statistics and hypothesis testing (Table 36) indicate that the national model needed to be calibrated.

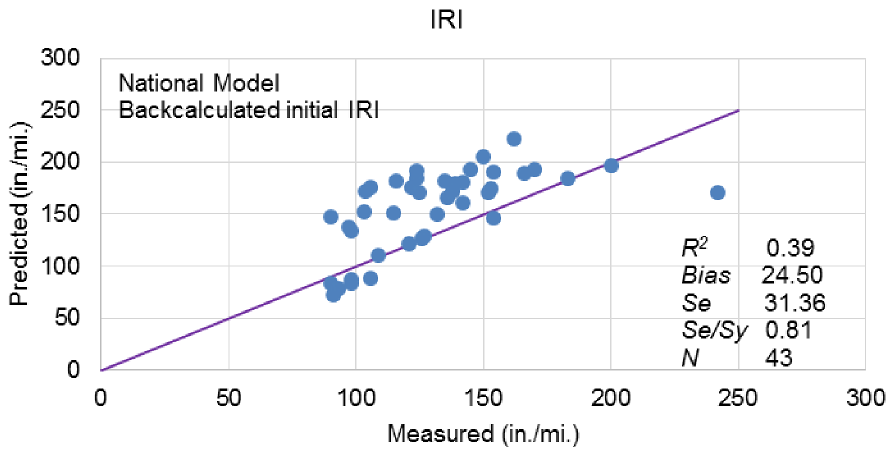


Figure 47

Predicted vs. measured IRI of rigid pavements (national model)

Table 36

Hypothesis analysis of IRI model for rigid pavements

Test	Null Hypothesis (H_0)	National Model		Local Calibrated Model	
		p-value	Result	p-value	Result
Test 1	H_0: mean (predicted-measured) = 0	1.43E-05	Rejected	0.226327	Accepted
Test 2	H_0: Slope = 1	1.21E-05	Rejected	1.71E-10	Rejected
Test 3	H_0: Intercept = 0	0.008956	Rejected	0.739955	Accepted

Before calibrating the IRI model coefficients, the coefficients for cracking and faulting models should be applied because the IRI model is an empirical model of cracking, faulting and site factor [equation (31) in Appendix B]. Therefore, coefficients in Table 33 and Table 35 were applied and Pavement ME was executed for each project. The predicted IRI was then compared with measured IRI as shown in Figure 48. It was found that predicted IRI with calibrated cracking and faulting model matched very well with measured IRI. Compared to Figure 47, the bias was reduced from 24.50 to -4.58 in/mi and the coefficient of determination was improved from 0.39 to 0.65. Hypothesis testing showed that the bias and intercept were accepted as zero. The slope was 1.0055, very close to 1.0. It appeared that there was no need to further adjust any coefficient in the IRI model.

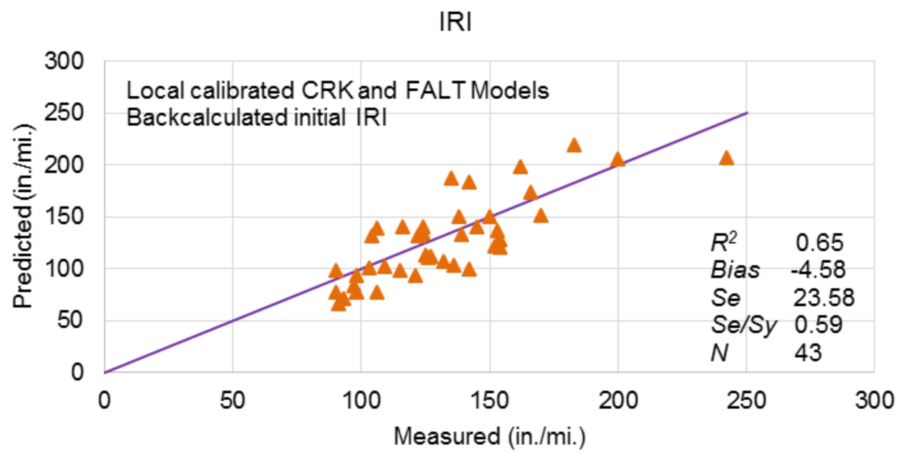


Figure 48

Predicted vs. measured IRI of rigid pavements (with calibrated cracking and faulting model)

Figure 49 shows an example of predicted IRI for project 451-04-0029, I-20 between Minden and Ruston, LA. The typical section of this four-lane road was 13 in. PCC surface, 2 in. HMA blanket base course and 6 in. cement treated base. The outside lane was 15 ft. with the travel lane marked at 12 ft. The shoulder was 7 ft. PCC. The initial IRI was backcalculated as 75 in/mi based on the IRI data in PMS. Figure 49 shows that the predicted IRI with calibrated cracking and faulting models matched well with the measured IRI in the field.

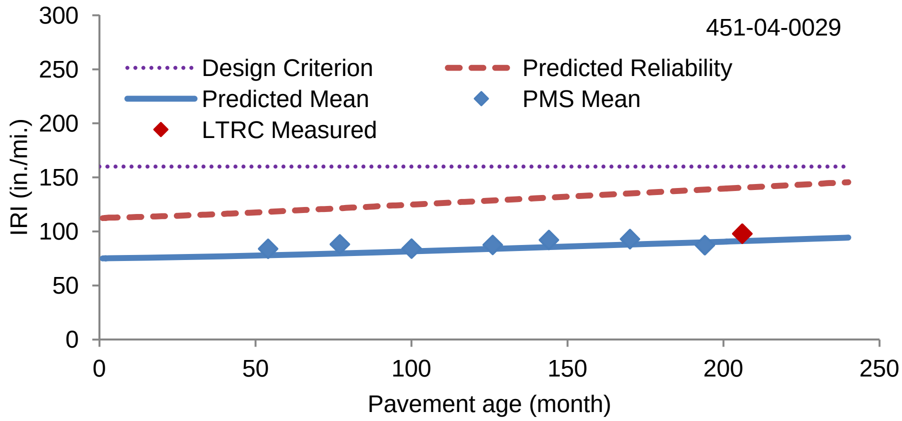


Figure 49

Predicted faulting with calibrated model for project 451-04-0029 (PCC over HMA blanket, slab widened to 15 ft., initial IRI 75 in/mi)

Figure 50 shows an example of predicted IRI for project 062-03-0019, LA 23 near West Point A La Hache, LA. The typical section for this four-lane road was 10 in. PCC surface and 8 in. stone base course. The travel lane was 12 ft. with a 10 ft. HMA shoulder. The initial IRI was backcalculated as 90 in/mi based on the IRI data in PMS. Figure 50 shows that the predicted IRI with calibrated cracking and faulting model matched well with the measured IRI in the field.

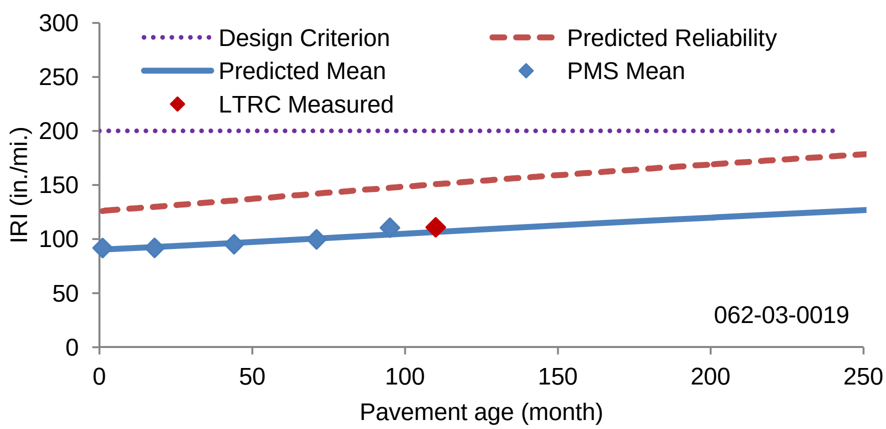


Figure 50

Predicted faulting with calibrated model for project 062-03-0019 (PCC over stone base, 12 ft. slab width, initial IRI 90 in/mi)

In summary, the results showed that the national models for rigid pavement need to be locally calibrated. After local calibration, all models (transverse cracking, faulting, and IRI) produced reasonable results that matched well with field measurement. Table 37 summarizes the calibration coefficients for Louisiana. This conclusion will be further verified by thickness comparison and design examples outside of the 43 evaluation sections using Pavement ME.

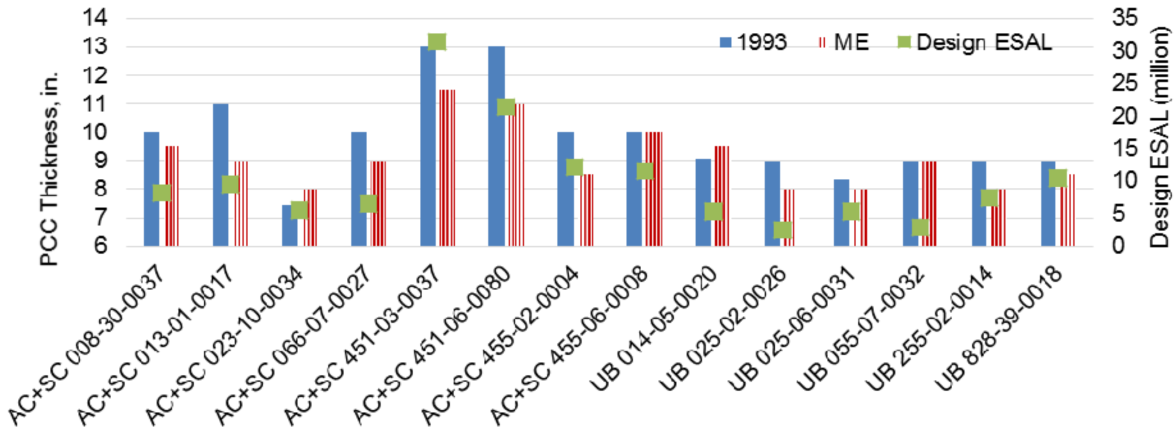
Table 37
Local calibration coefficients for new rigid pavements in Louisiana

Distress	Calibration coefficients	National default	Louisiana
PCC Cracking	C1	2	2.75
	C2	1.22	Default
	C4	1	1.16
	C5	-1.98	-1.73
	Standard deviation	Pow(5.3116*CRACK,0.3903) + 2.99	Default
PCC Faulting	C1	1.0184	1.5276
	C2	0.91656	Default
	C3	0.0021848	0.00262
	C4	0.000883739	Default
	C5	250	Default
	C6	0.4	0.55
	C6	1.83312	Default
	C8	400	Default
	Standard deviation	Pow(0.0097*FAULT,0.5178)+0.014	Default

Thickness Comparison

The calibrated Pavement ME was used to re-design the selected projects, using the same procedure as for flexible pavements. A thickness of 0.5 in. was the interval of adjustment until the predicted performance first failed any of the criteria. Note that this process of thickness design could be automated through the optimization tool in Pavement ME; but this study was conducted manually with the batch analysis tool to save time. The two approaches were confirmed to produce the same recommended thickness.

The recommended thickness from Pavement ME and 1993 Design Guide are compared in Figure 51. Overall, it is found that the recommended thickness from Pavement ME is comparable to 1993 Design Guide, with an average difference of 0.7 in. (ME requires thinner). Most projects were controlled by cracking distress because the faulting model was very sensitive to slab width— widened slab as used on interstates and other roadways significantly reduces the potential of joint faulting. Figure 51 also includes the design ESAL which represents the total amount of traffic loading in the design life. The general trend of "higher traffic, thicker pavement" is observed.



Note: AC+SC = Asphalt blanket layer and soil cement base, UB = unbound granular base

Figure 51

Recommended thickness from the 1993 design and Pavement ME design for rigid pavements

Validation and Design Examples

Five projects including Interstate, U.S. highways, and local roads outside of the evaluation pool were selected as design examples. Figure 52 presents the recommended PCC thickness from 1993 Design Guide and the Pavement ME design. It shows that the calibrated model suggests about one inch thinner concrete slab than the AASHTO 1993 Design Guide.

Note that the design criteria and recommended inputs for Louisiana (Appendix A) were applied. That is, the 28 day modulus of rupture for concrete is 600 psi; the coefficient of thermal expansion for concrete is 5.5×10^{-6} /°F; dowel bar diameter is 1.25 in. for slab thickness less than 10 in. and 1.5 in. for 10 in. and above. The major variables changed were slab width, slab thickness, dowel bar size, shoulder support, traffic, and climate station. It was learned that the cracking model is sensitive to slab thickness whereas the faulting model is not. Therefore, transverse cracking was the distress in control. In addition, based on predicted distresses, normal slab width without widening (12 ft.) is suitable for normal traffic such as local roads and US highways. But widening appears to be required for high volume roads such as the Interstate; otherwise, the required slab thickness would be impractically thick to meet the cracking and faulting criteria. Widening the slab or increasing the dowel bar size can significantly reduce joint faulting as well as transverse cracking.

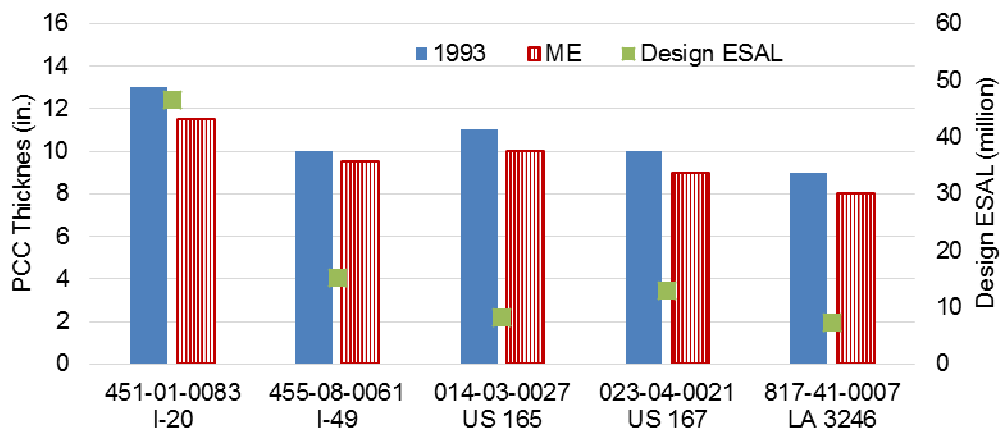


Figure 52

Required PCC slab thickness using different design models

Take project 014-03-0027 as an example. This project was to widen US-165 to a four lane road with a 14-ft. two way left turn center lane near Kinder, LA. The 1993 Design Guide requires 11 in. PCC surface on top of 12 in. Class II base course for the 20 years design life. Figure 53 shows that the national model predicted an erroneous transverse cracking of 114% but very low joint faulting. This is because the national model predicted extremely high tensile stress for the 20-ft. long slabs. After applying the local calibration coefficients, the predicted cracking and faulting seem to be normal in a reasonable range. In fact, the slab thickness could be reduced to 10 in. and still meet all criteria. But if the thickness was further reduced to 9.5 in., the predicted cracking would be more than 15% and faulting be more than 0.2 in. Therefore, the recommended thickness is 10 in., one inch less than the 1993 Design Guide. For this 3-mile-long project, the one inch difference would save concrete a total of 3,302 cu. yd., or $227,400 at a price of $75 per cu. yd. Figure 54 shows the predicted IRI, faulting, and slab cracking in the design life from Pavement ME at the recommended design thickness 10 in.

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	200.00	207.78	90.00	86.60	Fail
Mean joint faulting (in.)	0.20	0.05	90.00	100.00	Pass
JPCP transverse cracking (percent slabs)	15.00	114.35	90.00	0.00	Fail

(a) National model, PCC thickness = 11 in.

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	200.00	141.81	90.00	99.87	Pass
Mean joint faulting (in.)	0.20	0.13	90.00	99.86	Pass
JPCP transverse cracking (percent slabs)	15.00	8.69	90.00	99.29	Pass

(b) Local calibrated model, PCC thickness = 11 in.

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	200.00	145.63	90.00	99.78	Pass
Mean joint faulting (in.)	0.20	0.13	90.00	99.86	Pass
JPCP transverse cracking (percent slabs)	15.00	13.21	90.00	94.01	Pass

(c) Local calibrated model, PCC thickness = 10 in.

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	200.00	190.98	90.00	93.16	Pass
Mean joint faulting (in.)	0.20	0.23	90.00	77.81	Fail
JPCP transverse cracking (percent slabs)	15.00	17.09	90.00	83.99	Fail

(d) Local calibrated model, PCC thickness = 9.5 in.

Figure 53

Distress summary for project 014-03-0027 as reported by Pavement ME

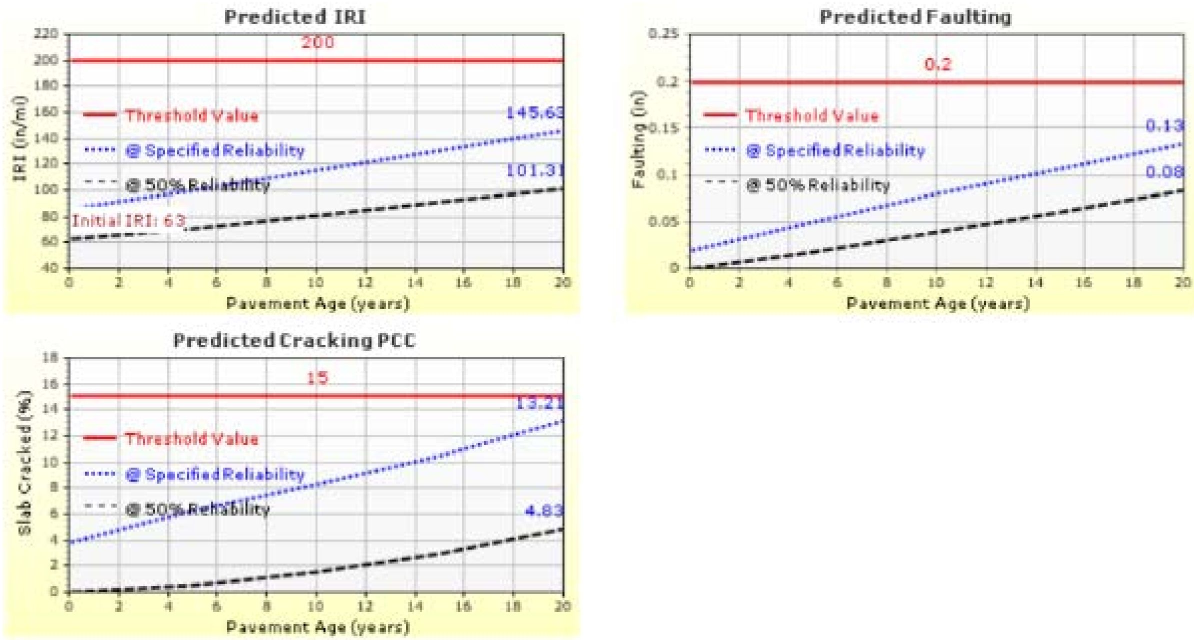


Figure 54

Pavement ME reported distress charts for project 014-03-0027 (at recommended PCC thickness of 10 in.)

Evaluate Pavement Rehabilitation Models

For AC overlay design, Pavement ME predicts both fatigue alligator cracking and reflective cracking. They are combined together and referred to as total cracking. To better understand the two models, fatigue cracking and reflective cracking were compared separately with field measurements in this study.

Fatigue Cracking

Figure 55 shows the comparison of predicted and measured alligator cracking for the three AC overlay subtypes. It was found that both measured and predicted fatigue cracking were at low levels. Except for project 025-01-0036 and 424-08-0030, all measured alligator cracking was lower than five percent. If the method of grouping was used to compare predictions with measurements, 95% of them would fall into the same group of 0~5% *[21]*. This indicated that alligator cracking seemed to not be a problem for AC overlays because in most cases a project was overlaid only when the existing structure was found structurally sound; otherwise, other maintenance methods would be needed. This was especially true considering that the overlay projects selected in this study were structural overlays instead of functional overlays.

The calibration coefficients for fatigue cracking as developed in new flexible pavement design were applied to overlays. The results are presented in Figure 56. Compared to Figure 55, the predicted fatigue cracking was increased. The maximum was increased from 5% to 20%. This was expected since the calibration coefficients were developed to increase predictions to better match the measured fatigue cracking as shown in Figure 25 and Figure 26. However, it should be pointed out that the predicted fatigue cracking was still low (close to zero) for most projects even after the calibration coefficients being applied. Data points with predicted cracking larger than one percent were from project 206-01-0011 and 037-02-0037.

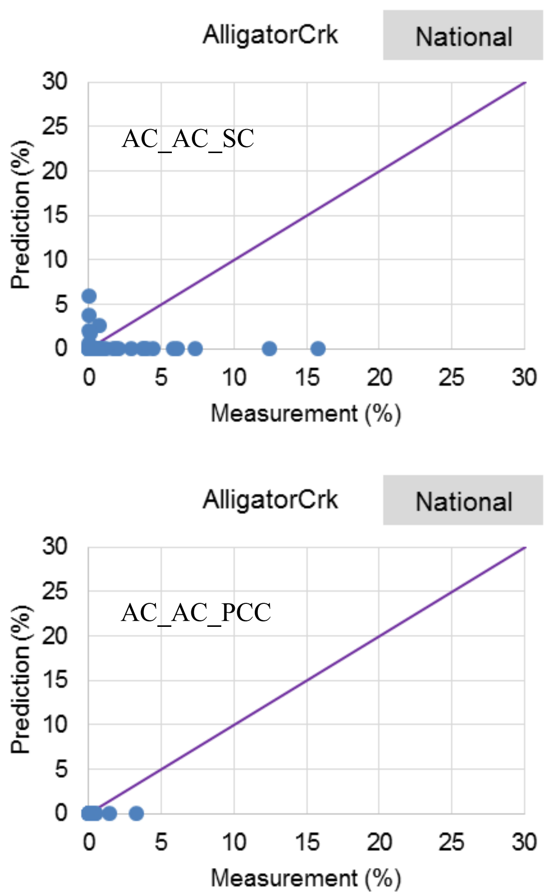


Figure 55

Evaluation of national models for overlay pavements: fatigue cracking

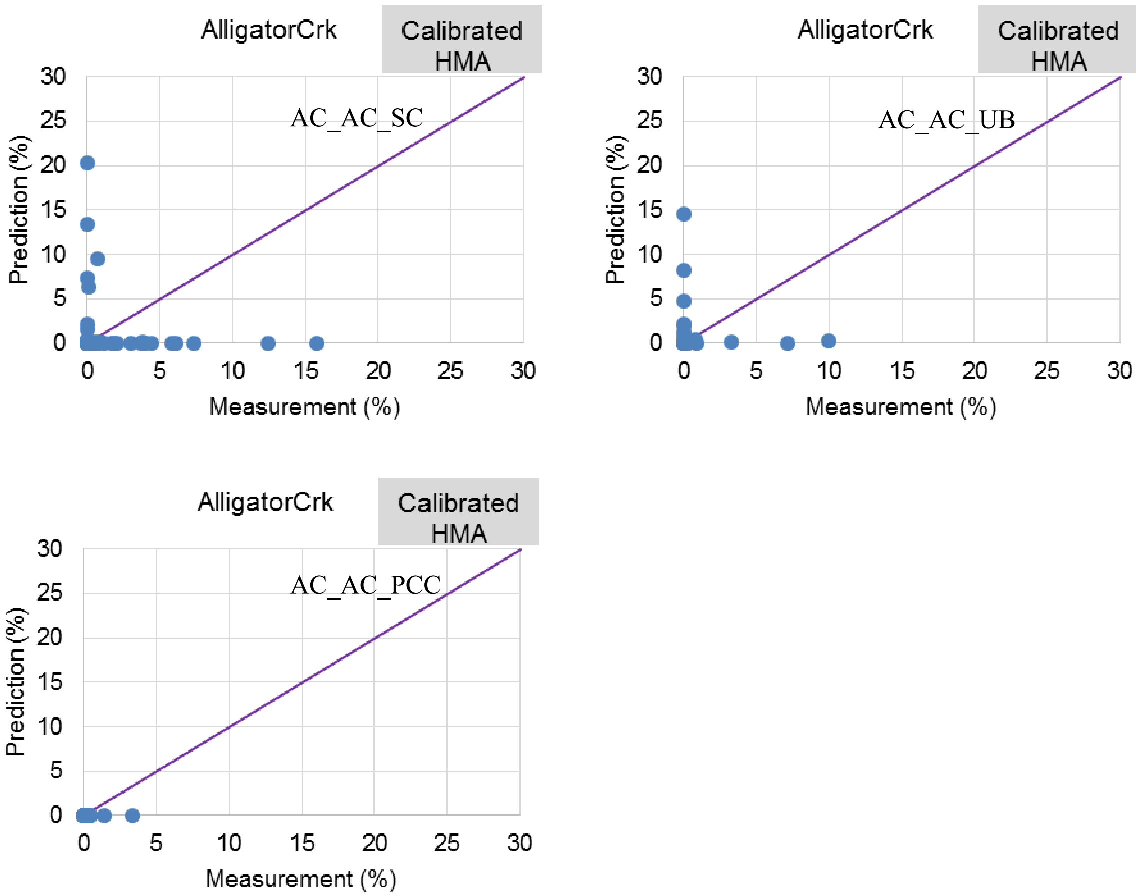


Figure 56

Evaluation of calibrated models for overlay pavements: fatigue cracking

Reflective Cracking

Pavement ME predicts reflective cracks in HMA overlays or HMA surfaces of semi-rigid pavements using an empirical equation. As explained in detail in Appendix B, this empirical equation predicts the percentage of area of cracks that propagate through the HMA as a function of time using a sigmoid function. The thickness of HMA overlay is the only parameter in this empirical model. A thicker overlay retards the occurrence of reflective cracks to a longer time than a thinner overlay could do.

The pavement management system only records the total length of transverse cracking in every 0.1 mile section. There is no separate recognition of reflective cracking. In other words, transverse cracking as recorded in PMS includes transverse cracking and reflective cracking. Therefore, certain assumptions had to be made to utilize the PMS data to evaluate the reflective cracking model in Pavement ME. This was decided as necessary because (1) reflective cracking in HMA overlays has been a serious concern and challenge associated with pavement rehabilitation *[69]*. Countless techniques and studies have been conducted to mitigate/retard the occurrence of reflective cracking in overlays. (2) Reflective cracking is the main distress of semi-rigid pavements (HMA on top of a cement treated or stabilized base layer) where shrinkage cracks in the base layer propagate to the surface layer *[70]*. Considering the wide use of soil cement and the warm weather in Louisiana, transverse cracking measured on the surface of such pavements are most likely reflective cracking rather than thermal cracking.

According to the *Manual of Practice,* the percent area of reflection cracking from Pavement ME is calculated by crack length multiply by a width of 1 ft. *[4]*. Hence, transverse cracking was converted to reflective cracking in this study according to equation (2).

Figure 57 compares the predicted reflective cracking and the estimated measured reflective cracking. To show the progression trend, predictions and measurements are plotted separately against time on two graphs. The increasing trend of reflective cracking as time progresses is found in both the prediction and measurement graphs. Quantitatively, the maximum cracking prediction is about 60% while the maximum cracking measurement is about 20%. Pavement ME over-predicted reflective cracking.

In addition, Figure 57 shows that the predicted reflective cracking for AC over composite pavement (AC_AC_PCC) is much less than it is for sections with soil cement and unbound base. Several trial analyses reveal that the prediction would be greatly reduced whenever the base is a chemical stabilized layer. This could be due to the different models Pavement ME uses to simulate stabilized layers and unbound layers or the higher modulus of stabilized layers compared to unbound layers. Although the reason is not clear, Figure 57 looks reasonable considering the fact that, in general, a sound pavement structure (such as AC over existing PCC) would have less cracking than a weaker pavement structure (such as AC over unbound base) would have. It should also be clarified that Pavement ME could not directly model the structure of AC over existing AC over PCC. The software only provides design options for AC over existing AC and AC over existing PCC. In this study, the former option was used by modeling PCC as a stabilized layer with a resilient modulus of 1,000 ksi and modulus of rupture of 600 psi. The modulus was determined as a conservative value since new PCC has a modulus of 4,200 ksi and rubblized PCC has a modulus of 200 ksi.

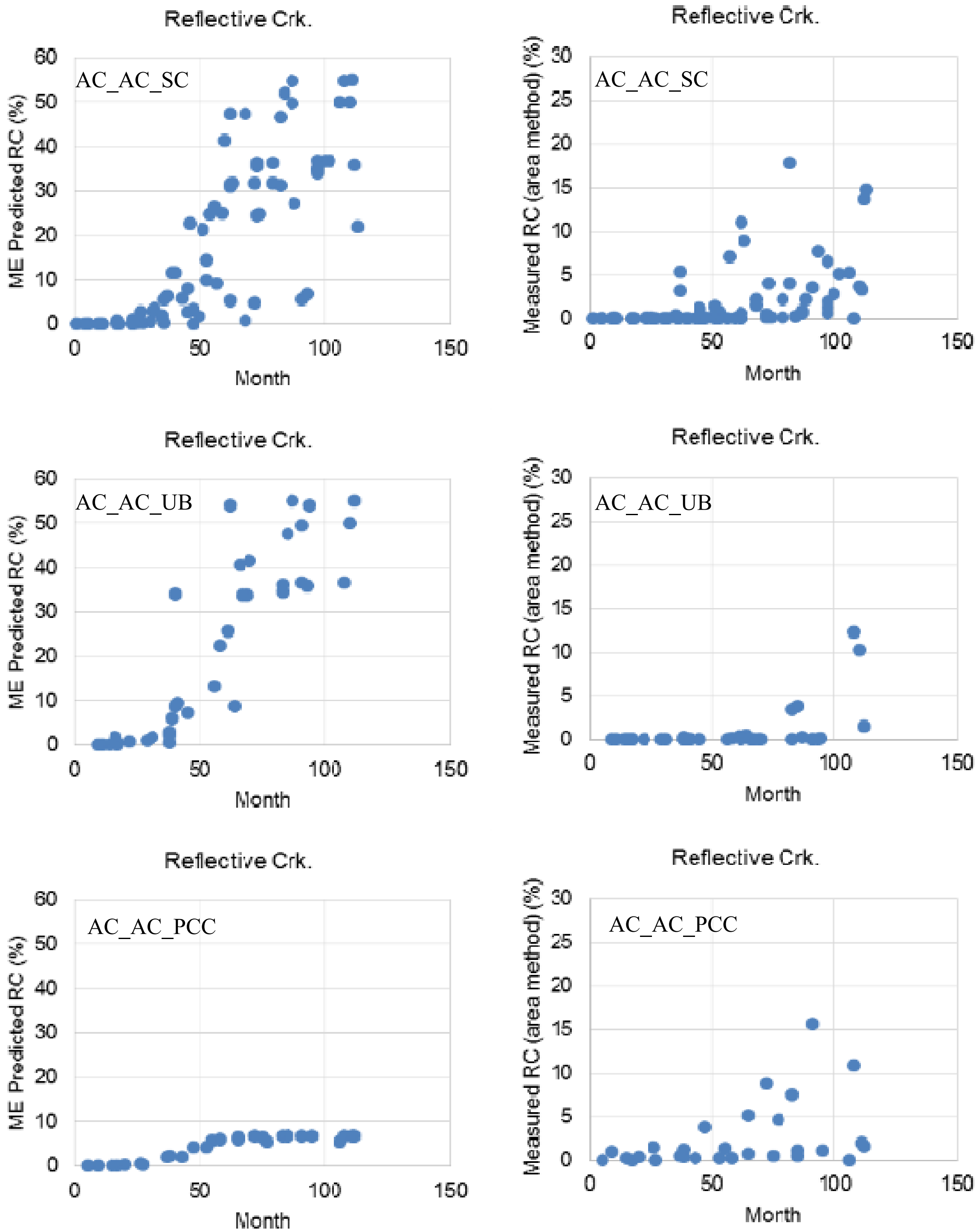
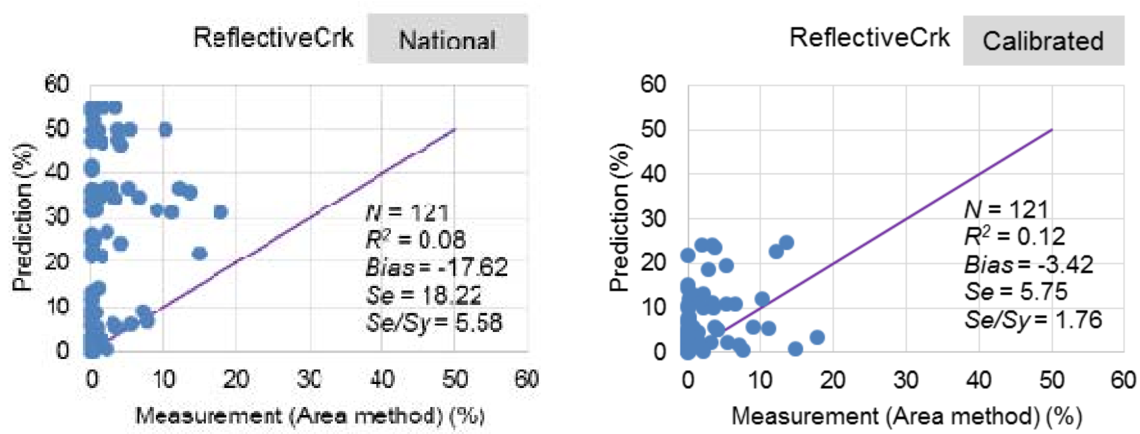


Figure 57

Evaluation of national models for overlay pavements: reflective cracking (left side: predicted from Pavement ME before local calibration; right side: estimated field measurements)

It should also be noted that existing soil cement was believed to have degraded and was assigned a modulus of 25,000 psi as recommended by MEPDG *[2]*. In addition, due to a software bug which did not allow a modulus lower than 100,000 psi for soil cement, this study modeled soil cement in rehabilitation projects as a crushed stone layer and assigned a modulus of 25,000 psi.

Local Calibration. The reflective cracking model has two calibration coefficients, *c* and *d*, which adjust the shape of the sigmoid curve in a similar way as the bottom-up fatigue cracking shown in Figure 24. Prediction and measurement data were assembled in Excel and the Solver function was used to determine the optimum values of *c* and *d* that minimized the error between predictions and measurements. The results were *c=0.72* and *d=0.30*. Figure 58 presents the comparison before and after local calibration.



(a) (b)

Figure 58

Predicted vs. measured reflective cracking: (a) national model, (b) calibrated model

Rutting

Pavement ME predicts rutting in each individual layer except the soil cement base. The PMS only provides the measured surface rutting, or total rutting. Figure 59 presents the comparison of predicted and measured total rutting. It shows that overall the national model over-predicted rutting. It also shows that most measured total rutting was less than 0.3 in. Considering the fact that these overlay projects were on average 9.5 years old, rutting seemed to be not a problem for AC overlays in Louisiana. This may indicate that the overlay thickness designed using the 1993 method seems adequate or even could be reduced.

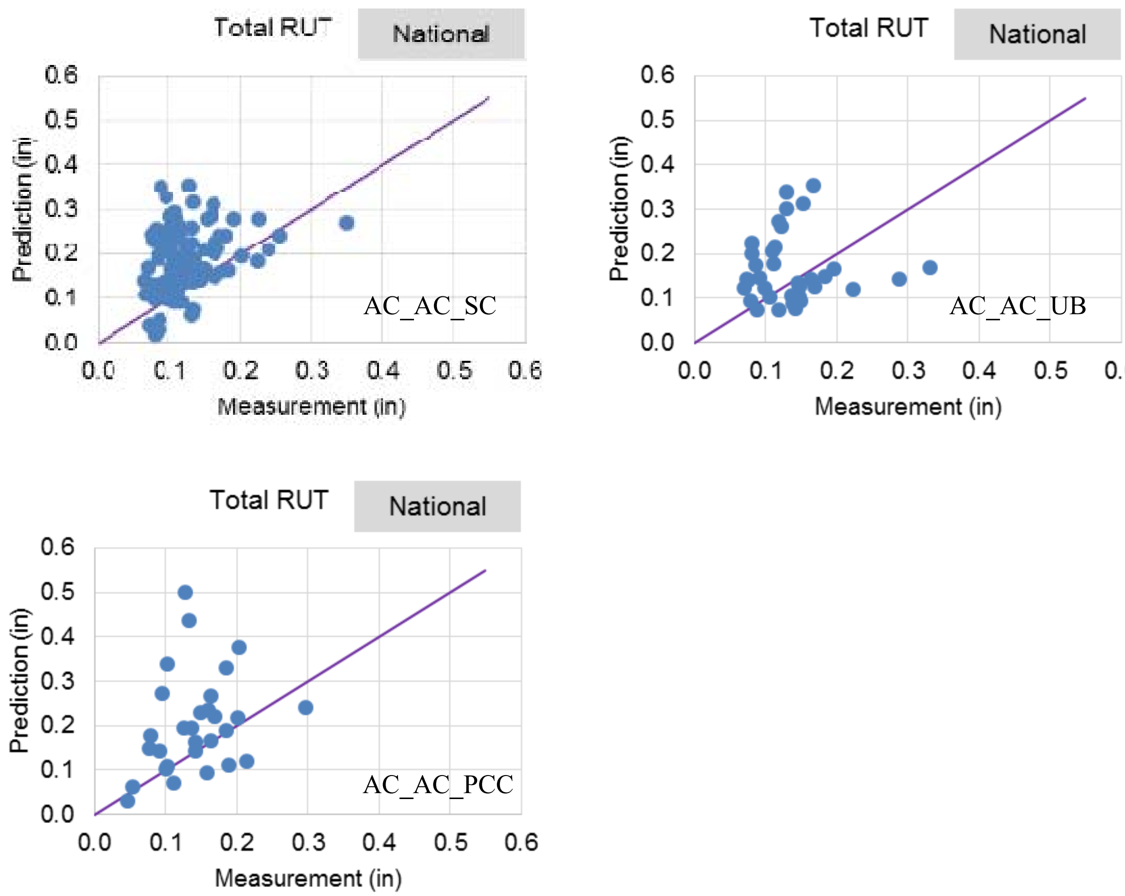


Figure 59

Evaluation of national models for overlay pavements: total rutting

The calibration coefficients for the rutting model as developed in new flexible pavement design were applied to overlays. The results are presented in Figure 60. When compared to Figure 59, it is clear that the predicted rutting was reduced. The maximum was decreased from 0.5 in. to 0.2 in. This was expected since the calibration coefficients were developed to reduce predicted rutting as shown in Figure 27 and Figure 29.

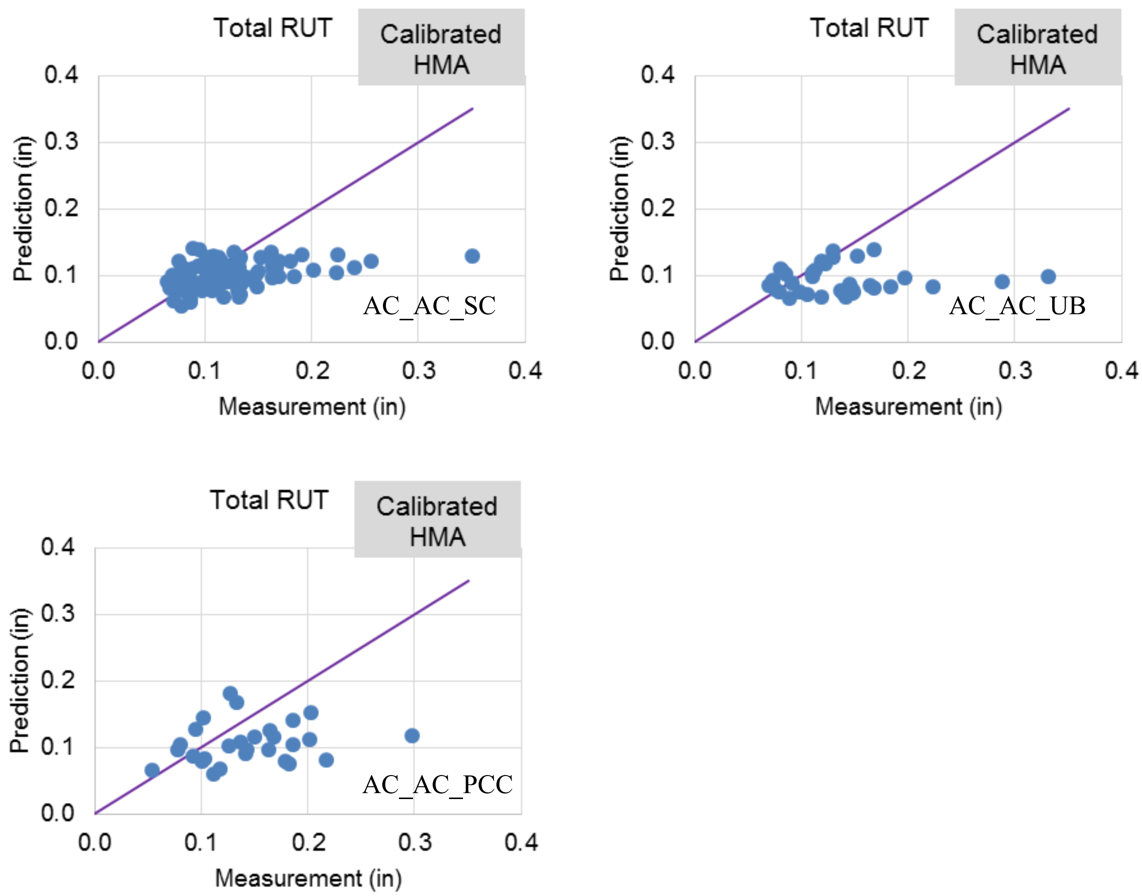


Figure 60

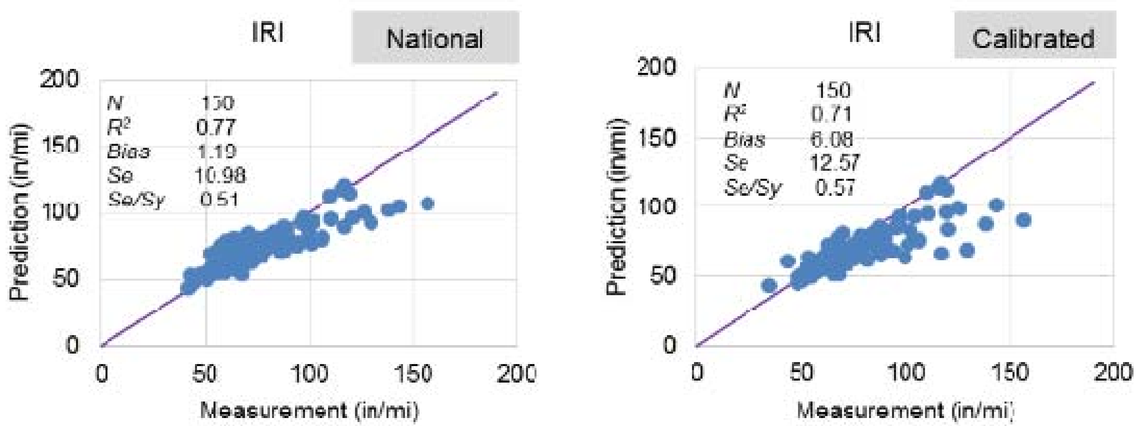
Evaluation of calibrated models for overlay pavements: total rutting

Further analysis on the total rutting was conducted to understand the estimated compositions in each layer. It was confirmed that no rutting was predicted in the soil cement or the PCC layers as the national model assumed. On average, subgrade contributed about 6% to 10% of total rutting. Total rutting was almost equal to AC rutting. In other words, rutting in AC overlay mainly came from the AC layer. The contribution from the existing AC layer and the new AC overlay was unknown based on the Pavement ME prediction because the software only outputted the subtotal rutting in all AC layers.

IRI

Figure 61a presents the comparison of predicted IRI from the national-calibrated Pavement ME and the measured IRI from PMS. It shows that measured average IRI matched well with predicted average IRI from the national model. The bias was 1.19 in/mi and the coefficient of determination was 0.77.

Similar to new flexible pavement design, the IRI model for AC overlay is an empirical model based on other distresses. Hence, the calibration coefficients of cracking and rutting model were applied and all projects were re-evaluated. Figure 61b presents the result of re-evaluation. It shows that predictions and measurements had a fair match. Compared to Figure 61a, the bias was slightly increased to 6.08 in/mi because the calibrated models predicted less rutting the national model did and hence reduced the predicted IRI. The R-square was 0.71. It seemed no additional calibration was needed for the IRI model.



(a) National model (b) Calibrated cracking and rutting model

Figure 61

Evaluation of national models for overlay pavements: IRI

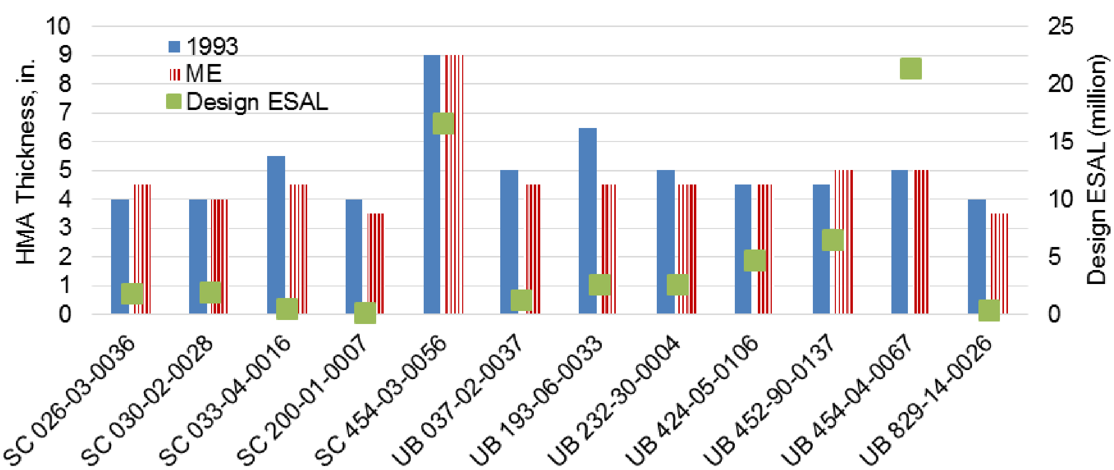
In summary, it was found that the national models performed similarly for AC overlay pavements and new flexible pavements: Pavement ME under-predicted fatigue cracking and over-predicted rutting. This was not a surprise because mechanistically both of them are asphalt surface pavements composed of HMA layers, base layers, and subgrade. The only difference is that all HMA layers are new mixtures in new flexible pavements and AC overlay pavements contain new HMA mixtures and existing HMA layers. Considering this similarity, this study did not conduct a separate calibration effort for AC overlay pavements. Instead, the calibration coefficients obtained for new flexible pavements were applied for overlay designs. Using only one set of calibration coefficients will also help implement Pavement ME because design engineers will not be confused by switching between different sets of coefficients.

Since reflective cracking could be the main distress for AC overlay projects, this study evaluated the reflective cracking model and found that Pavement ME over-predicted reflective cracking. Hence, the model was calibrated and a set of coefficients *c=0.72* and *d=0.30* were recommended. One should be aware of the limitation of this local calibration in which measured reflective cracking was converted from transverse cracking by assuming the transverse cracking measured in AC overlays were most likely cracks propagated from underneath stabilized base layers and existing AC layers.

After calibration coefficients for the cracking and the rutting models were applied, the predicted fatigue cracking was increased and the predicted rutting was reduced. The comparison was not perfect but acceptable. This conclusion was further validated by thickness comparison and design examples.

Thickness Comparison

To verify the suitableness of applying Pavement ME for design, overlay projects were re-designed using Pavement ME. Following the *Implementation Guidelines* and using the same calibrated coefficients as for new flexible pavements, the required overlay thickness was compared with the original 1993 Design Guide and presented in Figure 62. Comparing the recommended thickness from the two design methods, it was concluded that Pavement ME was comparable to the 1993 method for overlay design. On average, the difference was 0.3 in. with Pavement ME recommending slightly thinner overlays.



Note: SC = soil cement base, UB = unbound granular base

Figure 62

Recommended overlay thickness from AASHTO 1993 and Pavement ME

Besides comparing overlay thickness, it was worthwhile to investigate the relationship between total AC thickness and traffic level because the theory of "higher traffic, thicker pavement" should hold true for both new design and overlay designs. Figure 63 presents this relationship. Higher levels of traffic require thicker pavements. Statistically the correlation coefficient between total AC thickness and design ESAL was 89.0%.

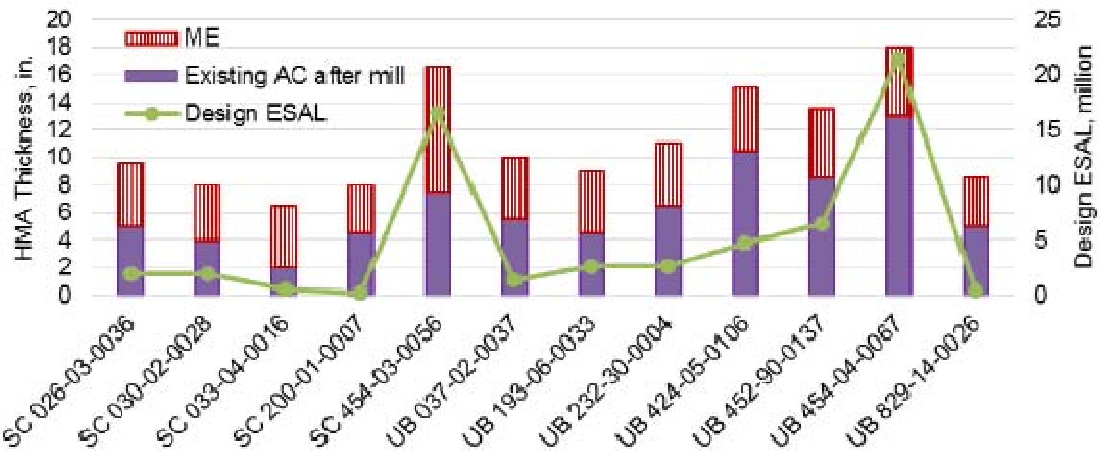


Figure 63

Total AC thickness and design traffic

Validation and Design Examples

Five projects out of the calibration pool were used to verify the usability and reasonableness of the local calibration coefficients and the design procedure (i.e., control distress and design criteria). Starting with the 1993 designed structure, overlay thickness was adjusted at 0.5 in. interval to approach the design criteria.

Figure 64 presents the recommended overlay thickness for these five projects. It shows that the design thicknesses from Pavement ME and 1993 were comparable. Four of the five projects required a thickness close to each other within ±0.5 in.

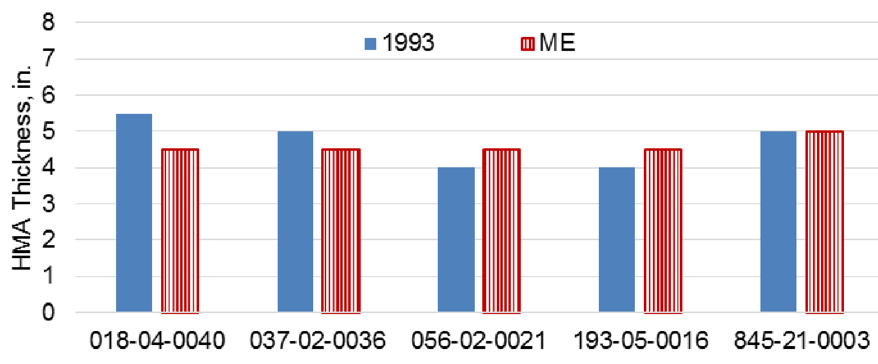


Figure 64

Recommended thickness for AC overlay design examples

Overall, the calibrated Pavement ME worked reasonably for the projects evaluated in this study. However, a concern was raised: the total rutting was almost equal to AC rutting in Pavement ME overlay design models. In other words, there was very little contribution of rutting from subgrade, stabilized base layers, and existing AC layers. The assumption is that subgrade has settled to a stable condition for old pavements. This may be true but it is worth a verification for Louisiana's clay-rich soil condition and low water table depth. Furthermore, the fact that total rutting almost equals AC rutting greatly escalates the importance of HMA parameters (gradation, effective binder content, air voids, and dynamic modulus) when using Pavement ME for overlay design. In this study the dynamic modulus was generated by Pavement ME software using mixture gradation, binder type, effective binder content and air voids. This study also evaluated a set of lab measured dynamic modulus with a few benefits, but none of them were project level data. Therefore, using HMA parameters as specific as possible to a design project is highly recommended.

Using project 037-02-0036 as an example, Figure 65 shows the distress summary from Pavement ME for the 1993 Design Guide structure and the ME recommended structure. It shows that all criteria were met with the 1993 designed overlay. By reducing the overlay thickness from 5.0 in. to 4.5 in., all design criteria were also met. But when the overlay was reduced to 4 in., the predicted total cracking failed the criterion. Therefore, the final thickness recommended by Pavement ME was 4.5 in. The distress charts in the 10 years design life for project 037-02-0036 are shown in Figure 66.

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	200.00	89.05	90.00	100.00	Pass
Permanent deformation - total pavement (in.)	0.50	0.19	90.00	100.00	Pass
Total Cracking (Reflective + Alligator) (percent)	25	14.75	-	-	Pass
AC thermal cracking (ft/mile)	700.00	27.17	90.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	25.00	1.46	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	5280.00	462.87	90.00	100.00	Pass
Permanent deformation - AC only (in.)	0.50	0.19	90.00	100.00	Pass

(a) Overlay thickness = 5.0 in.

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	200.00	89.67	90.00	100.00	Pass
Permanent deformation - total pavement (in.)	0.50	0.20	90.00	100.00	Pass
Total Cracking (Reflective + Alligator) (percent)	25	20.45	-	-	Pass
AC thermal cracking (ft/mile)	700.00	27.17	90.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	25.00	1.46	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	5280.00	616.37	90.00	100.00	Pass
Permanent deformation - AC only (in.)	0.50	0.20	90.00	100.00	Pass

(b) Overlay thickness = 4.5 in.

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	200.00	90.26	90.00	100.00	Pass
Permanent deformation - total pavement (in.)	0.50	0.22	90.00	100.00	Pass
Total Cracking (Reflective + Alligator) (percent)	25	27.37	-	-	Fail
AC thermal cracking (ft/mile)	700.00	27.17	90.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	25.00	1.46	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	5280.00	790.67	90.00	100.00	Pass
Permanent deformation - AC only (in.)	0.50	0.21	90.00	100.00	Pass

(c) Overlay thickness = 4.0 in.

Figure 65

Distress summary for project 037-02-0036 as reported by Pavement ME

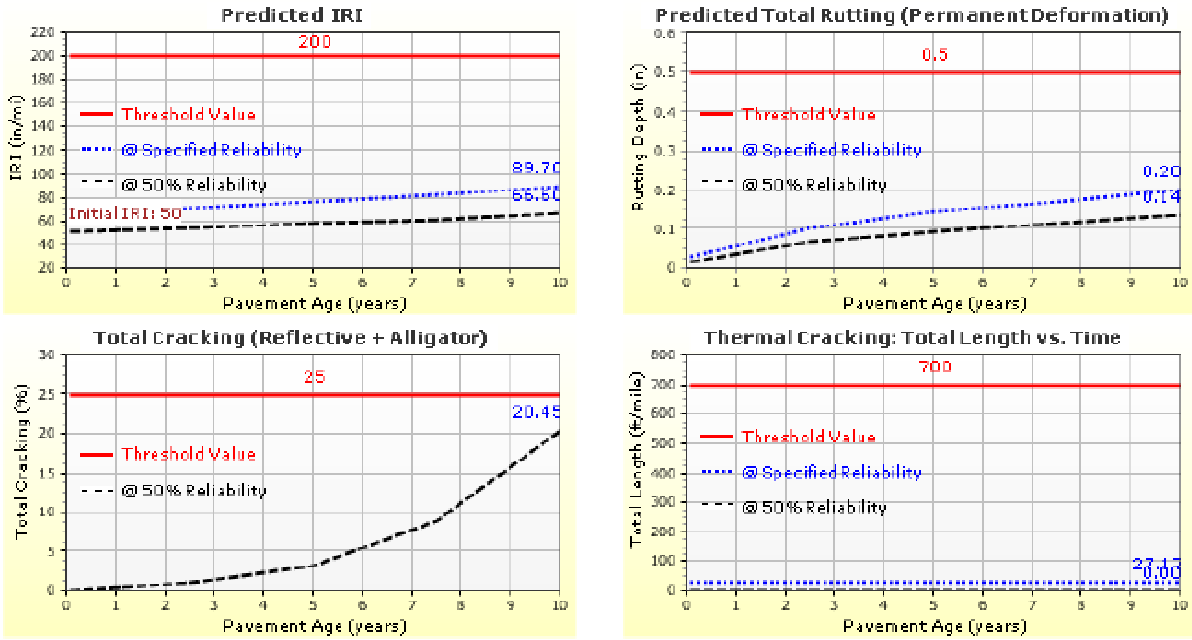


Figure 66

Distress charts reported by Pavement ME for project 037-02-0036 (Overlay thickness = 4.5 in.)

CONCLUSIONS

A total of 162 projects selected throughout Louisiana with different traffic volumes and subgrade stiffness were analyzed in this study using the newly developed AASHTOWare® Pavement ME™ software (version 2.0). The selected projects included three pavement types: flexible, rigid, and rehabilitation pavement. The design inputs were obtained from different database sources as outlined in the report. A set of Louisiana default inputs was developed based on Louisiana local conditions and practice. To evaluate the nationally calibrated models, pavement performance data retrieved from the LA-PMS database as well as data measured by the research team were compared against the predicted performance from Pavement ME. Local calibration was carried out to reduce bias and variation for flexible pavement bottom-up cracking and rutting models, rigid pavement cracking and faulting models, and the reflective cracking model for AC overlay projects. The recommended thicknesses using Pavement ME design were also compared with the original 1993 design. Design examples were then provided to verify the results from this study and showcase the implementation of Pavement ME for pavement design. Table 38 summarizes the result of evaluating Pavement ME for Louisiana. Models which can, with or without local calibration, generate predictions match with measurements are recommended to use, including fatigue cracking, rutting and IRI for new flexible pavements, cracking, faulting and IRI for new rigid pavements, fatigue cracking, reflective cracking, rutting, and IRI for AC overlay projects. However, models that were unable to be evaluated or pending upgrade from AASHTO are not recommended for use at this time. Finally, design guidelines were developed and included in Appendix A.

The following specific observations and conclusions can also be drawn from this study.

Flexible Pavements

- Pavement ME in general under-predicted fatigue cracking but over-predicted rutting.
- For AC over soil cement base, PMS data showed that alligator cracking, extreme transverse, and longitudinal cracking were measured, but Pavement ME predicted very little fatigue cracking probably due to the high modulus of stabilized base comparing to unbound base and the limitation of Pavement ME in analyzing stabilized materials.
- For AC over rubblized PCC pavements, Pavement ME predicted extreme rutting up to 1 in., which was by no means realistic. The measured rutting however were all less than 0.3 in. It was concluded that the rutting model needed to be calibrated to better distinguish between different levels of traffic loading.

Table 38

Summary of Pavement ME evaluation for Louisiana

Pavement Type	Model	Evaluation of national model	Recommendation
Flexible pavement	Bottom-up fatigue cracking	Under-predict	Calibrated, ready to use
	Top-down fatigue cracking	Not evaluated	Not use
	Thermal cracking	Not evaluated	Not use
	AC rutting	Over-predict	Calibrated, ready to use
	Total rutting	Over-predict	Calibrated, ready to use
	IRI	Reasonable	Use default
Rigid pavement	Mid-slab fatigue cracking	Over-predict	Calibrated, ready to use
	Joint faulting	Under-predict	Calibrated, ready to use
	IRI	Reasonable	Use default
Pavement rehabilitation	Bottom-up fatigue cracking	Under-predict	Same as new flexible pavement
	Top-down fatigue cracking	Not evaluated	Not use
	Thermal cracking	Not evaluated	Not use
	Reflective cracking	Over-predict	Calibrated, ready to use
	AC rutting	Over-predict	Same as new flexible pavement
	Total rutting	Over-predict	Same as new flexible pavement
	IRI	Reasonable	Use default

- IRI was slightly over-predicted by the national model, mainly due to severe over-prediction of rutting, but a good match was found after the fatigue cracking and rutting model were calibrated. Hence, no additional calibration was conducted on the IRI model.
- If Pavement ME was used to design flexible pavement, results showed that the recommended thickness from Pavement ME was comparable with the 1993 Design Guide with a difference ranging from -1 in. to 1 in. and an average of -0.3 in. (ME requires thinner).

Rigid Pavements

- Pavement ME significantly over-predicted slab cracking. One reason was that the 20-ft. joint spacing used in Louisiana was different from the normal value 15 ft. recommended in Pavement ME. Analysis showed that the cracking model was very sensitive to joint spacing.
- Joint faulting, however, was under-predicted. Field measured faulting was minimal at less than 0.1 in. except for a few projects in which the longitudinal cracking was not saw cut. The satisfactory performance of faulting might be attributed to Louisiana's mandatory application of dowel bars on all jointed concrete pavements and the common practice of widened slabs up to 15 ft.

- Field measurements revealed that the cracking data in PMS matched well with manual evaluation. However, faulting data were highly questionable. The cutoff criteria of 0.2 in. was definitely too large considering the fact that joint faulting in the field were mostly less than 0.1 in. and the design criteria in Pavement ME was 0.15 in. for interstate and 0.2 in. for primary highways. Calculations from the longitudinal profile using ProVAL software provided reasonable estimations for joint faulting.
- IRI was slightly over-predicted by the national model, mainly due to severe over-prediction of cracking, but a good match was found after the cracking and faulting model were calibrated. Hence, no additional calibration was conducted on the IRI model.
- Using field-measured cracking and faulting data, local calibration was conducted. Pavement ME with calibrated models recommended on average a 0.7-in. thinner PCC surface than the 1993 Design Guide.

Pavement Rehabilitation

- Similar results as for new flexible pavement were found – the national model under-predicted fatigue cracking and over-predicted rutting. By applying calibration coefficients developed for new flexible pavement, both fatigue cracking and rutting models were adjusted to match field measured performance. Hence, the need to further calibrate fatigue cracking and rutting model was minimal.
- Reflective cracking was over-predicted. Based on estimated field measurements, the empirical reflective cracking model was calibrated to better match field performance. However, it should be noted that existing soil cement was believed to have degraded and was assigned a modulus of 25,000 psi. In addition, due to a software bug which did not allow a modulus lower than 100,000 psi for soil cement, this study modeled soil cement in rehabilitation projects as a crushed stone layer and assigned a modulus of 25,000 psi.
- The IRI model for rehabilitation projects was found reasonable and hence was not calibrated separately.
- Pavement ME with calibration coefficients were found comparable with the 1993 Design Guide. The difference of recommended overlay thickness was within ±0.5 in. for most projects evaluated in this study with an average of 0.3 in. thinner than the 1993 Design Guide.

Calibration Coefficients

- Table 39 summarizes all calibration coefficients developed in this study. They are included in the *Implementation Guidelines* to assist engineers at DOTD conducting pavement design using the new M-E method.

Table 39

Local calibration coefficients for Louisiana M-E pavement design

Pavement Type	Distress	Calibration Factor	Calibration Coefficient
New flexible and AC overlays	AC bottom-up cracking	β_{f3}	1.05
		C_1	0.892
		C_2	0.892
	AC rutting	β_{r1}	0.80
		β_{r3}	0.85
	Subgrade rutting	β_{s1}	0.40
	Reflective cracking	c	0.72
		d	0.30
New rigid	Fatigue Cracking	C_1	2.75
		C_4	1.16
		C_5	-1.73
	Joint faulting	C_1	1.5276
		C_3	0.00262
		C_6	0.55

RECOMMENDATIONS

1. *Implementation Guidelines* (Appendix A) were prepared to help design engineers at DOTD use Pavement ME. A period of one year in-house practice is also recommended. This will (1) give engineers experience using the new procedure, (2) identify and troubleshoot any design issues, and (3) revise and modify any default design inputs or calibration coefficients to advance the implementation process.

2. Default inputs were proposed based on past research reports and Louisiana's practice. DOTD is sponsoring several studies to better characterize Louisiana's typical materials and traffic. Results from these research projects should be updated to the *Implementation Guidelines* when they become available.

3. Engineers should be aware that the *Implementation Guidelines* is not a comprehensive pavement design manual but rather an up-to-date implementation guide that moves Pavement ME from research to practice in Louisiana. If a large discrepancy is found between the two design methods, careful engineering judgment should be applied to develop a reasonable design. Such cases should be reported to LTRC for future update, modification and improvement of the *Implementation Guidelines*.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AADTT	Average Annual Daily Truck Traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ANOVA	Analysis of Variance
CRCP	Continuously Reinforced Concrete Pavement
CTE	Coefficient of Thermal Expansion
DARWin	Pavement Design, Analysis and Rehabilitation for Windows
DOTD	Louisiana Department of Transportation and Development
EICM	Enhanced Integrated Climate Model
ESAL	equivalent single axle load
FHWA	Federal Highway Administration
HMA	Hot Mix Asphalt
IRI	International Roughness Index
JPCP	Jointed Portland cement Concrete Pavement
LA-PMS	Louisiana Pavement Management System
LTPP	Long-term Pavement Performance
LTRC	Louisiana Transportation Research Center
MATT	Material Testing System
MEPDG	Mechanistic-Empirical Pavement Design Guide
M_r	Resilient modulus
NCDC	National Climate Data Center
NCHRP	National Cooperative Highway Research Program
PCC	Portland Cement Concrete
PMS	Pavement Management System
PSI	Present Serviceability Index
RPCC	Rubblized Portland Cement Concrete
$SEE(S_e)$	Standard error of estimate
SSE	Sum of squared errors
SSV	Soil Support Value
TAND	Highway Need System
TATV	Traffic Count ADT
TOPS	Tracking of Projects
TRB	Transportation Research Board
TTC	Truck Traffic Classification

WIM	Weigh-In-Motion
USGS	US Geological Survey
VCD	Vehicle class distribution

REFERENCES

1. AASHTO. *AASHTO Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Officials: Washington, D.C., 1993.
2. ARA, Inc. *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures.* ARA, Inc, ERES Consultants Division: Champaign, IL, 2004.
3. Pierce, L., and McGovern, G. *NCHRP Synthesis 457: Implementation of the AASHTO Mechanistic-Empirical Pavement Design Guide and Software.* Transportation Research Board: Washington D.C., 2014.
4. AASHTO. *Mechanistic-Empirical Pavement Design Guide, Interim Edition: A Manual of Practice.* American Association of State Highway and Transportation Officials: Washington, DC, 2008.
5. Quintus, H.L.V., Darter, M.I. and Mallela, J. *User Manual for the M-E Pavement Design Guide.* Applied Research Associates, Inc., 2007.
6. Buck, N., Chatti, K., Haider, S. W., and Manik, A. *Evaluation of the 1-37A Design Process for the New and Rehabilitated JPCP and HMA Pavements.* Michigan State University: East Lansing, MI, 2008.
7. Kim, Y. R., and Muthadi, N. R. *Implementation Plan for the New Mechanistic-Empirical Pavement Design Guide.* North Carolina State University: Raleigh, NC, 2007.
8. Mallela, J., Titus-Glover, L., Darter, M. I., Von Quintus, H. L., Gotlif, A., Stanley, M., and Sadasivam, S. *Guidelines for Implementing NCHRP 1-37A M-E Design Procedures in Ohio:Volume 1 - Summary of Findings, Implementation Plan, and Next Steps.* Applied Research Associates, Inc.: Champaign, IL, 2009.
9. Orobio, A., and Zaniewski, J. P. "Sampling-based sensitivity analysis of Mechanistic-Empirical Pavement Design Guide to material inputs." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2226, 2011, pp. 85-93.
10. Schwartz, C. W., Li, R., Kim, S. H., Ceylan, H., and Gopalakrishnan, K. *Sensitivity Evaluation of MEPDG Performance Prediction.* The National Cooperative Highway Research Program (NCHRP): Washington D.C., 2011.
11. Buch, N., Chatti, K., Haider, S. W., Baladi, G., Brink, W., and Harsini, I. *Preparation for Implementation of the Mechanistic-Empirical Pavement Design Guide in Michigan Part 2: Evaluation of Rehabilitation Fixes.* Michigan State University: East Lansing, MI, 2013.
12. Von Quintus, H. L., and Moulthrop, J. *Mechanistic-Empirical Pavement Design Guide*

Flexible Pavement Performance Prediction Models for Montana. Applied Research Associates, Inc.: Champaign, IL, 2008.

13. Muthadi, N., and Kim, R. Y. "Local Calibration of Mechanistic-Empirical Pavement Design Guide for Flexible Pavement Design." *Transportation Research Record: Journal of the Transportation Research Board* , No. 2087, 2008, pp. 131-141.
14. Banerjee, A., Aguiar-Moya, J. P., and Prozzi, J. A. "Texas Experience Using LTPP for Calibration of the MEPDG Permanent Deformation Models." *Transportation Research Board 88th Annual Meeting,* National Research Council, Washington, D.C., 2009.
15. Li, J., Preice, L., and Ulhmeyer, J. "Calibration of Flexible Pavement in Mechanistic-Empirical Pavement Design Guide for Washington State." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2095, 2009, pp. 73-83.
16. Kim, S., Ceylan, H., Gopalakrishnan, K., and Smadi, O. "Use of Pavement Management Information System for Verification of Mechanistic-Empirical Pavement Design Guide Performance Predictions." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2153, 2010, pp. 30-39.
17. Hall, K. D., Xiao, D. X., and Wang, K. C. P. "Calibration of the MEPDG for Flexible Pavement Design in Arkansas." *Transportation Research Record: Journal of the Transportation Research Board* , No. 2226, 2011, pp. 135-141.
18. Souliman, M., Mamlouk, M., El-Basyouy, M., and Zapata, C. Calibration of the AASHTO MEPDG for Flexible Pavement for Arizona Conditions. *Compendium of Papers of the 89th TRB Annual Meeting.* Washington, D.C., 2010.
19. Velasquez, R., Hoegh, K., Yut, I., Funk, N., Cochran, G., Marasteanu, M., and Khazanovich, L. *Implementation of the MEPDG for New and Rehabilitated Pavement Structures for Design of Concrete and Asphalt Pavements in Minnesota.* University of Minnesota: Minneapolis, MN, 2009.
20. Won, M. *Evaluation of MEPDG with TxDOT Rigid Pavement Database;* University of Texas at Austin: Austin, TX, 2009.
21. AASHTO. *Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide.* American Association of State Highway and Transportation Officials, 2010.
22. Haider, S. W., Buch, N., Brink, W., Chatti, K., and Baladi, G. *Preparation for Implementation of the Mechanistic- Empirical Pavement Design Guide in Michigan Part 3: Local Calibration and Validation of the Pavement-ME Performance Models.* Michigan State University: East Lansing, MI, 2014.

23. Guo, X. *Local Calibration of the MEPDG Using Test Track Data.* Auburn University: Auburn, AL, 2013.
24. Darter, M., Titus-Glover, L., Von Quintus, H., Bhattacharya, B. B., and Mallela, J. *Calibration and Implementation of the AASHTO Mechanistic-Empirical Pavement Design Guide in Arizona.* Arizona Department of Transportation: Phoenix, AZ, 2014.
25. Transportation Research Board. NCHRP 01-53. *Improved Consideration of the Influence of Subgrade and Unbound Layers on Pavement Performance*. http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=3625 Accessed Feb 18, 2015.
26. Azari, H., Mohseni, A., and Gibson, N. "Verification of Rutting Predictions from Mechanistic–Empirical Pavement Design Guide by Use of Accelerated Loading Facility Data." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2057, 2008, pp. 157–167.
27. Jadoun, F. M., and Kim, Y. R. "Calibrating Mechanistic-Empirical Pavement Design Guide for North Carolina: Genetic Algorithm and Generalized Reduced Gradient Optimization Methods." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2305, 2012, pp. 131-140.
28. Mallela, J., Titus-Glover, L., Sadasivam, S., Bhattacharya, B., Darter, M., and Quintus, H. V. *Implementation of the AASHTO Mechanistic-Empirical Pavement Design Guide for Colorado.* Colorado Department of Transportation: Denver, CO, 2013.
29. Mamlouk, M., and Zapata, C. E. "Necessary Assessment of Use of State Pavement Management System Data in Mechanistic-Empirical Pavement Design Guide Calibration Process." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2153, 2010, pp. 58-66.
30. Tarefder, R., and Rodriguez-Ruiz, J. I. "Local Calibration of MEPDG for Flexible Pavements in New Mexico." *Journal of Transportation Engineering,* Vol. 139, No. 10, 2013, pp. 981-991.
31. Wu, Z., and Yang, X. *Evaluation of Current Louisiana Flexible Pavement Structures Using PMS Data and New Mechanistic-Empirical Pavement Design Guide.* Louisiana Transportation Research Center: Baton Rouge, LA, 2011.
32. Li, J., Muench, S. T., Mahoney, J. P., Sivaneswaran, N., and Pierce, L. M. "Calibration of NCHRP 1-37A Software for the Washington State Department of Transportation: Rigid Pavement Portion." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1949, 2006, pp. 43-53.

33. Kim, S., Ceylan, H., Ma, D., and Gopalakrishnan, K. "Local Calibration Studies on DARWin-ME / Mechanistic-Empirical Pavement Design Guide Jointed Plain Concrete Pavement Performance Prediction Models." *Transportation Research Board 92nd Annual Meeting*, Washington D.C., 2013.

34. Bustosl, M., Cordo, O., Girardi, P., and Pereyra, M. "Calibration of Distress Models from the Mechanistic-Empirical Pavement Design Guide for Rigid Pavement Design in Argentina." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2226, 2011, pp. 3-12.

35. Ceylan, H., Kim, S., Gopalakrishnan, K., and Ma, D. *Iowa Calibration of MEPDG Performance Prediction Models.* Iowa Department of Transportation: Ames, IA, 2013.

36. Williams, C., and Shaidur, R. *Mechanistic-Empirical Pavement Design Guide Calibration for Pavement Rehabilitation.* Oregon Department of Transportation: Salem, OR, 2013.

37. Carvalho, R., and Schwartz, C. "Comparisons of Flexible Pavement Designs: AASHTO Empirical Versus NCHRP Project 1-37A Mechanistic-Empirical." *Transportation Research Record: Journal of Transportation Research Record,* No. 1947, 2006, pp. 167-174.

38. Li, J., Uhlmeyer, J. S., Mahoney, J. P., and Muench, S. T. "Updating the pavement design catalog for the Washington State Department of Transportation." *Transportation Research Record: Journal of Transportation Research Record,* No. 2154, 2010, pp. 124-129.

39. El-Badawy, S. M., Bayomy, F. M., Santi, M., and Clawson, C. W. "Comparison of Idaho Pavement Design Procedure with AASHTO 1993 and MEPDG Methods." *ASCE T & DI Congress 2011*, 2011, pp 586-595.

40. Gedafa, D. S., Mulandi, J., Hossain, M., and Schieber, G. "Comparison of Pavement Design Using AASHTO 1993 and NCHRP Mechanistic-Empirical Pavement Design Guides." *ASCE T& DI Congress 2011*, 2011, pp. 538-547.

41. McCracken, J. K., Vandenbossche, J. M., and Asbahan, R. E. "Effect of the MEPDG hierarchical levels on the predicted performance of a joint plain concrete pavement." *Proceedings of the 9th International Conference on Concrete Pavement*, San Francisco, CA, 2008, pp. 153-170.

42. Elfino, M., Nair, H. K., Gudimettla, J., and Crawford, G. "Design comparison between 1993 AASHTO and MEPDG using level 1 input for continuously reinforced concrete pavement: a case study in Virginia." *Compendium of Papers of the 89th Transportatoin*

Research Board Annual Meeting, Washington, D.C., 2010.

43. Vandenbossche, J. M., Mu, F., and Burnham, T. R. “Comparison of measured vs. predicted performance of jointed plain concrete pavements using the Mechanistic-Empirical Pavement Design Guide.” *International Journal of Pavement Engineering* Vol. 12, No. 3, 2011, pp. 239-251.
44. Boone, J. N. *Comparison of Ontario Pavement Designs Using the AASHTO 1993 Empirical Method and the Mechanistic-Empirical Pavement Design Guide Method.* University of Waterloo: Waterloo, Ontario, Canada, 2013.
45. Rasoulian, M., Becnel, B., and Keel, G. “Stone Interlayer Pavement Design.” *Transportation Research Record: Journal of the Transportation Research Board,* No. 1709*,* 2000, pp. 60-68.
46. Mohammad, L. N., Kim, M., Raghavendra, A., and Obulareddy, S. *Characterization of Louisiana Asphalt Mixtures Using Simple Performance Tests and MEPDG.* Louisiana Transportation Research Center: Baton Rouge, LA, 2014.
47. Schwartz, C. W., Li, R., Kim, S., Ceylan, H., and Gopalakrishnan, K. “Effect of Concrete Strength and Stiffness Characterization on Predictions of Mechanistic–Empirical Performance for Rigid Pavements.” *Transportation Research Record: Journal of the Transportation Research Board,* No. 2226, 2011, pp. 41-50.
48. Louisiana Department of Transportation and Development. *Louisiana Department of Transportation and Development Pavement Design Guide.* Louisiana Department of Transportation and Development: Baton Rouge, LA, 2013.
49. Shin, H.-C., and Chung, Y. *Determination of Coefficient of Thermal Expansion Effects on Louisiana’s PCC Pavement Design.* Louisiana Transportation Research Board: Baton Rouge, LA, 2011.
50. Gucunski, N., Sauber, R., Maher, A., and Rascoe, C. “Modulus of Rubblized Portland Cement Concrete from Surface Wave Testing.” *Transportation Research Record: Journal of the Transportation Research Board,* No. 2104, 2009, pp. 34-41.
51. Scullion, T. *Transportation Research Circular E-C087: Nondestructive Testing Results from the Rubblized Concrete Pavement on Interstate 10 in Louisiana.* Transportation Research Board of the National Academies: Washington, D.C., 2006.
52. Ishak, S., Shin, H.-C., and Sridhar, B. *Characterization and Development of Truck Load Spectra and Growth Factors for Current and Future Pavement Design Practices in Louisiana.* Louisiana State University: Baton Rouge, LA, 2009.
53. Khattak, M. J., Baladi, G. Y., Zhang, Z., and Ismail, S. “Review of Louisiana's Pavement

Management System: Phase I." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2084, 2008, pp. 18-27.

54. Louisiana Department of Transportation and Development. *Louisiana Cracking and Patching Protocol for Asphalt Pavements, Version 11*. Baton Rouge, LA, 2008.
55. Louisiana Department of Transportation and Development. *Louisiana Cracking and Patching Protocol for Concrete Pavements, Version 11.* Baton Rouge, LA, 2008.
56. Darter, M., Titus-Glover, L., and Von Quintus, H. *Implementation of the Mechanistic-Empirical Pavement Design Guide in Utah: Validation, Calibration and Development of the UDOT MEPDG User's Guide.* Utah Department of Transportation: Salt Lack City, UT, 2009.
57. AASHTO. *Standard Practice for Evaluating Faulting of Concrete Pavements.* The American Association of State Highway and Transportation Officials, 2013.
58. Mraz, A., Nazef, A., Lee, H., Holzschuher, C., and Choubane, B. "Precision of Florida Methods for Automated and Manual Faulting Measurements." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2306, 2012, pp. 131-137.
59. Miller, J. S., and Bellinger, W. Y. *Distress Identification Manual for the Long-Term Pavement Performance Program.* Federal Highway Administration: McLean, Virginia, 2003.
60. Agurla, M., and Lin, S. *Long-Term Pavement Performance Automated Faulting Measurement.* Federal Highway Administration: McLean, Virginia, 2015.
61. Moriasi, D. N., Arnold, J. G., Liew, M. W. V., Bingner, R. L., Harmel, R. D., and Veith, T. L. "Model Evaluation Guidelines for Systematic Quantification of Accuracy in Watershed Simulations." *Transactions of the ASABE*, Vol. 50, No. 3, 2007, pp. 885-900.
62. El-Basyouny, M. M., and Witczak, M. "Calibration of Alligator Fatigue Cracking Model for 2002 Design Guide." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1919, 2005, pp. 76-86.
63. Djakfar, L., and Roberts, F. L. "Performance Prediction of Louisiana Accelerated Loading Facility Test Sections." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1716, 2000, pp. 108-115.
64. Saxena, P., Tompkins, D., Khazanovich, L., and Balbo, J. T. "Evaluation of Characterization and Performance Modeling of Cementitiously Stabilized Layers in the Mechanistic-Empirical Pavement Design Guide." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2186, 2010, pp. 111-119.

65. Titi, H., Rasoulian, M., Martinez, M., Becnel, B., and Keel, G. "Long-Term Performance of Stone Interlayer Pavement." *Journal of Transportation Engineering* Vol. 129, No. 2, 2003, pp. 118-126.
66. Gaspard, K., Icenogle, P., Abadie, C., Zhang, Z., and Elseif, M. "Historical Performance of Rubblized Jointed Portland Cement Concrete Pavement Overlaid with Asphaltic Concrete in the State of Louisiana." *International Journal of Pavement Research and Technology,* Vol. 6, No. 3, 2013, pp. 165-174.
67. Wu, Z., Yang, X., and Zhang, Z. "Evaluation of MEPDG flexible pavement design using pavement management system data: Louisiana experience." *International Journal of Pavement Engineering,* Vol. 14, 2013, pp. 674-685.
68. Smith, K. D., Peshkin, D. G., Darter, M. I., Mueller, A. L., and Carpenter, S. H. *Performance of Jointed Concrete Pavements, Volume I, Evaluation of Concrete Pavement Performance and Design Features.* Federal Highway Administration: Washington D.C., 1990.
69. Elseifi, M., and Bandaru, R. *Cost Effective Prevention of Reflective Cracking of Composite Pavement.* Louisiana Department of Transportation and Development: Baton Rouge, LA, 2011.
70. Gaspard, K., and Zhang, Z. "Assessment of Shrinkage Crack Mitigation Techniques Performance in Soil Cement Base Courses." *Transportation Research Board 88th Annual Meeting*, Washington D.C., 2009.
71. Highway Research Board. *The AASHO Road Test Report 5: Pavement Research, Appendix F: The Pavement Serviceability-Performance Concept.* National Academy of Sciences: Washintong D.C., 1962.
72. Utah DOT. *Pavement design Manual of Instruction;* Utah Department of Transportation: Salt Lake City, UT, 2012.
73. Indiana DOT. *Design Manual;* Indiana Department of Transportation: Indianapolis, IN, 2013.

APPENDIX

APPENDIX A

Implementation Guidelines of AASHTOWare® Pavement ME™ Design for DOTD

Introduction

The AASHTO Mechanistic Empirical Pavement Design Guide (MEPDG) and its accompanying software, AASHTOWare® Pavement ME™ Design (hereafter called Pavement ME), provide the highway community with a state-of-the-practice tool for the design and analysis of new and rehabilitated pavement structures. To support the implementation of MEPDG in Louisiana, LTRC recently completed a research project (LTRC Project 12-4P): *Development of DARWin-ME Design Guideline for Louisiana Pavement Design*. The project was initiated to evaluate and validate the use of AASHTOWare Pavement ME (previously called DARWin-ME) based on Louisiana pavement conditions, and subsequently provide implementation guidelines for DOTD to use the Pavement ME in designing new flexible, rigid and rehabilitated pavement structures in Louisiana.

Project 12-4P has identified several new and rehabilitated pavement structure types commonly used by DOTD and evaluated their performance using the Pavement ME against those from the LA-PMS database. Therefore, this document provides the implementation guidelines of Pavement ME mainly useful for a pavement structural design of the following pavement types in Louisiana:

- AC over RPCC base,
- AC over soil cement base,
- AC over unbound base,
- AC over stone interlayer,
- AC over AC base,
- PCC over unbound base,
- PCC over stabilized base,
- PCC over HMA blanket base, and
- AC overlay over existing flexible pavement

It should be noted that, due to the lack of project level pavement performance data as well as Level I or II materials inputs, the recommended design inputs and local calibration coefficients of MEPDG's distress models obtained in Project 12-4P were all determined by extracting the required pavement design and performance data from different databases of DOTD. Therefore, all inputs recommended herein this guide should be considered as the

DOTD default inputs used in Pavement ME. It is recommended to use the DOTD's default input values and local calibration coefficients provided in this guideline document to conduct real project pavement design for approximately one year to (1) gain more experience with the new procedure, (2) identify and troubleshoot any design issues, and (3) revise and modify any default design inputs or calibration coefficients to advance the implementation process.

Overview of Pavement ME

Pavement design using Pavement ME is an iterative process. Outputs from the software are pavement distresses and smoothness, not layer thickness. A designer first considers site conditions (i.e., traffic, climate, subgrade, existing pavement condition for rehabilitation) and proposes a trial design for a new pavement or rehabilitation strategy. The trial design is then evaluated for adequacy against the performance criteria. If the design does not meet the desired performance criteria at a specified reliability level, it is revised and the evaluation process repeated until the criteria are met.

Pavement ME requires hundreds of inputs which can be grouped as general information, design criteria, traffic, climate, material properties and local calibration coefficients. A screenshot of the Pavement ME software is shown in Figure 67. Due to the large amount of inputs, of which many have not been tested before, designs using Pavement ME will start with Level 3 national default values. Then, parameters that are special for Louisiana will be changed according to this guideline.

The following sections provide the recommended design values which best represent Louisiana conditions, materials and construction procedures.

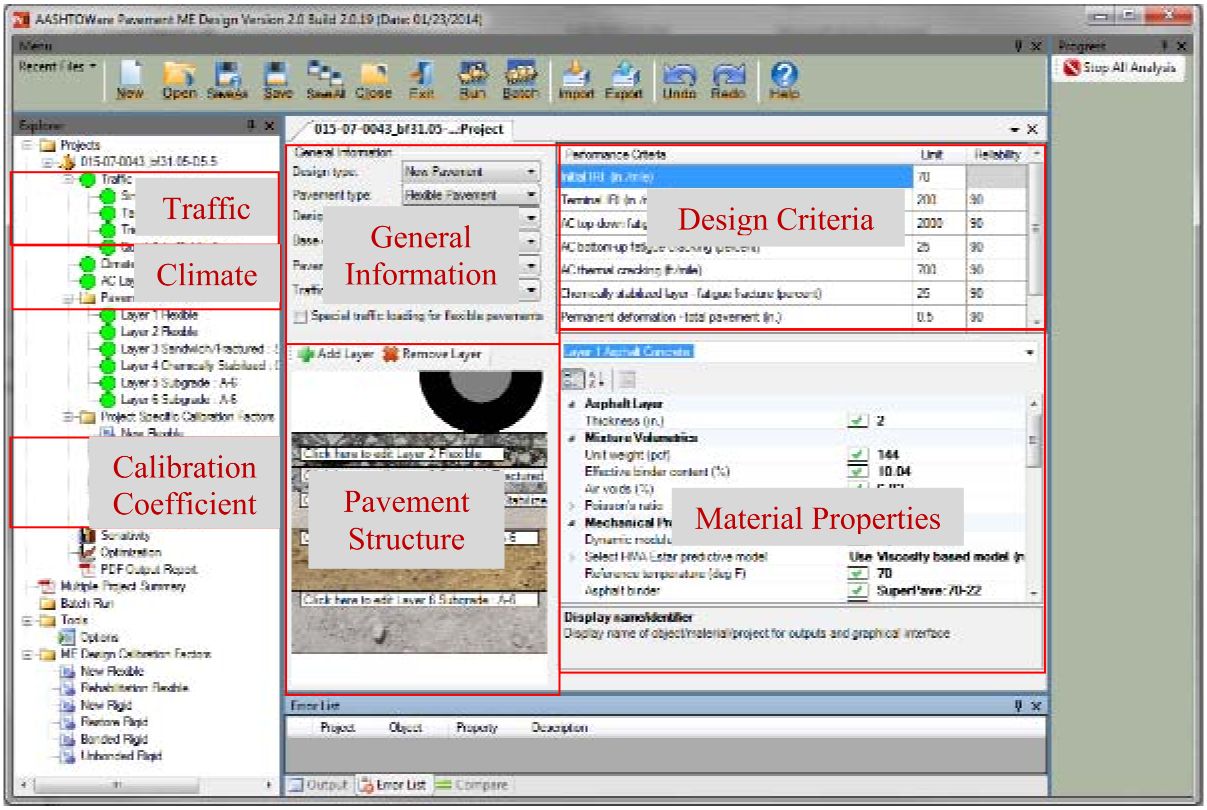


Figure 67
Main screen of AASHTOWare® Pavement ME™ software (version 2.0)

General Pavement Design Inputs

Design Type. Pavement ME software offers three types of design—New Pavement, Overlay, and Restoration.

Pavement Type. After selecting the Design Type, Pavement ME requires the designer to further identify the pavement type such as flexible pavement, jointed plain concrete pavement (JPCP), and AC over AC.

Design Life. Pavement design life is defined as the time from initial construction until the pavement has structurally deteriorated to a specified pavement condition -- the time when significant rehabilitation or reconstruction is needed. Table 40 lists the default design life for Louisiana.

Table 40
Pavement design life

Pavement Type	Design Life, years
New structure (AC and PCC)	20
Rubblize and overlay	15
Structural overlay	10

Base Construction. This is the year and month of the scheduled base construction time. If the exact time in unknown, it is assumed that the base construction will be in May for design purpose.

Pavement Construction. This is the year and month of the scheduled pavement construction time. Select June as the pavement construction month for design purpose if the exact time in unknown.

Traffic Opening. This is the year and month of the scheduled time that the pavement will be opened to traffic. For Louisiana, it is assumed that the open-to-traffic date will be in June for asphalt pavement and September for rigid pavements.

Design Criteria

Different from the *AASHTO 1993 Design Guide*, Pavement ME does not use present serviceability index (PSI). Instead, the software predicts individual distress such as alligator cracking, transverse cracking, rutting, faulting and smoothness (International Roughness Index, IRI). Design criteria are also closely tied with reliability level and pavement type. The design criteria shown in Table 41 are recommended for DOTD.

The initial IRI for flexible pavement and rigid pavement is 63 in./mi. as recommended by the *Manual of Practice*.

Table 41
Recommended design criteria for Pavement ME Design in Louisiana

Pavement Type	Distress	Interstate	Primary	Secondary
	[a]Reliability Level, %	95	90	80
New AC and AC overlay	Alligator cracking, %	15	25	35
	Total rutting, in.	0.40	0.50	0.65
	[b]AC rutting, in.	0.40	0.50	0.65
	Transverse cracking, ft/mi	500	700	700
	Reflective cracking, %	15	25	35
	IRI, in./mi.	160	200	200
New PCC	Faulting, in.	0.15	0.20	0.25
	Transverse cracking, %	10	15	20
	IRI, in./mi.	160	200	200

Note: a. Reliability level is not applicable to reflective cracking.
b. AC rutting uses the same criteria as total rutting.

Traffic Inputs

Pavement ME includes a comprehensive analysis of traffic, which requires a large number of input data from Weigh-in-Motion, automatic vehicle classification, and vehicle counts. It is recommended using the national default traffic inputs (e.g., traffic spectrum) except the following items:

Initial Two-way AADTT. The initial estimated two-way annual average daily truck traffic. This is calculated by multiplying the initial AADT with the truck percentage, both provided by the Traffic Division.

Number of Lanes. This is the number of lanes in the design direction, NOT the total lanes of both directions.

Percent Trucks in Design Direction. Usually between 50% and 60%. Based on the LTPP data for Louisiana, 55% can be used as the default. This value should be adjusted according to the traffic pattern of the design route.

Percent Trucks in Design Lane. Use these assumptions: (same as MEPDG default)

- One lane in design direction: 100%

- Two lanes in design direction: 90%
- Three lanes in design direction: 60%
- Four lanes in design direction: 50%

Operation Speed. Use the posted truck speed limit.

Design Lane Width. The width of each lane, in feet. For widened rigid pavements, only input the designated travel lane width (usually 12 ft.) as the "design lane width". The slab width (e.g., 15 ft.) will be an input under "JPCP design properties."

Vehicle Class Distribution. This can be calculated from the traffic estimation provided by the Traffic Division. Note that only trucks (Class 4 through 13 in the FHWA Vehicle Classification) are counted. Class 1 through 3 should not be included.

Growth Rate. Based on the traffic estimation provided by the Traffic Division. Use compound growth function. If data are not available, use 3% as the default.

The national default will be used for all other inputs such as monthly distribution, hourly distribution, truck configuration, axle per truck, single axle load distribution, tandem axle load distribution, tridem axle load distribution, and quad axle load distribution.

Climate Inputs

Longitude and Latitude. Provide the GPS coordinate of the center of the project in the format of decimal degrees.

Elevation. This can be determined from Google Earth by providing the GPS coordinate.

Depth of Water Table. Choose the available groundwater monitoring site near the project from United States Geological Survey (USGS) National Water Information System.

http://maps.waterdata.usgs.gov/mapper/index.html

It may also be available from soil borings for the project. When no information is available, assume a constant 5 ft. for Louisiana.

Climate Station. After inputting the longitude and latitude into Pavement ME, the designer can choose one of the nine climate stations closest to the project. The designer can also generate a virtual station from several nearby stations including stations located in neighboring states.

Materials Inputs

Asphalt Mixture. AC layer properties such as surface shortwave absorptivity, endurance limit and layer interface condition will be default values. Table 42 will be used as the default input for asphalt mixtures in Louisiana.

Table 42
Default AC material input parameters for typical AC mixtures in Louisiana

Design Input	Superpave	Superpave	Superpave
Asphalt Binder	PG 76-22	PG 70-22	PG 64-22
Cumulative % passing 3/4 inch sieve	95	96	89
Cumulative % passing 3/8 inch sieve	69	72	72
Cumulative % passing #4 sieve	48	52	54
% passing #200 sieve	5.1	5.6	5.3
Effective binder content (%)	9.49	9.46	9.17
In-place air void (%)	6.95	6.90	6.94
Total unit weight (pcf)	144	144	144

PCC Mixture and Rigid Pavement Design Features. Sensitivity analyses revealed that rigid pavement models in Pavement ME are very sensitive to coefficient of thermal expansion, PCC mixture strength, joint spacing, slab width, and slab thickness. Results also showed that the Level 3 input combination of modulus of rupture and elastic modulus could predict a better match with Level 1 input than using Level 3 compressive strength. Designers should refer to Table 43 for PCC mixture properties and rigid pavement design features. Inputs not listed will adopt national default values.

Table 43

PCC mixture properties and rigid pavement design features in Louisiana

Input	Louisiana Default
Aggregate type	Use limestone
PCC strength and modulus	28-day modulus of rupture: 600 psi 28-day elastic modulus 4,200,000 psi
Coefficient of thermal expansion (CTE)	5.5 x 10^{-6}/°F for default
PCC joint spacing	Use 20 ft.
Doweled joints	Always True and spacing at 12 in. Diameter depends on slab thickness (T). • Use 1.25 in. when T<10 in. • Use 1.5 in. when T≥10 in.
Tied shoulders	If PCC, tied shoulders with load transfer efficiency = 50%; Other than PCC, not tied shoulder
Erodibility index	Erosion resistant (3) for stabilized base Fairly erodible (4) for unbound base
PCC-base contact friction	Fully contact during the design life

RPCC. In general the modulus of rubblized PCC is higher than the modulus of a typical granular base and lower than the modulus of stabilized base materials. RPCC may have a large variation of modulus depending on the level of rubblization. The default value could be 200 ksi.

Stabilized Base Material. Base materials could be stabilized with different types of "binder" such as asphalt, cement, lime, fly ash, or a combination. Asphalt treated bases will be considered as "asphalt mixture" with properties listed in Table 42. Cement treated bases will be modeled as "chemically stabilized layer," including cement stabilized and soil cement. The default resilient modulus for 8.5 in. cement stabilized base is recommended as 100,000 psi, and 80,000 psi for 12 in. cement treated base. National defaults will be used for other parameters.

This recommendation is also applicable for other stabilized base materials such as lime cement fly ash and lime fly ash.

For rehabilitation and RPCC projects, the existing soil cement layer has most likely deteriorated under traffic loading. Therefore, the deteriorated modulus of 25,000 psi should be applied as recommended by the *AASHTO Manual of Practice*. In addition, due to a software bug which does not allow a modulus lower than 100,000 psi for soil cement, it is modeled as a crushed stone layer and assigned modulus of 25,000 psi for design purpose.

Unbound Granular Base Material. Limestone and recycled PCC are the most widely used unbound materials in Louisiana. The recommended resilient modulus for unbound granular base materials is:

- Use 30,500 psi for Kentucky limestone
- Use 27,000 psi for recycled PCC (crushed)
- Use 27,000 psi for default (unknown aggregate source)
- Use 23,500 psi for Mexican limestone

Specially, if an unbound stone layer is placed between two stabilized layers such as the case for stone interlayer, the modulus should be increased to 50,000 psi as a consideration of the increased confinement.

Subgrade. Subgrade data can be based on the soil survey for the design project. If a soil survey is not available, designers can estimate the subgrade type by locating to the GPS coordinate at this website http://nchrp923b.lab.asu.edu/

For resilient modulus, Louisiana will continue using values provided in the parish map at this time. Note that no extra strength will be given to a lime treated subgrade working table. In other words, the working table will follow the same resilient modulus listed in the parish map.

Existing Pavement Condition for Overlay Design. Choose Level 3 input. Total surface rutting and cracking should be measured from field testing or obtained from LA-PMS if possible. Then determine the level of existing condition according to Table 10-8 in the *Manual of Practice*. If not possible, assign poor as the default distress rating and 0.25 in. as the existing rutting for design purpose.

Interface Friction for Overlay Design. Full contact (friction coefficient = 1.0) should be used for all interfaces in all designs unless field investigation proves otherwise.

Local Calibration Coefficients

Results from Project 12-4P showed that the Pavement ME's national-calibrated distress/IRI models generally under-predicts bottom-up fatigue cracking and over-predicts rutting for flexible pavements. For rigid pavements, it was found that the Pavement ME's national-calibrated distress/IRI model over-predicts slab cracking but under-predicts joint faulting. A series of local calibration coefficients (Table 44) were developed under Project 12-4P.

Designers should apply these local calibration coefficients to different design modules of Pavement ME when performing a pavement design for DOTD.

Table 44
Local calibration coefficients for Louisiana M-E pavement design

Pavement Type	Distress	Calibration Factor	Calibration Coefficient
New flexible and AC overlays	AC bottom-up cracking	β_{f3}	1.05
		C_1	0.892
		C_2	0.892
	AC rutting	β_{r1}	0.80
		β_{r3}	0.85
	Subgrade rutting	β_{s1}	0.40
	Reflective cracking	c	0.72
		d	0.30
New rigid	Fatigue Cracking	C_1	2.75
		C_4	1.16
		C_5	-1.73
	Joint faulting	C_1	1.5276
		C_3	0.00262
		C_6	0.55

APPENDIX B

Distress Prediction Equations in Pavement ME™

A summary of distress models that were investigated in this study are presented here. For a detailed description please refer to the *AASHTO Manual of Practice* and MEPDG documents *[4], [2]*.

Flexible Pavements

Load-related Fatigue Cracking. Load-related fatigue cracking is the cracking in an asphalt concrete layer that is caused by repeated traffic loading. In Pavement ME, two types of load-related fatigue cracking are predicted for flexible pavements: bottom-up cracking (sometimes also referred as alligator cracking) and top-down cracking (also named as longitudinal cracking). The allowable number of axle-load applications needed for the incremental damage index approach to predict both types of load-related fatigue cracking is:

$$N_{f-HMA} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{HMA})^{k_{f3}\beta_{f3}} \tag{16}$$

where,

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

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

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

$k_{f1,f2,f3}$ = Global field calibration parameters (k_{f1} = 0.007566, k_{f2} = -3.9492, k_{f3} = -1.281);

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

C = 10^M

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

V_{be} = Effective asphalt content by volume, percent;

V_a = Percent of air voids in the HMA mixture; and

= Thickness correction term, dependent on type of cracking.

For bottom-up cracking:

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

For top-down cracking:

$$C_H = \frac{1}{0.01 + \frac{12}{1 + e^{(15.676 - 2.8186 H_{HMA})}}}$$

H_{HMA} = Thickness of HMA layer

Pavement ME calculates the amount of fatigue cracking of each type by the cumulative damage index DI. The cumulative damage index is determined by summing up the incremental damage indices over time:

$$DI = \sum (\Delta DI)_{j,m,l,p,T} = \sum \left(\frac{n}{N_f}\right)_{j,m,l,p,T} \tag{17}$$

where,

n	=	Actual number of axle load applications within a specific time period;
j	=	Axle load interval;
m	=	Axle load type (single, tandem, tridem, quad, or special axle configuration);
l	=	Truck type using the truck classification groups included in the MEPDG;
p	=	Month; and
T	=	Median temperature for the five temperature intervals or quartiles used to subdivide each month.

Bottom-up cracking is the fatigue cracking that initiates from the bottom of the HMA layer. It starts as a few short longitudinal or transverse cracks in the early stage and will develop into interconnected cracks with a chicken wire/alligator pattern. The unit for alligator cracking in Pavement ME is the percentage of total lane area.

The transfer function for bottom-up alligator cracking is:

$$FC_{Bottom} = \left(\frac{1}{60}\right)\left(\frac{C_4}{1 + e^{\left(C_1 C_1^* + C_2 C_2^* Log(DI_{bottom} * 100)\right)}}\right) \tag{18}$$

where,

FC_{Bottom} = area of alligator cracking, percentage of total lane area;

DI_{bottom} = cumulative damage index of alligator cracking;
$C_{1,2,4}$ = transfer function regression constants, C_1 = 1.00, C_2 = 1.00, C_4 = 6,000,
C_1^* = $-2C_2^*$; and
C_2^* = $-2.40874 - 39.748(1 + H_{HMA})^{-2.856}$.

Top-down cracking is another form of fatigue cracking that initiates at the surface of the HMA layer. It is often parallel to the pavement longitudinal centerline and does not develop into an alligator pattern. The unit for top-down cracking in Pavement ME is feet per mile.

The transfer function for top-down cracking is:

$$FC_{Top} = 10.56\left(\frac{C_4}{1 + e^{\left(C_1 - C_2 Log\left(DI_{Top}\right)\right)}}\right) \tag{19}$$

where,

FC_{Top} = length of longitudinal cracking, ft./mi.;
DI_{top} = cumulative damage index of longitudinal cracking; and
$C_{1,2,4}$ = transfer function regression constants, C_1 = 7.0, C_2 = 3.5, C_4 = 1,000.

Rutting (Permeant Deformation). Rutting is caused by permanent deformation developed in different pavement layers. Rut depth is defined as the maximum difference in elevation between the transverse profile of the HMA surface and a wire-line across the lane width. The unit for rut depth in Pavement ME is inches.

The transfer function for the AC layer is:

$$\Delta_{p(HMA)} = \varepsilon_{p(HMA)} h_{(HMA)} = \beta_{1r} k_z \varepsilon_{r(HMA)} 10^{k_{1r}} T^{k_{2r} * \beta_{2r}} N^{k_{3r} * \beta_{3r}} h_{(HMA)} \tag{20}$$

where,

$\Delta_{p(HMA)}$ = accumulated permanent or plastic vertical deformation in the HMA layer/sublayer, in.;
$\varepsilon_{p(HMA)}$ = accumulated permanent or plastic axial strain in the HMA layer/sublayer, in./in.;
$\varepsilon_{r(HMA)}$ = Resilient or elastic strain calculated by the structural response model at the mid-depth of each HMA sublayer, in./in.;

$h_{(HMA)}$ = Thickness of the HMA layer/sublayer, in.;
N = Number of axle-load repetitions;
T = Mix or pavement temperature, °F;
k_z = Depth confinement factor;
$k_z = (C_1 + C_2 D)0.328196^D$;
$C_1 = -0.1039(H_{HMA})^2 + 2.4868 H_{HMA} - 17.342$;
$C_2 = -0.0172(H_{HMA})^2 - 1.7331 H_{HMA} + 27.428$;
D = depth below the surface, in.;
H_{HMA} = Total HMA thickness, in.;
$k_{1r,2r,3r}$ = Global field calibration constants (k_{1r} = -3.35412, k_{2r} = 0.4791, k_{3r} = 1.5606); and
$\beta_{1r,2r,3r}$ = Local or mixture field calibration constants; for the global calibration, these constants were all set to 1.0.

The transfer function for rutting of the unbound layers (including unbound base and subgrade, but not stabilized base) is:

$$\Delta_{p(soil)} = \beta_{s1} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)^{\beta}} \quad (21)$$

where,

$\Delta_{p(soil)}$ = Permanent or plastic deformation for the layer/sublayer, in.;
N = Number of axle-load repetitions;
ε_0 = Intercept determined from laboratory repeated load permanent deformation tests, in./in.;
ε_r = Resilient strain imposed in laboratory test to obtain material properties ε_0, β and ρ, in./in.;
ε_v = Average vertical resilient or elastic strain in the layer/sublayer and calculated by the structural response model, in./in.;
h_{soil} = Thickness of the unbound layer/sublayer, in.;
k_{s1} = Global calibration coefficients; k_{s1} = 1.673 for granular materials and 1.35 for fine-grained materials;
β_{s1} = Local calibration constant for the rutting in the unbound layers; the local calibration constant was set to 1.0 for the global calibration effort;
$\log\ \beta$ = $-0.61119 - 0.017638 W_c$;
W_c = water content, percentage; and

$\rho \quad = \quad 10^9 \left[\frac{4.89285}{1-(10^9)^\beta}\right]^{\frac{1}{\beta}}.$

Smoothness (IRI). International Roughness Index (IRI) is used to define the pavement smoothness in Pavement ME. IRI is calculated based on an empirical function of other pavement distresses. The unit for IRI is in./mi. The equation for calculating IRI in new flexible pavements is:

$$IRI = IRI_0 + 0.015(SF) + 0.400(FC_{Total}) + 0.0080(TC) + 40.0(RD) \tag{22}$$

where,

IRI_0 = initial IRI after construction, in./mi.;
SF = site factor;
$SF = Age\big(0.02003(PI + 1) + 0.007947(Precip + 1) + 0.000636(FI + 1)\big)$
Age = pavement age, years,
PI = plastic index of the soil,
FI = average annual freezing index, degree F-days, and
$Precip$ = average annual precipitation or rainfall, in.
FC_{Total} = area of fatigue cracking (combined alligator, longitudinal, and reflection cracking in the wheel path), percent of total lane area, (longitudinal cracking is multiplied by 1-ft. to convert to an area basis);
TC = length of transverse cracking, ft./mi.; and
RD = average rut depth, in.

Transverse Cracking (Thermal Cracking). Transverse cracking is a non-load-related cracking, which is usually caused by low temperature or thermal cycling. The unit for transverse cracking in Pavement ME is feet per mile.

The transfer function for transverse cracking is:

$$TC = \beta_{t1} N \left[\frac{1}{\sigma_d}\left(\frac{C_d}{H_{HMA}}\right)\right] \tag{23}$$

where,

TC	=	amount of thermal cracking, ft./mi.;
β_{t1}	=	regression coefficient determined through global calibration (= 400);
N	=	standard normal distribution evaluated at $[z]$;
σ_d	=	standard deviation of the log of the depth of cracks in the pavement (= 0.769 in.);
C_d	=	crack depth, in.; and
H_{HMA}	=	thickness of HMA layers.

Rigid Pavements

The structural distresses considered for jointed plain concrete pavement are fatigue-related transverse cracking of PCC slabs and differential deflection related transverse joint faulting.

Transverse Slab Cracking (Bottom-Up and Top-Down) – JPCP. Transverse cracking of PCC slabs can initiate either at the top surface of the PCC slab and propagate downward (top-down cracking) or vice versa (bottom-up cracking) depending on the loading and environmental conditions at the project site, as well as material properties, design features, and the conditions during construction. Both top-down and bottom-up cracking are considered in Pavement ME.

The percentage of slabs with transverse cracks (including all severities) in a given traffic lane is used as the measure of transverse cracking and is predicted using the following global equation for both bottom-up and top-down cracking:

$$CRK = \frac{1}{1 + C_4(DI)^{C_5}} \tag{24}$$

where,

CRK	=	Predicted amount of bottom-up or top-down cracking;
DI	=	Fatigue damage;
C_4, C_5	=	Coefficients, default $C_4 = 1.0$, $C_5 = -1.98$

The calculation of fatigue damage follows Miner's hypothesis and is similar to flexible pavements as shown in equation (17). Fatigue damage is the accumulation of the applied number of load application divided by the allowable number of load application. The applied number of load application for rigid pavement is the same as it is for flexible pavement. But the allowable number of load application for rigid pavement depends on pavement structure and is calculated by

$$\log\left(N_{i,j,k,l,m,n}\right) = C_1 \left(\frac{MR_i}{\sigma_{i,j,k,l,m,n}}\right)^{C_2} \tag{25}$$

where,

$N_{i,j,k,...}$	=	Allowable number of load applications at condition *i, j, k, l, m, n*;
MR_i	=	PCC modulus of rupture at age *i*, psi;
$\sigma_{i,j,k,...}$	=	Applied stress at condition *i, j, k, l, m, n*;
C_1, C_2	=	Calibration constants, default $C_1 = 2.0$, $C_2 = 1.22$

The mechanistic model first calculates the critical stress σ for each condition, then the allowable number of load applications N can be determined, followed by the calculation of fatigue damage DI. Once bottom-up and top-down damage are estimated, the corresponding cracking is computed using equation (9) and the total combined cracking can be determined.

$$TCRACK = \left(CRK_{Bottom-up} + CRK_{Top-down} - CRK_{Bottom-up} \cdot CRK_{Top-down}\right) \cdot 100\% \tag{26}$$

where,

$TCRACK$	=	Total transverse cracking (percent, all severities);
$CRK_{Bottom-up}$	=	Predicted amount of bottom-up transverse cracking;
$CRK_{Top-down}$	=	Predicted amount of top-down transverse cracking;

Joint Faulting – JPCP. Repeated axle loads crossing transverse joints creates the potential for joint faulting. In Pavement ME, joint faulting is predicted month by month using an incremental approach. The faulting at each month is determined as a sum of faulting increments from all previous months in the pavement life from the traffic opening date. The current faulting level affects the magnitude of faulting increment. The following equations are used to predict faulting in Pavement ME:

$$Fault_m = \sum_{i=1}^{m} \Delta Fault_i \tag{27}$$

$$\Delta Fault_i = C_{34} * \left(FAULTMAX_{i-1} - Fault_{i-1}\right)^2 * DE_i \tag{28}$$

$$FAULTMAX_i = FALUTMAX_0 + C_7 * \sum_{j=1}^{m} DE_j * Log(1 + C_5 * 5.0^{EROD})^{C_6} \quad (29)$$

$$FAULTMAX_0 = C_{12} * \delta_{curling} * \left[Log(1 + C_5 * 5.0^{EROD}) * Log(\frac{P_{200} * WetDays}{p_s})\right]^{C_6} \quad (30)$$

where,

$Fault_m$	=	Mean joint faulting at the end of month m, in.;
$\Delta Fault_i$	=	Incremental change (monthly) in joint faulting during month i, in.;
$FAULTMAX_i$	=	Maximum mean transverse joint faulting for month i, in.;
$FAULTMAX_0$	=	Initial maximum mean transverse joint faulting, in.;
$EROD$	=	Base/subbase erodibility factor;
DE_i	=	Differential density of energy of subgrade deformation accumulated during month i;
$\delta_{curling}$	=	Maximum mean monthly slab corner upward deflection PCC due to temperature curling and moisture warping;
P_S	=	Overburden on subgrade, lb;
P_{200}	=	Percent subgrade material passing #200 sieve;
$WetDays$	=	Average annual number of wet days (greater than 0.1 in. rainfall);
$C_{1,2,3,4,5,6,7,12,34}$	=	Global calibration constants, default $C_1 = 1.29$, $C_2 = 1.1$, $C_3 = 0.001725$, $C_4 = 0.0008$, $C_5 = 250$, $C_6 = 0.4$, $C_7 = 1.2$ $C_{12} = C_1 + C_2 * FR^{0.25}$ $C_{34} = C_3 + C_4 * FR^{0.25}$
FR	=	Base freezing index defined as percentage of time the top base temperature is below freezing (32°F) temperature.

Smoothness (IRI) – JPCP. In Pavement ME, smoothness is predicted as a function of the initial as-constructed profile of the pavement and any change in the longitudinal profile over time and traffic due to distresses and foundation movements. The IRI model includes transverse slab cracking, joint faulting, joint spalling, and site factor for JPCP. The site factors include subgrade and climatic factor to account for the roughness caused by shrinking or swelling soils and frost heave conditions. IRI is estimated incrementally over the entire design period on a monthly basis.

$$IRI = IRI_0 + C_1 * CRK + C_2 * SPALL + C_3 * TFAULT + C_4 * SF \quad (31)$$

where,

IRI_0 = initial IRI after construction, in./mi.;
CRK = Percent slabs with transverse cracks (all severities);
$SPALL$ = Percentage of joints with spalling;
$TFAULT$ = Total joint faulting cumulated per mi, in.;
$C_{1,2,3,4}$ = Global calibration constants, default $C_1 = 0.8203$, $C_2 = 0.4417$, $C_3 = 1.4929$, $C_4 = 25.24$
SF = site factor;
$SF = AGE(1 + 0.5556 * FI)(1 + P_{200}) * 10^{-6}$
AGE = pavement age, years,
FI = average annual freezing index, degree F-days.

Rehabilitation Pavements

Reflective Cracking in HMA Overlays. Pavement ME predicts reflection cracking in HMA overlays using an empirical equation. This equation predicts the percentage of area of cracks that propagate through the HMA as a function of time using a sigmoid function.

$$RC = \frac{1}{1 + e^{a(c)+bt(d)}} \tag{32}$$

where,

RC = Percent of cracks reflected;
t = Time, year;
a, b = Regression fitting parameters defined through calibration process,
$a = 3.5 + 0.75(H_{eff})$
$b = -0.688684 - 3.37302(H_{eff})^{-0.915469}$
H_{eff} = Effective HMA overlay thickness;
c, d = User-defined cracking progression parameters.

APPENDIX C

Consensus Survey and Results Analysis

The objective of a consensus survey of engineers at different divisions of DOTD was to evaluate the design criteria recommended by the *Manual of Practice [4]* and, if necessary, to develop a set of criteria that match with the state-of-the-practice in pavement design for Louisiana.

This task was accomplished through a consensus survey of DOTD engineers to capture their experience and the state-of-practice. First, 32 (18 HMA-surfaced and 14 PCC-surfaced) representative distressed pavement images were selected from Visdata and LA-PMS. These sections were carefully selected to represent different highway classifications (interstate, arterial and collector), different traffic volumes (AADT ranging from 320 to 47,500), and different geographical locations in Louisiana. Furthermore, they were intentionally selected to include different distress types and severities such as fatigue cracking, longitudinal cracking, transverse cracking, and rutting.

An online survey of these 32 images was then developed and sent out through email to a wide variety of DOTD engineers, such as district engineers, design engineers, research engineers, management engineers, and maintenance engineers. In the survey, participants were asked to select one of the following choices: (1) Do nothing, (2) Overlay, and (3) Major rehabilitation for each pavement section based on their experience and the condition of each section. The survey results were then statistically analyzed against the measured pavement distress for the corresponding 0.1-mile section retrieved from LA-PMS. Besides regression analysis, the current DOTD PMS distress triggers were considered. Finally, a suite of design criteria was proposed for use in the implementation of Pavement ME in Louisiana.

Screenshots of the Consensus Survey

The following graphs are screenshots of the online survey as viewed by participants. The resolution is reduced to fit to the file size and page limit of this report. High resolution graphs of the survey are available upon request.

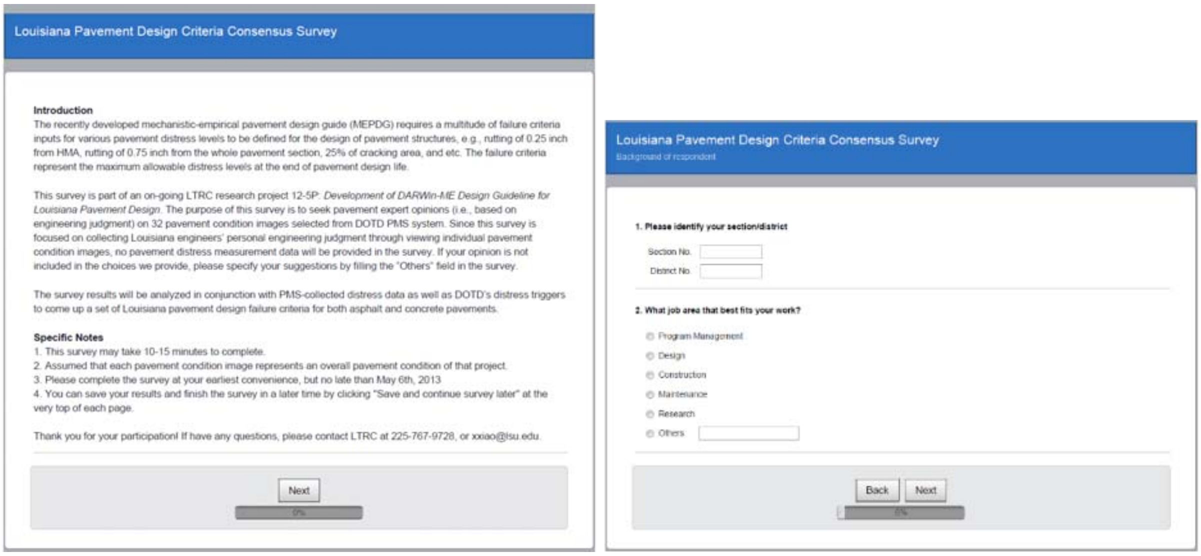


Figure 68

Consensus survey front page 1 and 2

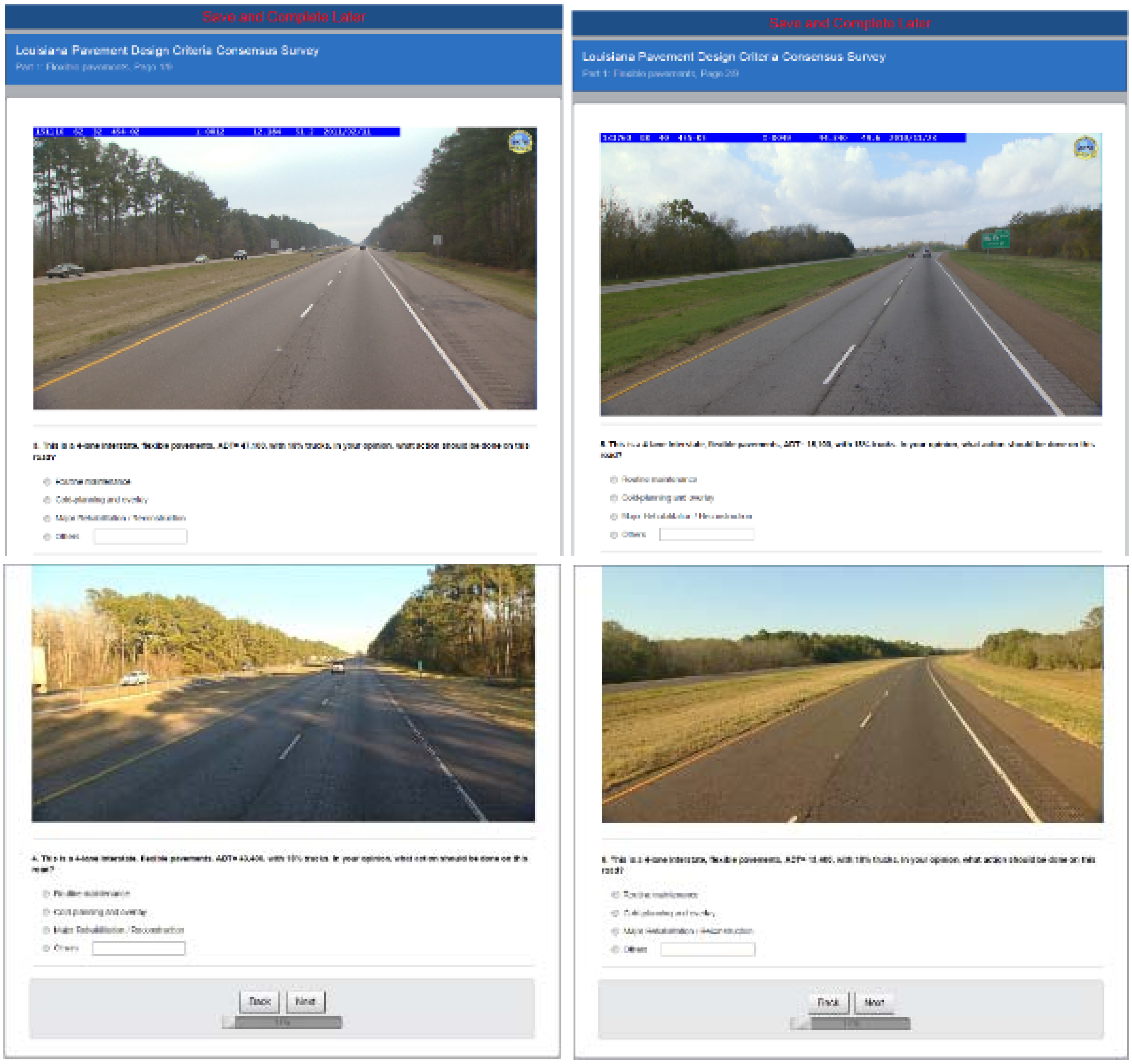


Figure 69

Consensus survey: flexible pavement page 1 and 2

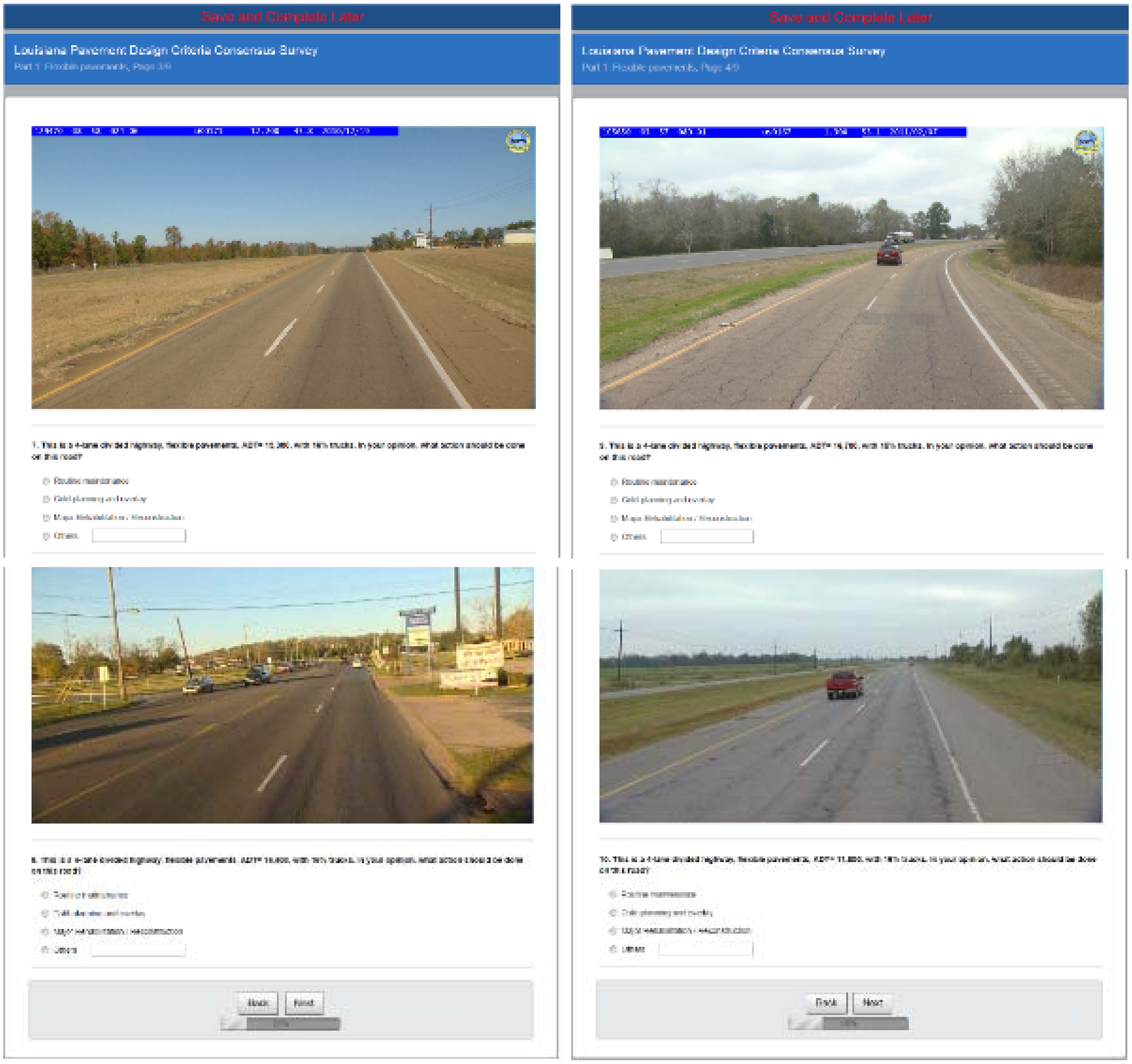


Figure 70

Consensus survey: flexible pavement page 3 and 4

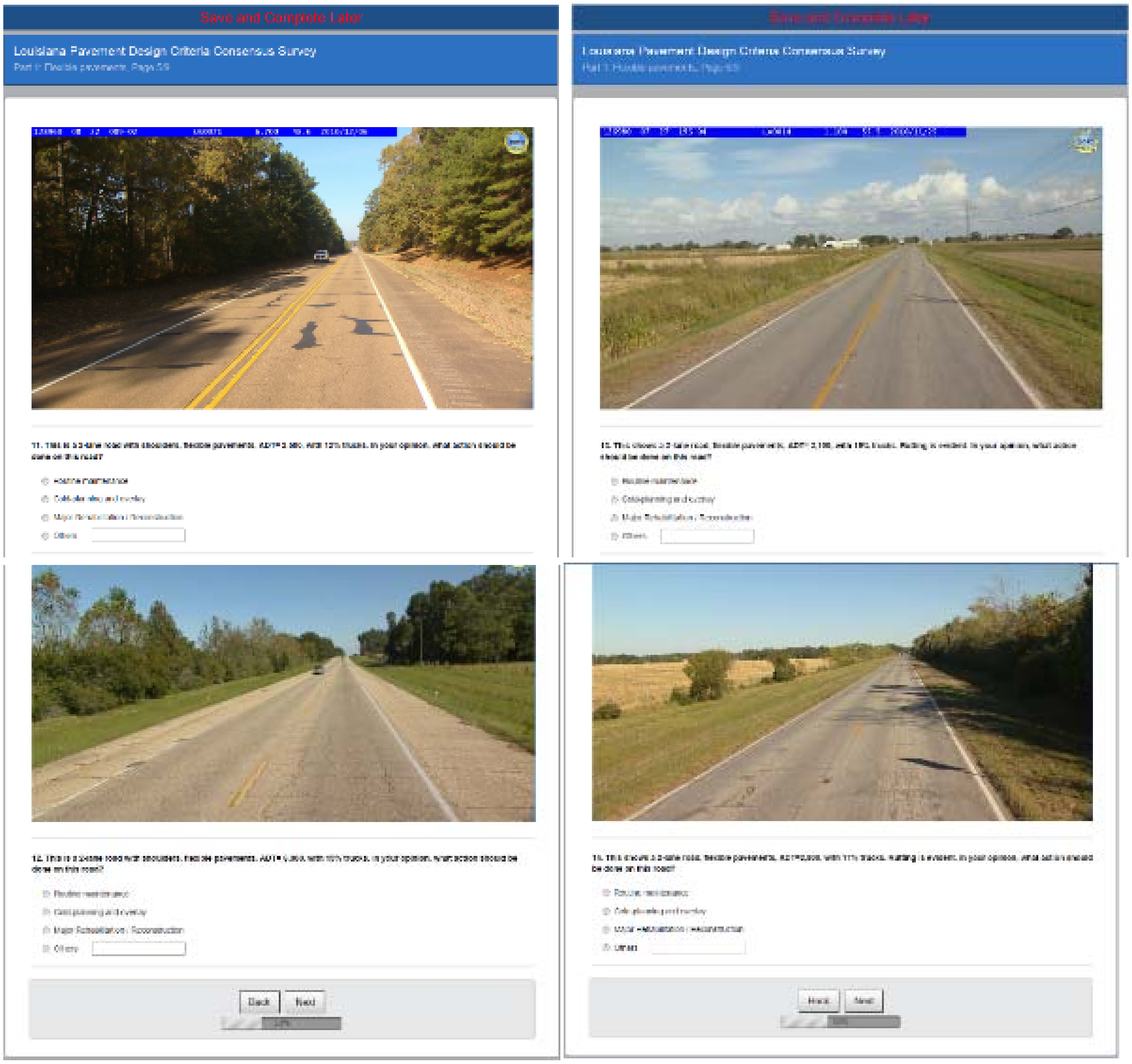


Figure 71

Consensus survey: flexible pavement page 5 and 6

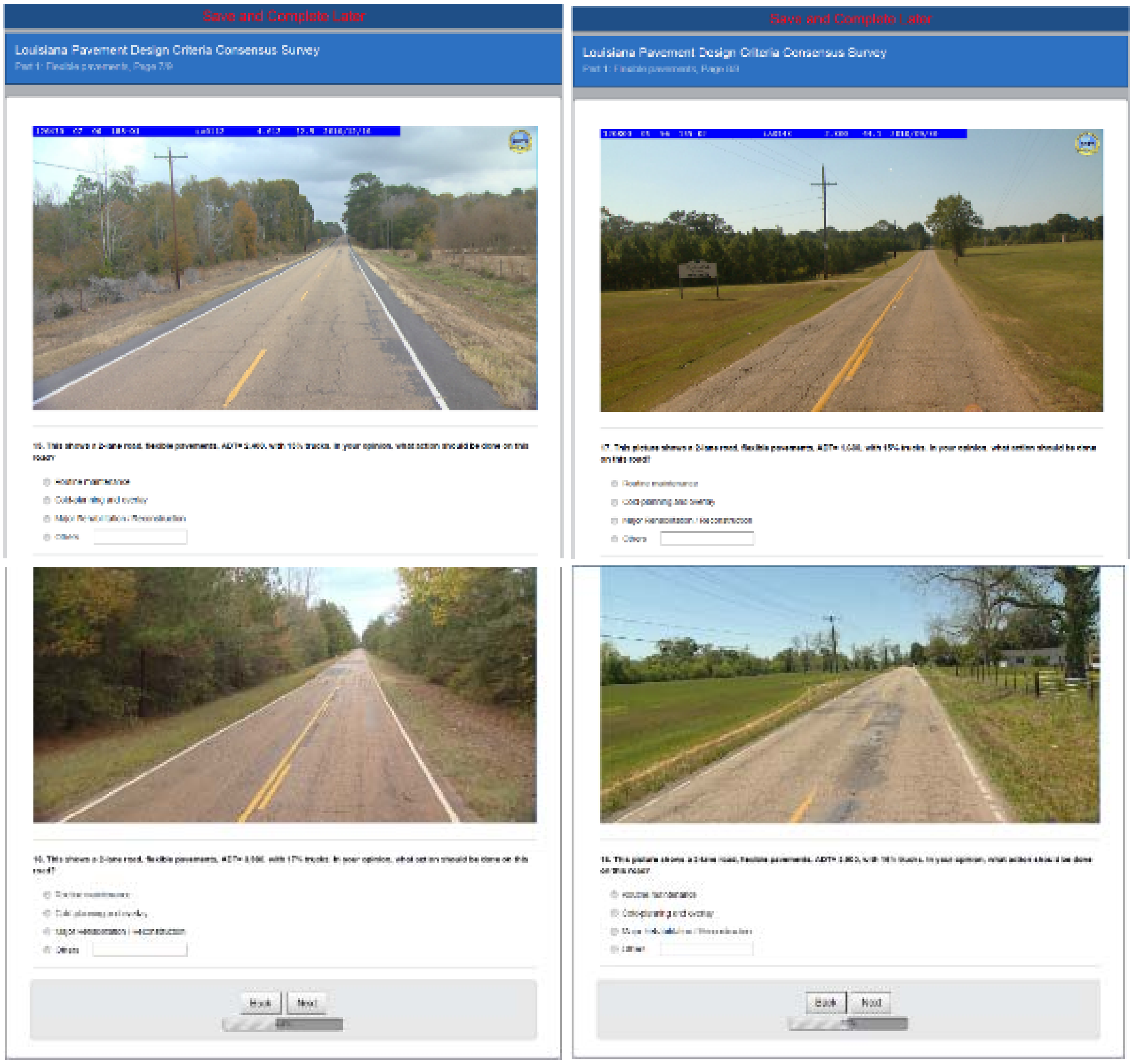


Figure 72

Consensus survey: flexible pavement page 7 and 8

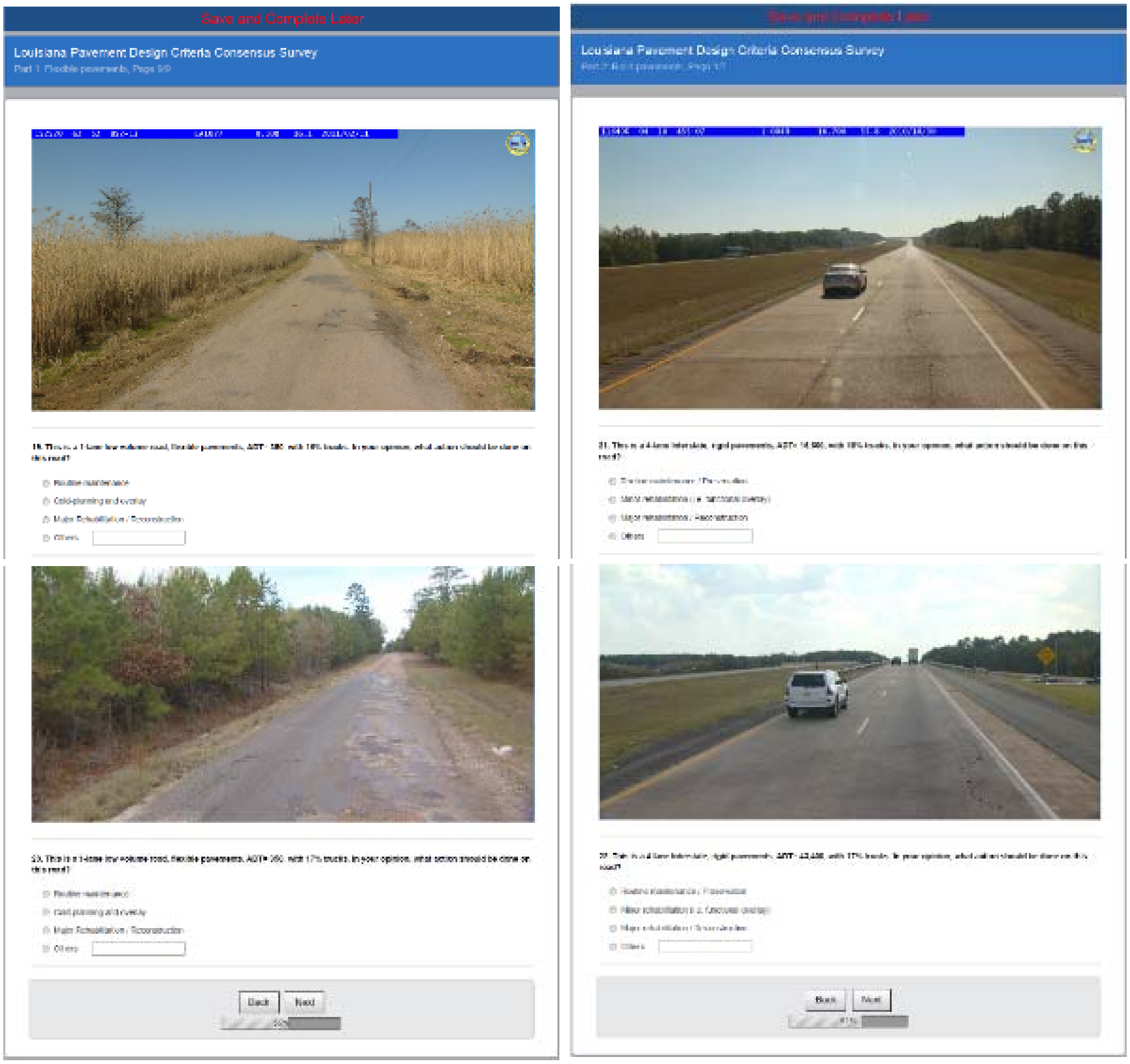


Figure 73

Consensus survey: flexible pavement page 9, rigid pavement page 1

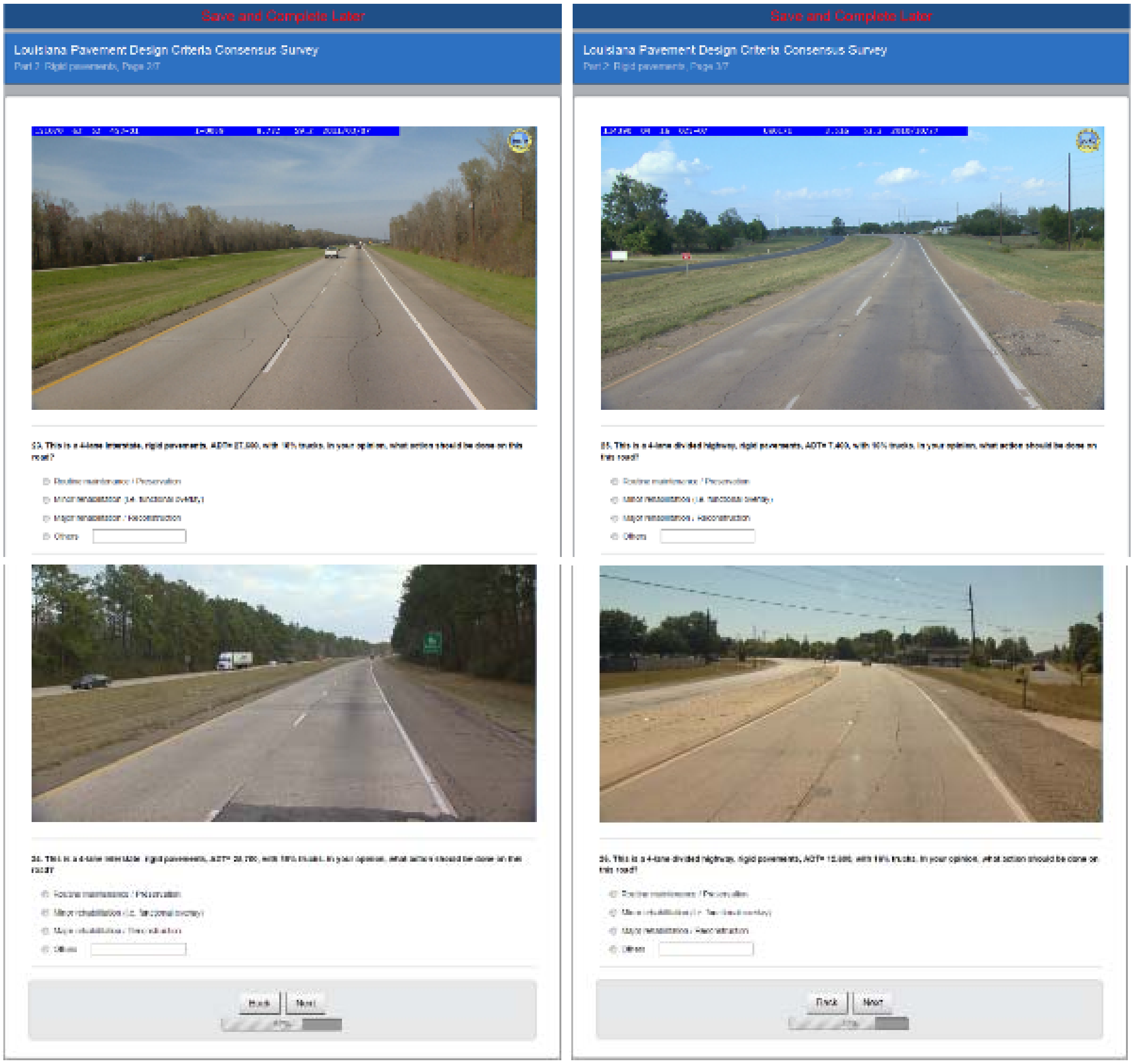


Figure 74

Consensus survey: rigid pavement page 2 and 3

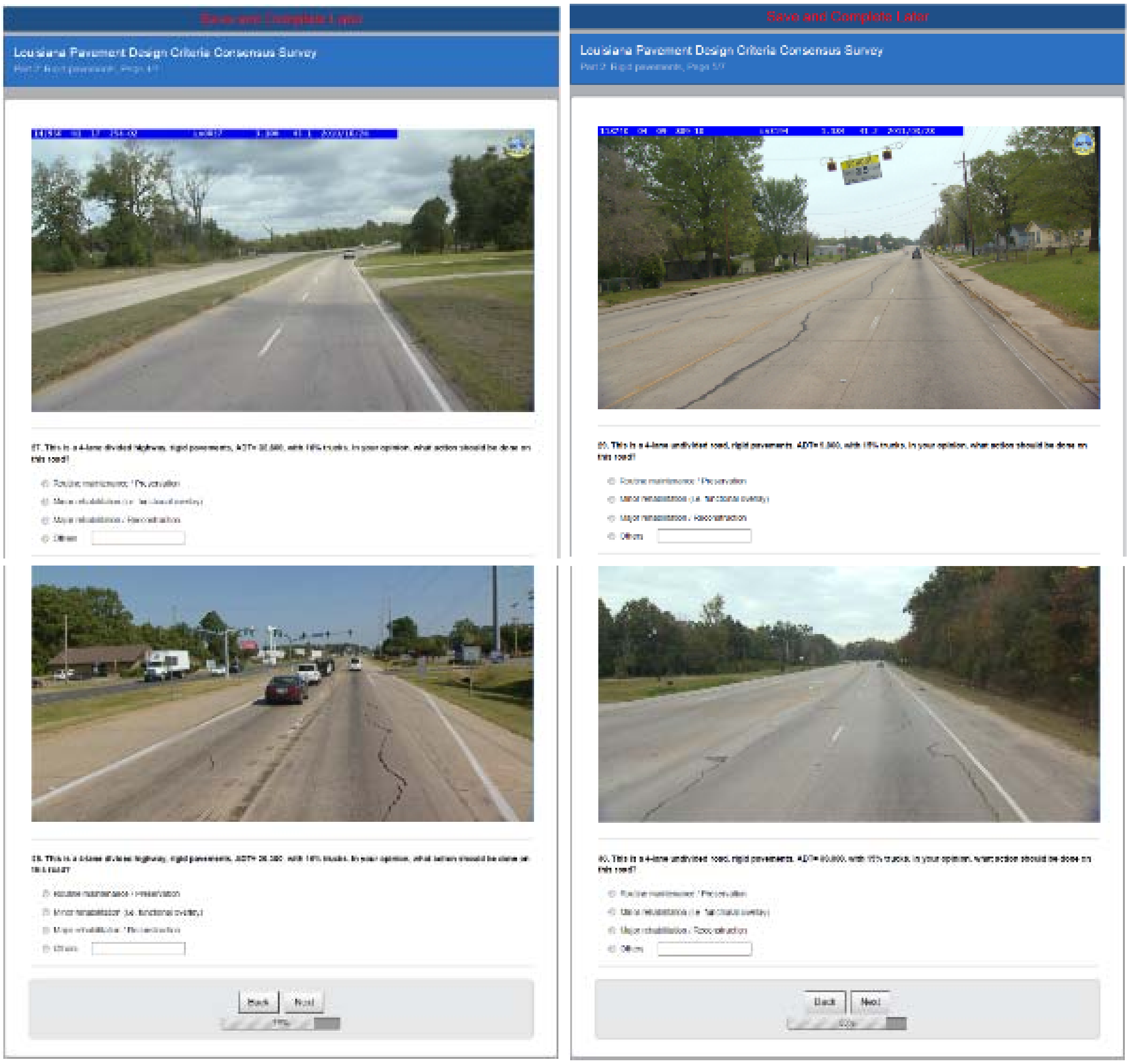


Figure 75

Consensus survey: rigid pavement page 4 and 5

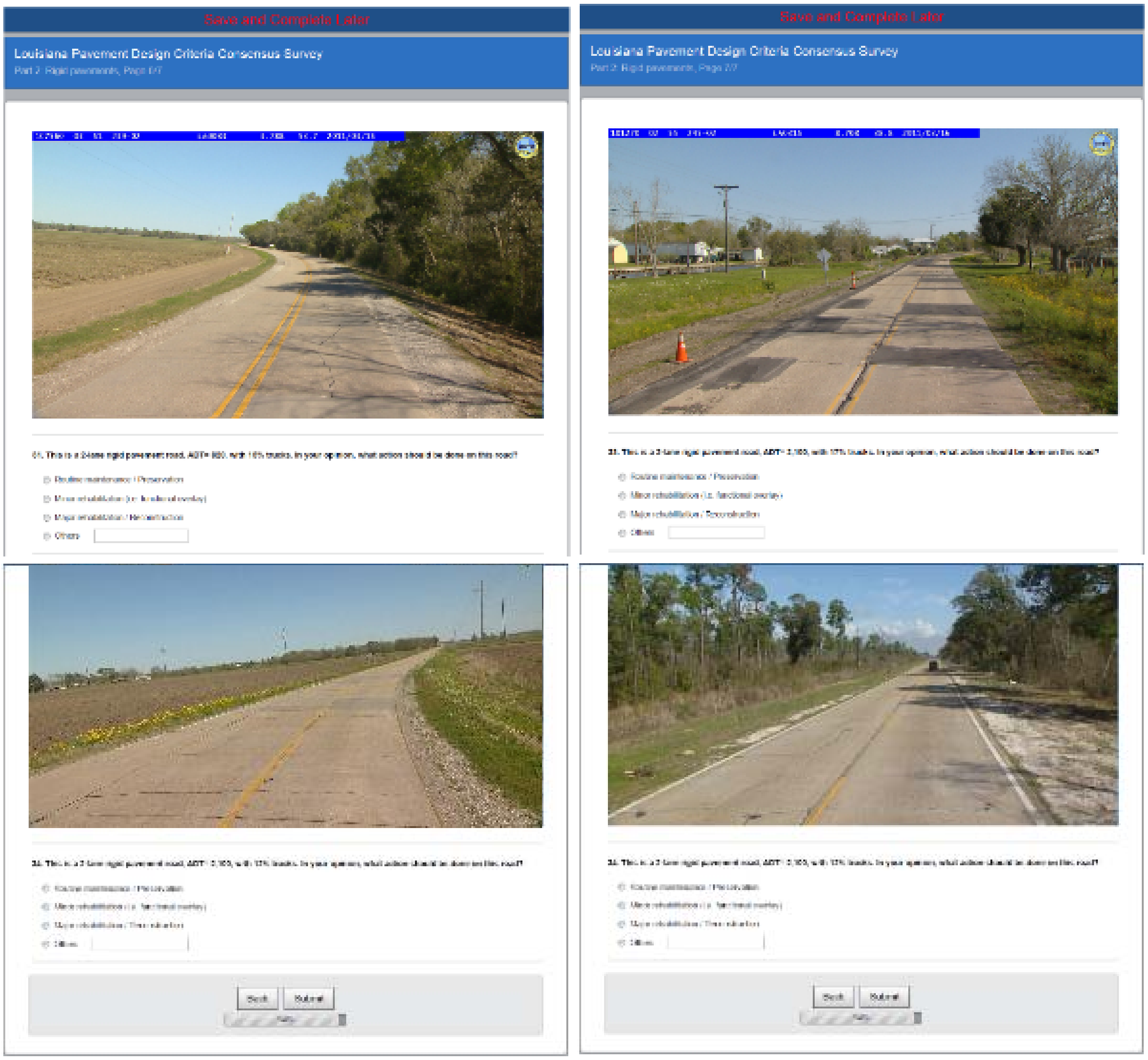


Figure 76

Consensus survey: rigid pavement page 6 and 7

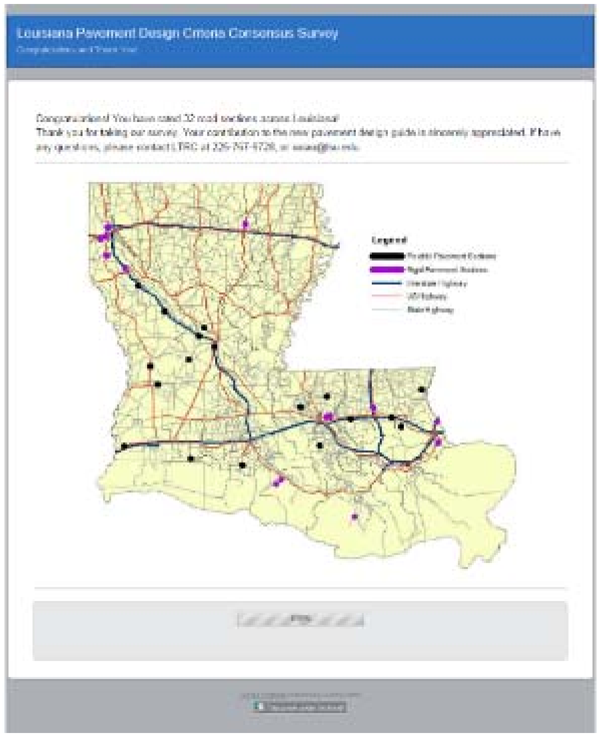


Figure 77

Consensus survey closing page

Survey Results

The survey received a total of 24 responses, in which six were from supervisors or managers, eight were from design engineers, nine were from construction engineers, and one was from a maintenance engineer. Table 45 lists the survey results. For each pavement section, a respondent recommended one and only one maintenance action, either routine maintenance or overlay or reconstruction. Since the criteria of failure are different from person to person, a pavement section may receive different suggestions. Taking Survey ID 1 for an example, 14 respondents (14/24 = 58%) thought routine maintenance was needed; 10 respondents (10/24 = 42%) recommended an overlay; none of them considered the pavement had reached the failure criteria that warrants reconstruction.

Pavement distresses corresponding to the survey sections were retrieved from the pavement management system. Other inventory and traffic data were collected from Track of Projects (TOPS) and the Mainframe database at DOTD. Table 46 lists primary distresses such as alligator cracking, rutting, IRI for flexible pavements, and Table 47 presents transverse cracking, faulting and IRI for rigid pavement. It can be found that the selected pavement sections covered a wide range of distress severities. For instance, alligator cracking ranged from 3.67% to 57.05%; rutting ranged from 0.12’’ to 0.70’’; IRI for flexible pavement had a range of 65 to 358 in/mi.

Table 45
Results of design criteria consensus survey

Pvmt Type	Survey ID	Control Section	Route	AADT	Survey Results		
					Routine Maintenance	Overlay	Reconstruction
ASP	1	454-02	I-0012	47,500	14	10	0
ASP	2	454-03	I-0012	42,800	6	13	5
ASP	3	455-05	I-0049	14,300	7	12	4
ASP	4	455-07	I-0049	12,300	12	10	2
ASP	5	024-06	US0171	12,000	8	12	3
ASP	6	014-06	US0165	16,300	1	12	10
ASP	7	080-01	US0167	15,700	0	14	9
ASP	8	050-06	LA0001	12,500	0	7	16
ASP	9	009-02	US0071	2,800	15	7	1
ASP	10	060-03	LA0067	8,400	4	15	3
ASP	11	196-04	LA0014	2,100	12	9	1
ASP	12	810-25	LA3063	2,700	8	14	1
ASP	13	188-01	LA0112	2,700	12	10	1
ASP	14	029-05	LA0121	3,000	4	12	7
ASP	15	155-02	LA0143	1,600	4	13	6
ASP	16	227-04	LA0413	2,700	2	6	15
ASP	17	852-13	LA1077	1,370	14	7	2
ASP	18	116-04	LA0478	320	8	7	8
JCP	21	455-07	I-0049	14,600	20	3	0
JCP	22	427-01	LA3132	38,400	14	8	1
JCP	23	453-01	I-0059	29,000	11	7	5
JCP	24	452-90	I-0055	25,500	7	9	7
JCP	25	025-07	US0171	6,500	9	8	6
JCP	26	809-08	LA0526	11,600	13	8	2
JCP	27	254-02	LA0037	31,600	9	8	6
JCP	28	016-01	US0165	28,200	4	4	15
JCP	29	809-10	LA3194	10,000	19	3	0
JCP	30	817-08	LA0946	32,300	3	10	10
JCP	31	239-02	LA0083	1,010	19	4	0
JCP	32	239-02	LA0083	1,010	14	8	1
JCP	33	245-02	LA0315	2,600	1	10	13
JCP	34	006-07	US0090	1,340	8	8	8

Table 46
PMS data for pavement sections in the consensus survey (flexible pavement)

Survey ID	Control Section	Route	AADT	ALCR (%)	LNCR (ft/mi)	TRCR (ft/mi)	RUT (in.)	IRI (in/mi)
1	454-02	I-0012	47,500	6.10	350	360	0.16	75
2	454-03	I-0012	42,800	3.67	326	1974	0.20	105
3	455-05	I-0049	14,300	14.33	665	916	0.18	65
4	455-07	I-0049	12,300	29.68	442	1235	0.12	89
5	024-06	US0171	12,000	15.42	738	1120	0.70	147
6	014-06	US0165	16,300	31.94	100	137	0.61	152
7	080-01	US0167	15,700	34.64	290	2093	0.31	175
8	050-06	LA0001	12,500	57.05	319	1413	0.48	229
9	009-02	US0071	2,800	27.07	149	884	0.29	157
10	060-03	LA0067	8,400	50.46	347	2260	0.25	133
11	196-04	LA0014	2,100	12.08	51	487	0.64	151
12	810-25	LA3063	2,700	7.88	38	629	0.41	168
13	188-01	LA0112	2,700	19.02	175	1277	0.16	144
14	029-05	LA0121	3,000	23.63	257	841	0.26	326
15	155-02	LA0143	1,600	32.20	661	1565	0.32	175
16	227-04	LA0413	2,700	46.42	752	1207	0.32	272
17	852-13	LA1077	1,370	7.99	212	37	0.47	151
18	116-04	LA0478	320	23.55	506	726	0.27	358
Max.			47,500	57.05	752	2260	0.70	358
Avg.			11,172	24.62	354	1065	0.34	171
Min.			320	3.67	38	37	0.12	65

Note: ALCR = alligator cracking, LNCR = longitudinal cracking, TRCR = transverse cracking.

Table 47
PMS data for pavement sections in the consensus survey (rigid pavement)

Survey ID	Control Section	Route	AADT	ALCR (%)	LNCR (ft/mi)	TRCR (%)	FAULT (in.)	IRI (in/mi)
21	455-07	I-0049	14,600	N/A	114	0.11	0.00	95
22	427-01	LA3132	38,400	N/A	298	16.84	0.24	177
23	453-01	I-0059	29,000	N/A	124	0.00	0.00	124
24	452-90	I-0055	25,500	N/A	123	23.25	0.40	177
25	025-07	US0171	6,500	N/A	229	2.42	0.25	176
26	809-08	LA0526	11,600	N/A	216	6.94	0.08	155
27	254-02	LA0037	31,600	N/A	294	0.00	0.23	186
28	016-01	US0165	28,200	N/A	557	15.47	0.34	225
29	809-10	LA3194	10,000	N/A	164	1.05	0.13	148
30	817-08	LA0946	32,300	N/A	240	8.73	0.23	174
31	239-02	LA0083	1,010	N/A	125	0.11	0.27	184
32	239-02	LA0083	1,010	N/A	110	1.05	0.24	253
33	245-02	LA0315	2,600	N/A	134	66.50	0.31	367
34	006-07	US0090	1,340	N/A	160	40.09	0.32	189
Max.			38,400		557	66.50	0.40	367
Avg.			16,690		206	13.04	0.22	188
Min.			1,010		110	0.00	0.00	95

Note: ALCR = alligator cracking, LNCR = longitudinal cracking, TRCR = transverse cracking.

Statistical Analysis

To facilitate statistical analysis, the recommended treatments (Table 45) were first normalized to percentages and then converted to a single value named Weighted Treatment Score which represents the overall suggestions from all 24 respondents.

$$WeightedTreatmentScore = \sum_{i=1}^{3} (Treatment_i * Weight_i) \tag{33}$$

Treatment weights were assigned as:

- Routine maintenance =1.0
- Overlay = 3.0
- Reconstruction =5.0

For example, the weighted treatment score for Survey ID 1 was calculated by

$$WeightedTreatmentScore = \frac{14}{24} * 1 + \frac{10}{24} * 3 + \frac{0}{24} * 5 = 1.83 \quad (34)$$

A score of 1.83 could be interpreted as "between routine maintenance and overlay but tilt towards routine maintenance". After this process, results from different respondents were converted to a single value between 1.0 and 5.0.

It is reasonable to assume that pavement distresses as recorded in PMS are related to the weighted treatment score from the consensus survey. Ideally if a pavement was in poor condition, the data in PMS should show it so as the recommended treatment would be on the side of reconstruction. Figure 78 and Figure 79 show the scatterplot of distress versus weighted treatment score for flexible pavement and rigid pavement, respectively. As expected, it is obvious to see the trend of "the worse the distress is, the higher the weighted treatment score is". Although R-squares are low from the perspective of statistics, one has to understand that 32 sections are very limited to cover pavements in a wide range of conditions (138 pavement sections were studied in the AASHO Road Test to develop the Present Serviceability Index) *[71]*.

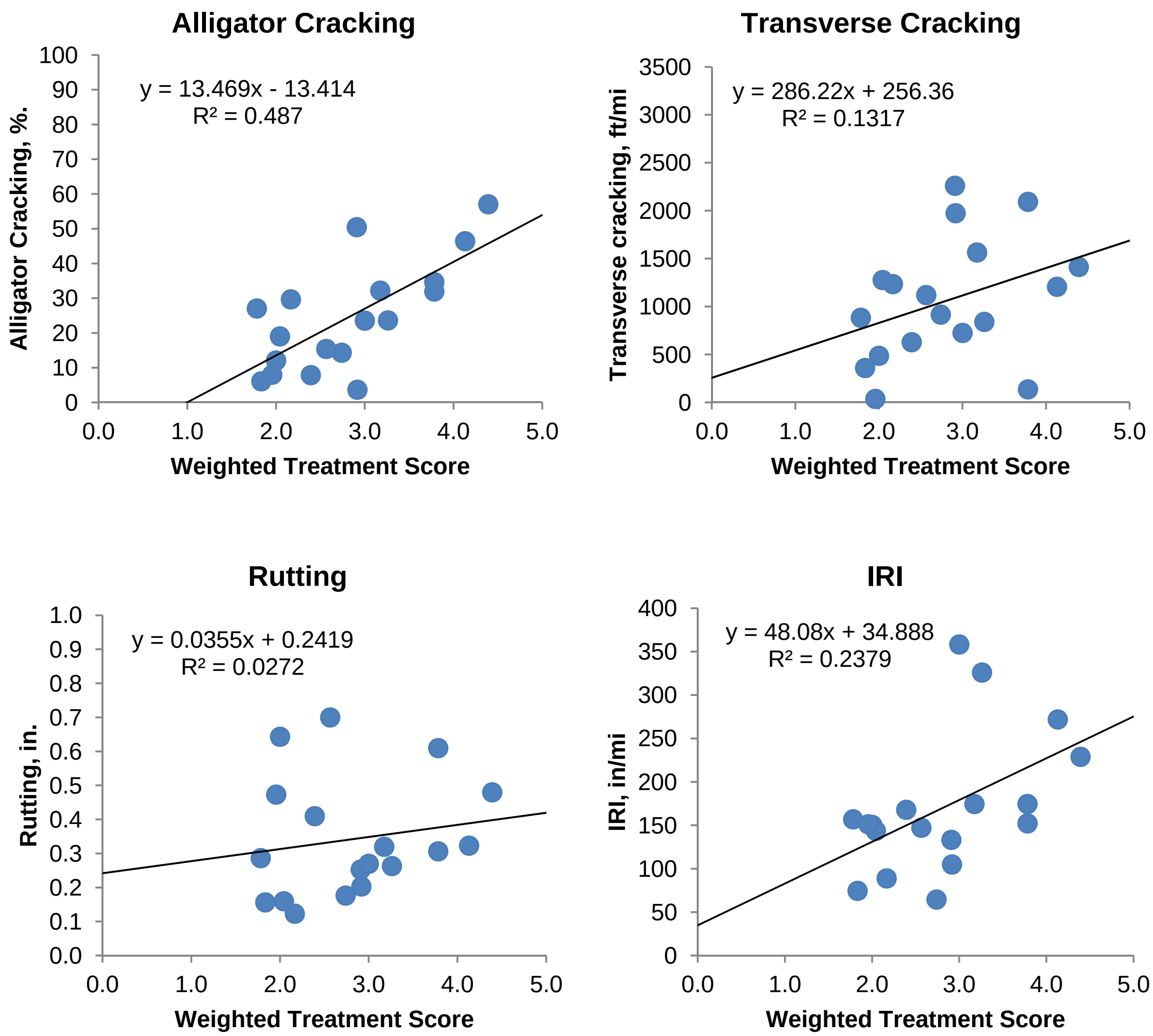


Figure 78

Relationship between pavement distresses and weighted treatment score (flexible pavements)

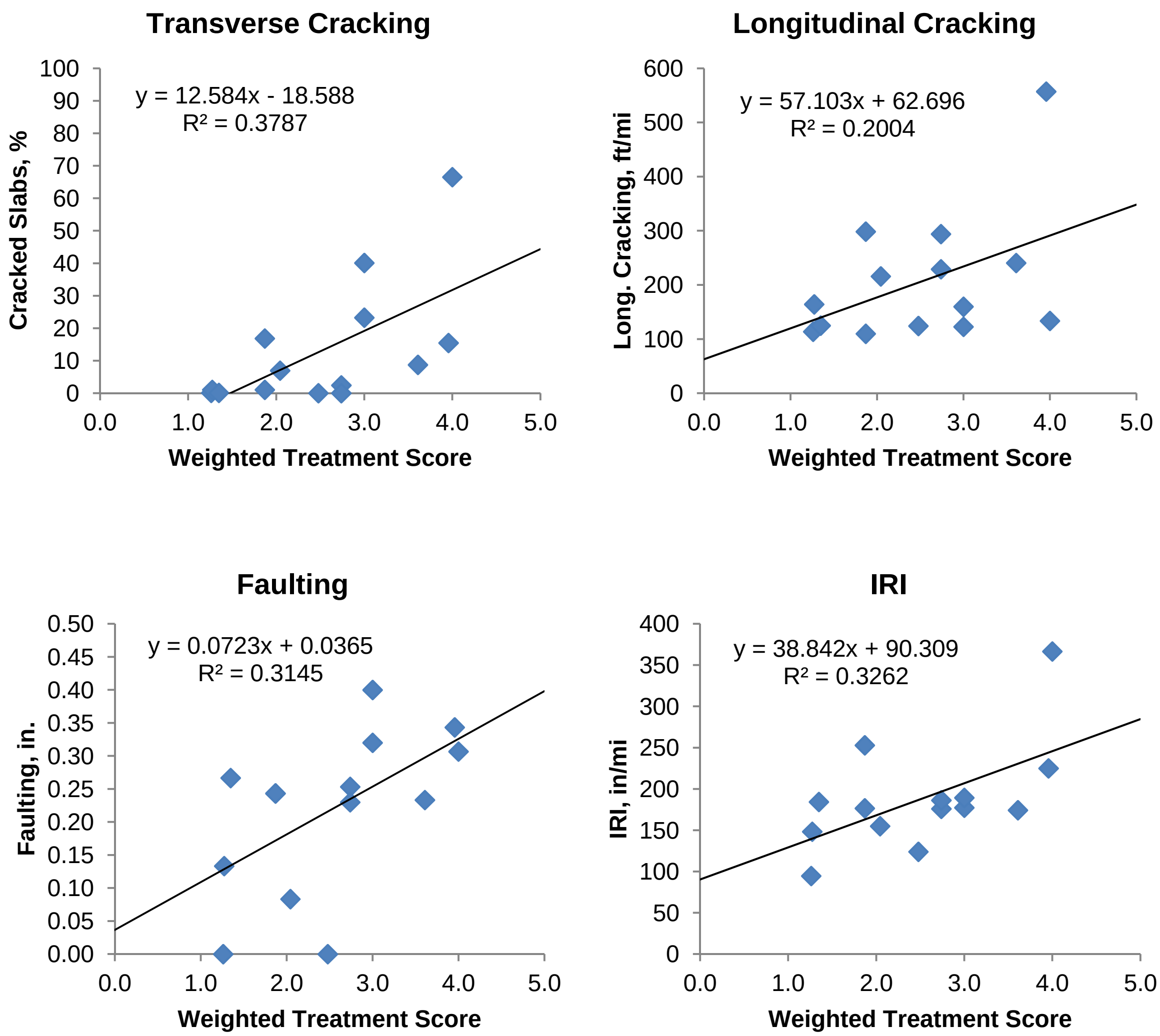


Figure 79

Relationship between pavement distresses and weighted treatment score (rigid pavements)

Design criteria are usually dependent on the functional classification of highways. Therefore, pavement distresses from PMS and the survey results were further analyzed for interstate, arterial and collector roads separately. Figure 80 shows scatterplots for interstate flexible pavements. Similar graphs for arterial roads, collector roads and rigid pavements are also available but not included in this report for concision. It can be found from Figure 80 that, as discussed before, (1) generally speaking, distress data in PMS have a positive relationship with the consensus survey, but (2) abnormal trends may occur due to the limited number of survey samples.

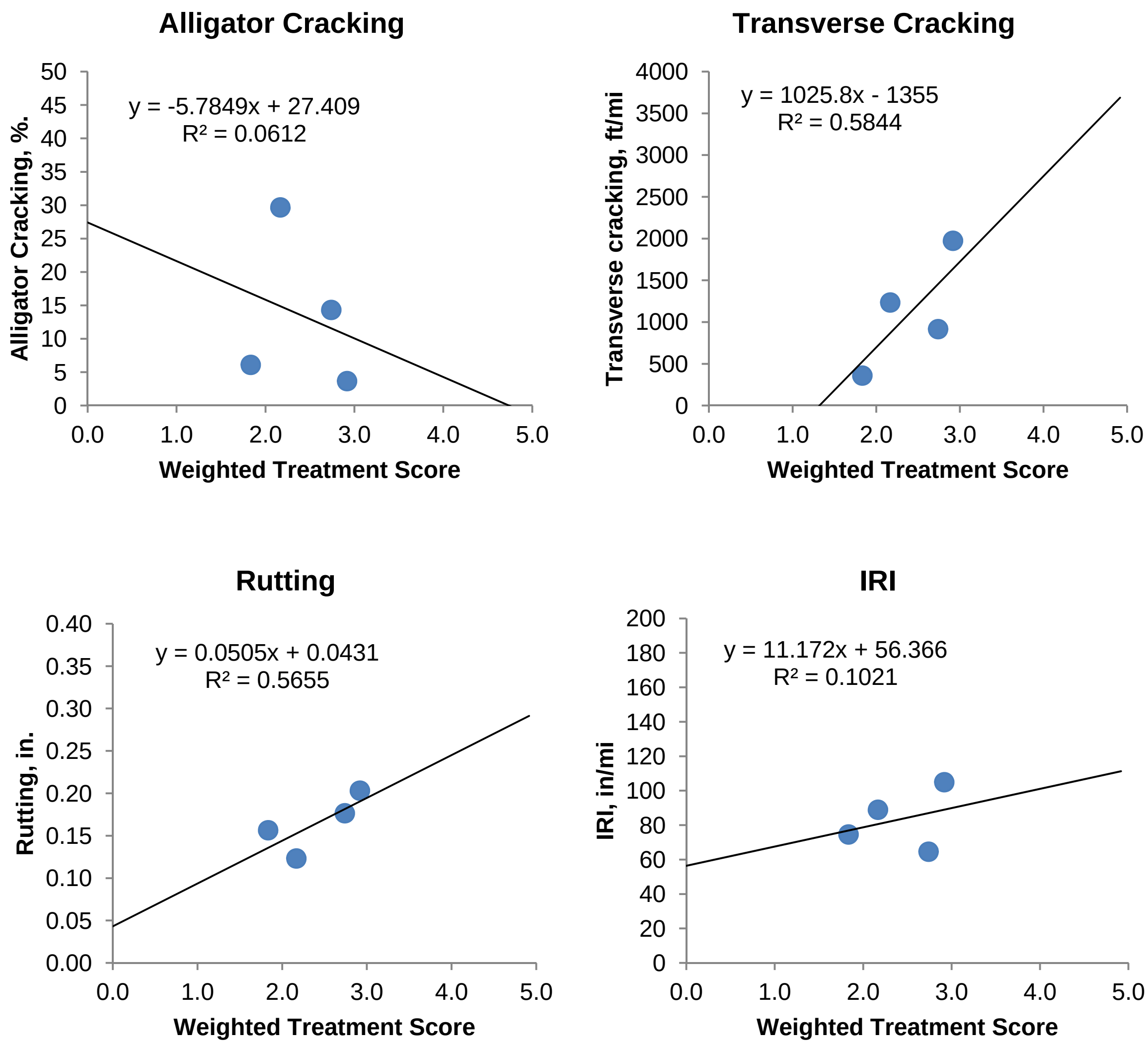


Figure 80

Estimation of design criteria for Interstate flexible pavements

Assuming a linear relationship between distress and the weighted treatment score, regression equations were developed, as shown in Figure 80. Since reconstruction was assigned as 5.0 in the weighted system, design criteria could be determined by extrapolating regression equations to a weighted treatment score of 5.0. For example, according to Figure 80, the rutting criterion for interstate flexible pavement can be calculated by

$$y = 0.0505 * 5.0 + 0.0431 = 0.30 \tag{35}$$

Although this value is smaller than the nationally recommended criterion 0.40 in., it is still a reasonable value. This process was applied to other functional classifications. The final result from regression analysis is listed in Table 48. For comparison purposes, the nationally recommended criteria are also listed as a reference. The following observations are found from Table 48.

In general, the criteria from the statistical analysis are larger than national recommendations. For example, the *Manual of Practice* suggests 10% cracked slab as the criterion for interstate rigid pavement, but the statistical analysis suggests 26% *[4]*. Furthermore, the difference is even larger for collector roads. It should be pointed out that these values are not the final recommended criteria to be used in pavement design but values determined from statistical analysis. The large values and inconsistent trends may be due to the limitation of the consensus survey (judge from pavement images) and the small sample size (e.g., only four data points to determine the criteria for interstate flexible pavements). Nevertheless, statistical analysis provides a starting point for further investigations.

Table 48
Design criteria from statistical analysis of the consensus survey

Pvmt Type	Distress	Interstate		Arterial		Collector	
		National	Louisiana	National	Louisiana	National	Louisiana
AC	Alligator cracking, %	10	NT	20	64	35	55
	Rutting, in.	0.4	0.30	0.5	0.47	0.65	NT
	Transverse cracking, ft/mi	500	3774	700	1363	700	1716
	IRI, in./mi.	160	112	200	228	200	328
PCC	Faulting, in.	0.15	0.45	0.2	0.56	0.25	0.37
	Transverse cracking, %	10	26	15	20	20	36
	IRI, in./mi.	160	205	200	244	200	333
Note: NT= negative trend.							

Discussion

The DOTD pavement management section is using a suite of pavement distress triggers to recommend different pavement rehabilitation actions. These triggers can be used as a reference in determining the criteria for pavement design. For example, as shown in Figure 81, a medium overlay for flexible pavement is triggered when the alligator cracking index is lower than 90, 85, and 75 for interstate, arterial, and collector pavements respectively. If these triggers are adopted, the corresponding design criteria for the total fatigue cracking in the ME design will be 15%, 20% and 30% for interstate, arterial, and collector flexible pavements.

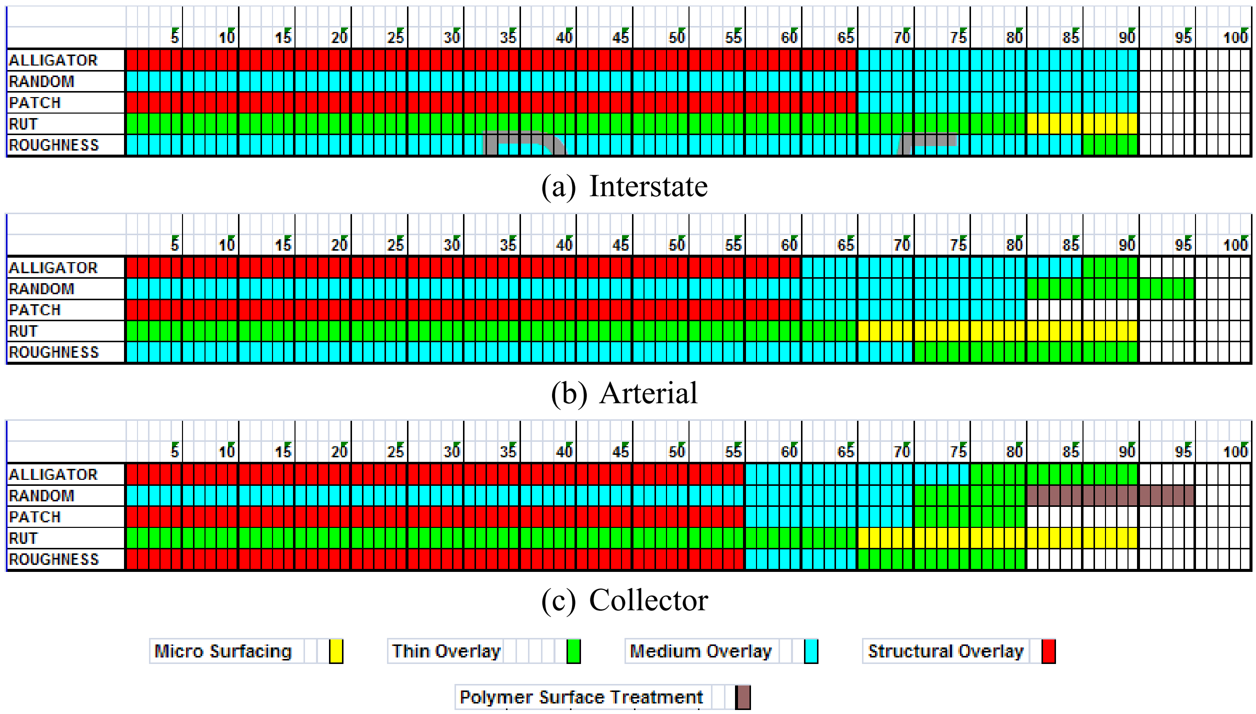


Figure 81

Distress triggers for flexible pavements currently used by DOTD

Table 49 summarizes the design criteria converted from DOTD pavement distress triggers. It should be noted that criteria for many distress types are not available from PMS triggers. For example, as shown in Figure 81, structural overlay is only initiated based on alligator cracking and patch for interstate and arterial roads; alligator cracking, patch and roughness for collector roads. In other words, rutting will never trigger a structural overlay. Similarly, faulting is not used as a trigger for rigid pavement maintenance in the current PMS.

Listed in Table 49 are also design criteria used for mechanistic empirical pavement design by the state of Utah and Indiana *[72], [73]*.

Table 49

Design criteria from PMS and other states

Pvmt Type	Distress	Interstate			Arterial			Collector		
		LA PMS	UT	IN	LA PMS	UT	IN	LA PMS	UT	IN
AC	Alligator cracking, %	15	10	10	20	20	25	30	45	35
	Rutting, in.	n/a	0.4	0.4	n/a	0.5	0.4	n/a	0.75	0.4
	Transverse cracking, ft/mi	n/a	905	50	n/a	1267	60	n/a	1267	60
	IRI, in./mi.	n/a	169	160	n/a	169	200	275	223	200
PCC	Faulting, in.	n/a	0.12	0.15	n/a	0.2	0.22	n/a	0.25	0.25
	Transverse cracking, %	6	10	10	21	15	10	53	20	10
	IRI, in./mi.	200	169	160	n/a	169	200	n/a	223	200
Note: LA= Louisiana; UT=Utah; IN = Indiana										

Proposed Design Criteria

Taking all aforementioned factors in consideration, the design criteria proposed for Louisiana ME pavement design are listed in Table 50. This criteria system (reliability and failure limit) was applied to all design examples in this study.

Table 50

Recommended design criteria of ME pavement design for Louisiana

Pavement Type	Distress	Interstate	Primary	Secondary
	Reliability Level, %	95	90	80
AC	Alligator cracking, %	15	25	35
	Total rutting, in.	0.40	0.50	0.65
	Transverse cracking, ft/mi	500	700	700
	IRI, in./mi.	160	200	200
PCC	Joint faulting, in.	0.15	0.20	0.25
	Cracked slab, %	10	15	20
	IRI, in./mi.	160	200	200

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