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FINAL REPORT ~ FHWA-OK-16-04

DEVELOPMENT AND IMPLEMENTATION OF AN MEPDG FOR RIGID PAVEMENTS – PHASE 3

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DEVELOPMENT AND IMPLEMENTATION OF AN MEPDG FOR RIGID PAVEMENTS – PHASE 3

FINAL REPORT ~ FHWA-OK-16-04
ODOT SP&R ITEM NUMBER 2208

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16. ABSTRACT

The objective of the Phase III of this project is to validate and calibrate the MEPDG for the concrete pavement design in Oklahoma. Both jointed plain concrete pavement (JPCP) and continuously reinforced concrete pavement (JPCP) were considered. Seven LTPP segments and 50 PMS pavement segments, including 30 JPCPs and 20 CRCPs, were selected and used in the local validation/calibration analyses. The suitability of the national default design models in the MEPDG were assessed by comparing the measured field pavement performance recorded in the PMS and the predicted pavement performance by the MEPDG. It was found that the national default MEPDG under-predicted the transverse joint faulting for doweled JPCPs in Oklahoma. The faulting model has been calibrated based on the Oklahoma PMS data. The national default transverse cracking model seems to be adequate, but more data are needed to confirm this observation. There is also not enough information to evaluate the suitability of the CRCP punchout model due to the limited number of CRCPs in Oklahoma. Both the transverse cracking model and the punchout model need to be revisited when more pavement distress data become available. Finally, recommendations were made on using the MEPDG to design concrete pavements in Oklahoma.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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1 Introduction

1.1 Background

The Mechanistic-Empirical Pavement Design Guide (MEPDG) is the new pavement design guide released by the American Association of State Highway and Transportation Officials (AASHTO). Compared to the previous 1993 AASHTO Pavement Design Guide, which is purely empirical, pavement design using the MEPDG represents a major change in nearly all aspects. As of the date of this report, the latest version of the MEPDG design program is Pavement ME v2.2. which is released in August 2015.

The design models for flexible and rigid pavements in the MEPDG were calibrated based on the long-term pavement performance (LTPP) database which represents the nationwide-averaged conditions. Before the local implementation, AASHTO suggests each state highway agency to validate and, if necessary, calibrate the MEPDG design models based on their local conditions. In recent years, many other state highway agencies have started to locally validate and calibrate the MEPDG for both flexible and rigid pavements. That the localized design models are expected to produce more reliable designs than the default design models in the MEPDG.

Oklahoma Department of Transportation (ODOT) is currently under the process of accepting the MEPDG to replace the 1993 AASHTO pavement design guide. In 2008, ODOT and the Oklahoma Transportation Center (OkTC) jointly sponsored the research project SPR 2208 "Development and Implementation of a Mechanistic and Empirical Pavement Design Guide for Rigid Pavements" in order to help the ODOT pavement design division in the transition. In the first two phases of the project, a comprehensive research work has been carried out which involved laboratory tests, road section instrumentation on I-44, and sensitivity analysis. The research revealed valuable information regarding local climate data and concrete material properties of Oklahoma concrete pavements. that are required inputs. Meanwhile, some initial work was conducted to validate the MEPDG design model with the performance of the continuously reinforced concrete pavement (CRCP) in Oklahoma.

For the implementation of the MEPDG, ODOT extended the SPR 2208 project to a Phase III which focuses on the local validation and calibration of the MEPDG models for Oklahoma concrete pavements. The Phase III project started on October 1, 2014 and ended on Oct 31, 2016. The purpose of this report is to summarize the research activities and findings of the Phase III of the project.

1.2 Objective and Scope

The primary objective of the project is to validate and calibrate the MEPDG for the design of typical Oklahoma rigid pavements. The secondary objectives of the proposed research are (1) to continue to monitor the field performance of the instrumented road section on I-44 and (2) to investigate the slab/base friction properties of typical Oklahoma rigid pavement structures.

This project will be limited to the design of new concrete pavements, that is, the jointed plain concrete pavement (JPCP) and the continuously reinforced concrete pavement (CRCP). Concrete overlay design, such as concrete pavement overlay, is out of the scope of the current project.

Since ODOT is not intended to design non-doweled JPCP in the future, the current research focuses on the doweled joint concrete pavement (DJCP) as the only type of JPCP. However, the term JPCP is used in this report instead of DJCP to be consistent with the language of the MEPDG.

1.3 Methodology and Tasks

The methodology of the project followed in general the MEPDG local calibration guideline [1] by the AASHTO. The research was carried out in six tasks. A brief summary about the work completed in each task is presented below.

- **Task 1**: Collect information and select road segments for the local calibration
  The research team conducted a literature review about other states' experience on the local validation/calibration of the MEPDG for concrete pavements. Findings from the literature review is presented in Chapter 2 of this report. The research team also reviewed all the available data sources in Oklahoma regarding concrete pavement structure, material, traffic, and climate. Finally, 30 JPCP and 20 CRCP segments were selected for the local validation/calibration of the MEPDG design models in Oklahoma. Detailed information about the selected pavement segments are listed in Chapter 3 of this report.
- **Task 2**: Decide the hierarchical input strategy for each MEPDG input
  The objective of this task is to determine the hierarchical input strategy for the local calibration. The decisions were made based on the availability of information and a sensitivity analysis performed in the earlier phase of the project. The hierarchical levels for design input parameters are listed in Chapter 4 of this report. Some slab/base friction tests were performed and the result was presented in Chapter 4 as well.
- **Task 3**: Continue to monitor the I-44 road section in Tulsa
  Field curling of an instrumented CRCP section on I-44 in Tulsa were periodically recorded and analyzed. The findings from the field monitoring are presented in Chapter 5.
- **Task 4**: Perform local calibration of the MEPDG based on the collected information
  The selected JPCP and CRCP segments were analyzed with the Pavement-ME v2.2 program along with the associated material and traffic data collected. The research team compared the predicted pavement distress by the design program and measured pavement distress from the Oklahoma pavement management system (PMS). Local

calibration was performed on JPCP faulting model and the CRCP transverse cracking model. The comparison of the predicted and measured pavement distresses and the resulted local calibration factors are presented in Chapter 6.

- **Task 5**: Perform a cost benefit analysis of using the MEPDG in rigid pavement design.
  Four typical pavement structures were used to compare M-E (before and after the local calibration). The results of the cost benefit analysis are presented in Chapter 7.
- **Task 6**: Develop design examples and material database files.
  Two design examples (one JPCP and one CRCP) were developed to demonstrate procedure of running a M-E pavement design using the Pavement-ME program the JPCP and a CRCP. The design examples are presented in Appendix A.

2 Literature Review

2.1 Introduction

A comprehensive literature review about concrete pavement design and the MEPDG has been conducted in the previous two phases of this project. Therefore, in Phase III, the literature review focused on other states' experiences on local validation/calibration of the MEPDG. This chapter summarizes the findings from the literature review.

2.2 MEPDG Local Calibration for the Concrete Pavements

Many state highway agencies have sponsored research studies to locally validate/calibrate the MEPDG for rigid pavements [2]. Mellela et al. conducted a study for Ohio DOT to validate the MEPDG for JPCP using the local LTPP data [3]. It was found that the MEPDG prediction generally agreed well with the field measured transverse cracking, faulting, and IRI. Won evaluated the punchout model using data collected from 27 roadway sections in Texas [4]. He found that the MEPDG significantly over-predicted the amount of punchout observed in the field. Missouri DOT conducted local calibration of the JPCP pavement design module in the MEPDG using a combined database of LTPP and the pavement management system (PMS) [1]. 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. The results showed that transverse cracking and the IRI models in the MEPDG was adequate for JPCP. However, the MEPDG over-predicted the faulting for JPCPs that are not doweled or have long joint spacings (> 20 ft.). In general, previous studies found that field distresses of rigid pavement sections are low. A longer service period is needed for rigid pavements to show sufficient amount of distresses in order to fully validate the MEPDG model. A recent local calibration research was conducted in Colorado by the Applied Research Association, Inc. [5]. It was shown that locally calibrated MEPDG produces similar or slightly thinner design thicknesses to that determined by the 1993 design guide.

According to the AASHTO local calibration guide, the first step for the local calibration is to select hierarchical input level for each input parameter. This is a policy making process. If a different input strategy is to be used in the pavement design, the standard error of the MEPDG design models changes, and local calibration should be conducted again [1]. MEPDG accepts three hierarchical input levels: project level (Level-1), state default (Level-2), and national default (Level-3). Project level inputs are determined for each project at the design stage and is the most accurate, whereas national default inputs are typically the default input values in the design software and are the least accurate. Each design agency needs to determine the hierarchical input level to use based on the significance of the input parameter.

In Phases I and II of the SPR 2208 project, regional default inputs for typical Oklahoma concrete pavement materials have been determined from the lab and field tests [6].

3 Selection of Pavement Segments

3.1 Introduction

This chapter documents the selection process of Oklahoma JPCP and CRCP segments for the local validation/calibration of the MEPDG.

3.2 LTPP Segments Overview

The default design models in the MEPDG were calibrated against the pavement performance data in long-term pavement performance (LTPP) database. The LTPP program is a large ongoing project initiated in early 1990s. The program monitors the pavement performance of around 2,500 road segments in the US and Canada. Each LTPP segment is 500 ft long in one direction of the road. The pavement performance data are stored in the LTPP database along with other information such as traffic, pavement structure, and material. The database is available to public through the LTPP InfoPave website (infopave.fhwa.dot.gov/). The LTPP database is the most accurate data source for the MEPDG local calibration, because the pavement condition survey in all LTPP segments follows the same protocol and is consistent with the distresses definitions in the MEPDG.

Oklahoma has 67 LTPP segments including 7 JPCP segments and 4 CRCP segments (one of which is an overlay pavement which is not interested in this project). These LTPP segments have the most complete and accurate information for the MEPDG analysis. However, all the 7 JPCP segments in the LTPP database in Oklahoma are non-doweled JPCP constructed before 1990, which does not reflect the current pavement design practice of ODOT.

Table 3.1 LTPP segments (JPCP and CRCP only) in Oklahoma

Section	Route, Direction	County	Pavement	Date of Construction	AADTT	Status
40-3018	I-240, WB	Oklahoma	JPCP (non-doweled)	06/01/1976	283	Inactive
40-4157	US-69, NB	Mayes	JPCP (non-doweled)	05/01/1986	1,040	Active
40-4160	SH-3, WB	Pontotoc	JPCP (non-doweled)	06/01/1979	178*	Active
40-4162	US-62, EB	Comanche	JPCP (non-doweled)	06/01/1985	196**	Inactive
40-A410	SH-3, WB	Pontotoc	JPCP (non-doweled)	06/01/1979	185*	Inactive
40-A420	SH-3, WB	Pontotoc	JPCP (non-doweled)	06/01/1979	185*	Inactive
40-A430	SH-3, WB	Pontotoc	JPCP (non-doweled)	06/01/1979	185*	Inactive
40-4155	US-75, NB	Washington	CRCP over CRCP	06/01/1970	256	Active
40-4158	US-75, SB	Washington	CRCP	06/01/1989	602	Active
40-4166	US-69, NB	Pittsburg	CRCP	05/01/1990	1383	Active
40-5021	SH-33, WB	Mayes	CRCP	10/01/1987	493	Active

*1983 traffic data

**1990 traffic data

3.3 PMS Segments Overview

Due to the limited data available in the Oklahoma LTPP, the research team decided to select more pavement segments from the ODOT PMS database. The ODOT PMS division keeps the pavement type and performance information of the entire highway network in Oklahoma. Each PMS segment is associated with one ODOT roadway construction project which should have a designed pavement structure and a construction date. The pavement condition survey is conducted by external contractors every two years. A large volume of data is available. However, PMS data have several drawbacks. First, pavement performance data were measured by automatic survey vehicles based on the Oklahoma pavement survey protocol. This protocol is not always consistent with the protocol used by the LTPP. Second, the ODOT PMS does not store traffic, pavement structure, and material information associated to each road segment. Supplemental information has to be collected from other sources. The quality of the data depends on the accuracy of the data source and is usually less accurate than the LTPP.

A suitable PMS segment for the local calibration/validation of the MEPDG should have a complete design, construction, and traffic record, and it should have no major rehabilitation in the past that affects its normal accumulation of pavement distress. In this project, PMS segments constructed before 1990 were first excluded based on an earlier discussion with the ODOT pavement design engineer. The reason for this is twofold. First, these pavement segments may not represent the typical pavement structure and material used today. Second, these segments may have been overlaid even though the record shows they have not. Pavement segments with less than 10 years of service life were also excluded, because these segments often do not show enough distress to be compared with the MEPDG models. The above procedure narrowed the pool of candidate segments down to a total of 124 JPCP segments and 104 CRCP segments. The overall condition of these pavement segments are described below.

Based on the 2014-2015 cycle PMS data, 88 out of the 124 JPCP segments have developed non-zero average transverse joint faulting. The statistical distribution of the transverse joint faulting of the JPCP segments in Oklahoma is shown in Figure 3.1. Only two segments (one on I-44 and the other on US-412) showed faulting above the design limit of 0.12 inches. Most of the JPCP segments showed less than 0.06 inches of average faulting. This indicates an overall good condition of the JPCP joints in Oklahoma probably due to the use of dowel bars.

Thirty-four (34) out of the 124 JPCP segments showed at least one transverse crack over the entire length of the segment. The statistical distribution of the transverse cracking of the JPCP segments are shown in Figure 3.2. The worst transverse cracking was observed from a segment on I-40 that has about 8% of concrete slabs cracked. That is about half of the MEPDG design limit (15% cracked slabs). Most of the JPCP segments have less than 3% of their slabs cracked. This also indicates an overall good quality of the JPCP slabs in Oklahoma.

The statistical distribution of the average international roughness index (IRI) of the 124 JPCP segments are shown in Figure 3.3. The IRI of JPCP segments ranged from 51 to 237 in./mi. Fifteen (15) JPCP segments showed IRI over 150 in./mi, of which 9 segments exceeded the roughness design limit of 173 in./mi.

Of the 104 CRCP segments, 59 segments showed at least one punchout over the entire length of the segment. The statistical distribution of the average punchouts per mile of the CRCP segments are shown in Figure 3.4. It is shown that only three segments (two on I-35 and one on I-40) have developed excessive amount of punchouts above the MEPDG design limit (10 punchouts per mile). There are also a few segments where the amount of punchouts developed are approaching the design limit.

The statistical distribution of the average IRI of the 104 CRCP segments are shown in Figure 3.5. The IRI of CRCP segments ranged from 56 to 165 in./mi. Three CRCP segments showed IRI of over 150 in./mi. None of the CRCP segments exceeded the roughness design limit of 173 in./mi.

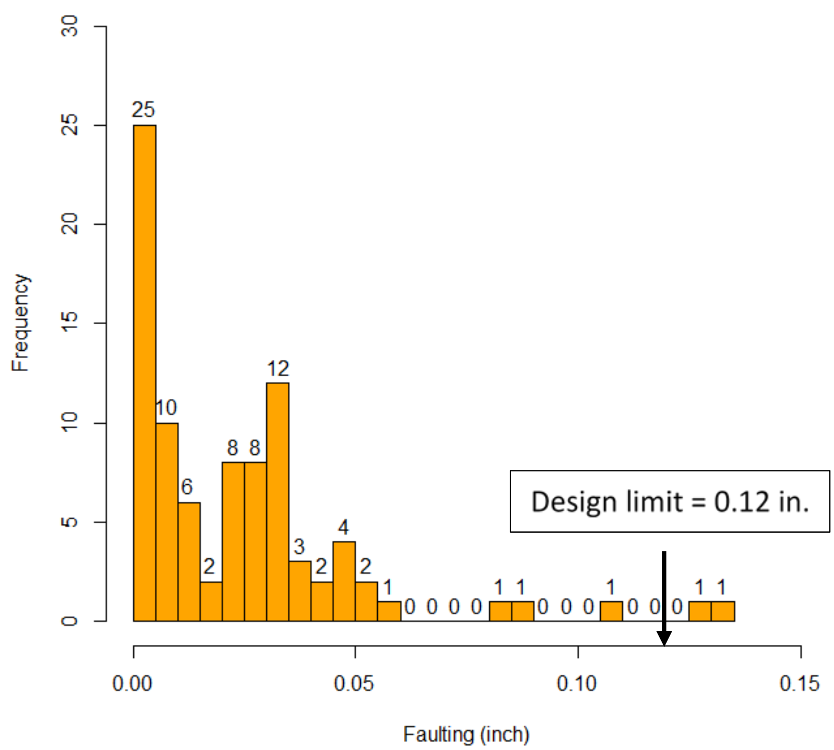


Figure 3.1 Transverse joint faulting of the 124 JPCP segments in Oklahoma

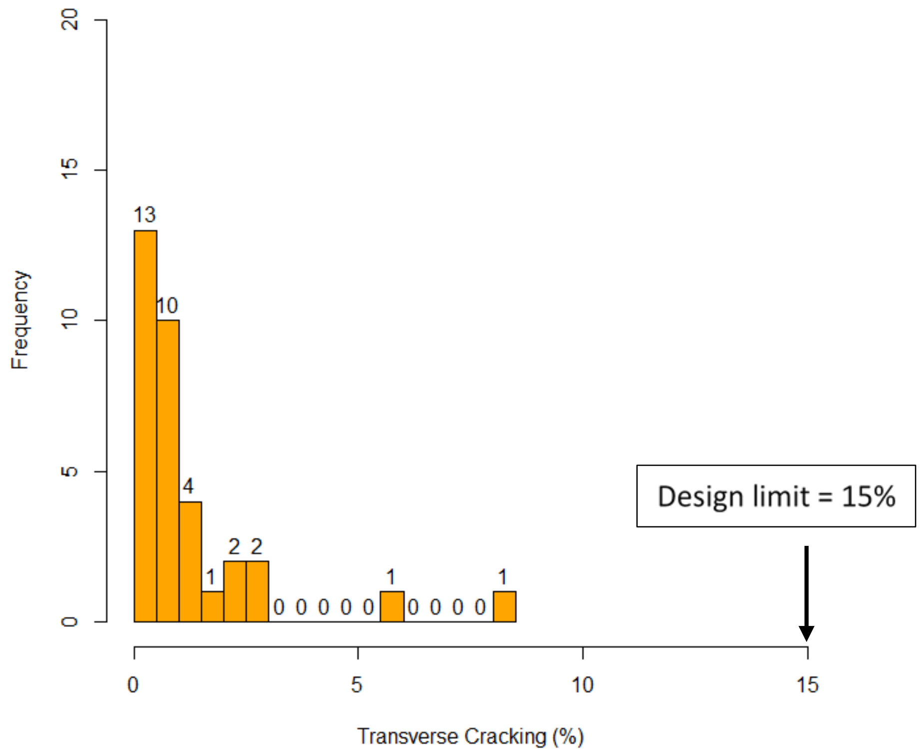


Figure 3.2 Transverse cracking of the 124 JPCP segments in Oklahoma

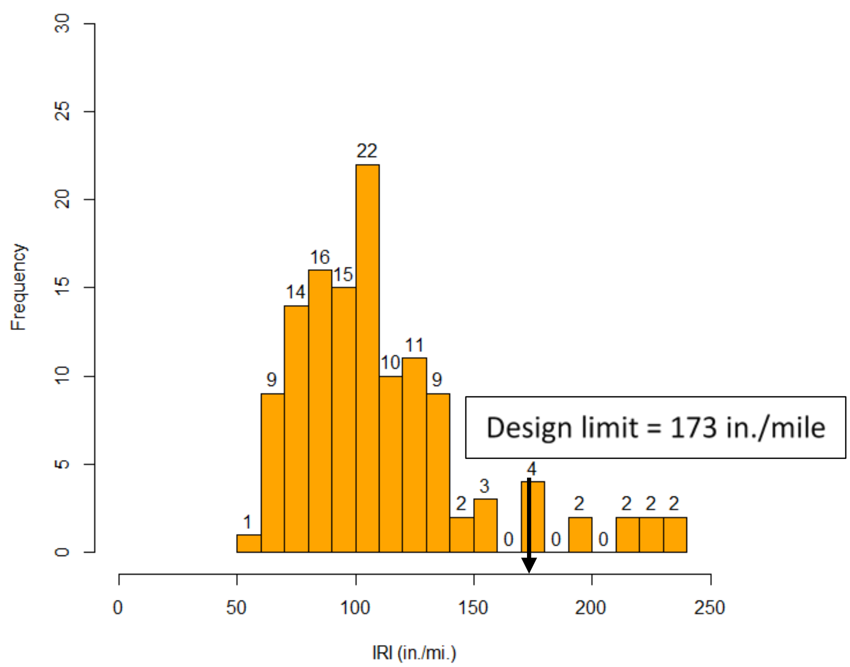


Figure 3.3 IRI of the 124 JPCP segments in Oklahoma

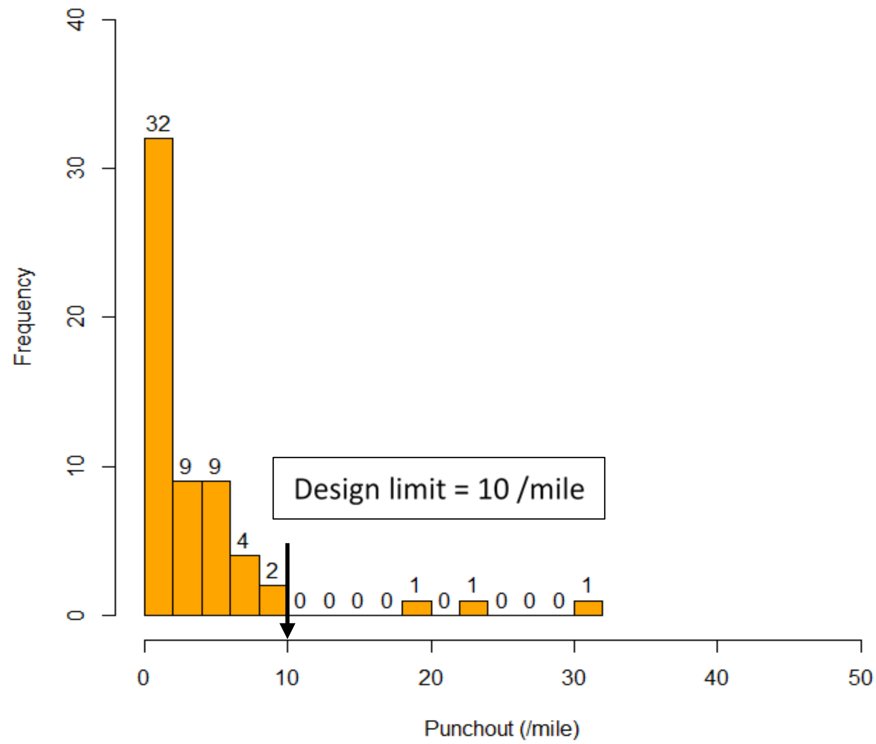


Figure 3.4 Transverse cracking of the 104 CRCP segments in Oklahoma

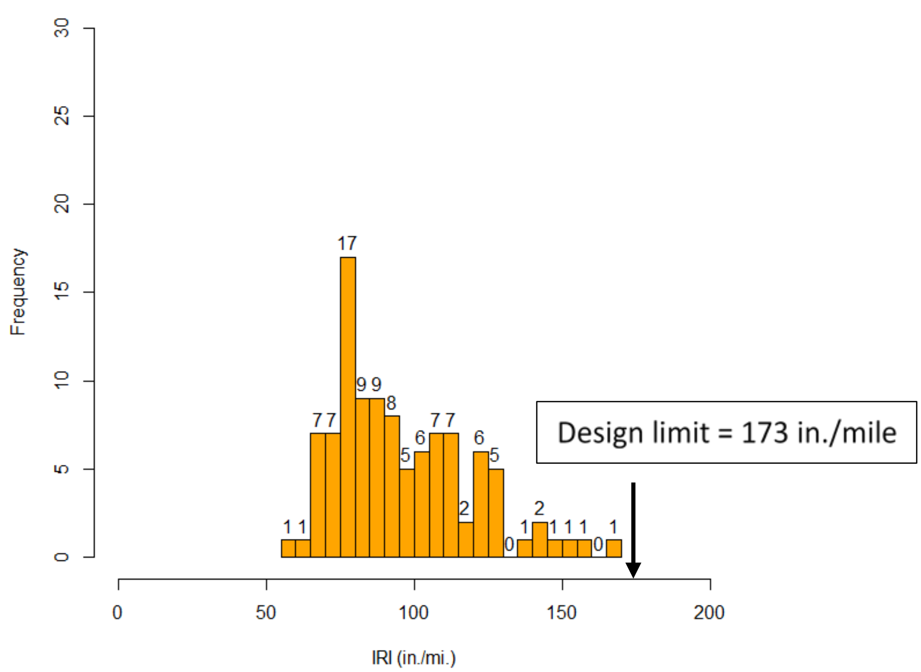


Figure 3.5 IRI of the 124 JPCP segments in Oklahoma

3.4 Selection of JPCP and CRCP Segments

For the local validation/calibration of the MEPDG, the selected pavement segments have to develop enough pavement distresses, ideally, comparable to the design limit. However, as discussed previously, most of the JPCP and CRCP segments in the Oklahoma are in relatively good conditions. Therefore, the research team sorted JPCP and CRCP segments based on each type of pavement distress and finally selected 30 JPCP and 20 CRCP segments which developed the most severe pavement distresses. Sometimes there are multiple segments in the same control section and with the same pavement structure, same traffic condition, and very close construction dates. In that case, only one (usually the one with the most pavement distress) was selected to avoid duplicate data points. When the road segment has two directions, the average pavement distress (such as faulting) was calculated from the direction with the higher distress.

Finally, the research team checked the maintenance record of each control section to further confirm that no major maintenance or rehabilitation had been done on these road segment. The location of the selected JPCP and CRCP segments are mapped in Figure 3.6 and Figure 3.7, respectively. Detailed information about these segments are provided in Appendix A.

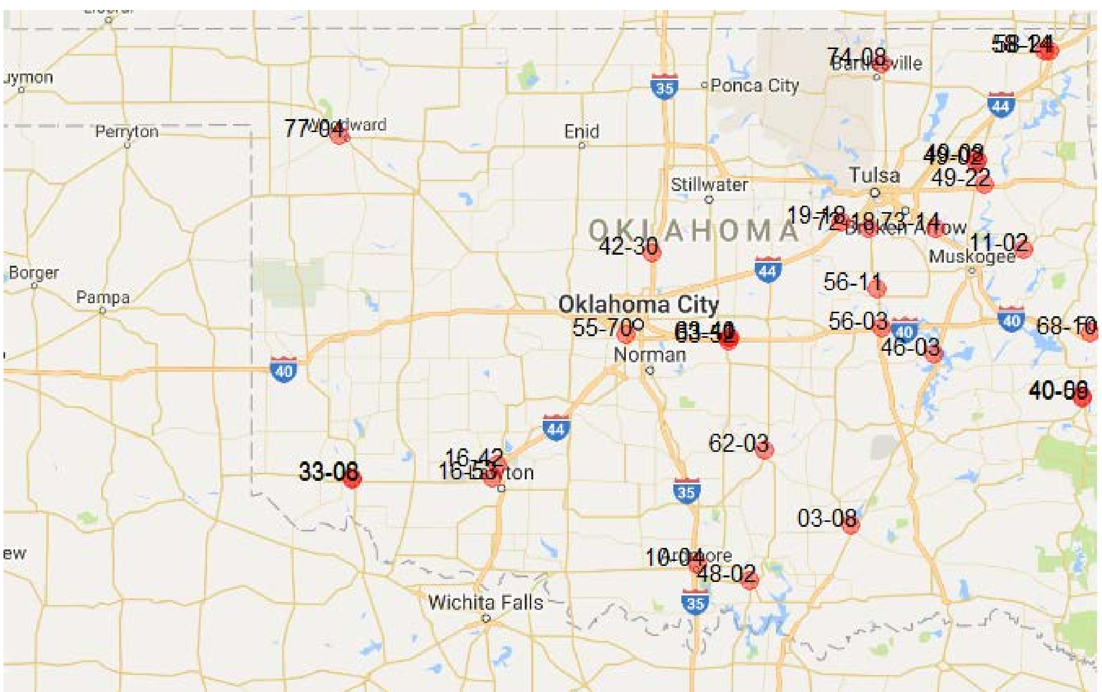


Figure 3.6 Selected JPCP segments from the PMS

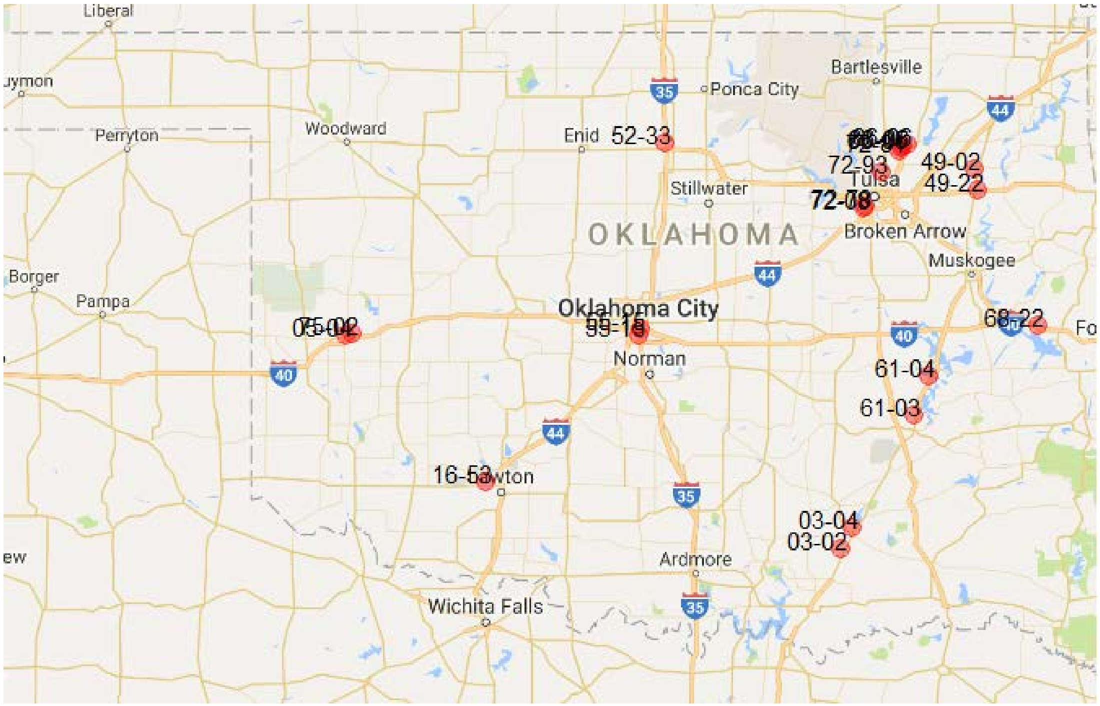


Figure 3.7 Selected CRCP road segments from the PMS

3.5 Pavement Performance Data

In this research, pavement distress data were collected from the latest PMS data collected during the 2014-2015 cycle. Pavement distresses related to the MEPDG design are the transverse cracking (in % of cracked slabs) and transverse joint faulting (in inches) for JPCP and punchouts (in count/mile) for CRCP. The international roughness index (IRI) is also a design output for both JPCP and CRCP in the MEPDG. The prediction of IRI is based on the initial IRI of the pavement and an incremental function of predicted pavement distresses (cracking, faulting, and punchout). In this research, since the initial IRI of the pavement as-constructed was not available for any PMS segments, the local validation/calibration effort focused on the transverse cracking and transverse joint faulting models of the JPCP and the punchout model of the CRCP.

4 Design Input Strategy

4.1 Introduction

This chapter introduces the input strategy to be used in the local validation/calibration analysis of the MEPDG. The concrete pavement design in the MEPDG requires three categories of input parameters: (i) traffic input parameters, (ii) climate input parameters, and (iii) pavement structure and material input parameters. The MEPDG allows hierarchical input (from Level-1 to Level-3) for many design parameters, with Level-1 being the most accurate project-level information and Level-3 being the nationwide typical value. In addition, the researchers conducted a set of slab/base friction tests to evaluate the friction characteristics between the concrete slab and three types of base materials.

4.2 Slab/Base Friction

In the previous phases of this project [6], it was found that the concrete pavement design using the MEPDG is very sensitive to the slab/base friction input parameters. For the JPCP design, MEPDG assumes the concrete slab and base course are fully bonded until a certain month, at which point the slab-base bond is lost completely and the interface becomes completely smooth. The number of months until the total loss of slab/base bond is a required input for the JPCP design. For the CRCP design, a different slab/base interface model is adopted. The coefficient of friction μ between the concrete slab and the base course is a required input. During the development of the MEPDG, the above mentioned slab/base friction parameters are back-calculated to match the field performance of the LTPP segments. The real interaction between the concrete slab and the base course has rarely been evaluated from experiments.

In this project, the number of months until the total loss of slab/base bond was set to the end of the design life as suggested by the MEPDG. Some laboratory friction tests were conducted to evaluate the coefficient of friction μ for the CRCP design. Since it is not the primary focus of this project, only two types of base course materials were tested:

- Section 1: ODOT Type A unbound aggregate base (UAB)
- Section 2: ODOT cement stabilized base (CSB) with bond breaker*

*The bond breaker fabric used in this research is the Mirafi 1160N non-woven geotextile provided by Tencate, Inc.

There is no standard procedure available for running a laboratory slab/base friction test. The test procedure adopted in this research aims to produce a slab/base interface that closely imitates the field condition. Figure 4.1 shows the friction test setup. The test procedure is summarized as follows:

i. Compact the base material in a 2 ft. x 2 ft. x 4 in. thick wood box. The ODOT aggregate base was compacted at the optimum moisture content and 95% of the maximum dry density. The cement stabilized base was compacted according to the typical mix design provided by the ODOT Material Division. The weight of water and dry materials were pre-calculated to ensure the consistency among different sections.
ii. Pour concrete slurry into a 1 ft. x 1 ft. x 4 in. tall wood frame positioned at the center of the section. Cover the concrete and the base immediately with a thick plastic membrane to preserve the moisture content. The concrete was cured for 28 days before the friction test.
iii. Position the push machine in front of the concrete slab. The piston of the push machine should point to the center of the front face of the concrete slab. The push machine used in this research was built by the technician at the OSU Civil Engineering laboratory. It is able to generate up to 300-lb push force at a controlled speed. After positioning the push machine, a 5000-lb load cell and three 1-in LVDTs are installed to measure the friction force and the displacement of the slab. In this project, the load and displacement were recorded every 0.25 seconds by a Campbell CR 6 data logger.
iv. Switch on the push machine and push the concrete slab with a horizontal displacement rate of ~0.36in/min until the measurement range of the LVDT (1 inch) is reached. The load displacement curve is then plotted with a computer.
v. Move the concrete slab to the original location and repeat the friction test by placing one or two more concrete slabs on top of the original concrete slab. This procedure allows measurement of friction force at three different normal forces (50 lb, 100 lb, and 150 lb)
vi. Calculate the coefficient of friction μ by finding the slope of the interface strength vs. normal force plot.

Figure 4.1 Slab/base friction test setup

The load-displacement curves measured from Sections 1 and 2 are shown in Figures 4.2 and 4.3, respectively. It was observed from both sections that the slip occurred at the interface between the concrete slab and the base. When a bond breaker is used (Section 2) the slip occurred between the bond breaker and the base, and top surface of the bond breaker was always firmly bonded to concrete slab

On the ODOT UAB, the interface friction strength F measured is proportional to the normal force N (N equals to the total weight of the concrete slabs) applied (See Figure 4.2). The measured coefficients of friction μ from this section is 0.84.

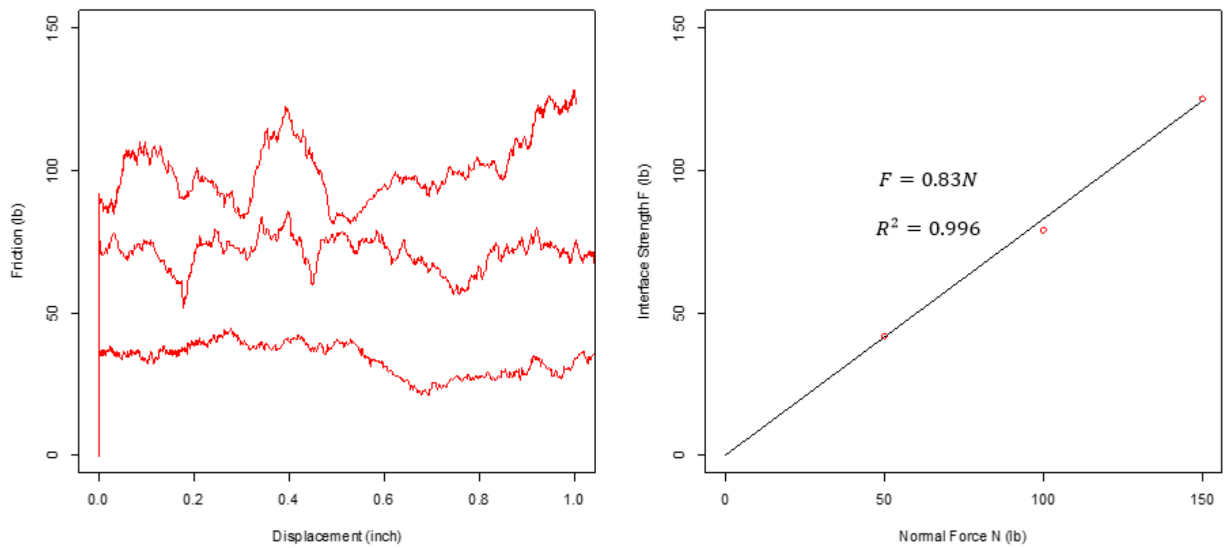


Figure 4.2 Slab/base friction test on Section 1 (ODOT UAB)

On the ODOT CSB, the interface friction strength F measured shows a linear relationship with the normal force N, but with a non-zero intercept (Figure 4.3). This result indicates that the slab/base interaction in this section has two components, the adhesion and the friction. The adhesion component is due to the bonding between the fabric and the cured cement in the base, which is measured as 57 lb/ft^2 in this study. This component has nothing to do with the normal force, thus it shown as a constant intercept in the test result. However, the current CRCP design model in the MEPDG adopted a simple friction relationship ($F = \mu N$) which does not account for the adhesion component. Assuming a 10-inch thick concrete slab is placed on the base, an equivalent coefficient of friction can be estimated as 1.31.

Table 4.3 summarizes the μ values suggested by the 1993 AASHTO Design Guide and the MEPDG for different types of base/subgrade materials. Note that none of these values were determined by matching the field pavement performance rather than from experimental results. The μ values measured from the laboratory friction tests (0.83 for aggregate base and 1.31 for cement stabilized base with a bond breaker) are lower than the suggested default value in the Pavement-ME 2.2. When using these values in the MEPDG design, the design program predicts unreasonably high punchout even at early service periods. Therefore, the researchers decided to use the suggested μ values in the design program rather than the measured values, so that to generate more reasonable results.

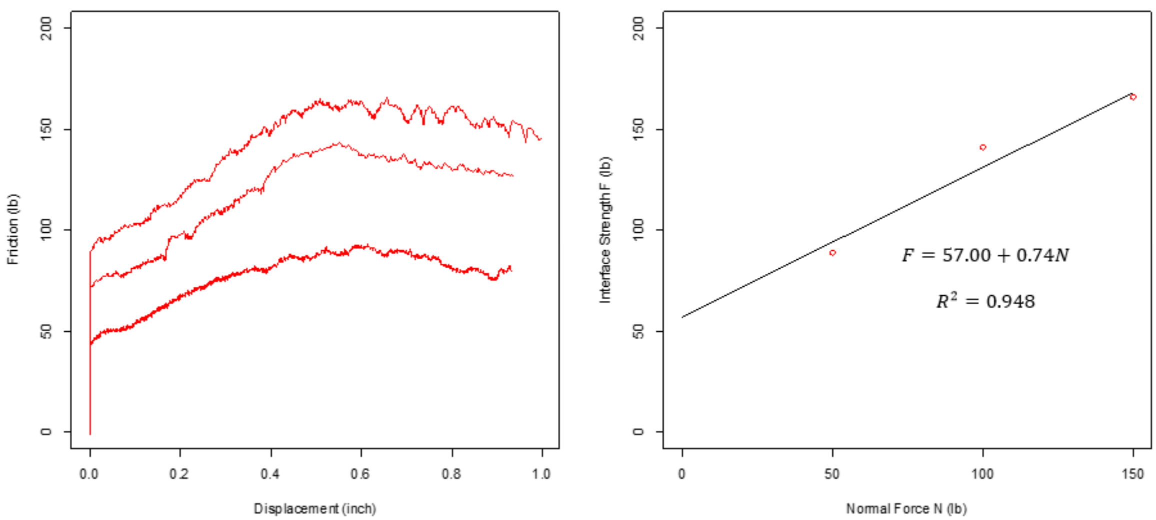


Figure 4.3 Slab/base friction test on Section 2 (ODOT CSB with a bond breaker)

Table 4.3 Slab/base coefficient of friction μ measured from this research

	1993 AASHTO Guide			NCHRP 1-39A			Pavement-ME 2.2
Base or subgrade material	**Low**	**Medium**	**High**	**Low**	**Medium**	**High**	**Default**
Fine grained soil	0.5	1.3	2.0	0.5	1.1	2.0	1.1
Sand	0.5	0.8	1.0	0.5	0.8	1.0	2.5
Aggregate	0.7	1.4	2.0	0.5	2.5	4.0	2.5
Fabric bond breaker*	0.5	0.6	1.0	--	--	--	--
Lime-stabilized clay	3.0	n/a	5.3	3.0	4.1	5.3	8.9
Cement-treated gravel	8.0	34	63	3.5	8.9	13	8.9
Asphalt treated gravel	3.7	5.8	10	2.5	7.5	15	7.5
Asphalt concrete	--	--	--	--	--	--	7.5
Lean concrete with single or double wax curing compound	3.5	--	4.5	3.5	8.5	20	--

*Pavement-ME does not explicitly consider the fabric bond breaker

4.3 Design Input Strategy

Based on the availability of information and a sensitivity analysis performed in the previous phase of the research, the research team decided an input strategy for the JPCP and CRCP MEPDG analysis (Table 4.4). Default values from the Pavement-ME program will be used for all other inputs.

Table 4.4 JPCP MEPDG input strategy for Oklahoma PMS segments

Input	Level*	From
AADTT (Average Annual Daily Truck Traffic)	1	Plan file
Truck factor	1	Plan file
Growth rate	1	Plan file
Speed	1	Plan file
Lane	1	Plan file
Weather station	1	Select the nearest one weather station
Depth of groundwater table	3	10 ft
PCC thickness	1	Plan
Dowel diameter	2	1.25 in. (#10)
Slab joint spacing	2	15ft
CTE of concrete	2	4.5 x 10^{-6} in/in/F
28d modulus of rupture	2	620 psi
Cement content	2	600 lb/yd^3
Erodibility index	2	1 for cement treated and asphalt concrete base 2 for aggregate base 3 for lime stabilized subgrade
Slab/Base Friction	2	8.9 for cement stabilized base with bond breaker 7.5 for asphalt concrete base with bond breaker 2.5 for unbound aggregate base
Base and Subbase	2	Aggregate base: Crushed gravel (Mr = 25000 psi) Asphalt concrete base: Asphalt concrete (Binder Grade = PG 64-22) Cement treated base: Cement stabilized base (E = 750000 psi)
Subgrade modulus	3	A-6 (Mr = 5500 psi)

5 Local Validation and Calibration of the MEPDG

5.1 Introduction

This chapter presents the local validation and calibration of the MEPDG model based on the LTPP and PMS data in Oklahoma. Seven LTPP segments and 50 PMS segments were analyzed. The latest version of MEPDG design program (Pavement-ME v2.2) was used to predict the pavement distress. The predicted distress was then compared with measured pavement distress in each segment based on the 2014-2015 PMS data.

5.2 LTPP segments

In this research, 4 JPCP and 3 CRCP segments in the LTPP database were analyzed. Three other JPCP segments (40-A410, 40-A420, and 40-A430) were excluded because these segments are in the same project as Segment 40-4160 in Pontotoc County. One other CRCP segment was excluded because it is an overlay pavement which is out of the scope of this research.

It should be noted that all the four LTPP segments analyzed here are non-doweled JPCPs which doesn't reflect the current design practice of the ODOT roadway division. These JPCP segments are analyzed here but not included in the faulting model calibration.

5.2.1 Section 40-3018

The pavement structure of the segment is shown in Figure 5.1. The pavement consists of an 8.9 in. Portland cement concrete (PCC) over a 3.6 in. sand asphalt base over a 6.1 in. lime stabilized subgrade underlain by an A-7-6 subgrade soil.

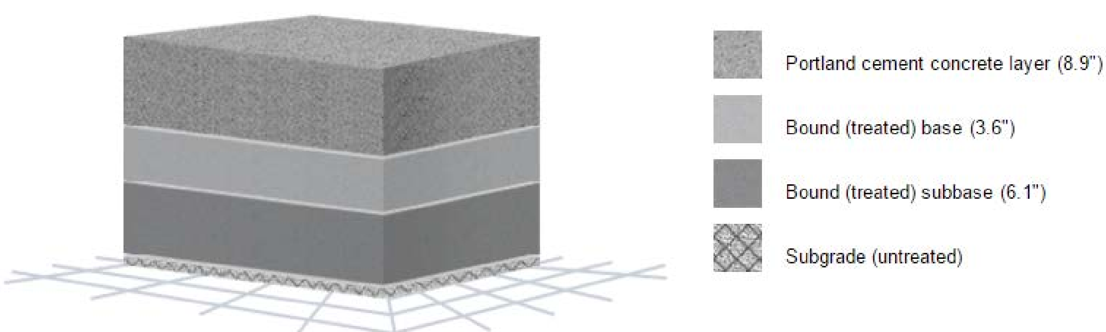


Figure 5.1. Pavement structure of LTPP Segment 40-3018

The measured and predicted pavement distresses are plotted together in Figures 5.2. It is shown that the MEPDG design program predicted significant amount of transverse cracking after 20 years of service life, whereas the almost no transverse cracking was observed in the segment in nearly 30 years. Meanwhile, the MEPDG design program under-predicted the field faulting and slightly under-predicted the IRI of the pavement.

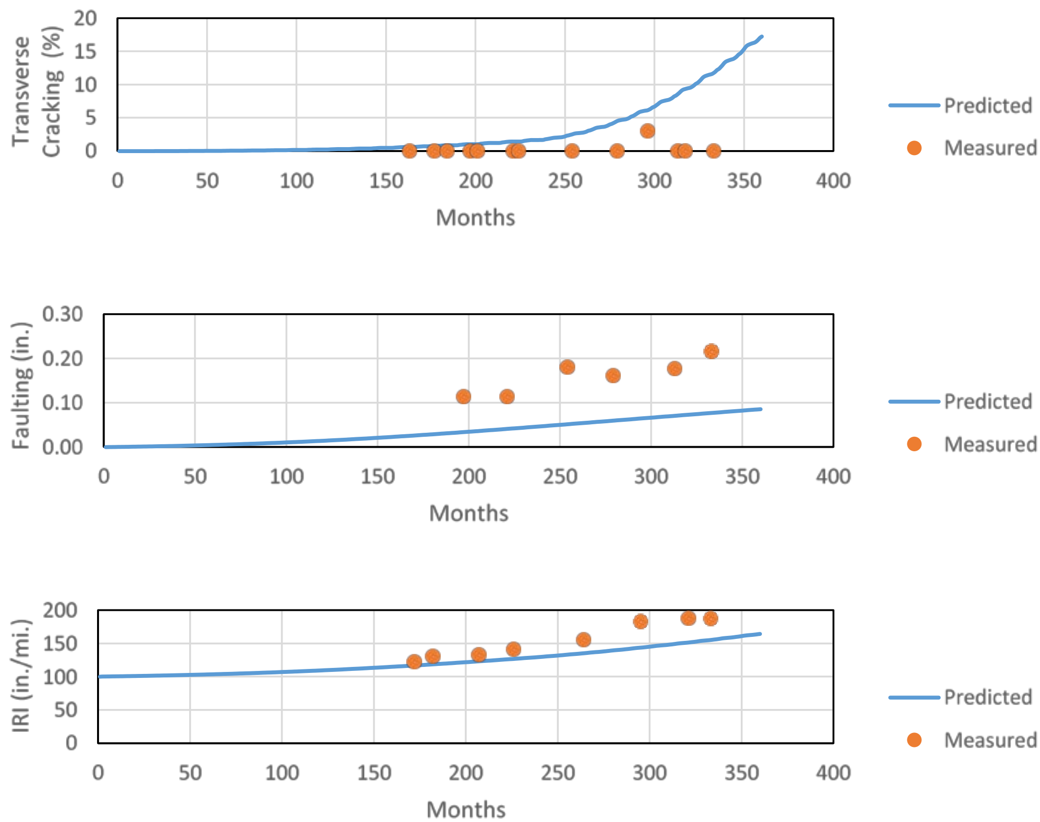


Figure 5.2 Predicted vs. measured pavement performance in LTPP Segment 40-3018

5.2.2 Section 40-4157

The pavement structure of the segment is shown in Figure 5.3. The pavement consists of 9.1 in. PCC over a 3.8 in. hot mix asphalt concrete (HMAC) base over a silty sand (A-2-4) subgrade soil.

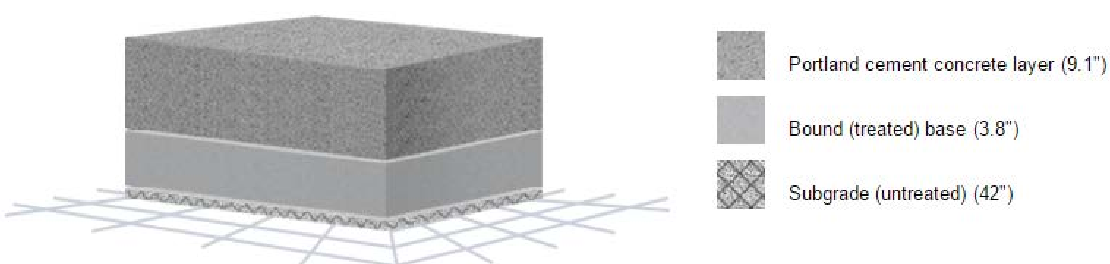


Figure 5.3 Pavement structure of LTPP Segment 40-4157

The measured and predicted pavement distresses are plotted together in Figures 5.4. For this pavement, the MEPDG design program also predicted significant amount of transverse cracking after 20 years of service life, whereas the almost no transverse cracking was observed. Meanwhile, the MEPDG design program over-predicted the field faulting and the IRI of the pavement.

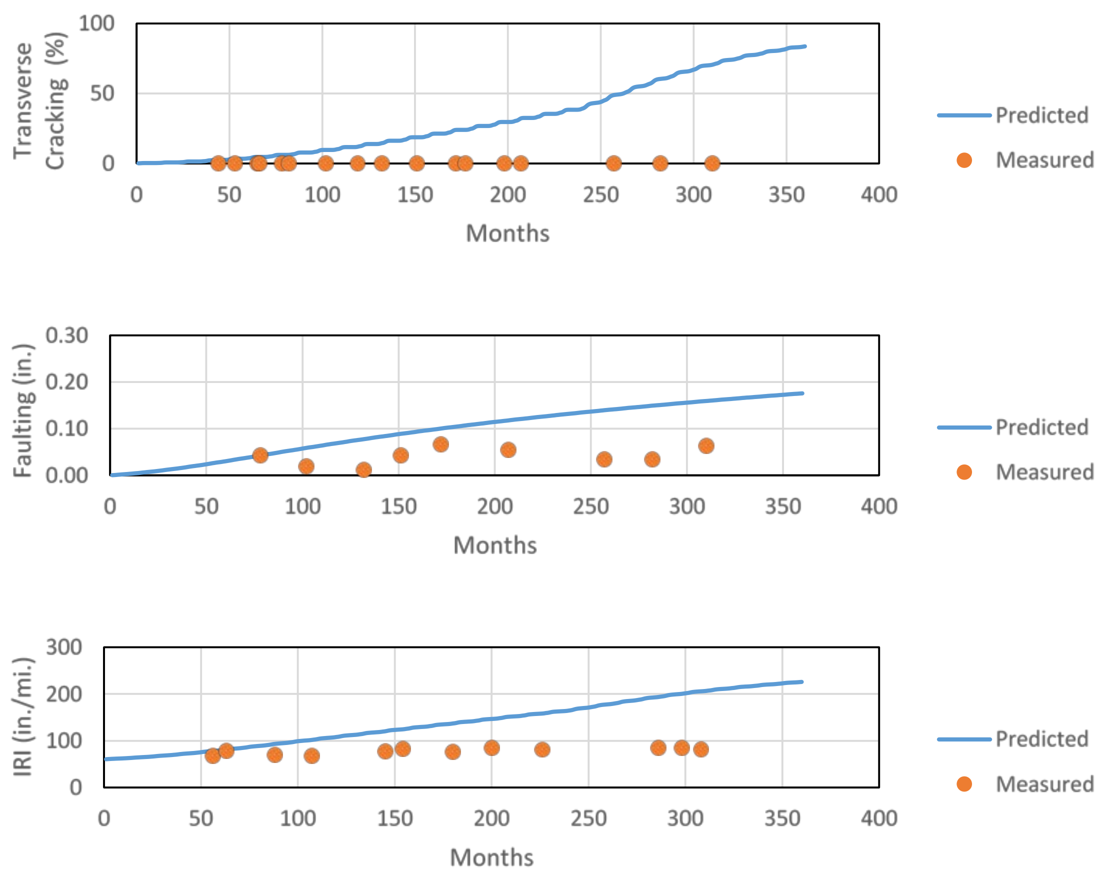


Figure 5.4 Predicted vs. measured pavement performance in LTPP Segment 40-4157

5.2.3 Section 40-4160

The pavement structure of the segment is shown in Figure 5.5. The pavement consists of a 9.2 in. PCC over a 2.2 in. sand asphalt base over a 12 in. unbounded granular subbase over an A-7-6 subgrade soil.

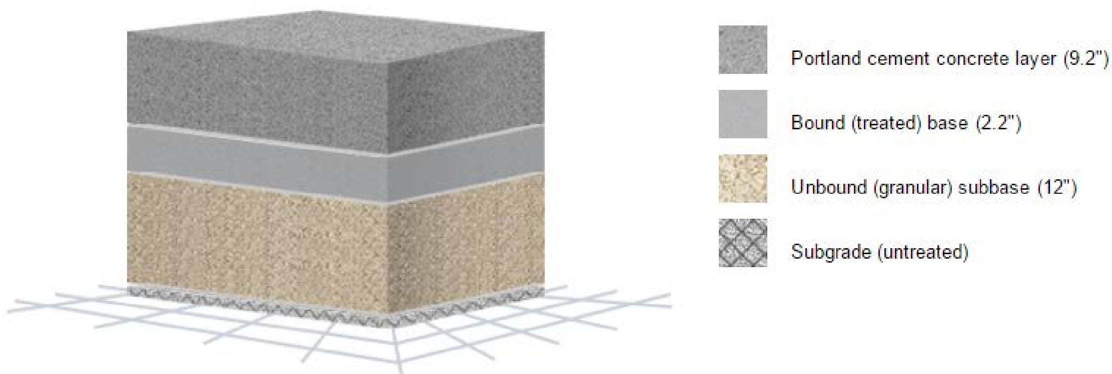


Figure 5.5 Pavement structure of LTPP Segment 40-4160

The measured and predicted pavement distresses are plotted together in Figures 5.6. For this pavement, the MEPDG design program also predicted significant amount of transverse cracking after 20 years of service life, whereas the almost no transverse cracking was observed. Meanwhile, the MEPDG design program slightly under-predicted the field faulting. The predicted IRI of the pavement seems to agree well with the field measurement.

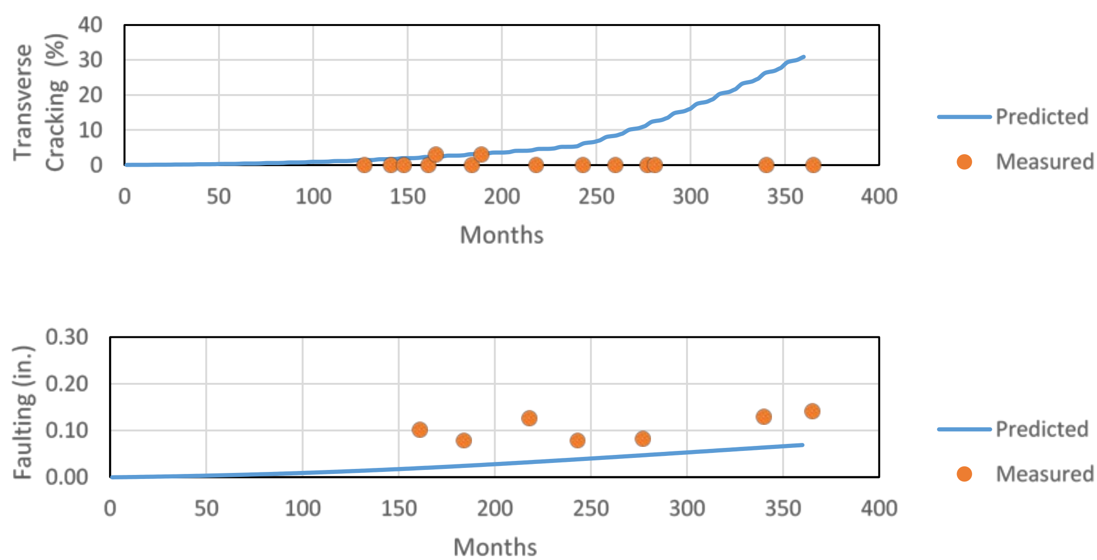


Figure 5.6 Predicted vs. measured pavement performance in LTPP Segment 40-4160

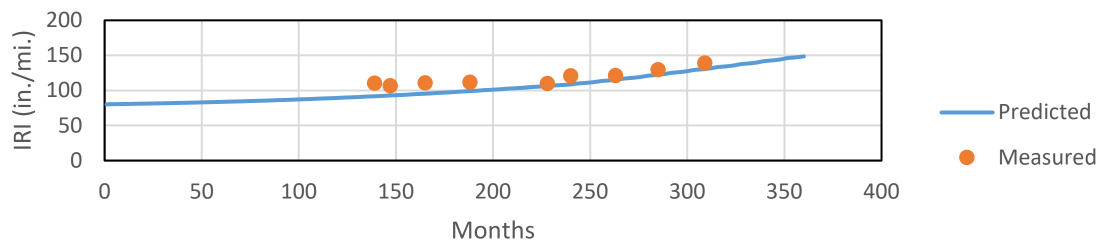


Figure 5.6 Predicted vs. measured pavement performance in LTPP Segment 40-4160 (cont.)

5.2.4 Section 40-4162

The pavement structure of the segment is shown in Figure 5.7. The pavement consists of a 9 in. PCC over a 2.9 in. HMAC base over an A-4 subgrade soil.

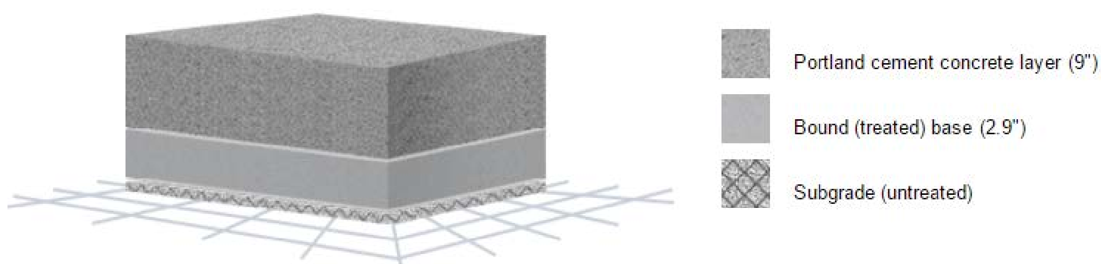


Figure 5.7 Pavement structure of LTPP Segment 40-4162

The measured and predicted pavement distresses are plotted together in Figures 5.8. For this pavement, the MEPDG design program again predicted significant amount of transverse cracking after 20 years of service life. The LTPP record shows zero transverse cracking in 13 years of service life. Meanwhile, the MEPDG design program slightly under-predicted the field faulting. The predicted IRI of the pavement seems to agree well with the field measurement.

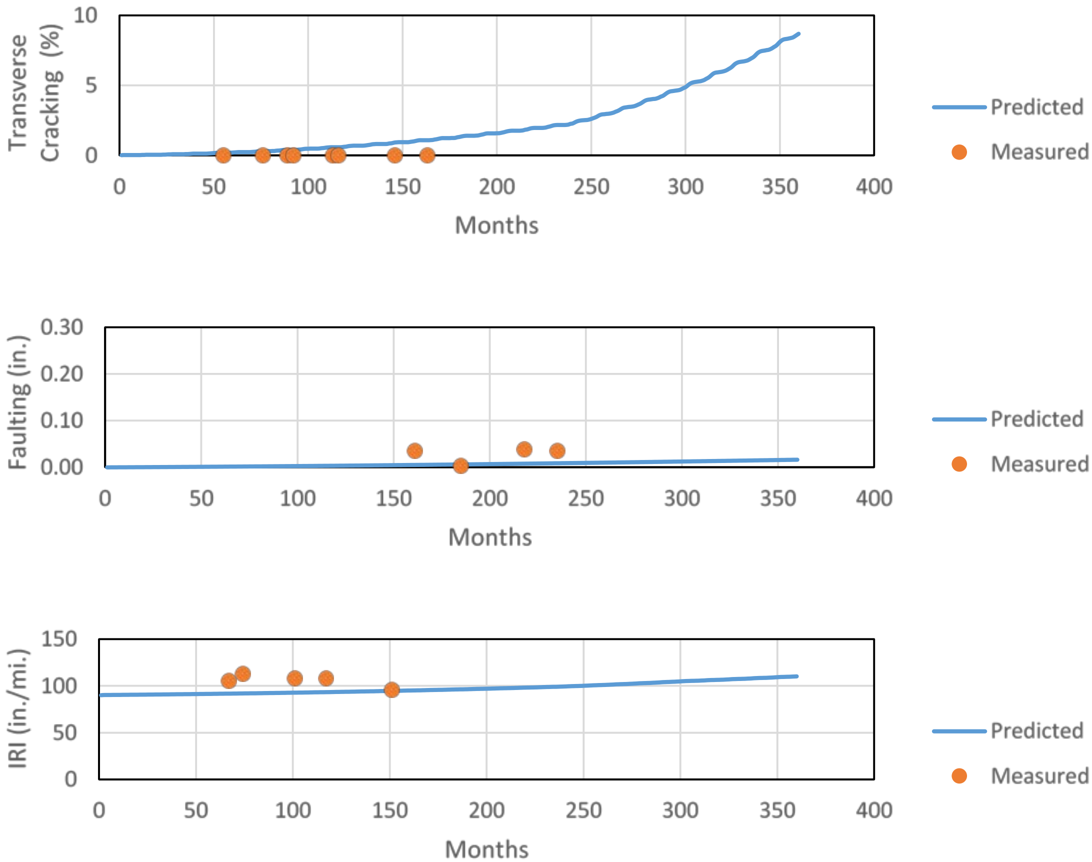


Figure 5.8 Predicted vs. measured pavement performance in LTPP Segment 40-4162

5.2.5 Section 40-4158

The pavement structure of the segment is shown in Figure 5.9. The pavement consists of a 10.3 in. PCC over a 4.4 in. HMAC base over an A-2-4 subgrade soil.

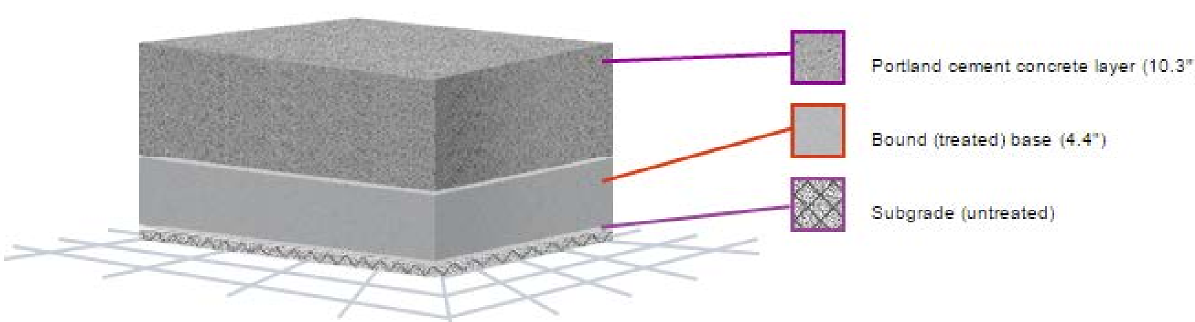


Figure 5.9 Pavement structure of LTPP Segment 40-4158

The measured and predicted pavement distresses are plotted together in Figures 5.10. Both the measured and the predicted punchouts are near zero. The measured IRI of the road does not change much for over 20 years of service life. The IRI model seems to be adequate.

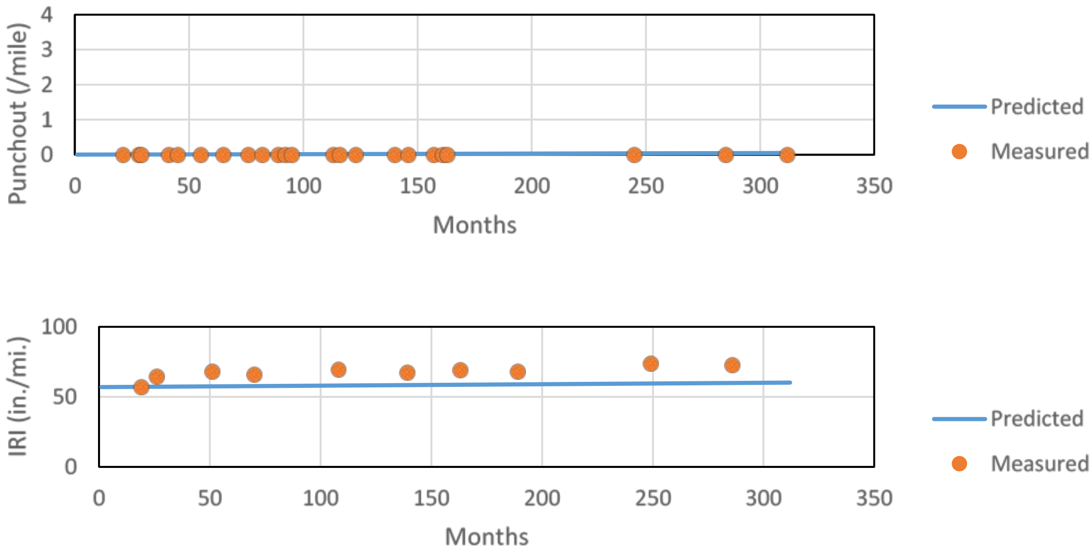


Figure 5.10 Predicted vs. measured pavement performance in LTPP Segment 40-4158

5.2.6 Section 40-4166

The pavement structure of the segment is shown in Figure 5.11. The pavement consists of a 9.9 in. PCC over a 4.1 in. cement stabilized base over an A-2 subgrade soil.

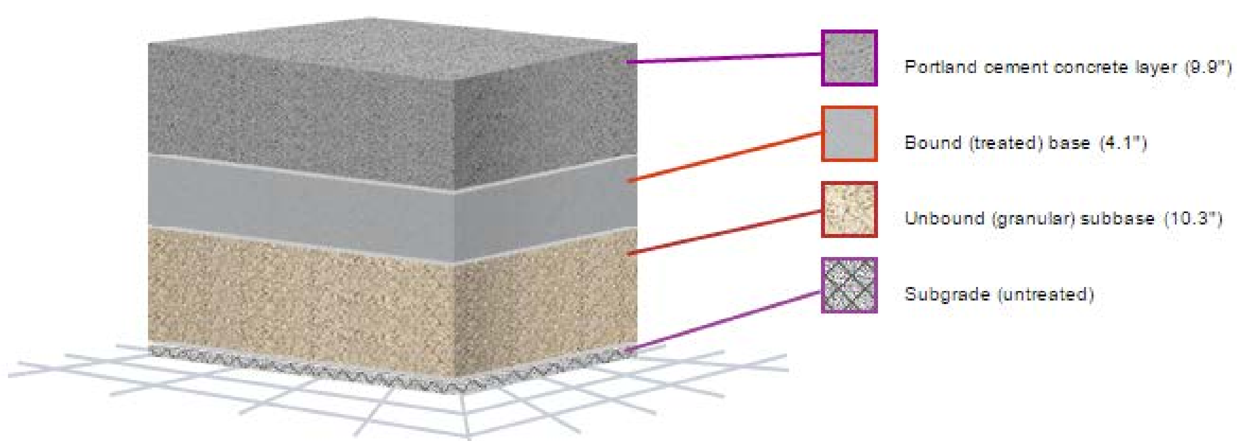


Figure 5.11 Pavement structure of LTPP Segment 40-4166

The measured and predicted pavement distresses are plotted together in Figures 5.10. For this pavement, the MEPDG predicted nearly zero punchout for over 20 years of service life. The LTPP record showed a few punchouts in some of the surveys after 10 years and then zero punchout in the last two surveys. This could be due to minor maintenance that did not show on the record. The predicted IRI matched well with the LTPP record.

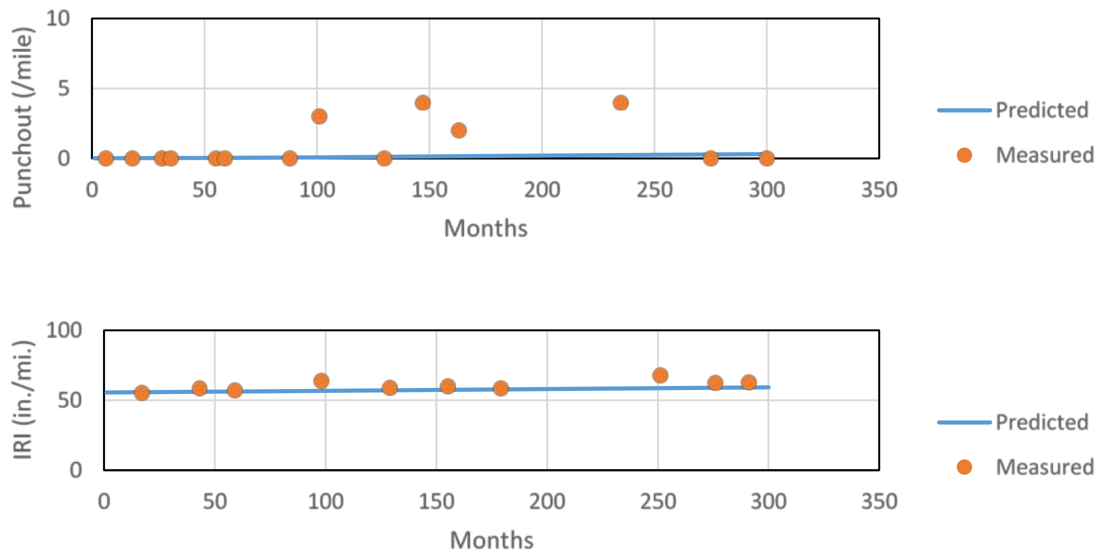


Figure 5.12 Predicted vs. measured pavement performance in LTPP Segment 40-4166

5.2.7 Section 40-5021

The pavement structure of the segment is shown in Figure 5.11. The pavement consists of a 9.4 in. PCC over a 3.5 in. HMAC base over an A-2-4 subgrade soil.

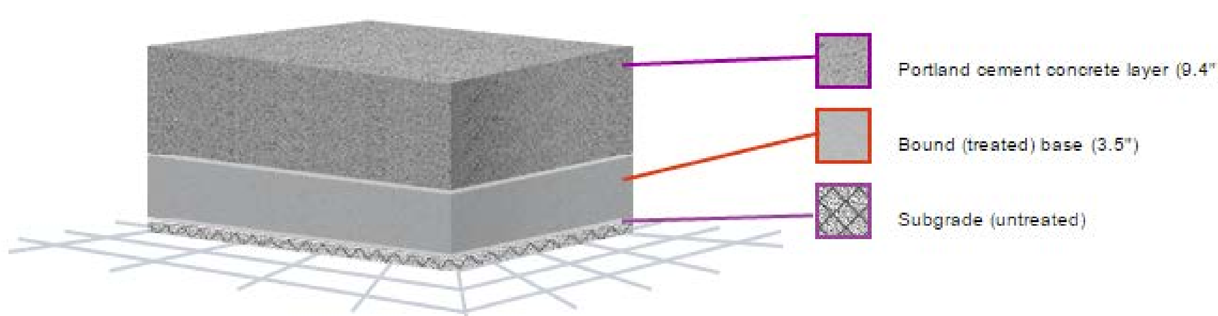


Figure 5.13 Pavement structure of LTPP Segment 40-5021

The measured and predicted pavement distresses are plotted together in Figures 5.14. Both the measured and the predicted punchouts are near zero. The measured IRI of the road does not change much for over 20 years of service life. The IRI model seems to be adequate.

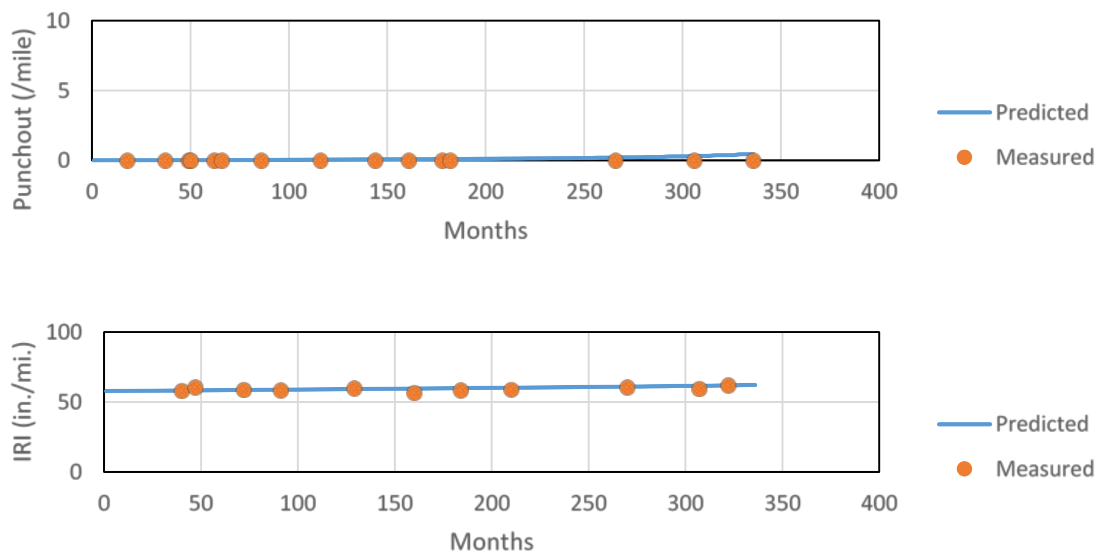


Figure 5.14 Predicted vs. measured pavement performance in LTPP Segment 40-5021

5.2.8 Summary of LTPP segments

Based on the LTPP data, all the four non-doweled JPCP segments in Oklahoma showed near-zero transverse cracking in 20 to 30 years of service period, whereas the MEPDG design program predicted significant amount cracking during the same service period. It seems that the MEPDG transverse cracking model tends to over-predict the field cracking for non-doweled JPCPs in Oklahoma. The MEPDG design program under-predicts the field faulting for three of the four JPCP segments, except for Segment 40-4157. The predicted IRI by the MEPDG design program agrees well with the field IRI for three of the four JPCP segments, except for Segment 40-4157.

The CRCP punchout model predicted nearly zero punchout for all three CRCP segments, which compared well with the LTPP records except for Segment 40-4166. This segment showed a few punchouts after 10 years of service life but then zero punchout in the last two surveys. Both the predicted and the measured IRI for the three CRCP segments changed little over the nearly 30 years of service life. The IRI model for the CRCP seems to be adequate.

The above observations were made from only four JPCP segments and three CRCP segments. The amount of data is not enough to draw a firm conclusion about the accuracy of the MEPDG design models. In addition, the JPCP segments in the Oklahoma LTPP were all constructed in

1970s-1980s without doweled joints, which does not reflect the current practice of the ODOT. Further analysis on doweled JPCPs needs to be performed based on the PMS segments.

5.3 PMS Segments

A total of 30 JPCP (all with doweled joints) and 20 CRCP segments were selected for the local validation/calibration of the MEPDG. These segments represent a wide variety of traffic, climate, and subgrade conditions in Oklahoma. The selection process has been discussed in Chapter 3 in this report. The latest version of Pavement-ME (v2.2) program was used to predict the pavement condition in Year 2014 or 2015, depending on survey date of the segment. The predicted pavement distress was then compared with the measured pavement distress from the PMS pavement condition survey. When a clear deviation between the predicted and measured pavement distress was observed, the corresponding distress model in the MEPDG would be calibrated to match the local pavement performance.

5.3.1 Local Validation

Figure 5.15 shows the comparison between the predicted and measured transverse joint faulting for the selected doweled JPCP segments. All data points fall below the line of equality, which indicates that the predicted faulting is consistently lower than the measured faulting in the field for all the selected segments. Although there are only a few segments with significant faulting near or above the design limit (0.12 inches), the national calibration model in MEPDG seems to under-predict the field faulting for doweled JPCP in Oklahoma based on the current data. A local calibration of the model is needed.

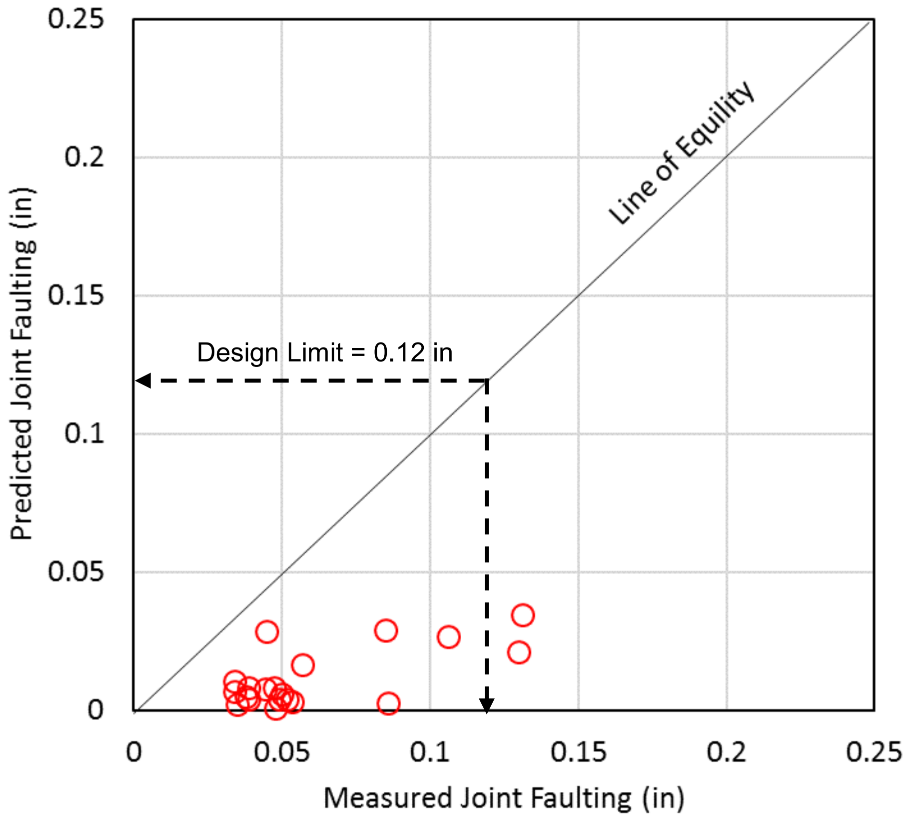


Figure 5.15 Measured and predicted transverse joint faulting for JPCP

Figure 5.16 shows the comparison between the predicted and measured transverse cracking (reported by the percentage of slabs cracked) for the selected doweled JPCP segments. For most of the selected segments, both predicted and measured transverse cracking are less than 3% except for four segments (as marked out in Figure 5.9). These four segments showed conflicting results, with two over-predictions and two under-predictions. In lack of segments with significant amount of measured or predicted cracking, it is difficult to judge the accuracy of the transverse cracking model based on the current data.

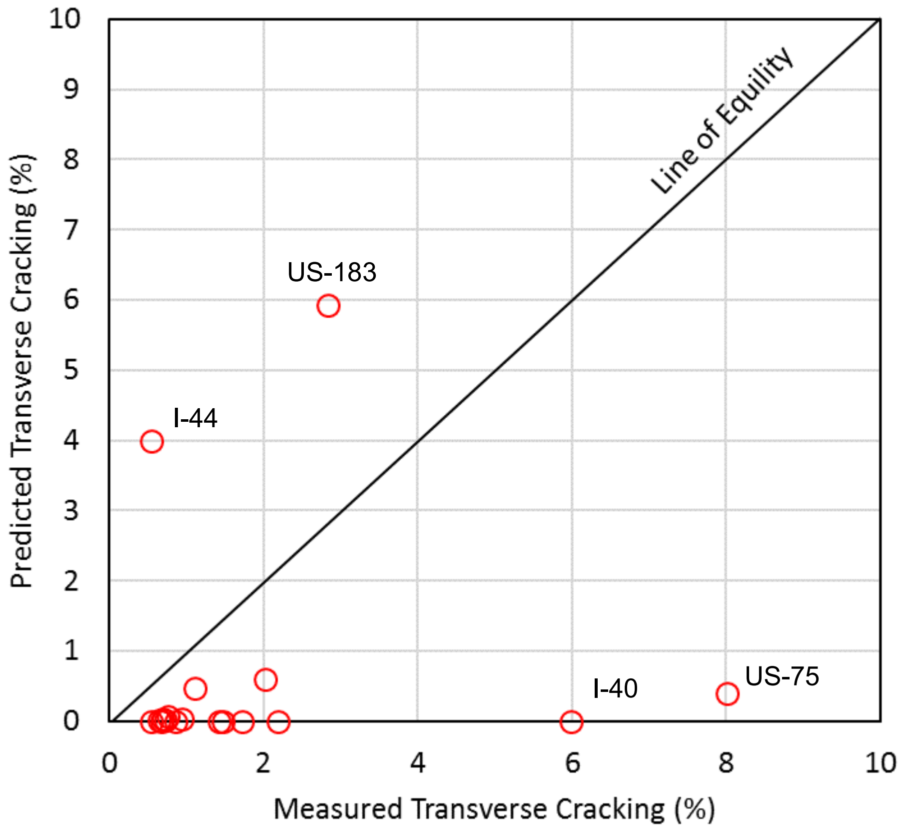


Figure 5.16 Measured and predicted transverse cracking for JPCP

Another way evaluate the transverse cracking model is to compare the measured cracking with the predicted cumulated damage (reported in percentage), as shown in Figure 5.17. The cumulated damage defined as the ratio of axle load cycle n at certain time to the axle load cycle N needed to damage the pavement. The calculated cumulated damage can be found in the output file of the Pavement-ME program.

It should be noted that the most critical part of the national default curve (solid line in Figure 5.17) is the part where it starts to curve up and intersects with the design limit. The location of this critical part will determine the design thickness of the pavement. With only a few data points on the right side of the chart (Figure 5.17), it is difficult to evaluate the accuracy of the national default MEPDG model. However, the overall trend of the Oklahoma data seems to be consistent to the national default model. Therefore, it is recommended that ODOT accepts the current national default transverse cracking model for now and re-visit this model when more transverse data become available.

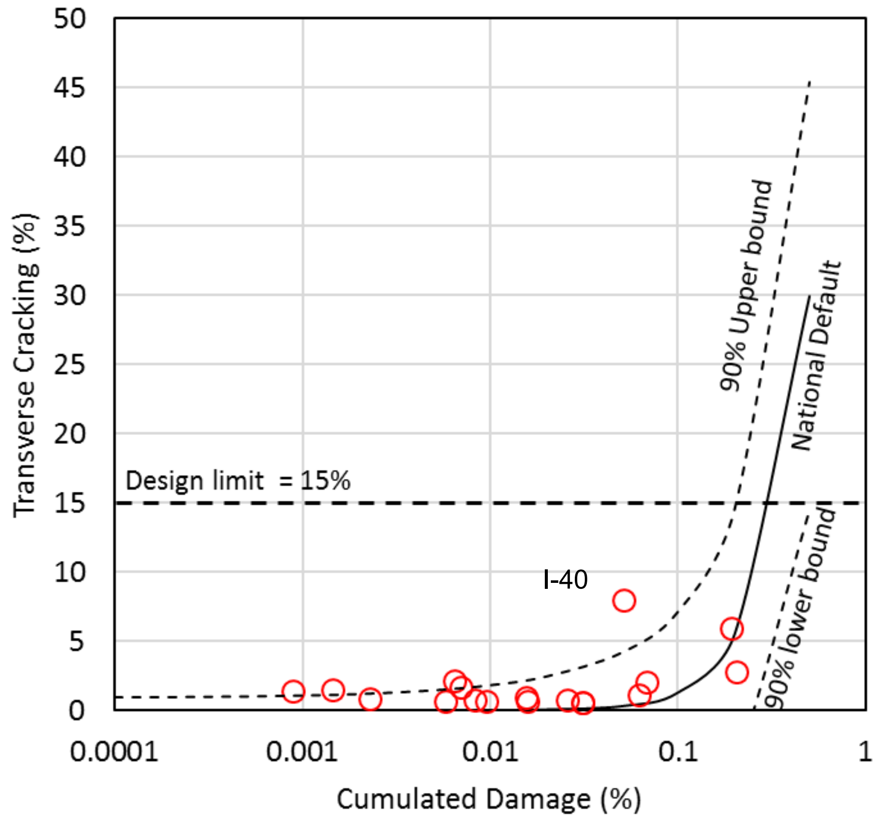


Figure 5.17 Measured transverse cracking and predicted cumulated damage

Figure 5.19 compares the predicted and measured IRI for the 30 selected JPCP segments. The predicted IRI ranged from 55 to 86 in./mi. whereas the field measured IRI ranged from 68 to 237 in./mi. Overall, the MEPDG showed a trend to under-predict the field IRI in Oklahoma, which indicates a need for local calibration.

The predicted IRI in the MEPDG is calculated based on the severity of the distress for the pavement. Because there is not enough data to validate the transverse cracking model, this makes it not possible to validate the IRI model either. Therefore, it is recommended that ODOT accepts the current national default IRI model and re-visit this model after the transverse cracking model has been validated or calibrated.

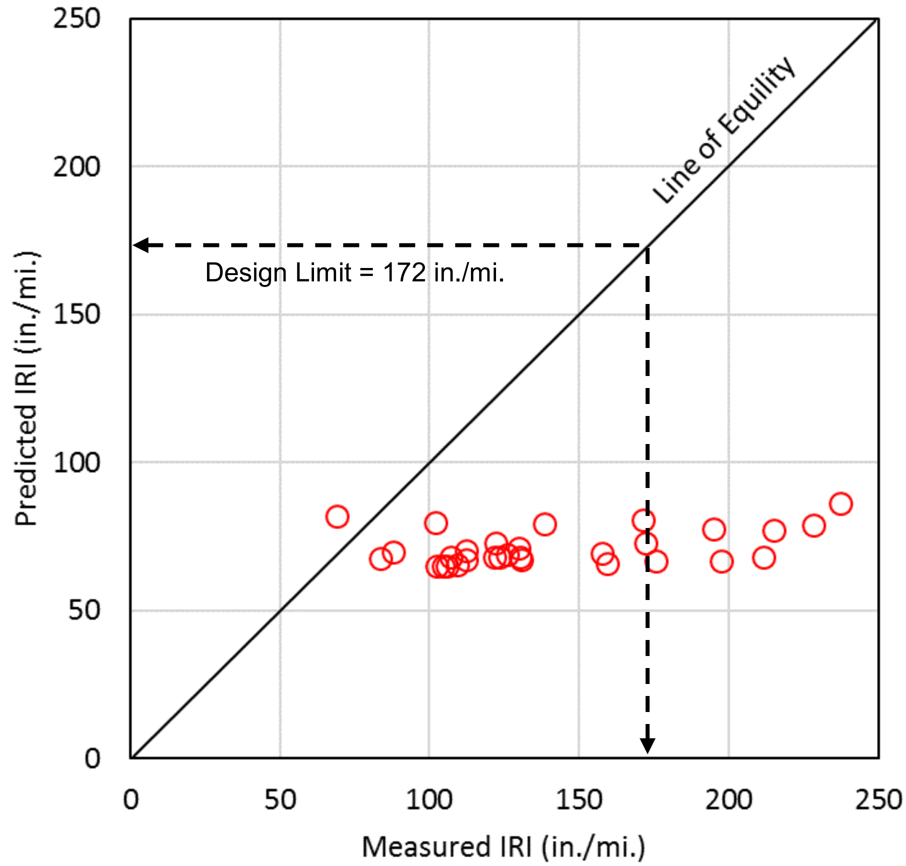


Figure 5.18 Measured and predicted IRI for JPCP

Figure 5.19 shows the comparison between the predicted and measured punchouts (reported by the number of punchouts per mile) for the selected CRCP segments. It is shown that, for most except for three of the CRCP segments, both the predicted and the measured punchouts are less than 10 per mile. These three segments showed conflicting results, with two over-predictions and one under-prediction. Similar to the JPCP transverse cracking model, further comparison between the measured punchout and the predicted cumulated damage is necessary. This comparison is shown in Figure 5.19. Data points from the three LTPP segments are also added into this figure.

Although many of the data points on the left side of Figure 5.19 fall above the national default model line, these data points are not important in judging the accuracy of the MEPDG. It is mainly due to the way these 20 CRCP segments were selected, that is, by sorting the distress and picking the most severe. In this way, some of the segments with the most severe early stage punchouts will be picked.

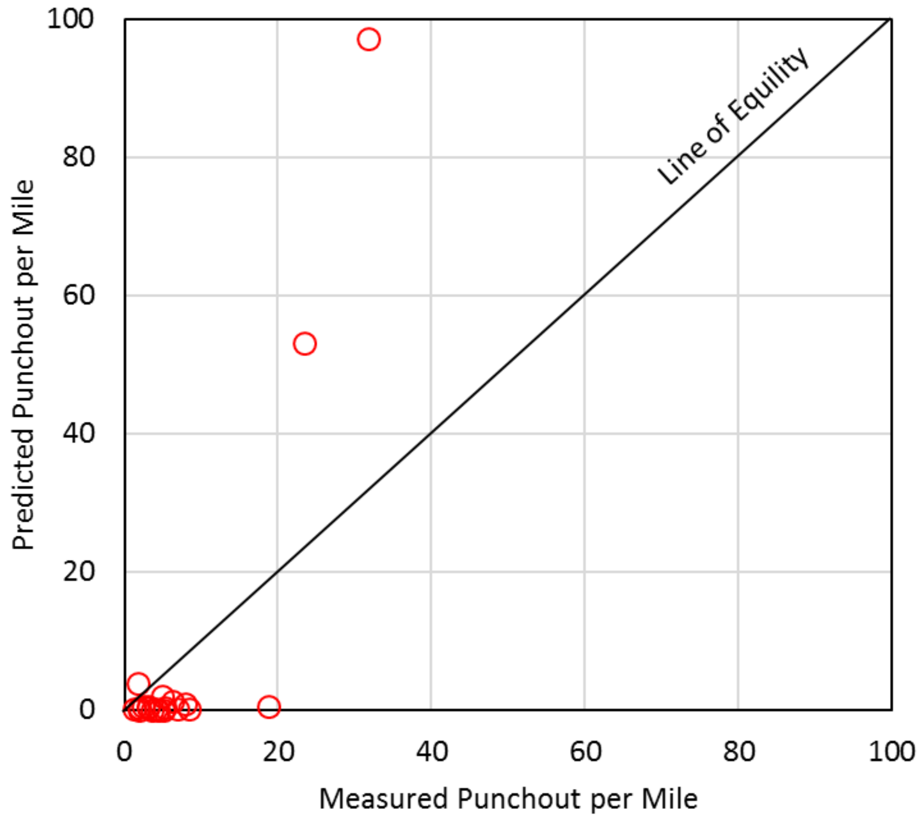


Figure 5.19 Measured and predicted punchouts for CRCP

As discussed earlier, the most important portion of design curve is the part where it starts to curve up and intersects with the design limit (10 punchouts per mile). In order to evaluate accuracy of the MEPDG model, more data points are needed in the critical area circled out in Figure 5.20. With only two data points on the right side of the chart and no data points in the critical area, it is difficult to determine the accuracy of the national default CRCP punchout model for Oklahoma pavements. Based on the above consideration, the researchers recommends re-visit the CRCP punchout model in the future when more pavement distress data become available.

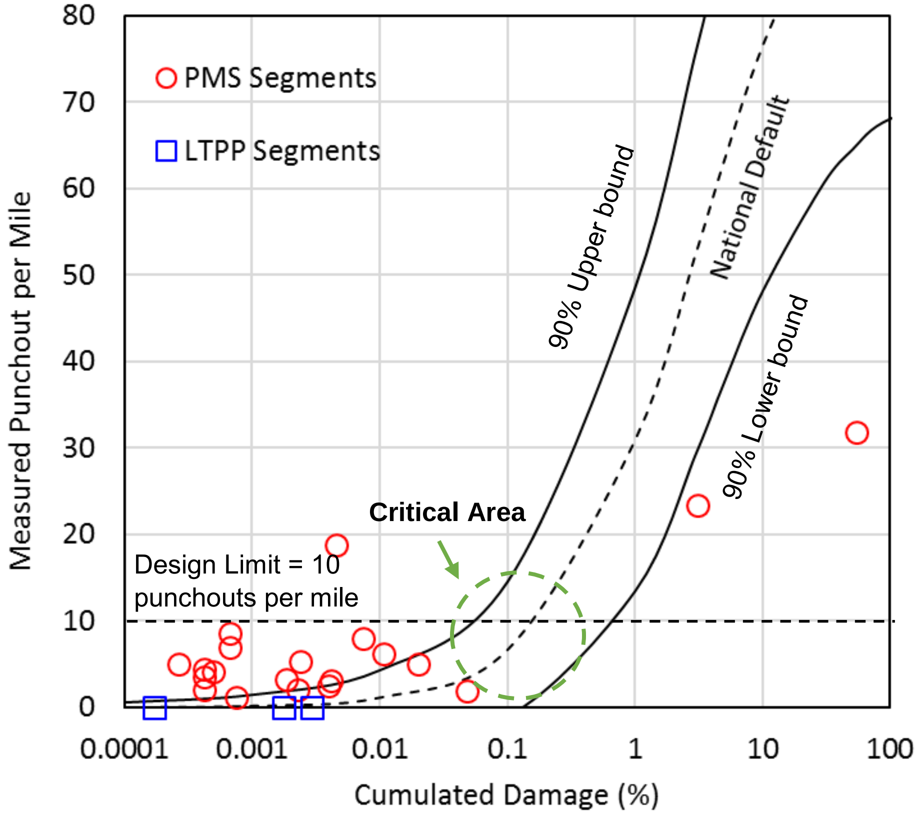


Figure 5.20 Measured punchout and predicted cumulated damage

Figure 5.21 compares the predicted and measured IRI for the 20 selected CRCP segments. The predicted IRI ranged from 65 to 86 in./mi. except for two segments where the MEPDG significantly overpredicted the field punchout.

In the MEPDG, the predicted IRI for CRCP is calculated from the predicted punchout per mile. Currently there is not enough information to validate the punchout model. Therefore, it is recommended that ODOT accepts the current national default IRI model for now and re-visit this model after the punchout model has been validated or calibrated.

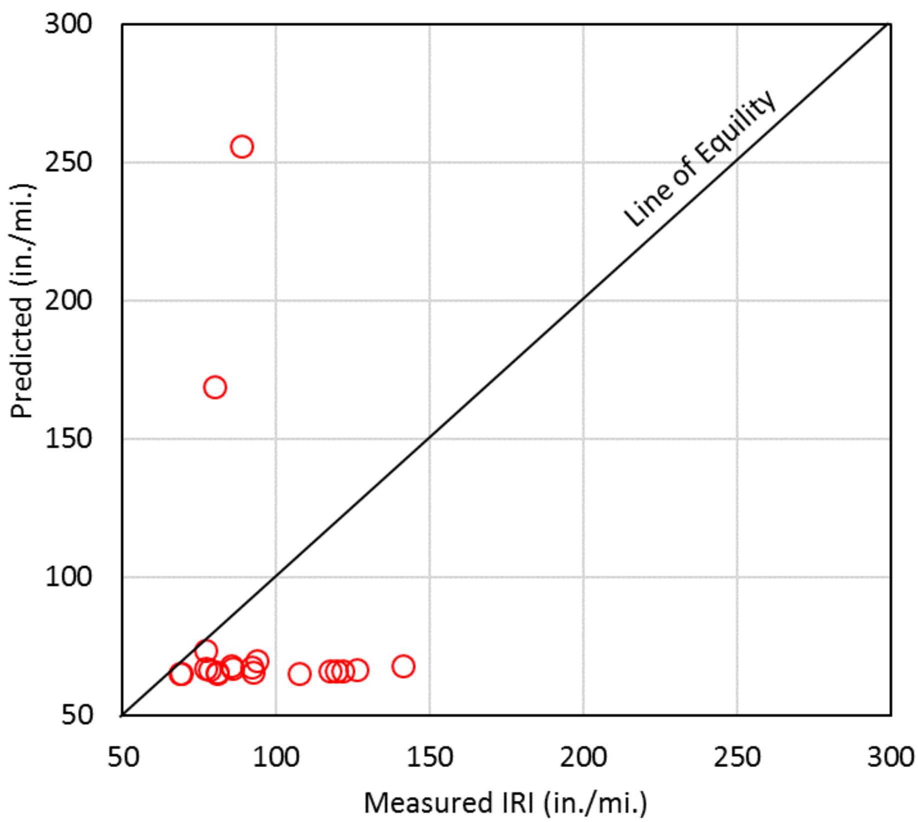


Figure 5.21 Measured and predicted IRI for CRCP

5.3.2 Local Calibration

Based on the local validation results in the previous section, the MEPDG faulting model needs to be calibrated for doweled JPCPs in Oklahoma. The local calibration was conducted by first examine the effect of each local calibration factor on the design output.

The JPCP faulting model in the MEPDG has 8 local calibration factors (C1 through C8), 7 of which (except C8) will affect the predicted performance of a doweled JPCP. The first four calibration factors (C1 through C4) have the most significant effect (proportional to the predicted faulting). These four factors were adjusted by trial and error until the standard error of estimation (SEE) of the system is minimized.

The local calibration factors for the JPCP faulting model determined from the above process are listed along with the national calibration factors in Table 5.1. After the local calibration, all the selected JPCP segments were re-analyzed with the local calibration factors. The predicted and measured faulting, before and after calibration, are compared in Figures 5.22. The 90% reliability band is also plotted. The local calibration reduced the bias of the model, and the predicted and measured faulting matched better with each other. Meanwhile, the majority of the data points now fall into the 90% reliability band.

Table 5.1 Local calibration factors for the JPCP faulting model

Calibration Factors	National	Oklahoma
C1	0.595	0.9044
C2	1.636	2.4867
C3	0.00217	0.003298
C4	0.00444	0.006749
C5	250	250
C6	0.47	0.47
C7	7.3	7.3
C8	400	400

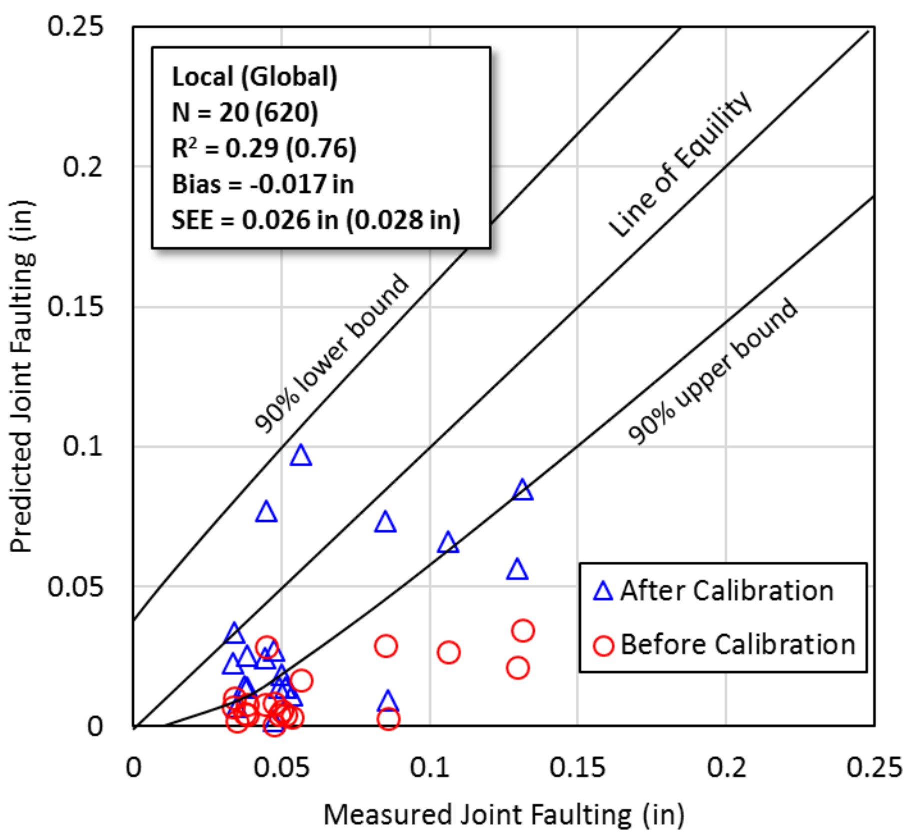


Figure 5.22 JPCP transverse joint faulting model before and after calibration

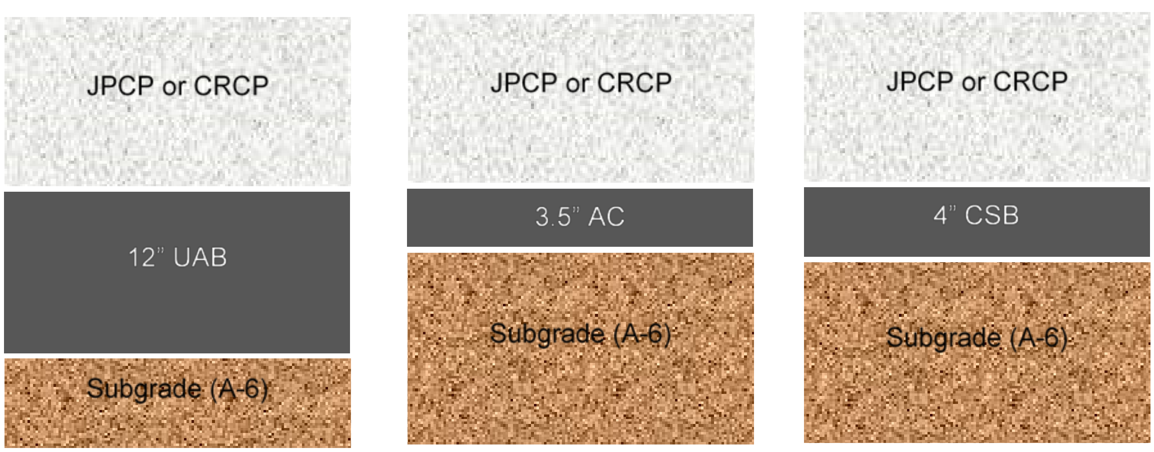
6 Comparison Analysis

6.1 Introduction

In this chapter, the design thicknesses of the Portland cement concrete (PCC) slab and the associated design costs based on the 1993 AASHTO Design Guide (DARwin v3.1), the nationally calibrated MEPDG (Pavement-ME v2.2), and the locally calibrated MEPDG are compared side by side. For comparison purposes, the design cases are based on a conventional 12-ft wide JPCP or CRCP without tied shoulder in the Oklahoma City area. The subgrade resilient modulus is fixed at 5500 psi for all design cases, which is typical in Oklahoma. The traffic volume input (i.e., annual average daily truck traffic, AADTT) is set as a variable ranging from 1000 to 4000 with a fixed yearly growth factor of 3.0%.

Three types pavement structures were considered with different base courses: (1) a 12-inch unbound aggregate base (UAB), (2) a 3.5-inch S-3 asphalt concrete (AC) base, and (3) a cement stabilized base (CSB). Figure 6.1 presents the three pavement structures used in the comparison analysis. The material and construction costs information of each pavement layer were collected from the ODOT roadway division.

The cost benefit analysis performed in this study only accounts for the change of initial construction cost due to the design difference. It is different from a life cycle cost analysis (LCCA) which considers the maintenance schedule and the associate costs.



(a) PCC over Aggregate Base (b) PCC over AC Base (c) PCC over CSB

Figure 6.1 Pavement Structures used in the comparison analysis

6.2 JPCP over Unbound Aggregate Base

The thicknesses of the JPCP and the construction costs of the pavement structures designed by different methods are compared in Table 6.1 and Figure 6.2.

For all the cases evaluated, the PCC thicknesses designed by the MEPDG are 1 to 3 inches thinner than that designed by the 1993 AASHTO Guide. The national default and the locally calibrated MEPDG produce the same pavement thicknesses for truck volume up to AADTT = 2500. This is because the design in these cases is controlled by the transverse cracking rather than faulting. At AADTT = 3000, the transverse joint faulting becomes the critical pavement distress in this type of pavement. In this case, the local calibration factors start to take effect on the design thickness. Compared to the 1993 AASHTO Design Guide, the construction cost of the pavement decreases by 5.04% to 15.11% when using the locally calibrated MEPDG to design the pavement.

Table 6.1 Comparison between 1993 AASHTO Guide and MEPDG (JPCP over Aggregate Base)

AADTT	1993 AASHTO Guide		Nationally Default MEPDG		Locally Calibrated MEPDG	
	PCC thickness* (inches)	Estimated cost (x $1000 per lane mile)	PCC thickness* (inches)	Estimated cost (x $1000 per lane mile)	PCC thickness* (inches)	Estimated cost (x $1000 per lane mile)
500	10	346.1276	9	325.5948	9	325.5948
1000	11	366.6603	9	325.5948	10	346.1276
1500	11	366.6603	10	346.1276	10	346.1276
2000	12	387.1931	10	346.1276	10	346.1276
2500	13	407.7259	10	346.1276	10	346.1276
3000	13	407.7259	10	346.1276	12	387.1931

* The design thicknesses of the PCC are rounded to 1".

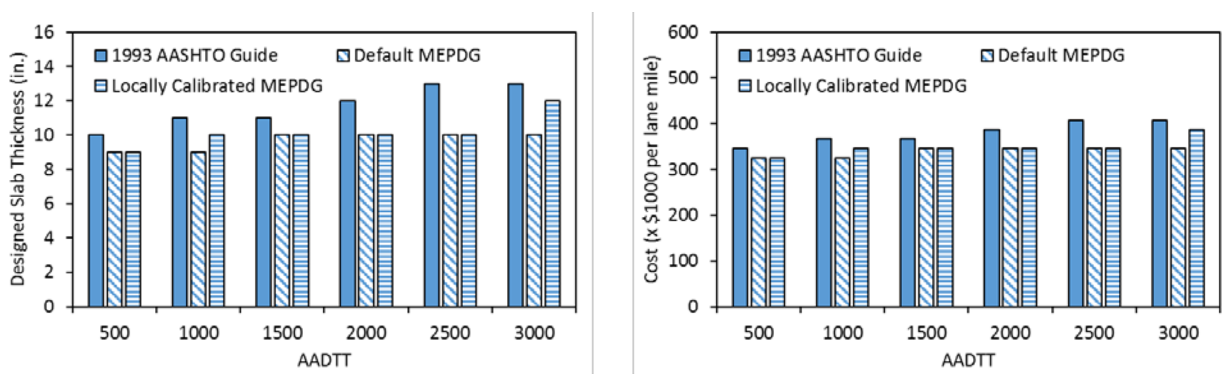


Figure 6.2 Comparison Analysis for JPCP over UAB

6.3 JPCP over Asphalt Concrete Base

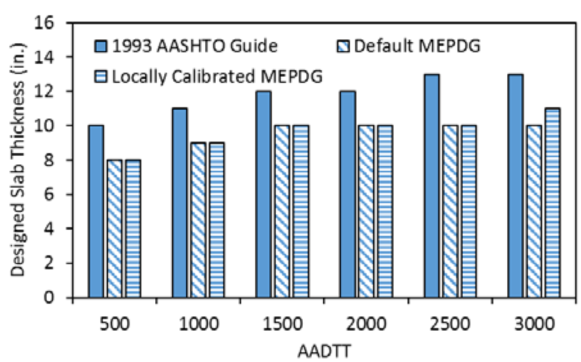
The thicknesses of the JPCP and the construction costs of the pavement structures designed by the two computer programs are compared in Table 6.2 and Figure 6.3.

For all the cases evaluated, the PCC thicknesses designed by the MEPDG are 1 to 3 inches thinner than that designed by the 1993 AASHTO Guide. The national default and the locally calibrated MEPDG produce the same pavement thicknesses for truck volume up to AADTT = 2500. This is because the design in these cases is controlled by the transverse cracking rather than faulting. At AADTT = 3000, the transverse joint faulting becomes the critical pavement distress in this type of pavement. In this case, the local calibration factors start to take effect on the design thickness. Compared to the 1993 AASHTO Design Guide, the construction cost of the pavement decreases by 8.75% to 13.13% when using the locally calibrated MEPDG to design the pavement.

Table 6.2 Comparison between 1993 AASHTO Guide and MEPDG (JPCP over AC Base)

AADTT	1993 AASHTO Guide		Nationally Default MEPDG		Locally Calibrated MEPDG	
	PC thickness (inches)	Estimated cost (x $1000 per lane mile)	PCC thickness* (inches)	Estimated cost (x $1000 per lane mile)	PC thickness (inches)	Estimated cost (x $1000 per lane mile)
500	10	407.7165	8	366.651	8	366.651
1000	11	428.2493	9	387.1837	9	387.1837
1500	12	448.782	10	407.7165	10	407.7165
2000	12	448.782	10	407.7165	10	407.7165
2500	13	469.3148	10	407.7165	10	407.7165
3000	13	469.3148	10	407.7165	11	428.2493

* All design thickness of the PCC is rounded to 1".



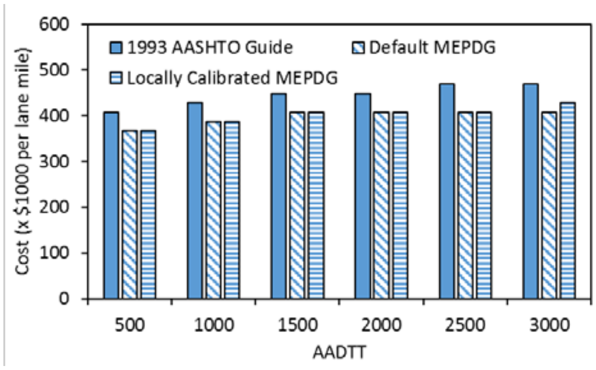


Figure 6.3 Comparison Analysis for JPCP over AC Base

6.4 JPCP over Cement Stabilized Base

The thicknesses of the JPCP and the construction costs of the pavement structures designed by the two computer programs are compared in Table 6.3 and Figure 6.4.

For all the cases evaluated, the PCC thicknesses designed by the MEPDG are 1 to 3 inches thinner than that designed by the 1993 AASHTO Guide. The national default and the locally calibrated MEPDG produced identical design thicknesses for all the design cases. This is because the design is controlled by the transverse cracking rather than faulting in all these cases. Compared to the 1993 AASHTO Design Guide, the construction cost of the pavement decreases by 5.59% to 14.36% when using the locally calibrated MEPDG to design the pavement.

Table 6.3 Comparison between 1993 AASHTO Guide and MEPDG (JPCP over Cement Stabilized Base)

AADTT	1993 AASHTO Guide		Nationally Calibrated MEPDG		Locally Calibrated MEPDG	
	PC thickness (inches)	Estimated cost (x $1000 per lane mile)	PC thickness (inches)	Estimated cost (x $1000 per lane mile)	PC thickness (inches)	Estimated cost (x $1000 per lane mile)
500	10	367.2476	9	346.7148	9	346.7148
1000	11	387.7803	9	346.7148	9	346.7148
1500	11	387.7803	10	367.2476	10	367.2476
2000	12	408.3131	10	367.2476	10	367.2476
2500	12	408.3131	10	367.2476	10	367.2476
3000	13	428.8459	10	367.2476	10	367.2476

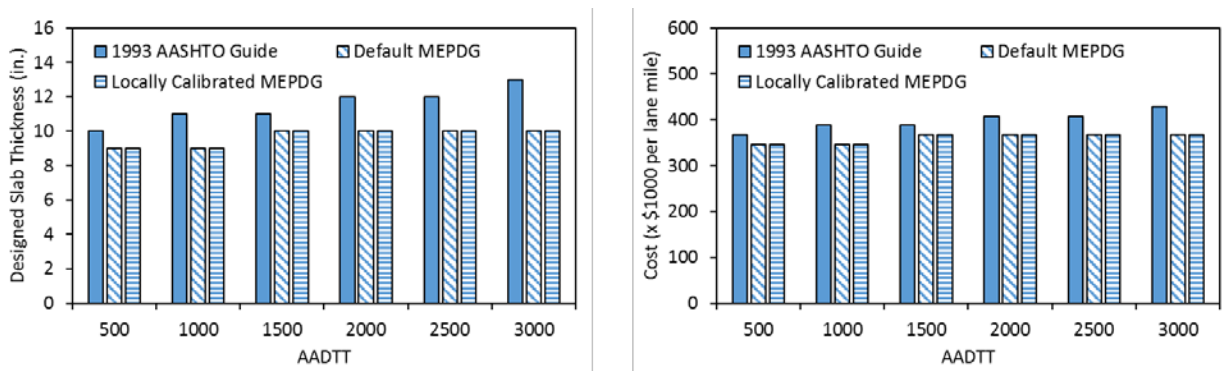


Figure 6.4 Comparison Analysis for JPCP over CSB

6.5 CRCP over Unbound Aggregate Base

The thicknesses of the CRCP and the construction costs of the pavement structures designed by different methods are compared in Table 6.4 and Figure 6.5.

The default MEPDG produces slightly thicker (by 1 inch) PCC thicknesses than the 1993 AASHTO Guide at truck volume AADTT <= 1000. The two design methods produce the same PCC thicknesses when 1500 <= AADTT <= 3000 for all the traffic volumes evaluated. Compared to the 1993 AASHTO Design Guide, the construction cost of the pavement slightly increases by 0 to 5.93% when using the default MEPDG to design the pavement.

Table 6.4 Comparison between 1993 AASHTO Guide and MEPDG (CRCP over UAB)

AADTT	1993 AASHTO Guide		Default MEPDG	
	PCC thickness* (inches)	Estimated cost (x $1000 per lane mile)	PCC thickness* (inches)	Estimated cost (x $1000 per lane mile)
500	10	484.5786	11	513.3245
1000	11	513.3245	12	542.0703
1500	12	542.0703	12	542.0703
2000	13	570.8162	13	570.8162
2500	13	570.8162	13	570.8162
3000	13	570.8162	13	570.8162

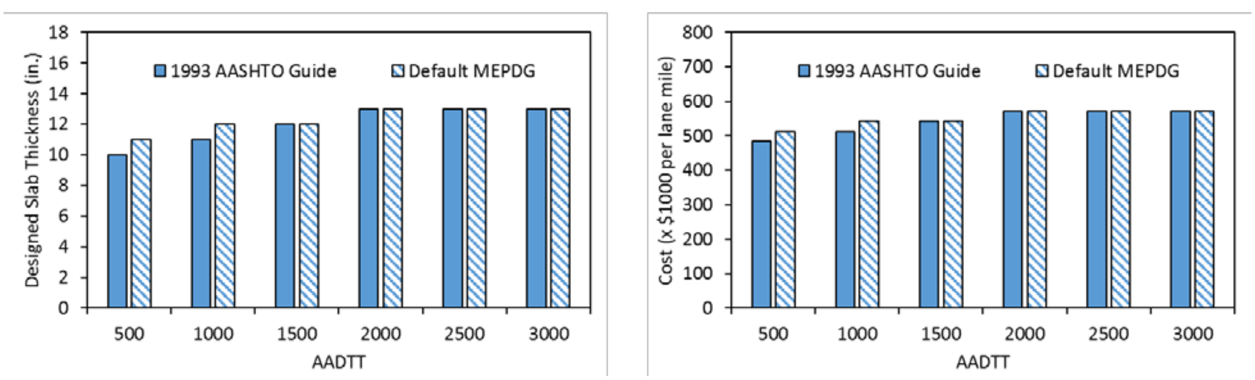


Figure 6.5 Comparison Analysis for CRCP over UAB

6.6 CRCP on Asphalt Concrete Base

The thicknesses of the CRCP and the construction costs of the pavement structures designed by different methods are compared in Table 6.5 and Figure 6.6.

For this type of pavement, the default MEPDG produces thinner (by 2 to 4 inches) PCC thicknesses than the 1993 AASHTO Guide. The MEPDG design model is not as sensitive to the truck volume as the 1993 AASHTO Guide. Compared to the 1993 AASHTO Guide, the construction cost of the pavement decreases by 9.59% to 17.50% when using the default MEPDG to design the pavement.

Table 6.5 Comparison between 1993 AASHTO Guide and MEPDG (CRCP over AC Base)

AADTT	1993 AASHTO Guide		Nationally Calibrated MEPDG	
	PCC thickness* (inches)	Estimated cost (x $1000 per lane mile)	PCC thickness* (inches)	Estimated cost (x $1000 per lane mile)
500	10	570.8031	8	513.3114
1000	11	599.549	9	542.0572
1500	12	628.2948	9	542.0572
2000	12	628.2948	9	542.0572
2500	13	657.0407	9	542.0572
3000	13	657.0407	9	542.0572

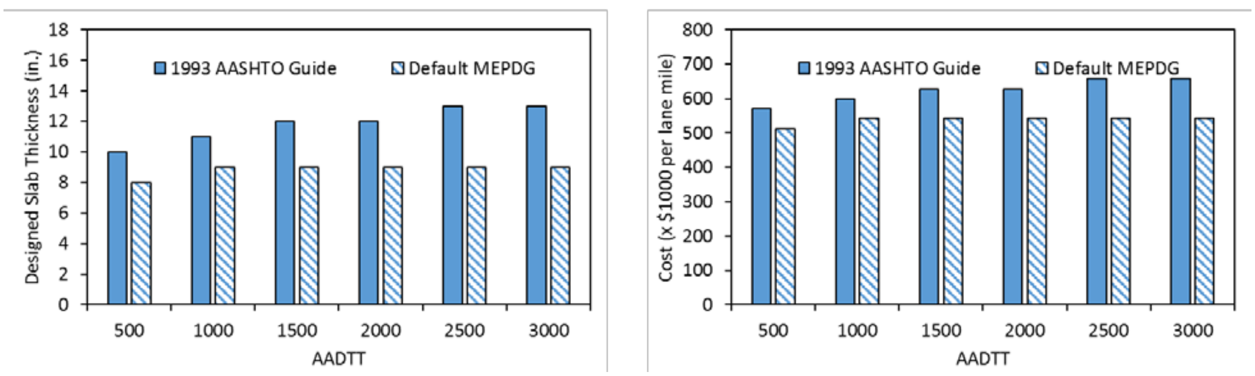


Figure 6.6 Comparison Analysis for CRCP over AC Base

6.7 CRCP on Cement Stabilized Base

The thicknesses of the CRCP and the construction costs of the pavement structures designed by different methods are compared in Table 6.6 and Figure 6.7.

For this type of pavement, the default MEPDG produces thinner (by 1 to 4 inches) PCC thicknesses than the 1993 AASHTO Guide. The MEPDG design model is not as sensitive to the truck volume as the 1993 AASHTO Guide. Compared to the 1993 AASHTO Guide, the construction cost of the pavement decreases by 9.59% to 17.50% when using the default MEPDG to design the pavement.

Table 6.6 Comparison between 1993 AASHTO Guide and MEPDG (CRCP over CSB)

AADTT	1993 AASHTO Guide		Nationally Calibrated MEPDG	
	PC thickness (inches)	Estimated cost (x $1000 per lane mile)	PC thickness (inches)	Estimated cost (x $1000 per lane mile)
500	10	514.1466	9	485.4008
1000	11	542.8925	9	485.4008
1500	12	571.6383	9	485.4008
2000	12	571.6383	9	485.4008
2500	13	600.3842	9	485.4008
3000	13	600.3842	9	485.4008

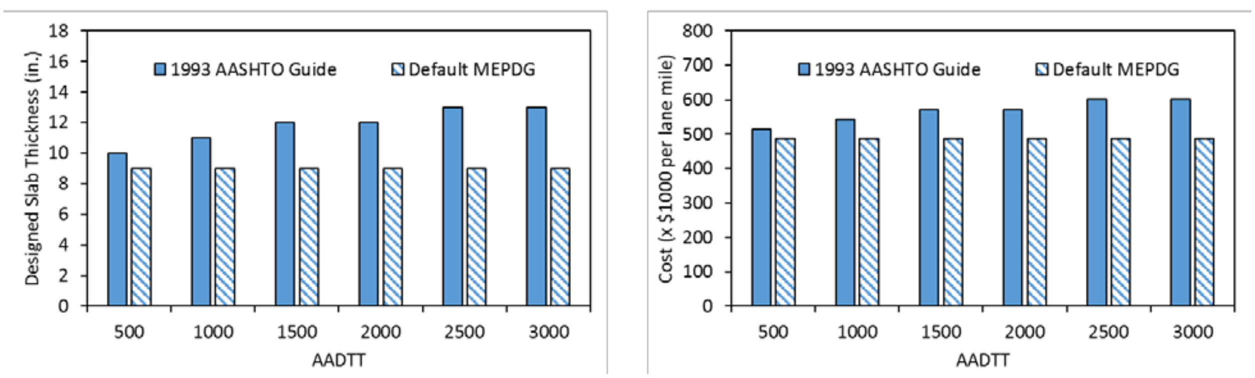


Figure 6.7 Comparison Analysis for CRCP over CSB

6.8 Summary

For doweled JPCP design, the default and the locally calibrated MEPDG both produce thinner PCC thicknesses than the 1993 AASHTO Guide. More reduction is shown at higher truck volumes. Transverse cracking is the critical distress (which will control the design) for doweled JPCPs at lower truck volumes. At higher truck volumes, faulting may start to control the design, in which case the locally calibrated MEPDG will produce thicker PCC thicknesses than the national default model.

For CRCP over unbound aggregate base pavement, the national default MEPDG model produces similar design thicknesses to the 1993 AASHTO Guide for all the truck volumes evaluated. However, much thinner (up to 4 inches) PCC thicknesses were determined by the MEPDG when AC or cement stabilized bases are used.

Due to the lack of pavement performance data to validate the JPCP transverse cracking model and the CRCP punchout model (as shown in Chapter 5), there is not enough information to justify a thickness reduction of more than 3 inches. At this moment, the researchers recommend ODOT start to use the MEPDG (with the locally calibrated faulting model) and use the 1993 Guide as a reference. When the PCC thickness designed by the MEPDG is 3 inches or more different from the 1993 design, the 1993 design thickness should be used in the project.

7 Instrumentation on I-44 and Lewis

7.1 Introduction

In order to learn more about the impact of different curing methods on the curling of concrete pavements 62 different sensors were installed on a continuous reinforced concrete pavement on Highway I-44 & Lewis in Tulsa, Oklahoma in August of 2013. The pavement was 12” in depth and 14’ in width.

7.2 Instrumentation

The goal of the research was to compare four different curing methods and the impact they have on the curling of concrete pavements. Figure 7.1 shows the truck mixer and the paver used on the project.

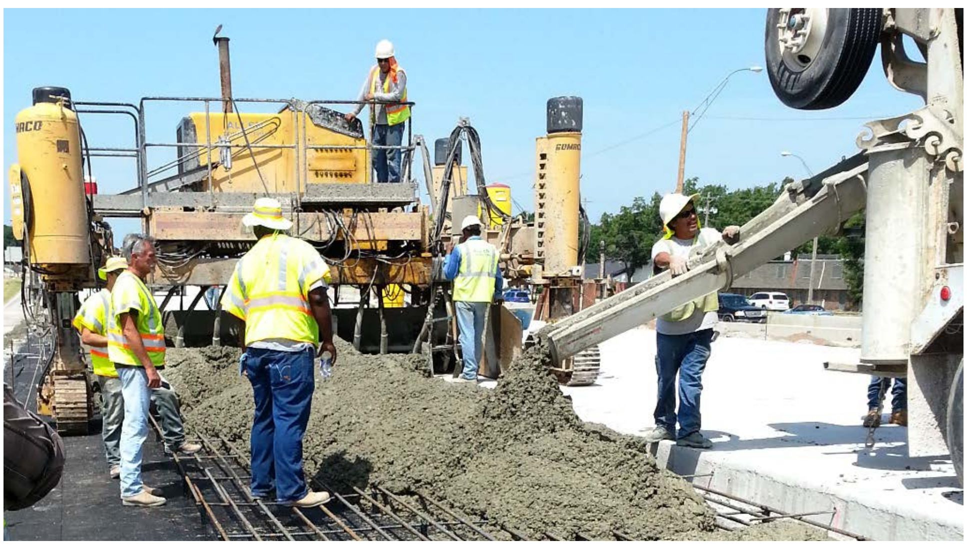


Figure 7.1 Casting the concrete pavement

The following curing methods were investigated as shown in Figure 7.2: wet cure with wet burlap for 5 days, water-wax and PAMS curing compound, and misting provided to the surface of the pavement every hour for 24 hours. Nine strain gages and seven RH sensors are used to measure the strain and RH profiles at three locations for each section of pavement.

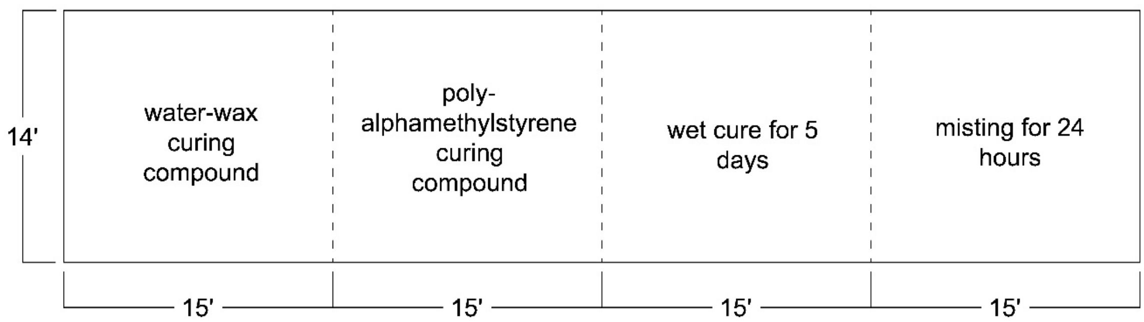


Figure 7.2 Curing materials and methods used on the pavement

Figure 7.3 shows the arrangement of the strain gages and rebar. Each gage can measure strain and temperature simultaneously. These gages were chosen because they were robust and have been used for long term monitoring of concrete structures. These strain gages were tied to a vertical stand with feet at the bottom to help hold the bars in place. This stand was then tied to the traverse reinforcing bars. Care was taken in the field to ensure that each gage was at the reported height from the concrete base shown in Figure 7.3 However, it was not possible to measure these sensors after the fresh concrete was placed and consolidated and so it may be possible that small changes occurred that were not captured. It should also be noted that the actual pavement constructed was 13” instead of 12” at the point of construction. This slightly changes the strains measured in the pavement.

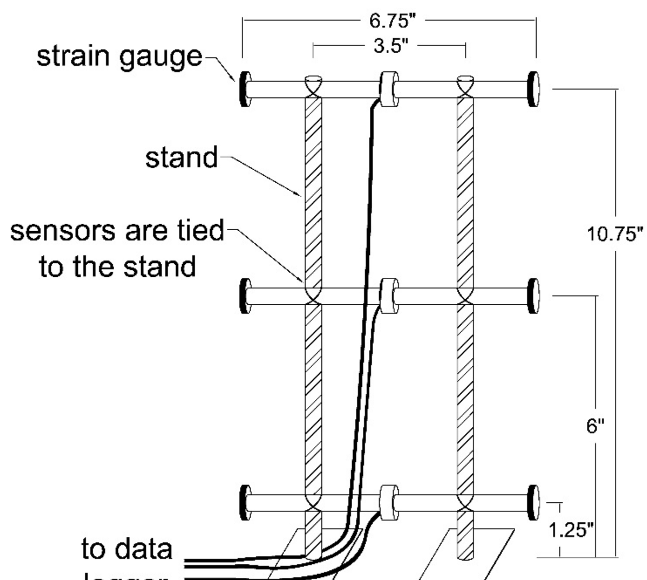


Figure 7.3 A strain gage tied to the steel bars

The RH gages were placed at two different spots, which are the centerline between each two sets of strain gages. RH is measured at different depths: 1”, 3.5”, 6”, and 11” as shown in Figure 7.4.

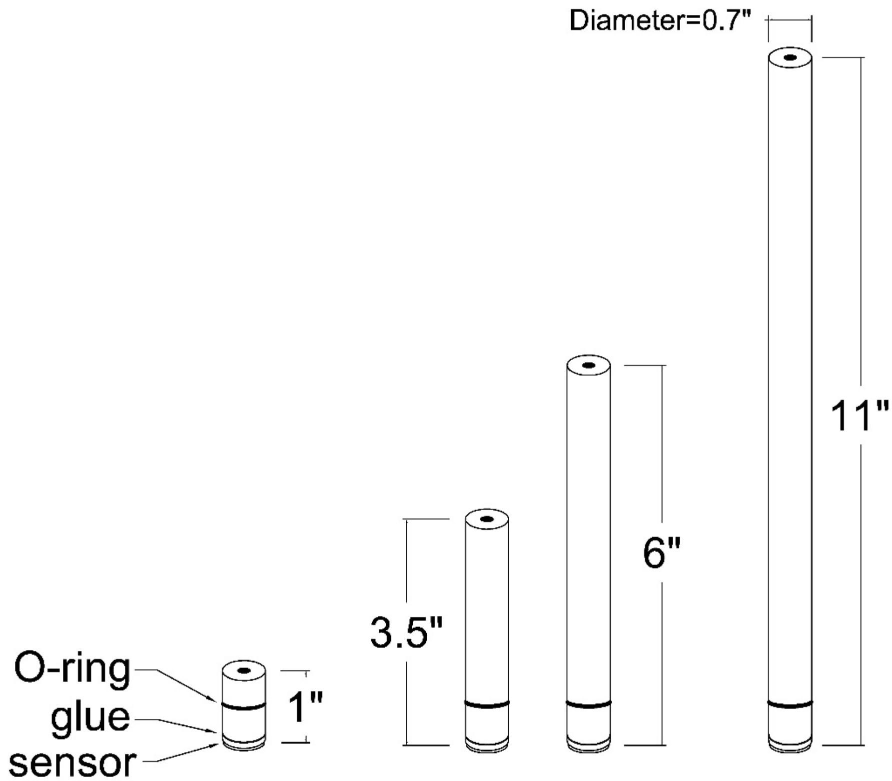


Figure 7.4 RH sensors glued to rods

The sensors at 11” were measured only for wet curing and the water-wax based curing compound. The mentioned depths were drilled in the pavement five days after paving. Sensors were glued to rods that were embedded into the drilled holes. These rods had an O-ring at the end to seal between the concrete and the rod to ensure that the readings were only coming from the concrete near the sensor. Grease and caulk was also used to help seal the rod. A typical installation is shown in Figure 7.4.

Strain gages were embedded in the pavement at three different spots. Nominally the gages were attempted to be placed at the center, edge, and the quarter point of the pavement. Unfortunately, because of the rebar layout and pavement vibrators these locations had to be varied slightly. As shown in Figure 7.5 for a typical slab, the gages were oriented in the transverse direction of the pavement in order to measure curling and shrinkage of the pavement. A gage was used at the top and bottom of the pavement so that it could show any differential strain between these two locations. This strain differential will be tied to the amount of curvature or bending that occurs in the pavement. The strain gage at the center of the pavement will help examine the strain profile and the amount of uniform shrinkage that occurs in the pavement. By combining these measurements with the temperature measurements and humidity, it should be possible to compare the overall strain profiles in the pavement and determine whether these can be attributed to differentials in shrinkage, temperature, or humidity.

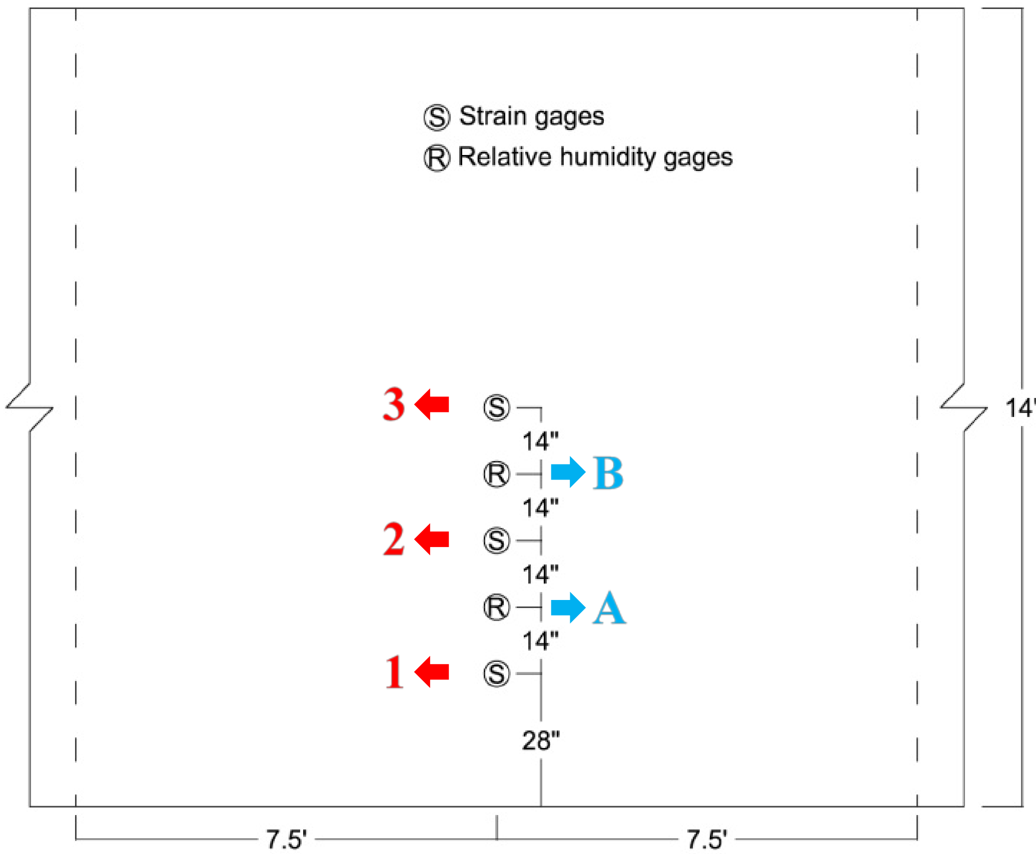


Figure 7.5 Top view of a typical slab to show the coordination of all the gages

The gages as placed in the pavement are shown in in Figure 7.6.

Figure 7.6 The strain gage locations for each curing method

7.3 Application of the Curing Methods

Both of the curing compounds were manually applied to the surface of the pavement. The water-wax based curing compound was sprayed manually by the contractor and the PAMS was sprayed by the researchers. Examples of their sprayed surfaces is shown in Figures 7.7 and 7.8. Figure 7.9 shows the wet burlaps under the white plastic sheet to cure this section of the pavement for five days. After five days and before drilling, the curing was terminated and the burlaps were still moist. The last section of this field experiment was cured by using misting. This was done manually every hour for the first 24 hours after placement of the concrete.

Figure 7.7 The water-wax based curing compound applied by the contractor approximately 30 minutes after paving

Figure 7.8 PAMS curing compound applied by the research team

Figure 7.9 Wet curing for five days under white plastic sheets

While constructing the shoulder concrete on the project a heavy rainstorm destroyed the surface finish of the concrete. The contractor had to remove and replace the shoulder. In doing this the wiring for three gages were destroyed.

7.4 Results and Discussion

Using a running average for every 6 hours from paving the temperature profiles have been calculated from the measured data. The running average has been used to reduce the fluctuations over time. Temperature profiles of the slab are shown in Fig. 7.10, 7.11, and 7.12 respectively for locations 1, 2, and 3 in Fig. 7.5. These locations are at 28”, 56”, and 84” from the edge of the slab, respectively. The profiles are for ages 0.1, 30, and 100 days after paving.

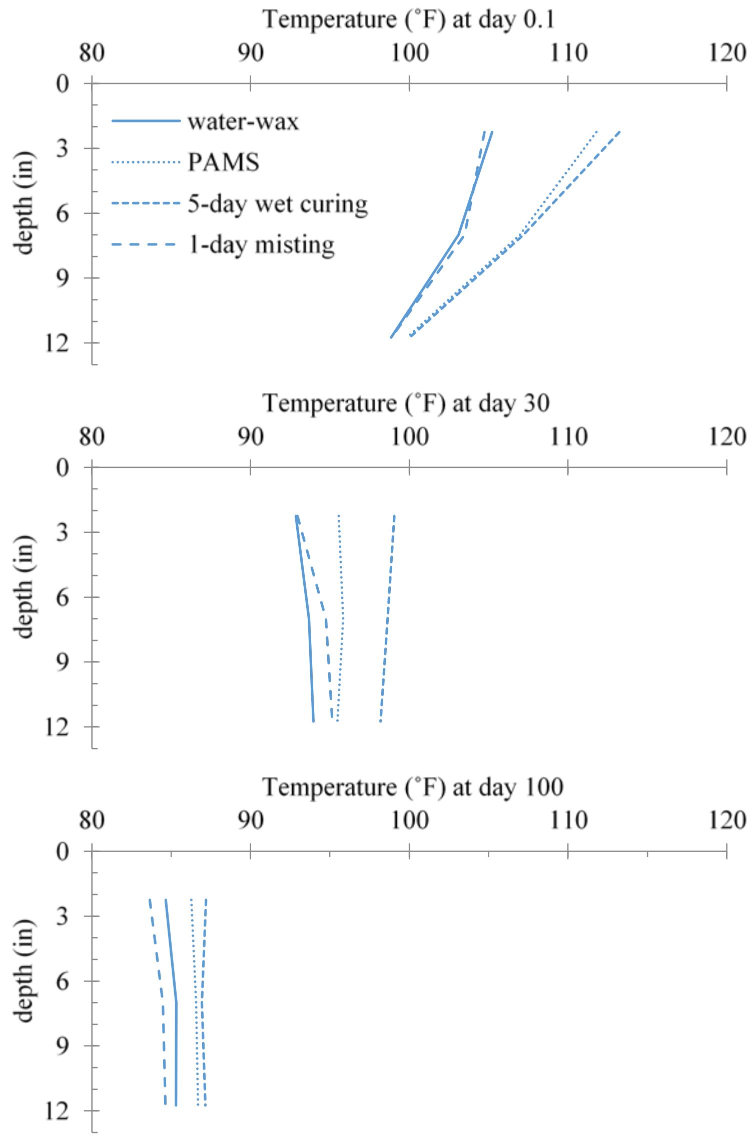


Figure 7.10 - Temperature profiles for *location 1* at 0.1, 30, and 100 days after paving

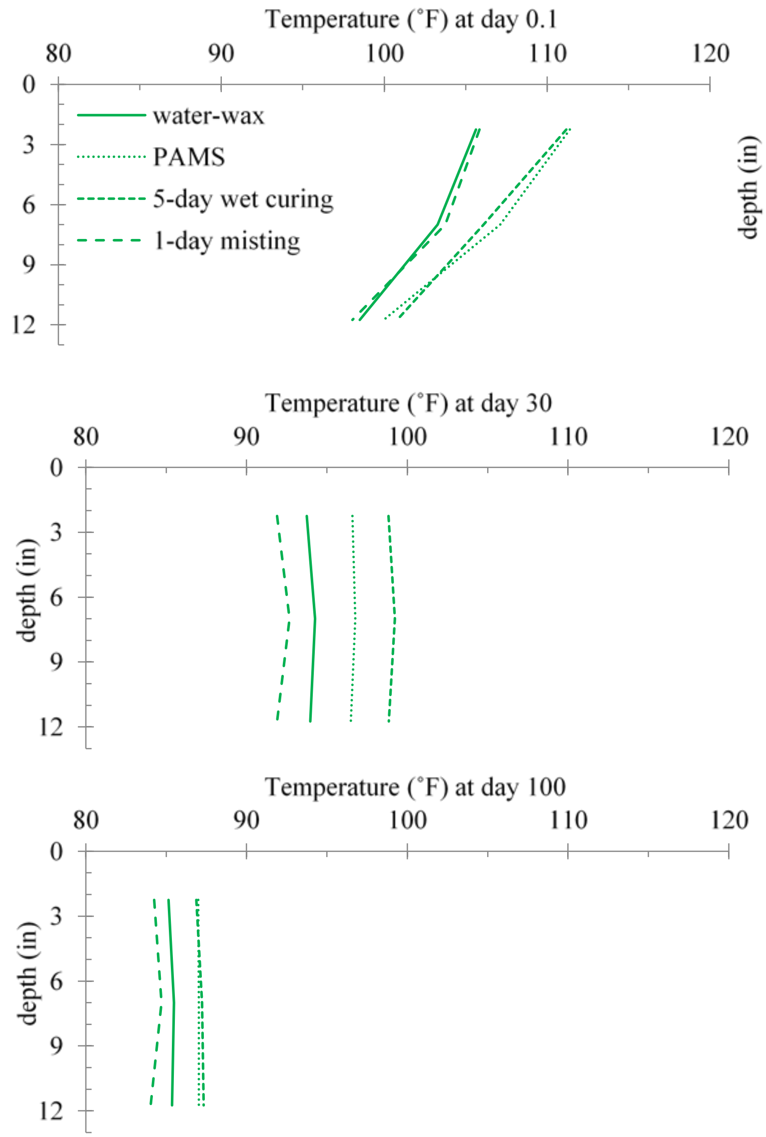


Figure 7.11 - Temperature profiles for *location 2* at 0.1, 30, and 100 days after paving

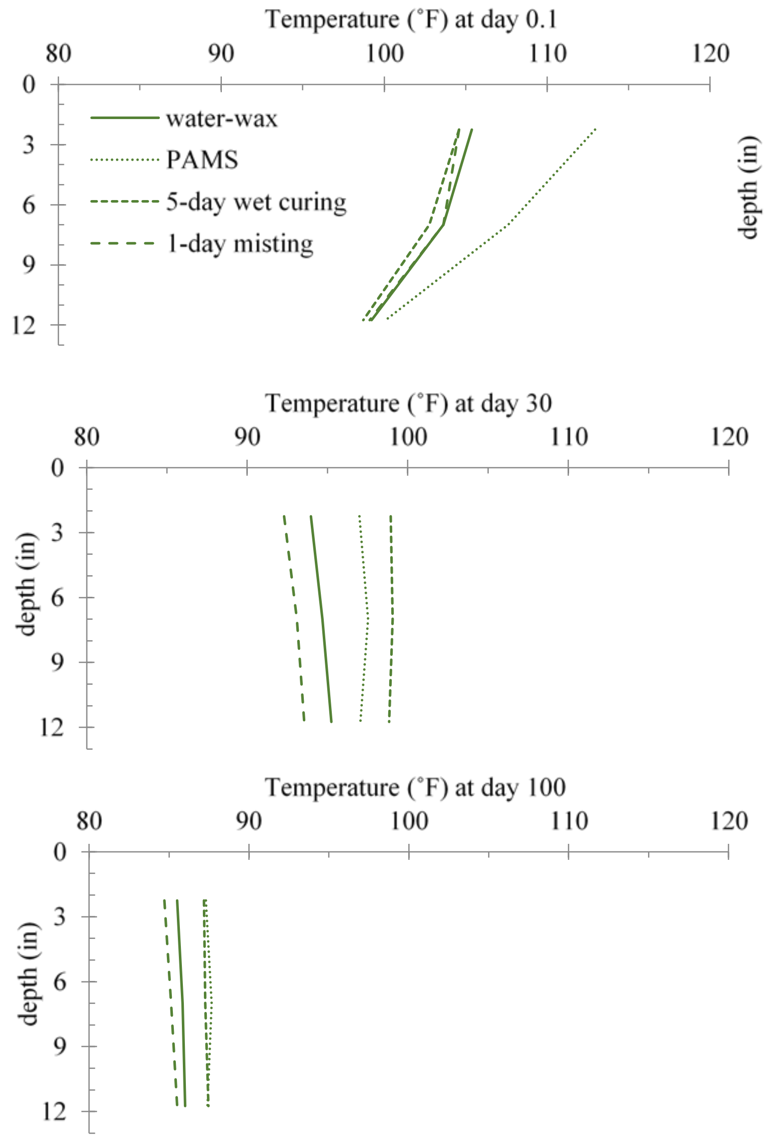


Figure 7.12 - Temperature profiles for *location 3* at 0.1, 30, and 100 days after paving

The temperature gradient is the highest at 0.1 day after paving. However, the gradient becomes negligible over the time for all curing techniques. This low temperature gradient suggests that the slab movement would be caused by differences in moisture. Also, gradients at all of the sections are comparable at a given time. This means that the same temperatures were experienced in the three different locations in the slab at comparable time periods.

Figures 7.13, 7.14, and 7.15 show the strain profiles of the slabs cured with different techniques for locations 1, 2, and 3 of Fig. 7.5. These locations are at 28", 56", and 84" from the edge of the slab, respectively. A running average for every 6 hours has been used. The profiles show the strain gradients within the slab for 0.1, 30, and 100 days after paving. The positive values show the swelling and negative values are for shrinkage.

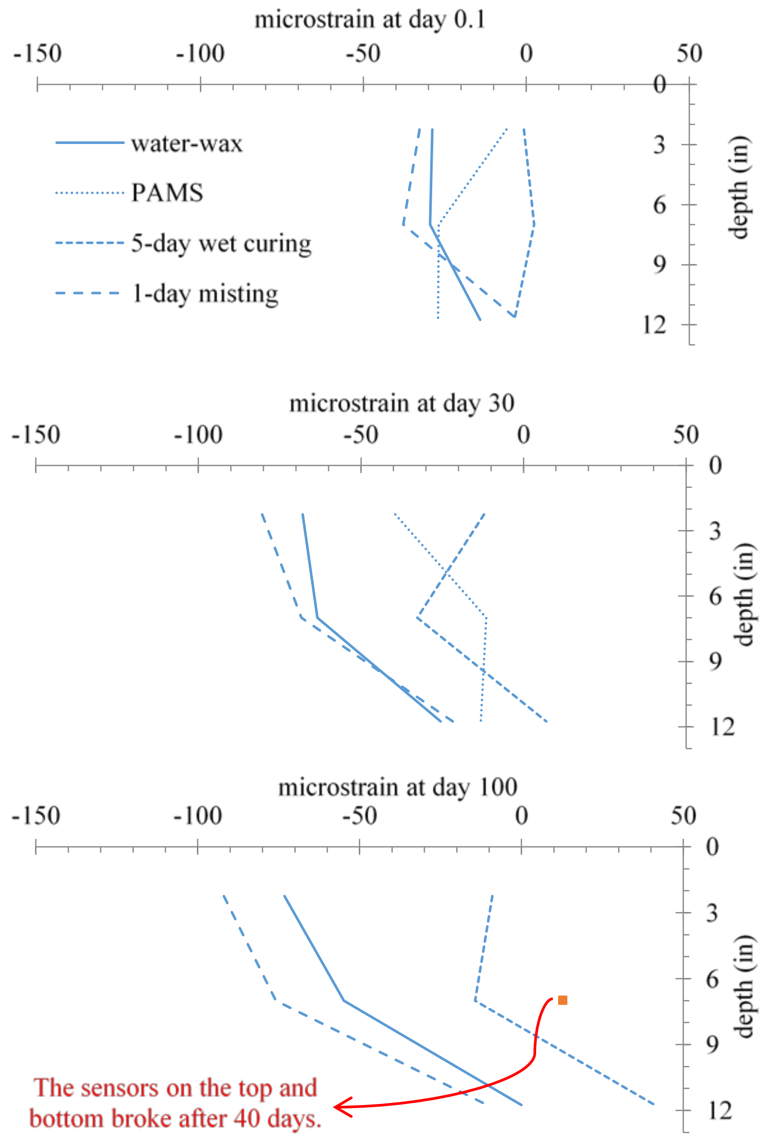


Figure 7.13 - Strain profiles for *location 1* at 0.1, 30, and 100 days after paving

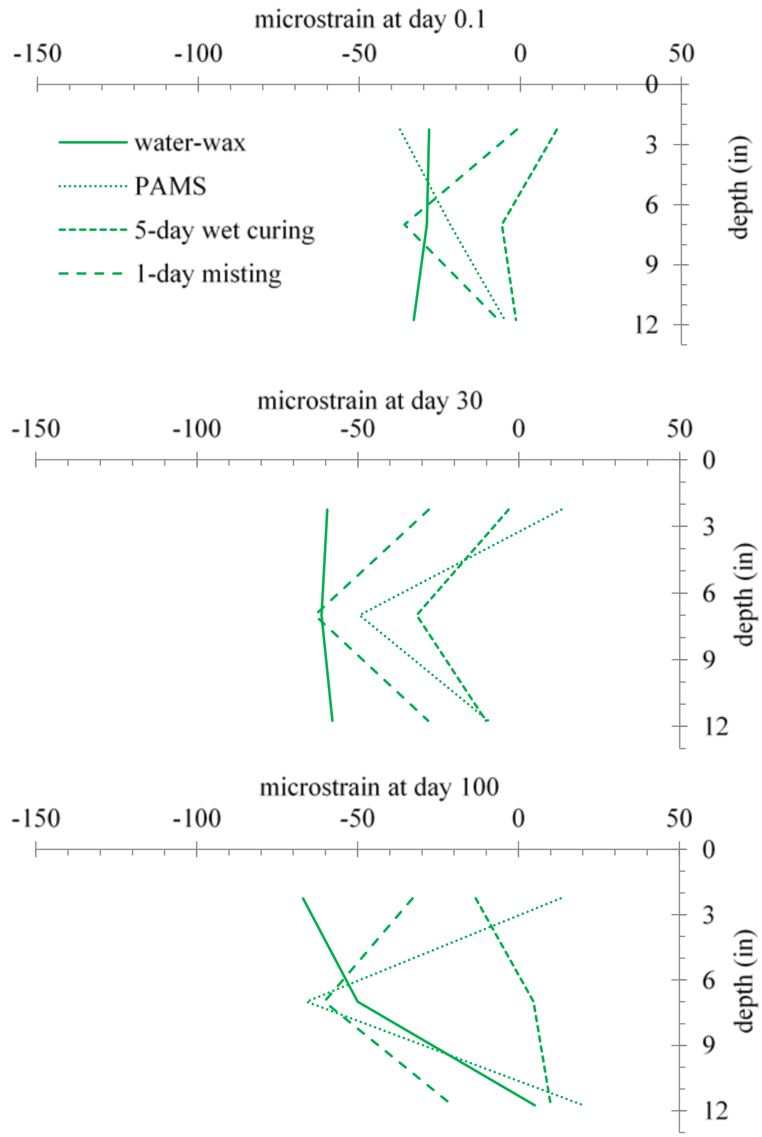


Figure 7.14 - Strain profiles for *location 2* at 0.1, 30, and 100 days after paving

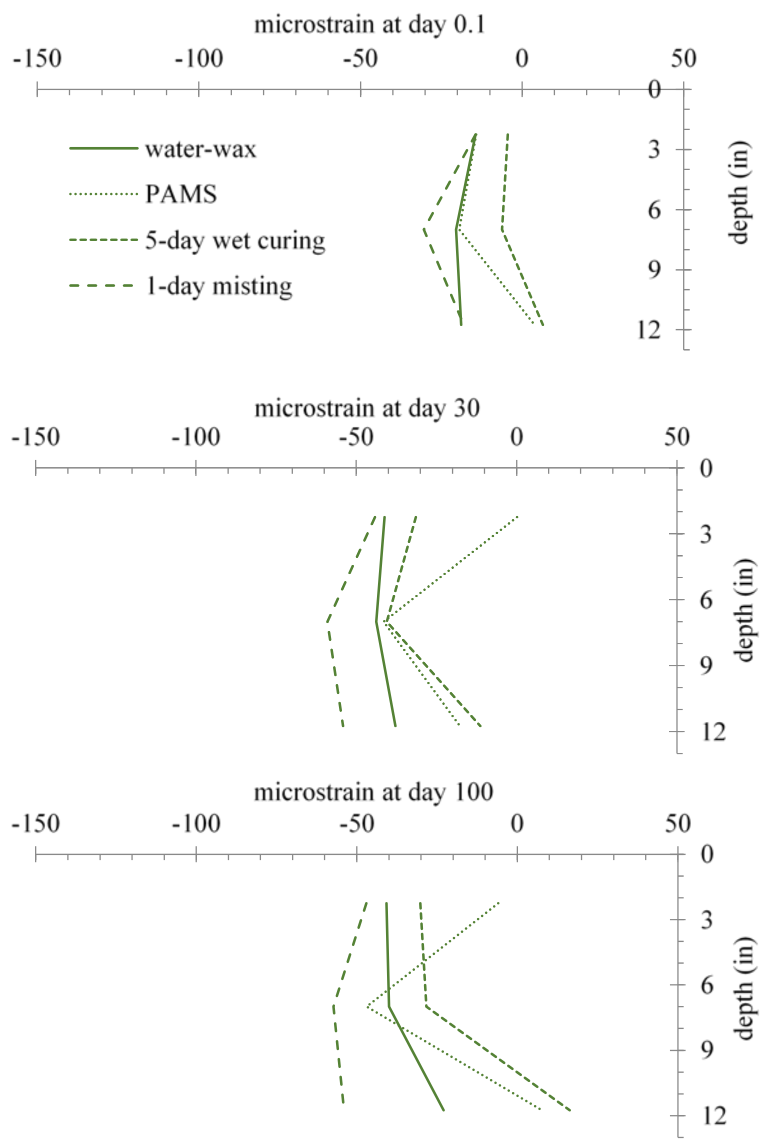


Figure 7.15 - Strain profiles for *location 3* at 0.1, 30, and 100 days after paving

Based on Figs. 7.13, 7.14, and 7.15 it appears that there is significantly more strain gradient and this will in-turn cause additional curling at the edge of the pavement than is happening in the middle. Also, the magnitudes of shrinkage are not more than 100 microstrain. This is not a large amount of shrinkage. As stated previously, the temperature gradients at these sections are almost constant and so this difference in strain is likely caused by differences in moisture. Also, there does not seem to be large differences in the gradients in the pavements. This reinforces the previous work that the type of curing does not seem to have an impact on the curling of the pavement. During construction the water table was observed to be high in this region. This means that the bottom of the pavement is likely swelling while the top dries. This can create a strain gradient in the pavement and leads to the increased curling that is observed. This curling may become a problem over time and should be watched closely.

As mentioned earlier in the instrumentation section, several *RH* sensors at different depths of the slab were used at the marked locations A and B in Fig. 7.5. These sensors broke due to the very high *RH* over the depth of slab after about 30 days. This might be due to either a very high water level that caused the sensors to become saturated and fail or the penetration of the water through the drilled holes.

7.5 Summary

In this chapter the instrumentation of a concrete pavement has been reviewed. The pavement was instrumented to measure strain, temperature, and relative humidity at the pavement center, edge, and then at an intermediate point. The data shows that the temperature gradient is almost constant in the pavement. However, the strain gradient is not. The strain at the edge of the pavement is much higher than the middle. This higher strain gradient at the edge of the pavement may be caused by less support at the edge. Also, this gradient is likely caused by a significant moisture gradient in the pavement that is caused by a very high water table.

These findings confirm the previous laboratory and field research on previous phases of this project that showed that in Oklahoma the RH is high and so the pavement does not experience significant drying. Furthermore, the work proves that the primary slab movement is caused by moisture gradients and not temperature changes. Finally, the work has confirmed that the curing method used for the pavement does not have a significant impact on the amount of curling that occurs, at least not in a moist environment such as Oklahoma.

8 Conclusions and Recommendations

8.1 Conclusions

The phase 3 of this research project focused on the local validation and calibration of the MEPDG for concrete pavement design in Oklahoma. The following conclusions can be drawn from this phase:

1. There are only a small number of LTPP segments in Oklahoma with concrete pavements. The local validation/calibration of the MEPDG in Oklahoma has to rely on PMS data.
2. This project focused on the Oklahoma PMS segments that have 10 to 25 years of service. Overall, these concrete pavements in Oklahoma highway network are in good conditions, with very few segments approaching the design limit condition. This observation to some extent indicates that the current pavement design method used by ODOT is at least on the conservative side, but it also makes it challenging to evaluate the MEPDG due to the overall low distress level in Oklahoma concrete pavements.
3. The JPCP transverse joint faulting model in the MEPDG seems to be adequate for non-doweled JPCPs based on the 4 LTPP JPCP segments in Oklahoma. However, the data collected from the PMS showed that the MEPDG under-predicts transverse joint faulting for dowed JPCPs in Oklahoma.
4. The transverse cracking model in the MEPDG seems to be adequate from the PMS data, although this conclusion should be revisited as more transverse cracking data becomes available in the PMS.
5. Currently, there is not enough pavement performance data to determine the accuracy of the MEPDG punchout model for Oklahoma CRCPs. More segments are needed with close to 10 punchouts per mile or more. The CRCP punchout model in the MEPDG need to be revisited in the future.
6. The JPCP transverse joint faulting model have been calibrated using the Oklahoma PMS data. With the local calibration factors, the predicted faulting for the doweled JPCPs matched better with the field measured faulting data.
7. Comparison analysis was performed by designing typical JPCP and CRCP structures in Oklahoma using the 1993 AASHTO guide, the national default MEPDG, and the locally calibrated MEPDG. The required concrete slab thickness was determined using the three methods at six different truck traffic volumes and three different base courses. In most of the design cases, both the default and the locally calibrated the MEPDG produced thinner designed concrete thicknesses than the 1993 AASHTO Guide, except for CRCP over unbound aggregate base.
8. At lower truck volumes (AADTT<=2500), doweled JPCP design in the MEPDG is controlled by transverse cracking at lower truck volumes. At higher truck volumes, faulting may start to control the design, in which case the locally calibrated MEPDG will produce thicker PCC thicknesses than the national default model.

9. For CRCP over unbound aggregate base pavement, the national default MEPDG model produces similar design thicknesses to the 1993 AASHTO Guide. However, much thinner (up to 4 inches) PCC thicknesses were determined by the MEPDG when AC or cement stabilized bases are used.
10. Data collected from the I-44 instrumented section proved that the primary slab movement is caused by moisture gradients and not temperature changes. The work also confirmed that the curing method used for the pavement does not have a significant impact on the amount of curling that occurs, at least not in a moist environment such as Oklahoma.

8.2 Recommendations

1. To facilitate the local calibration/validation and other research on the MEPDG, The PMS office should continue to collect pavement condition data. It is important to ask the contractor use the same definition of pavement distress as the LTPP survey protocol. For example, the transverse joint faulting values that are lower than 0.2 inch should also be reported.
2. Due to the limited data available to this research, the JPCP transverse cracking model and the CRCP punchout model need to be revisited in the future as more pavement distress data become available. It is recommended that ODOT continue to update pavement distress data for the 50 selected segments after each survey cycle. Once the JPCP transverse cracking model and the CRCP punchout model are calibrated, the IRI model for both JPCP and CRCP can be validated and calibrated.
3. The ODOT pavement design office may start to use the MEPDG to design JPCPs and CRCPs and use the 1993 Guide as a reference. The local calibration factors for the faulting model should be used when designing doweled JPCPs. When the PCC thickness determined by the MEPDG is 3 inches or more different from the 1993 design, the 1993 design thickness should be used.
4. Concrete pavement design using the MEPDG is very sensitive to the slab/base friction parameters. The suggested input values in the MEPDG were determined by matching the field pavement performance instead of experimental results. In this project, some preliminary laboratory work was done to measure the coefficient of friction between concrete and two Oklahoma base materials. The measured coefficient of friction μ values are lower than the default input values suggested by the MEPDG. However, when using the measured μ values in the CRCP analysis, MEPDG predicted unreasonably high punchouts. Due to the limitation of the laboratory tests (e.g., not representing long term field condition), the researchers recommend ODOT use the default input values in the MEPDG in CRCP designs.

9 References

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Appendix A Selected Pavement Segments

Table A.1 Selected JPCP Segments

Control	Constr. Date	Project	AADTT	Structure	Crack (%)	Fault (In.)	IRI (in./mi.)
03-08	1/1/1992	BRF236(112)	405	10" DJCP + 3" AC	0	0.086	175.7
10-04	4/1/1993	MAF89(036)	928	10" DJCP + 4" AC	0.646	0	122.2
11-02	9/1/1996	SAP11(335)	640	9" DJCP + 4" CSB	0.554	0.038	83.7
16-42	2/1/2004	S897(3)	386	10" DJCP + 4" AC	0.861	0	102.5
16-53	8/1/1995	MAM7780(001)	142	9" DJCP + 2.5" AC	0.768	0.054	197.5
19-18	3/1/1992	MBZY119B(030)	200	11" DJCP + 2" CSB	0	0.048	159.1
33-06	6/1/1997	STP33A(208)	596	10" DJCP + 4" AC	0	0.034	112.4
33-08	10/1/1997	STP33A(207)	596	10" DJCP + 4" AC	0	0.047	157.6
40-06	6/1/2003	STPY140A(001)	792	10" DJCP + 3" AC	0.728	0	130.3
40-59	6/1/2003	STPY140A(007)	748	10" DJCP + 3" AC	0	0.034	106.9
42-30	6/1/1998	n/a	1828	10" DJCP + 4" AC	0	0.045	102.1
46-03	10/1/2003	NHY13N(022)	3581	12" DJCP + 3" AC	1.421	0.085	138.3
48-02	12/1/1993	F59(076)	720	10" DJCP + 4" AC	0	0.039	87.8
49-02	11/1/2000	CIP149N(019)	2480	12" DJCP + 2" AC + 6" Agg	0	0.106	228.3
49-08	9/1/2000	STPY49A(336)	910	10" DJCP + 4" AC	2.185	0	211.5
49-22	8/1/1990	F398(035)	1340	10" DJCP + 4" AC	0	0.130	214.9
55-70	10/1/1999	CIP155N(058)	4203	9" DJCP + 4" AC + 12" Agg	0.546	0.131	237.3
56-03	6/1/2000	IMY40-6(236)240	3360	10" DJCP + 2" AC + 6" Agg	8.012	0	68.8
56-11	7/1/2001	CIP156N(013)	1593	10" DJCP	5.981	0.057	194.9
58-14	7/1/1997	MASTP08(054)	660	10" DJCP + 4" AC	0.676	0.050	123.5
58-24	12/1/1997	BRF-8(045)	1430	9" DJCP + 4" AC	1.107	0	126.1
62-03	2/1/2004	CMAY021N(005)	933	10" DJCP + 2" AC	0.702	0	109.4
63-40	1/1/1998	IMY040-05(325)	1776	10" DJCP + 4" CSB	2.029	0.039	130.7
63-41	1/1/1998	IMY040-05(325)	1776	10" DJCP + 4" CSB	0	0.052	112.2
63-52	10/1/1996	DSB63B(376) ?	824	8" DJCP + 3" AC	0	0.044	171.3
68-10	5/1/2001	STP068C(237)	10	10" DJCP + 4" AC	1.480	0	106.0
72-18	11/1/1997	n/a	1911	10" DJCP + 4" AC	0.947	0	129.7
73-14	5/1/2004	BHFY173B(014)	588	10" DJCP + 4" AC	0	0.035	104.6
74-08	10/1/1997	NH481(069)	842	10" DJCP + 4" AC	1.728	0	121.6
77-04	8/1/1990	F282(193)	302	8" DJCP + 3" AC	2.841	0.049	172.2

Table A.2 Selected CRCP Segments

Control	Constr. Date	Project	AADTT	Structure	Punchouts per mile
03-02	5/1/2001	NHY13N(005)	405	12" CRCP + 4" Agg	5.276
03-04	1/1/1992	F236(112)	405	10" CRCP + 4" Agg	5.000
05-04	8/1/1995	IM40-1(062)25	3604	10" CRCP + 4" CSB	18.817
16-53	2/1/1998	STP16A(278)	416	9" CRCP + 6" AC	5.051
49-02	5/1/1991	BRF593(236)	3000	10" CRCP + 3" AC	7.971
49-22	8/1/1990	F398(035)	1340	10" CRCP + 4" AC	4.054
52-33	5/1/1990	MAIR 35-4(111)	1575	10" CRCP + 4" CSB	1.982
55-15	9/1/1994	IR-353(049)125	4290	10" CRCP + 4" Agg + 12" Agg	31.818
55-15	2/1/2001	IR-353(049)125	5645	10" CRCP + 4" Agg + 12" Agg	23.360
61-03	12/1/1990	MAF 186 183	2700	10" CRCP + 4" CSB	3.125
61-04	9/1/1993	MAF 186 180	2970	10" CRCP + 4" CSB	2.432
66-05	10/1/1998	NH030N(001)	1422	10" CRCP + 4" CSB + 6" Agg	4.444
66-06	10/1/1998	NH030N(001)	1422	10" CRCP + 4" CSB + 6" Agg	3.571
68-22	1/1/1991	IR-40-6 (222)298	3255	10" CRCP + 4" CSB + 6" Agg	3.150
72-08	5/1/1991	IR-44-2(328)221	4800	12" CRCP + 4" CSB	8.491
72-74	1/1/1994	STPY 72C (404)	555	10" CRCP + 4" Agg	1.176
72-78	5/1/1991	IR-44-2(328)221	4800	12" CRCP + 4" CSB	6.897
72-81	10/1/1998	NH 30N (001)	1422	10" CRCP + 4" CSB + 6" Agg	1.961
72-93	9/1/1990	F 15 218	1960	9" CRCP + 4" CSB	1.828
75-02	8/1/1993	IM40-2(119)	3800	10" CRCP + 4" CSB + 4" Agg	6.250

11 Appendix B Design Examples

Two design examples (one JPCP and one CRCP) are provided in this appendix. The suggested input values for concrete material properties and the slab/base friction are built in to two material database files: “JPCP-OK.xml” and “CRCP-OK.xml”. These design parameters can be imported into the Pavement-ME program.

11.1 Example 1. New JPCP over Cement Stabilized Base

Design input:

Project:
Type: New Jointed Plain Concrete Pavement (JPCP)
Design life: 20 years
Design Reliability: 90%
Pavement Construction: October 2016
Traffic Opening: November 2016
Traffic Input:
AADTT: 1000
Truck in the design direction: 50%
Truck in the design lane: 90%
Growth factor: 3.0%
Structure and Materials:
Layer 1: JPCP: (thickness to be determined)
Layer 2: 4” cement stabilized base: Mr = 100,000 psi
Layer 3: 10” Subgrade: Mr = 5,500 psi (MEPDG requires to have at least two unbound layers)
Layer 4: Subgrade: Mr = 5,500 psi
Climate:
Weather station: Tulsa (53908)

Design procedure:

1. Open the Pavement-ME program and create a new project.
2. Input the above parameters in the appropriate windows. Leave all other inputs as default.
3. Right click “Layer 1 PCC: JPCP Default” and select “Import”. Locate the “OK JPCP” material file and import the input parameters to the layer. (See Figure A.1)

4. Determine the design thickness of JPCP by gradually increasing the PCC thickness until all distress criteria are passed.

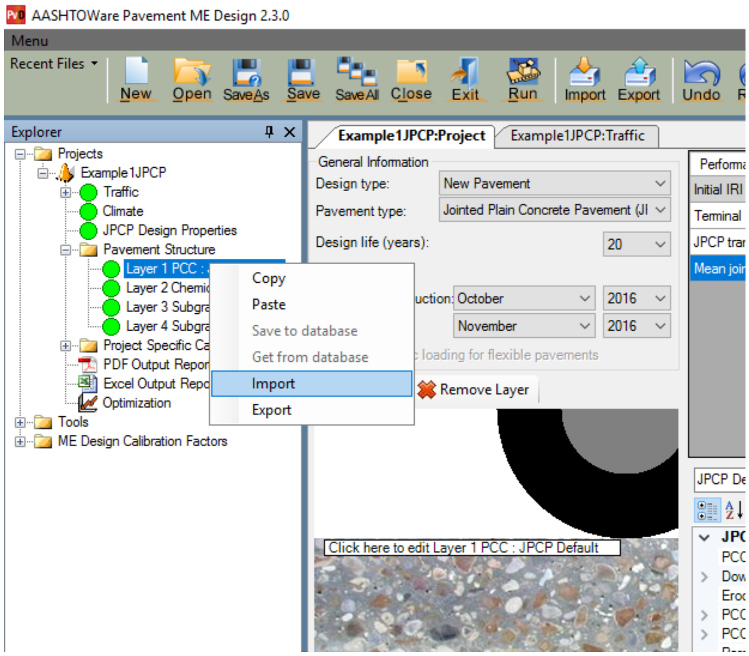

Figure A.1 Import the concrete material input

Design output (10” JPCP):

After gradually increasing the JPCP thickness from 8 inches, 1 inch at a time, the minimum design thickness was determined as 10”. The design output is shown in Figure A.2.

Distress Prediction Summary — JPCP thickness = 8"

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	226.36	90.00	56.96	Fail
Mean joint faulting (in)	0.12	0.08	90.00	99.67	Pass
JPCP transverse cracking (percent slabs)	15.00	117.20	90.00	0.00	Fail

Distress Prediction Summary — JPCP thickness = 9"

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	160.63	90.00	94.30	Pass
Mean joint faulting (in)	0.12	0.07	90.00	99.89	Pass
JPCP transverse cracking (percent slabs)	15.00	52.16	90.00	5.24	Fail

Distress Prediction Summary — JPCP thickness = 10"

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	116.20	90.00	99.95	Pass
Mean joint faulting (in)	0.12	0.06	90.00	99.97	Pass
JPCP transverse cracking (percent slabs)	15.00	7.33	90.00	99.83	Pass

Figure A.2 Design output at different JPCP thicknesses

11.2 Example 2. CRCP over Asphalt Concrete Base

Design input:

<u>Project:</u>
Type: New Continuously Reinforced Concrete Pavement (CRCP)
Design life: 20 years
Design Reliability: 90%
Pavement Construction: October 2016
Traffic Opening: November 2016
<u>Traffic Input:</u>
AADTT: 1000
Truck in the design direction: 50%
Truck in the design lane: 90%
Growth factor: 2.5%
<u>Structure and Materials:</u>
Layer 1: JPCP: (thickness to be determined)
Layer 2: 3” Asphalt Concrete Base: Binder grade = PG 64-22
Layer 3: 10” Subgrade: Mr = 5,500 psi (MEPDG requires to have at least two unbound layers)
Layer 4: Subgrade: Mr = 5,500 psi
<u>Climate:</u>
Weather station: Tulsa (53908)

Design procedure:

1. Open the Pavement-ME program and create a new project.
2. Input the above parameters in the appropriate windows. Leave all other inputs as default.
3. Right click “Layer 1 PCC: CRCP Default” and select “Import”. Locate the “OK CRCP” material file and import the input parameters to the layer. (See Figure A.3)
4. Determine the design thickness of JPCP by gradually increasing the PCC thickness until all distress criteria are passed.

Design output:

After gradually increasing the CRCP thickness from 8 inches, 1 inch at a time, the minimum design thickness was determined as 9”. The design output is shown in Figure A.4.

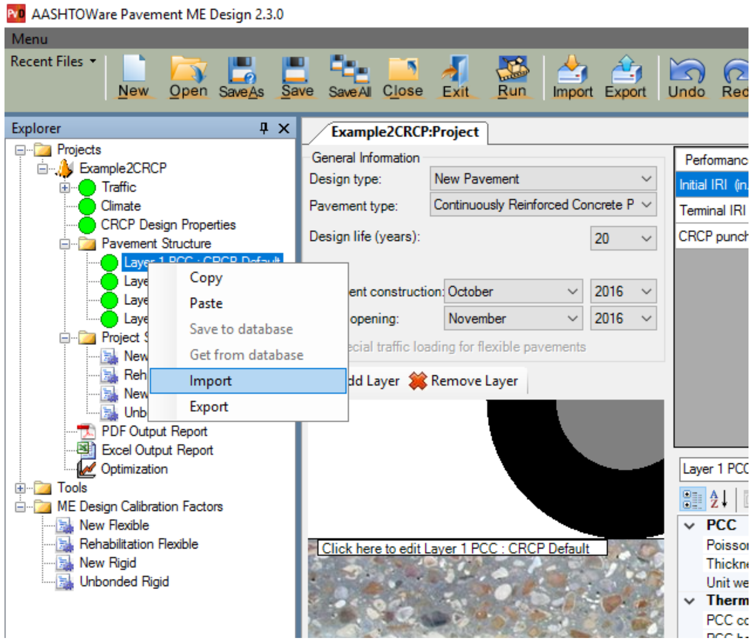

Figure A.3 Import the concrete material input

Distress Prediction Summary JPCP thickness = 8”

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	128.57	90.00	99.69	Pass
CRCP punchouts (1/mile)	10.00	23.44	90.00	37.88	Fail

Distress Prediction Summary JPCP thickness = 9”

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	91.48	90.00	100.00	Pass
CRCP punchouts (1/mile)	10.00	2.03	90.00	100.00	Pass

Figure A.4 Design output at different CRCP thicknesses