Technical Paper

ISSN 1997-1400 Int. J. Pavement Res. Technol. 7(6):405-416

Evaluation of AASHTO Mechanistic-Empirical Pavement Design Guide for Designing Rigid Pavements in Louisiana

Zhong Wu[1+], Danny X. Xiao[1], Zhongjie Zhang[1], and William H. Temple[2]

Abstract: This paper presents a recent study on using AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) design software (Pavement ME™) to evaluate the performance of typical Louisiana rigid pavement structures as compared to the existing pavement performance data available in the pavement management system (PMS). In total, 19 projects with two pavement structure types, Portland cement concrete (PCC) over unbound base and PCC over asphalt mixture blanket, were analyzed. Results show that the national model over-predicts transverse cracking and under-predicts joint faulting. Therefore, a preliminary calibration was conducted to adjust Pavement ME for Louisiana's condition. In addition to comparing the measured and predicted performance, the recommended thickness from the current and the new design methods was also compared. It was found that the two design methods are comparable with an average difference of 2 cm or 7 percent (Pavement ME requires a thinner pavement). At the end of this paper, problems, challenges and possible solutions for fully implementing the new design method are discussed.

DOI: 10.6135/ijprt.org.tw/2014.7(6).405

***Key words**: Local calibration; MEPDG; Pavement management system; Rigid pavement.*

Introduction

Many state highway agencies in the United States are moving towards the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) by conducting sensitivity analysis, material characterization, traffic data preparation, local calibration, parallel design, etc. Following this trend, the State of Louisiana has completed the evaluation of flexible pavement models in MEPDG with Louisiana typical structures, material inputs and climate conditions [1]. This paper presents the results of a continuing effort to evaluate rigid pavement models towards the implementation of MEPDG in Louisiana.

Some states have evaluated rigid pavement models for their local conditions. But literature presents a mixed result. Some states found that rigid pavement models in MEPDG fit reasonably well with their local conditions [2]. While some others concluded that models had to be local calibrated before final implementation [3, 4].

Washington State department of transportation (WSDOT) completed their initial calibration of MEPDG rigid pavement models in 2006 in aim to find a suitable modeling tool for the purpose of prioritizing rehabilitation and reconstruction efforts [5]. It was found that calibration factors for WSDOT were quite different from the national default values. In addition, more than one set of calibration factors were needed for the entire network. Overall, the MEPDG was calibrated, not perfectly but acceptable, to predict the performance of rigid pavements in Washington. Because previous calibrations were conducted using old MEPDG software, a recalibration effort was made to update the pavement design catalog for WSDOT [6]. Only the cracking model was assigned a set of calibration factor that was different from the national default values. Default calibration factors for faulting and roughness models were used and resulted in estimates that matched well with the pavement management system (PMS) data in magnitude.

In the Missouri study, a total of 24 jointed plain concrete pavement (JPCP) projects were selected covering a wide range of design characteristics and distress levels [2]. The study found that there were too few Long Term Pavement Performance (LTPP) and PMS segments with appreciable transverse fatigue cracking to validate the global calibration values using statistical methods. Instead, the transverse cracking model was evaluated by categorizing measured and predicted slab transverse cracking into eight groups and determining how often the measured and predicted cracking would fall within the same group. It was confirmed that 74 percent of them fell within the same group, and hence, the JPCP transverse cracking model was reasonably well for Missouri's pavements. A significant bias was found for joint faulting. It was further revealed that (1) doweled JPCP with joint spacing less than 6 m exhibited very little faulting, and (2) MEPDG over-predicted faulting for JPCP that are not doweled or have long joint spacing (i.e., joint spacing > 6 m). The smoothness (International Roughness Index, IRI) model with national calibrated coefficients performed very well for Missouri.

North Carolina conducted a preliminary evaluation using three pavement sections [3]. It was found that the national model over-predicted both transverse cracking and faulting. Calibration could reduce the bias. But additional pavement sections and performance data were needed prior to NCDOT's consideration for adoption of the recommended calibration coefficients.

Iowa evaluated rigid pavement models using 35 JPCP sections, of which 70% were utilized for calibration and 30% were used for verification [4]. Historical performance data were retrieved from the pavement management information system. The study found that the national calibrated DARWin-ME (now named Pavement ME)

[1] Louisiana Transportation Research Center, 4101 Gourrier Avenue, Baton Rouge, LA 70808, USA.
[2] Concrete & Aggregates Association of Louisiana, 18212 E. Petroleum Drive, Bldg. 7, Baton Rouge, LA 70809, USA.
[+] Corresponding Author: E-mail zhongwu@ltrc.lsu.edu
Note: Submitted April 12, 2014; Revised August 8, 2014; Accepted August 10, 2014.

under-predicted joint faulting but over-predicted transverse cracking and IRI. Researcher then adjusted local calibration coefficients using both linear and nonlinear optimization approaches. The accuracy of all models was successfully improved by local calibration.

Regarding the impact on design using the AASHTO 1993 Design method and the new M-E method, most studies found that slab thickness is slightly thinner when designed using MEPDG [7, 8]. Timm [9] concluded that slab thicknesses from the new design approach were typically 9 percent thinner.

As one of the states that have adopted MEPDG, Indiana prepared a design manual that provides default values (general information, materials and traffic) for design engineers to use [10]. Although no calibration coefficient needs to be adjusted, MEPDG has worked quite well for the state of Indiana. INDOT reported a great amount of cost saving because MEPDG generally requires a thinner pavement as compared to the AASHTO 1993 Design [11].

Overall, literature suggests the necessity of evaluating MEPDG to local conditions before adopting the new method as the standard design tool. In addition, such an effort is even more important for the state of Louisiana because no LTPP section in Louisiana was available during the national calibration of MEPDG rigid pavement models [12].

Objectives

The objectives of this research were (1) to evaluate, and calibrate if needed, the performance of typical Louisiana rigid pavements using the latest AASHTO MEPDG software (Pavement ME™ version 1.3) and pavement management system data, and (2) to assess the difference between the current design method and the new mechanistic-empirical method on designing rigid pavements in Louisiana.

Data Collection and Input Strategy

Ideally MEPDG should be evaluated with project-level data such as LTPP and accelerated pavement testing, as recommended by the MEPDG Local Calibration Guide [2]. But Louisiana has only four LTPP sections, among which only one is rigid pavement, and the only one project is a joint reinforcement concrete pavement (JRCP), a type that is no longer widely used. Some states who faced a similar problem usually supplemented the LTPP sections with data from their pavement management system [2, 4, 13]. Otherwise, the evaluation effort has to rely on field distress survey on a limited number of sections [14] and a couple of years of data preparation before any conclusion could be made [15]. Like many states, Louisiana has a comprehensive pavement management system with data back to 2001. Although with some limitations [16], the PMS data represent to some extent the general pavement performance under local conditions and therefore could serve as the data source for an evaluation of MEPDG models. This study was completed by utilizing data from the pavement management system stored at Lousiana Department of Transportation and Development (LADOTD).

Evaluation Sections

Starting from all available rigid pavement sections in Louisiana, this study screened them to find suitable sections for further evaluation. These criteria were followed during this process:

- Since plan changes during construction may alter the initial plan, only projects with as-built plan were included in this study.
- The history of construction, maintenance and rehabilitation within a control section was checked. This was done by retrieving data from the tracking of projects (TOPS) database and reviewing historical right-of-way images through a software named Visiweb.
- Performance data after major rehabilitation were excluded for further analyses. Projects with less than three performance data points between 2001 and 2011 were excluded.
- Pavements constructed before 1970 were not included.
- Projects outside of a city limit were treated with high priority. Special attention was given to projects inside a city to assure a consistent pattern of traffic.
- Right-of-way images were reviewed to select a section with consistent characteristics. A suitable section should not have any major intersections or bridges.

In total, 19 projects were determined as suitable evaluation sections (Fig. 1), including nine Interstate sections, eight US highway sections and two LA state highway sections. Although more than 100 projects were initially identified as candidates, most of them were eliminated due to several reasons including (1) being a city street with many intersections and possibly a complicated traffic pattern; (2) although the tracking of projects database indicates a project as rigid pavement, right-of-way images show it as asphalt pavement; and (3) projects rehabilitated after 2005 have less than three data points in PMS. All analyses in this study were based on the selected 19 projects.

Traffic

Traffic data were retrieved from several database in LADOTD. Traffic volume (annual average daily traffic, AADT) was available in a database named TATV. Direction distribution, lane distribution and vehicle class distribution were based on the original traffic assignment, which was stored in an electronic document management system called Content Manager.

Regarding to load spectra, this study evaluated both the national default and the LA default which was derived from portable weigh in motion (WIM) data [17]. It was found that the developed LA spectra had lighter loading than the national default and only generated about 25% equivalent single axle load (ESAL) comparing to the original AASHTO 1993 design. This was mainly due to the short data collection time (48 hours) and the limitation of portable WIM sensors (extremely sensitive to temperature). On the contrary, ESAL from the national default matched well with the ESAL used in the original design. Considering the limitations of portable WIM data [17], a decision was made during this study to use the national default load spectra until better quality data are made available. Louisiana is implementing a strategic plan which will install several permanent WIM stations throughout the state [17]. Other traffic data such as hourly adjustment, monthly adjustment, axle per truck and axle configuration were national defaults.

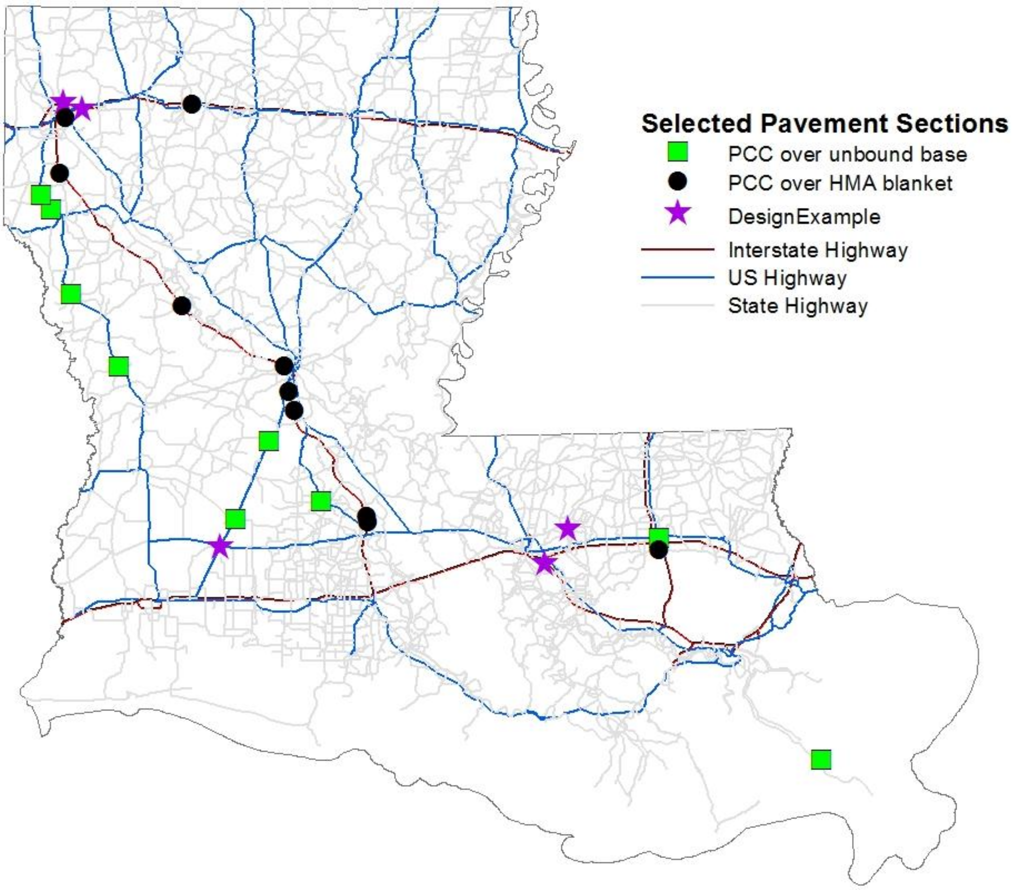


Fig. 1. Location of the Selected Projects.

Table 1 lists the year opened to traffic, initial two-way AADT, and initial annual average daily truck traffic (AADTT) in the design lane. Among the 19 projects, 11 of them were opened to traffic between 1980 and 1999; six projects were built in 2000s. As of the end of 2013, the average age of these projects is 21 years with a standard deviation of 9 years. Initial two-way AADT ranges from 4,600 to 23,700 vehicle per day. Truck percentage ranges from 5% to 35%.

Climate

The location information (longitude, latitude, and elevation) was obtained from the PMS at the mid-point of each roadway segment. The climate station close to the project or a virtual station generated from multiple nearby climate stations was determined based on the GPS coordinate. Water table depth was assumed to be 1.5 m.

Pavement Structure and Materials

Pavement structure was obtained from as-built plans. Lane width, slab width and shoulder type were retrieved from the highway need system and verified from right-of-way images. Although there is a database named MATT in LADOTD that stores materials information, very few data related to MEPDG design inputs were found available. The MATT System was built in 1970s before the implementation of Superpave. After LADOTD adopted Superpave in 2000s, very few data were entered to the system since the database was not updated to accommodate Superpave parameters. Therefore, this study started with national recommended material inputs and adjusted them to Louisiana's practice. In detail, special attention was given to the following:

- Joint spacing of 6 m has been used in Louisiana for many years.
- Dowel bars are required for all jointed concrete pavements in Louisiana. The dowel diameter is 31.75 mm if slab is less than 25.4 cm thick. Otherwise, 38.1 mm dowel bars should be used.
- Aggregates for PCC slab are limestone.
- The coefficient of thermal expansion is 9.9×10^{-6} /°C.
- 28-day modulus of rupture for PCC slab is 4.1 MPa.
- 28-day elastic modulus for PCC slab is 29.0 GPa.
- Resilient modulus for soil cement is 689 MPa.
- Resilient modulus for unbound base or rubblized PCC is 186 MPa.
- Erodibility index was assumed to be level 3 erosion resistant.
- PCC slab and base has a full friction with friction loss at 600 months (50 years).

During the process to determine materials inputs, sensitivity analysis of a typical Louisiana rigid pavement structure was conducted. It was found that MEPDG is very sensitive to coefficient of thermal expansion, slab thickness, joint spacing and PCC strength. In addition, results showed that the Level 3 input combination of modulus of rupture and elastic modulus could predict a better match with Level 1 input than using Level 3 compressive strength. This agrees with the recommendation by Schwartz et al. [18]. Hence, this

Table 1. General Information about Selected Projects.

Structure Type	Project ID	District	Parish	Route	Open to Traffic Date	$AADT_0$	$AADTT_0$ Per Lane
PCC over unbound base	013-08-0015	62	53	US 190	3/3/1975	8,660	226
	014-03-0028	07	2	US 165	5/30/2003	6,100	414
	014-05-0020	08	40	US 165	1/14/2005	9,500	844
	025-01-0027	08	58	US 171	3/29/2005	9,350	1,319
	025-03-0025	08	43	US 171	9/5/2003	6,600	897
	025-06-0027	04	16	US 171	7/19/2002	6,300	395
	025-06-0031	04	16	US 171	1/23/2003	4,600	649
	062-05-0018	02	38	LA 23	8/1/1974	5,900	382
	066-07-0030	03	20	US 167	6/29/1995	14,150	739
PCC over HMA blanket	451-04-0029	04	7	I-20	7/5/1996	22,600	4,133
	452-90-0039	62	53	I-55	8/1/1990	11,400	1,310
	455-02-0003	03	49	I-49	11/1/1983	10,200	1,141
	455-02-0004	03	49	I-49	9/29/1987	9,900	1,293
	455-05-0021	08	40	I-49	5/14/1992	9,386	981
	455-05-0022	08	40	I-49	3/27/1991	9,385	981
	455-05-0026	08	40	I-49	7/24/1992	23,700	1,857
	455-06-0008	08	35	I-49	2/2/1988	12,000	1,129
	455-07-0012	04	16	I-49	9/25/1986	12,000	1,442
	808-07-0029	04	8	LA 3105	4/6/1993	15,190	555

Note: $AADT_0$ = initial annual average daily traffic, $AADTT_0$ = initial annual average daily truck traffic

study used the combination of modulus of rupture and elastic modulus instead of using compressive strength.

Table 2 lists the pavement structure of the selected 19 projects. Two major pavement structure types are identified: PCC over unbound base, and PCC over HMA blanket (Fig. 2). In general, it appears that HMA blanket was used for high volume roads (e.g. Interstates) and unbound base was used for roads with less traffic (e.g. state roads). The thickness of PCC slab ranges from 23 to 33 cm. The majority is 25 and 28 cm. It is also noticed that most of rigid pavements in Louisiana are widened to 4.3 or 4.6 m. In terms of shoulder types, eight projects have tied PCC shoulder and others have either asphalt shoulder or curb and sidewalk.

Design criteria and reliability levels recommended by AASHTO were used in this study [19]. US highways were considered as principal arterials and LA highways as collectors.

Interpreting Measured Performance Data

LADOTD began collecting pavement distress data by windshield surveys in the early 1970s. Since 1995, LADOTD has used the Automatic Road Analyser (ARAN) to conduct network-level pavement condition surveys once every two years [20]. Pavement distress data collected for rigid pavements include IRI, faulting, longitudinal cracking, transverse cracking, and patching.

Estimating the Percentage of Cracked Slabs

The LTPP database includes not only the length of transverse cracking in low, moderate and high severity but also the number of transverse cracking in the three severities. In the national calibration, percent slabs cracked was computed by summing the total number of transverse cracks observed (all severities) for a given test section and dividing it by the number of slabs within the test section [12]. Unfortunately, LA-PMS does not report the number of transverse cracks. In addition, LA-PMS does not indicate the location of a transverse crack—may or may not be mid-slab crack, as defined in Louisiana cracking protocol [21]:

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

To match the definition of MEPDG, this study assumed (1) transverse cracking in PMS are mid-slab cracks, and (2) each slab has only one transverse crack. Eq. (1) was used to estimate the percentage of cracked slab in a 161-m section.

$$PercentCrackedSlab = \frac{(TransL + TransM + TransH)/SingleCrackLength}{161/JointSpacing} * 100 \quad (1)$$

By manually evaluating pavement images of several sections, it was determined reasonable to assume 3.6 m as the single crack length. In fact, it was found that longitudinal cracking was more prominent in Louisiana than transverse cracking. Nevertheless, pavement images show that the crack would pass through the whole slab if a mid-slab transverse crack was observed. In case the calculated percentage was over 100 percent, the value was capped at 100 percent.

Estimating Joint Faulting

LADOTD collects faulting for concrete pavements in the outer lane. ARAN reports faulting for concrete pavements at both joints and transverse cracks, wherever an elevation difference is detectable.

Table 2. Pavement Structure of Selected Projects.

Structure Type	Project ID	Pavement Structure			Slab Width (m)	Shoulder Type
		PCC (cm)	Base	Subbase		
PCC over unbound base	013-08-0015	22.9	Granular Base 15 cm		3.7	Curb
	014-03-0028	27.9	Crushed Stone or recycled PCC 20 cm		4.3	PCC
	014-05-0020	25.4	Stone or Recycled PCC 20 cm		4.3	Curb
	025-01-0027	27.9	Stone or Recycled PCC 20 cm		4.6	PCC
	025-03-0025	27.9	Stone or Recycled PCC 25 cm		4.6	PCC
	025-06-0027	27.9	Class I Stone base 20 cm		4.6	PCC
	025-06-0031	27.9	Stone or Recycled PCC 20 cm		4.6	PCC
	062-05-0018	27.9	Stone or Recycled PCC 20 cm		3.7	HMA
	066-07-0030	25.4	Stone or Recycled PCC 15 cm		4.3	Curb
PCC over HMA blanket	451-04-0029	33.0	HMA Type 5B 5 cm	Selected Material 5 cm	4.6	PCC
	452-90-0039	30.5	HMA Type 5B 5 cm	Cement Treatment 22 cm	4.6	HMA
	455-02-0003	25.4	HMA Type 5B 5 cm	Selected Material 15 cm	4.6	HMA
	455-02-0004	25.4	HMA Type 5B 5 cm	Cement Treatment 15 cm	4.6	PCC
	455-05-0021	25.4	HMA Type 5B 5 cm	Embankment 20 cm	4.6	HMA
	455-05-0022	25.4	HMA Type 5B 5 cm	Embankment 28 cm	4.6	HMA
	455-05-0026	33.0	HMA Type 5B 5 cm	Lime Treated Subgrade Working Table 15 cm	4.6	PCC
	455-06-0008	25.4	HMA Type 5B 5 cm	Cement Treatment 15 cm	4.6	HMA
	455-07-0012	25.4	HMA Type 5B 5 cm	Cement Treatment 15 cm	4.6	HMA
	808-07-0029	27.9	HMA Type 5B 5 cm	Subgrade Treatment 22 cm	4.3	Curb

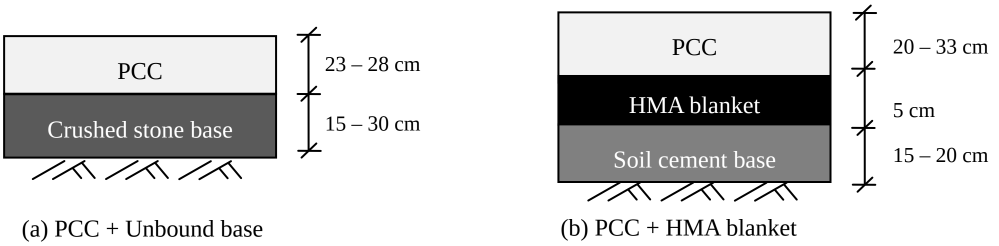


Fig. 2. Typical rigid pavement structures in Louisiana.

The fault minimum was set at 5 mm during data collection; therefore, only faulting over 5 mm were reported and stored in the pavement management system. A zero value in PMS could mean either a perfect joint or a small fault that was less than 5 mm.

For every 161 m interval, the pavement management system includes five faulting-related data points: the average faulting, the maximum positive faulting, the maximum negative faulting, the number of positive faulting, and the number of negative faulting. Different from IRI, the variation (standard deviation) of faulting is not reported to PMS. During this research, efforts were made to estimate an appropriate average value for faulting between 0 and 5 mm by comparing faulting from PMS and from profile measurement on ten selected sections, a method used by Utah DOT [13]. However, no obvious improvement was observed by doing so. Hence, the average faulting of each evaluation section was calculated by taking the average of the raw data available in PMS.

Calculating the Standard Deviation of IRI

Among all distresses types, IRI can be considered as an objective measurement. The mean and standard deviation of IRI in m/km are reported for every 161 m subsection. The definition and unit in LA-PMS are the same as they are in MEPDG. Hence, no unit conversion was needed.

Besides the mean value, this study also calculated the overall standard deviation of IRI for each selected evaluation project by averaging the pooled mean values. From the theory of statistics, the total sum of squares (TSS) equals to the sum of the within sample sum of squares (SSW) and the sum of squares between samples (SSB).

$$TSS = SSW + SSB \tag{2}$$

or

$$\sum_{i,j}\left(y_{ij} - \bar{y}\right)^2 = \sum_{i,j}\left(y_{ij} - \bar{y_i}\right)^2 + \sum_{i,j}\left(\bar{y_i} - \bar{y}\right)^2 \tag{3}$$

where, y_{ij} = the jth individual distress (IRI) measurement in the i-th subsection,
$\bar{y}_i$ = the pooled mean distress in the ith subsection,
$\bar{y}$ = the total mean distress of the entire project.

Let n_i be the number of 161-m subsections in a project and n_j be the number of distress measurements in each subsection. Eq. (3) can be re-written in terms of total standard deviation s and pooled standard deviation s_i of the ith 161-m subsection.

$$(n_i n_j - 1)s^2 = (n_j - 1)s_i^2 + \sum_i n_j(\overline{y_i} - \overline{y})^2 \quad (4)$$

Eq. (4) can be used to calculate the total standard deviation s of IRI of a particular project. The number of distress measurements in each subsection n_j depends on the configuration of the surveyor. Based on a communication with the PMS section in Louisiana DOTD, n_j = 100 was used for IRI measurement in this study [1].

Evaluate the Selected Typical Pavement Structures

This study evaluated MEPDG national models from two perspectives:

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

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

At project level, Excel spreadsheet was prepared to compare the time series performance curves from Pavement ME and PMS. If the two curves matched well with each other, the model would be deemed as good. Otherwise, Pavement ME would over-predict if the predicted performance curve was on top of the measured performance curve; and vice versa.

After data for each project were compiled together and compared, all data for each pavement type were gathered together. At this level, predictions and measurements were compared as pairs but regardless of time. The ideal situation is that all data would line up on the 45° line of equality if a model worked perfectly well. Otherwise, a model over-predicted the performance if predictions were larger than measurements; and vice versa.

Besides visual examination, goodness-of-fit statistics and hypothesis tests [2] were used to assess the bias of the MEPDG prediction model.

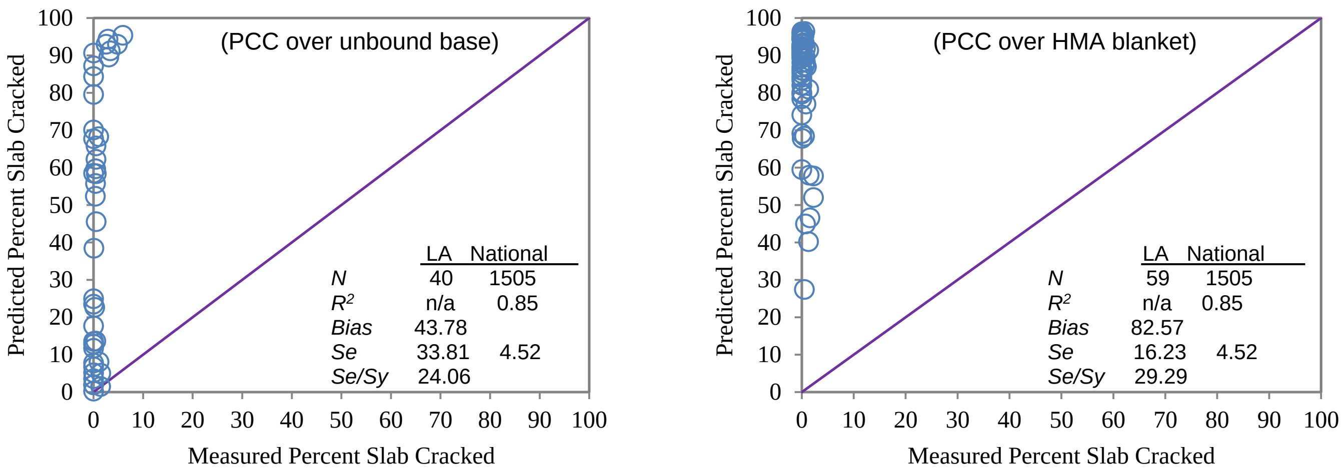
In addition to comparing measured mean performance with predicted performance at 50% reliability, this study compared field measured distresses at the mean-plus-one-standard-deviation level (corresponds to about 85-percentile by assuming measured distresses are normally distributed) was compared with the predicted distresses at 85% reliability level [1, 12]. The purpose of this comparison was to examine any difference between the default variation in MEPDG and the variation in field measurement. For instance, if the predicted IRI matched well with the measured IRI at the mean level but was less than the measured IRI at reliability level, it would indicate that the default variation in MEPDG was less than the variation of measured IRI in field. When comparing the predicted pavement distresses at the reliability level and the field variation of pavement distresses, only the pavement condition data from the latest survey were used. The predicted and measured pavement condition at the early life were not compared, because in most of the cases the trend would be similar except that both the measured and the predicted distresses were smaller than those at the latest survey point.

Cracked Slabs

Fig. 3 presents the comparison of percentage slab cracked from Pavement ME and PMS. Data are lined on the y-axis, which means predictions are more than measurements. In other words, Pavement ME over-predicts slab cracking for both PCC over unbound base and PCC over HMA blanket pavements. This finding agrees with other studies [4]. Summary statistics are also included in Fig. 3. Unsurprisingly, statistics confirm the finding from visual examination. Considering the significant bias, cracking model for rigid pavement needs to be calibrated for Louisiana.

Fig. 4 shows the comparison of slab cracking at reliability levels. One can find that all predictions (circles) are above measurements (diamonds). Pavement ME over-predicts slab cracking for rigid pavements.

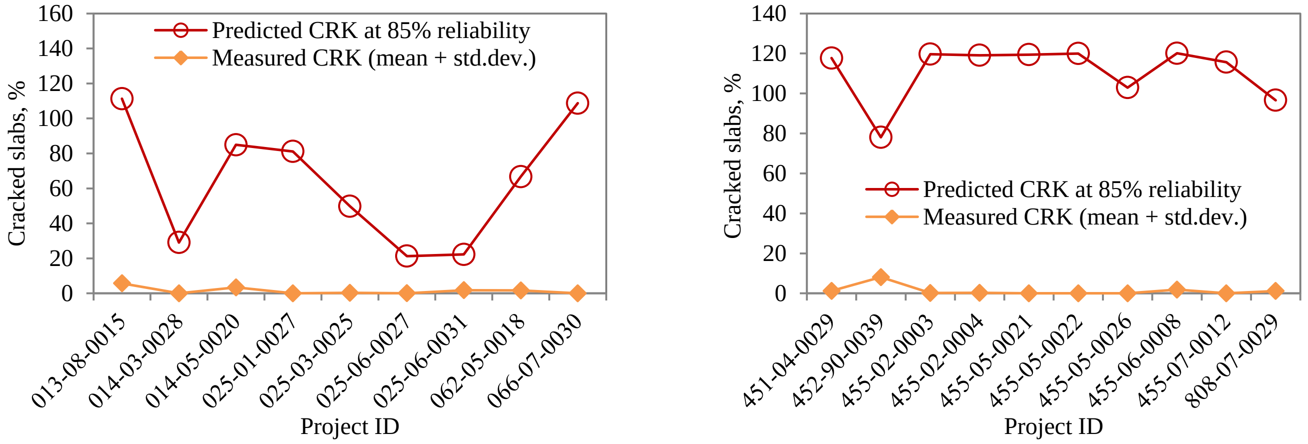
It is worth noting that measured slab cracking are very low for all of the 19 selected projects (the maximum mean value is 5.9% and the maximum mean-plus-standard-deviation is 11.8%). The research team first questioned about the method of converting transverse cracking from length in PMS to percentage as used in Pavement ME. Since it was assumed that each transverse cracking was 3.6 m long and one slab had only one crack, this means that a project with 10% cracked slab would have on average 96.6 m transverse cracking for



(a) PCC over Unbound Base

(b) PCC over HMA blanket

Fig. 3. Predicted vs. Measured Mean Slab Cracking.



(a) PCC over Unbound Base (b) PCC over HMA blanket

Fig. 4. Predicted vs. Measured Slab Cracking at Reliability Level.

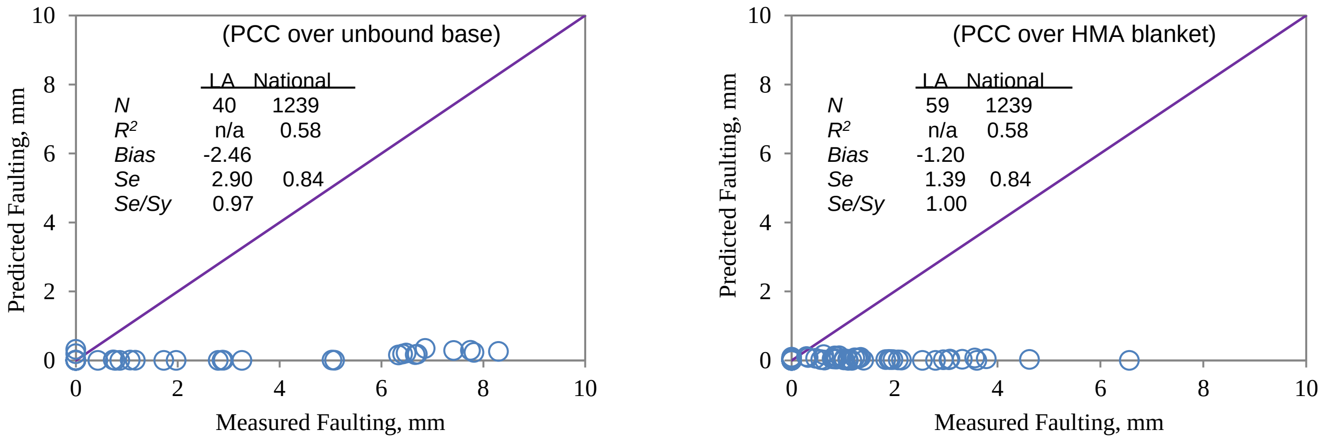
every 161-m section. This amount of transverse cracking is very rare for rigid pavement in the PMS database. During this study, pavement images were reviewed for many projects in which it was confirmed that (1) transverse cracking is not common on rigid pavements (in fact, the chance to spot a longitudinal crack is much higher than seeing a transverse crack), (2) if a slab has a mid-slab transverse cracking, the crack usually extends the full width. Furthermore, engineers at LADOTD also confirmed that although joint spacing at 6 m is the common practice in Louisiana, transverse cracking has not been an issue. Therefore, it was concluded that measured slab cracks were indeed low and Pavement ME over-predicted slab cracking for Louisiana.

It should also be pointed out that "less than 10% measured slab cracking" in this study does not indicate that the pavement is definitely in a good condition. Corner cracking and longitudinal cracking may exist in these pavements. As aforementioned, this study only considers mid-slab transverse cracking to match the definition of the load related fatigue cracking for rigid pavement in MEPDG [12]. Currently MEPDG does not include the prediction of longitudinal cracking. Future development in this area appears to be necessary.

For the purpose of local calibration, however, if a model is only calibrated to the lower level of distress, the ability for the model to predict higher level of distress would be questionable and not guaranteed. Therefore, finding other sections with more transverse slab cracking is strongly recommended.

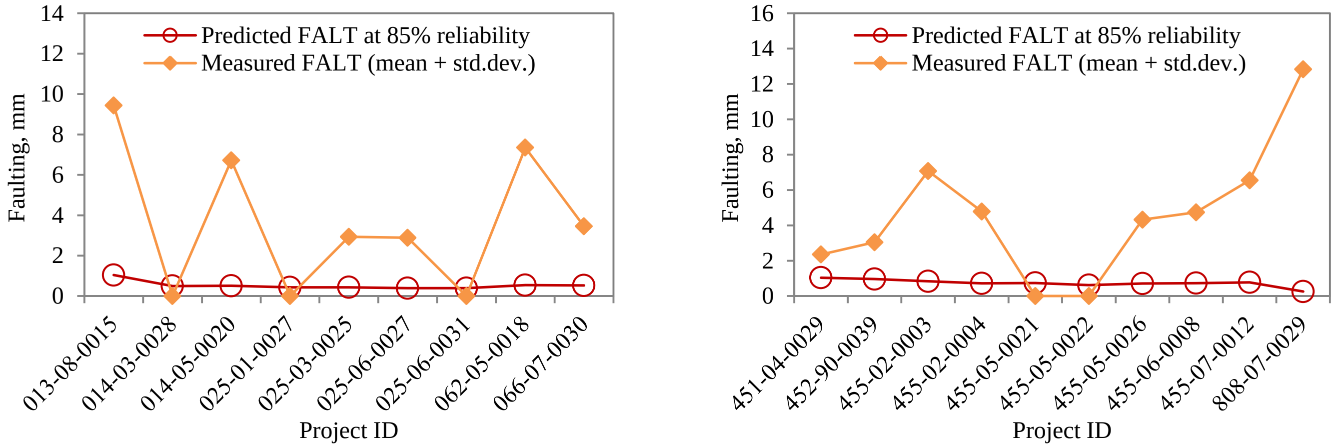
Joint Faulting

The comparison of joint faulting is presented in Figs. 5 and 6. It is found that, on the contrary of slab cracking, all faulting data are lined on the x-axis, which means that Pavement ME under-predicts joint faulting. Measured faulting ranges from zero to 10 mm, but predictions from Pavement ME are lower than 1.3 mm. Fig. 6 shows that measured faulting (diamond) are higher than predicted faulting (circle) except five projects which have zero measured faulting in PMS. As formerly discussed, zero faulting in the PMS does not necessarily mean there is no faulting in the field. It is very likely these five projects have some faulting in the range of zero and 5 mm. Therefore, it could be concluded that Pavement ME under-predicts joint faulting for both PCC over unbound base and PCC over HMA blanket pavements. Thus, the faulting model has to be calibrated for Louisiana.



(a) PCC over Unbound Base (b) PCC over HMA blanket

Fig. 5. Predicted vs. Measured Mean Joint Faulting.



(a) PCC over Unbound Base

(b) PCC over HMA blanket

Fig. 6. Predicted vs. Measured Joint Faulting at Reliability Level.

IRI

Comparison of predicted and measured IRI is presented in Fig. 7 and 8 for at the mean level and at the reliability level, respectively. It is found that predictions match well with measurements for PCC over unbound base, but not well for PCC over HMA blanket pavement. Summary statistics confirm that predictions and measurements have a good correlation (R-square=0.90) for PCC over unbound base; the bias is 0.16 m/km and 0.87 m/km for PCC over unbound base and PCC over HMA blanket, respectively. Considering the magnitude of IRI (normal between 1.6 and 3.2 m/km), the bias for PCC over unbound base could be deemed as small but the bias for PCC over HMA blanket is indeed a concern.

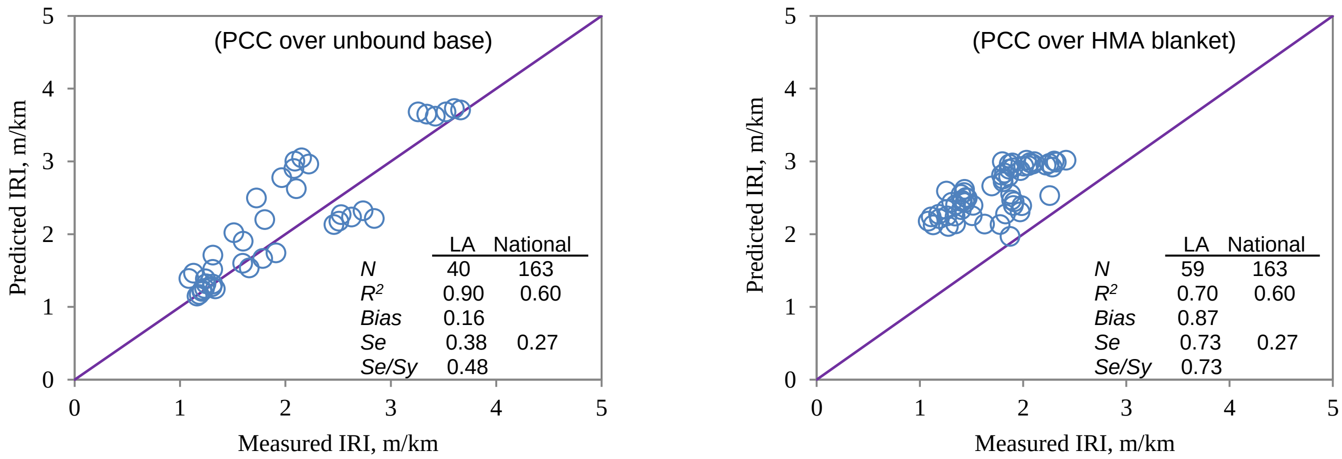
Fig. 8 shows the comparison at the reliability level. One can see that predicted IRI at 85% reliability level (circle) and measured mean-plus-standard-deviation (diamond) are twirled together for PCC over unbound base pavement, meaning they are close to each other. But for PCC over HMA blanket pavement, the line of prediction is on top of the line of measurement except one data point, meaning that the predicted IRI is higher than the measured IRI.

Because the IRI model for rigid pavements is an empirical function of slab cracking, joint faulting, joint spalling and site factor, it needs to be re-evaluated if any of the component distress model is changed. Hence, the need to calibrate IRI model only for the bias of PCC over HMA blanket pavement is not warranted.

Preliminary Local Calibration

MEPDG has four, eight and four calibration coefficients for the transverse cracking model, joint faulting model and IRI model, respectively [19]. The Local Calibration Guide provides recommendations for adjusting different coefficients to reduce bias and standard error. Li et al. [5] used elasticity to evaluate the relative impact of each factor to the model estimation. Based on the relative impact, coefficients could be adjusted accordingly. Similar methods were also used by Bustosl et al. [14] and Kim et al. [4].

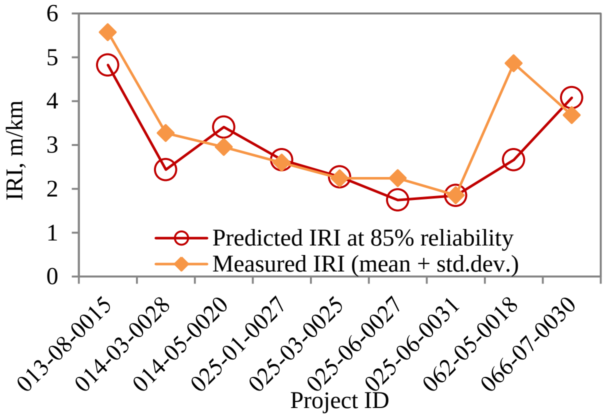
Relying on former studies, this research conducted a preliminary local calibration by adjusting two of the most sensitive coefficients: C1 of the cracking model and C6 of the faulting model. C1 is related to PCC modulus of rupture and stress. Hence changing C1 could dramatically influence the allowable number of load applications a PCC slab can carry, which in turn impact the estimated transverse cracking. C6 is the index of the power function



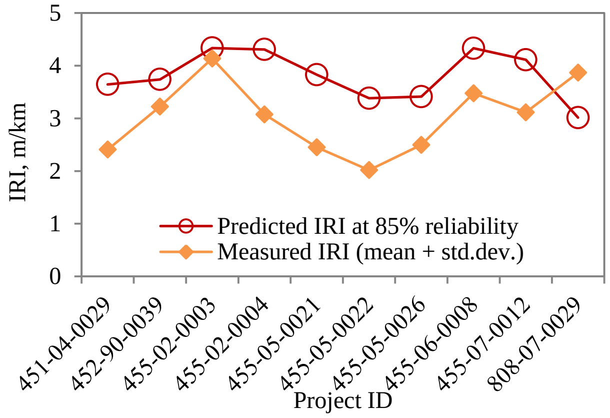
(a) PCC over Unbound Base

(b) PCC over HMA blanket

Fig. 7. Predicted vs. Measured Mean IRI.



(a) PCC over Unbound Base



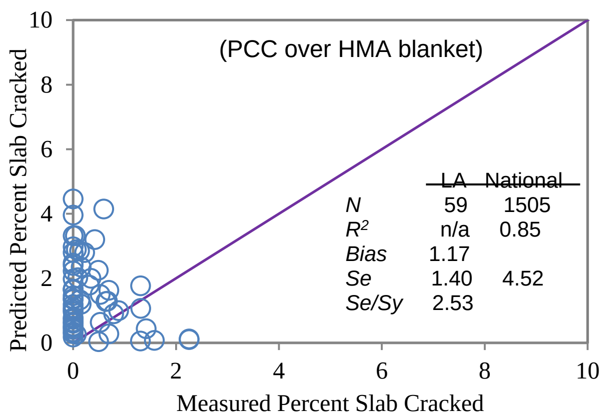
(b) PCC over HMA blanket

Fig. 8. Predicted vs. Measured IRI at Reliability Level.

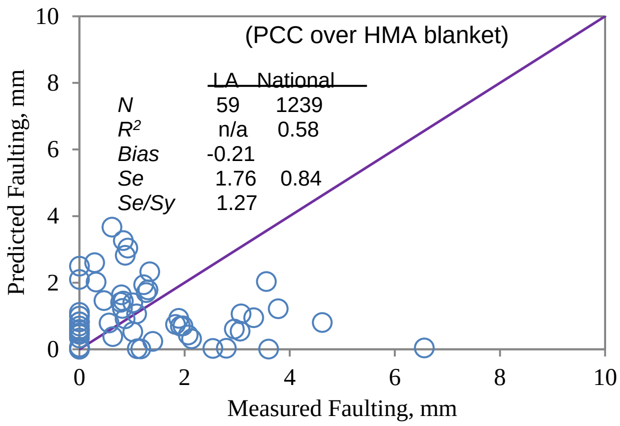
that connects joint faulting with erodibility factor, number of wet days and subgrade load. Keeping the calibrated coefficients in the literature as a reference, the method of trial and error was applied to minimize the difference between predictions and measurements.

C1 = 2.6 and C6 = 1.2 were determined as the calibration coefficients. Comparison of measured and predicted performance using the calibrated model is shown in Fig. 9 for PCC over HMA blanket pavements. Goodness of fit, bias, and standard error are also presented. The predicted slab cracking is greatly reduced so that it matches better with the PMS data. Statistically, the hypothesis that the mean of predicted cracking is equal to the mean of measured cracking cannot be rejected at a 95% reliability level. Comparing to Fig. 3, the bias is reduced from 82.57% to 1.17%. Furthermore, the standard error is reduced from 16.23% to 1.40%, which is lower than the national calibration.

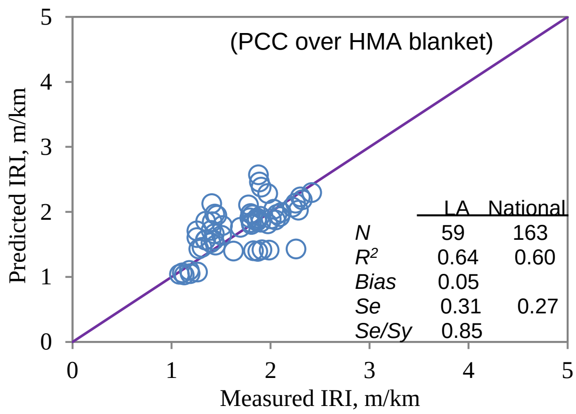
The joint faulting model and IRI model are also improved by local calibration. For faulting, bias is reduced from -1.20 mm to -0.21 mm. For IRI, the bias is reduced from 0.87 m/km to 0.05 m/km. Similar results were found for PCC over unbound base pavements, but not presented here due to page limit.



(a) Cracked slabs



(b) Joint faulting



(c) IRI

Fig. 9. Predicted vs. Measured Distresses after Local Calibration.

Thickness Comparison

LADOTD is currently using the AASHTO 1993 Guide as the standard design tool. To assess the possible difference between the current and the new method, the selected 19 rigid pavement sections were re-designed as new pavements using Pavement ME. Starting with the 1993 designed structure, the PCC surface thickness was increased or decreased according to whether the predicted performance (cracking, faulting and IRI) met the design criteria. Fig. 10 presents the result of this process by comparing the recommended PCC thickness from Pavement ME with the original designed thickness. It shows that the two design methods are comparable with an average difference of 2 cm (Pavement ME requires less). In terms of percentage, the new design method seems to require 7 percent thinner on average, which agrees with Timm's finding [9]. Except three projects, other 16 projects have a difference ranges between -3.8 and 5 cm. A close check with the three projects reveals that the three projects have widened slab up to 4.6 m and tied PCC shoulders. Both feathers are known to reduce the potential of joint faulting and mid-slab transverse cracking. Hence, Pavement ME requires only 18 cm of PCC slab instead of the 1993 designed 28 cm. For other projects with HMA shoulders and un-widened slabs, Pavement ME seems to work reasonably.

Validation and Design Examples

Five projects outside of the evaluation pool were selected as design examples. Table 3 shows the traffic and pavement structure; Fig. 11 presents the recommended PCC thickness. The graph shows that the national model would require much thicker PCC surface to a level of impractical. This is mainly because the national model over-predicts slab cracking (Fig. 3). On the contrary, the calibrated model estimates a similar or slightly thinner concrete slab than the AASHTO 1993 design. It is also important to clarify that the distress in control is faulting for interstates and transverse cracking for US and LA highways. This leads to a hypothesis that the cracking model is dramatically impacted by slab thickness but the faulting model depends more on traffic level. At low traffic level and thin slab thickness, faulting is not a problem so the distress in control is transverse slab cracking; while at high traffic level and a thick slab, cracking is not the prominent issue (a thick slab greatly reduces the tensile stress which then reduces the potential of transverse cracking) but joint faulting accumulates high under repetitive loading. In addition, widened slab is designed for project 450-10-0108, 451-02-0078 and 014-03-0027 to reduce the predicted faulting. The joint faulting is found to be sensitive to slab width, shoulder type, dowel diameter and joint spacing. This example illustrates the advantage of Pavement ME for providing such a tool to estimate the influence of different features of a design beyond slab thickness.

Discussion

To evaluate and calibrate MEPDG, many states have to make the best use of their pavement management system. However, PMS was traditionally designed for monitoring the existing pavement

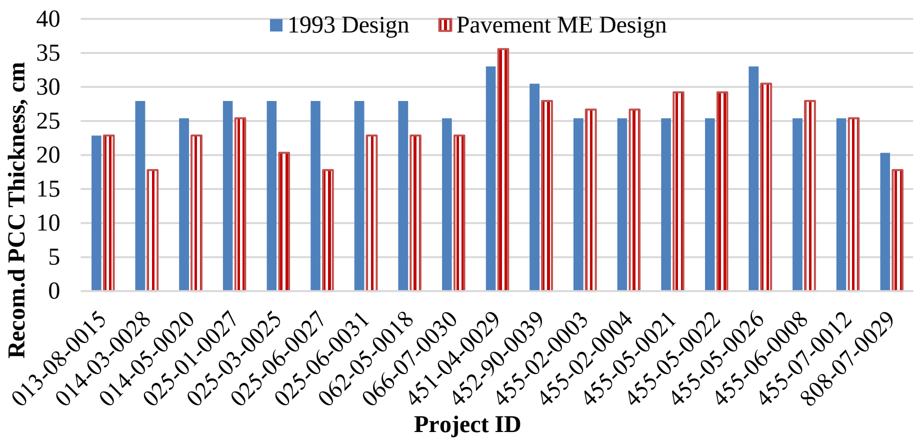


Fig. 10. Recommended Thickness from AASHTO 1993 and Pavement ME.

Table 3. Traffic and Structure Information of Design Examples.

Project ID	450-10-0108	451-02-0078	014-03-0027	262-02-0023	808-07-0040
Route	I-10	I-20	US-165	LA-16	LA-3105
Initial AADT	121,100	48,205	12,000	22,750	24,200
$AADTT_0$ per lane	7,369	2,471	914	1,283	679
Growth rate %	2.5	1.0	2.5	2.0	4.5
20-years ESAL (Millions)	85.2	19.38	7.19	10.2	4.5
Surface (Original Design)	PCC 33 cm	PCC 30 cm	PCC 28 cm	PCC 28 cm	PCC 25 cm
Base	Class II Stone 25 cm	Class II Stone 4 cm Soil Cement 23 cm	Class II Stone 30 cm	Class II Stone 20 cm	Class II Stone 20 cm
Subgrade	Subgrade (A-6)	Subgrade (A-7-6)	Subgrade (A-6)	Subgrade (A-6)	Subgrade (A-7-6)

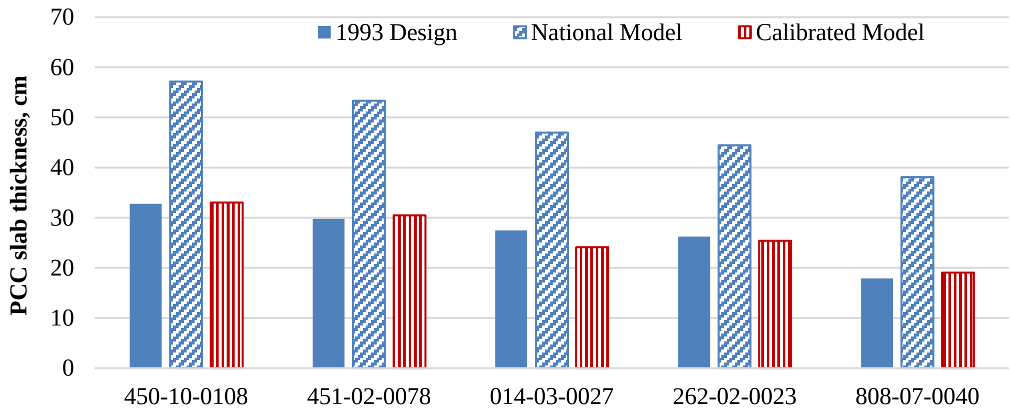


Fig. 11. Required PCC Slab Thickness Using Different Design Models.

network, and network-level optimization of resources. It was not designed for project-level applications. In addition, the definition of distresses may not be the same in PMS as it is in MEPDG. The accuracy of using network-level data to calibrate project-level models therefore becomes a challenge. For example, this study had to convert transverse cracking from total length to percentage by assuming each transverse crack is 3.6 m and each slab has only one crack. WSDOT had to assume that two-thirds of all cracks were longitudinal cracking because PMS did not differentiate between longitudinal and transverse cracks [5]. Similar challenges on converting distresses were also found in other states [16, 22].

Two suggestions may help solving this problem. First, conduct project-level distress survey on historical pavement images. This is a cumbersome work and may still not achieve the accuracy of field distress evaluation (e.g. fine cracks may not be visible in pavement images), but it will provide more accurate data than directly converting PMS data from length to percentage. Similarly, project-level data analysis could be carried out on the original profile data to interpret faulting and IRI. Although this raises another challenge on storing massive data and accurately indexing them, it would provide the most accurate historical data that other methods could not offer. The second solution is to initiate statewide mini-LTPP program. By a careful experimental design and giving special attention during PMS data collection, this would prepare data for future endeavours on model evaluation, calibration, and other research activities. This approach would provide high quality data and reduce the burden on massive data management. But the disadvantage is that it requires several years of data collection without generating any tangible products.

This study revealed another challenge on collecting high quality traffic data for MEPDG design. Different from flexible pavements, rigid pavements are usually constructed within a city limit. In fact, the majority of rigid sections in Louisiana are in Baton Rouge, Shreveport, New Orleans and principle roads across a city or town. Considering the complexity of traffic patterns in a city and the high dependency of pavement performance on traffic data, further studies on the installation, management and analysis of traffic data is highly recommended.

Although the calibrated MEPDG software does not predict pavement performance that match the measured performance in PMS for every single project, the overall performance is promising. Furthermore, MEPDG acts as a catalyst and conveyor that stimulate the communication between different divisions in a highway agency. For instance, this study assembled data and requested help from pavement management, planning, material testing, construction, maintenance and computer service. Problems such as missing data, inconsistent records, and future requirements were reported. These problems were further discussed among administrators, academia, and consultants. Overall, it is promising that future efforts could better facilitate the required data and business operations to finally implement the new mechanistic empirical design method.

Conclusions and Recommendations

This paper presents the evaluation of MEPDG rigid pavement models in the state of Louisiana. Details are given on the procedure of selecting calibration sections, and collecting structure, material, and traffic data from different database. Assumptions to interpret PMS data are explained. National models are first evaluated followed by a preliminary local calibration. Design examples are also provided to verify the usefulness of the calibrated model. In summary, the following conclusions are reached:

- The two major structure types for rigid pavements in Louisiana are PCC over unbound base and PCC over HMA blanket.
- General speaking, the national model over-predicts transverse cracking and under-predicts joint faulting. IRI shows a good match between the predicted and measured performance.
- Both the cracking and faulting model are improved by adjusting calibration coefficients. Design examples show that the PCC thickness from MEPDG is comparable or slightly thinner (2 cm or 7 percent on average) than that of the AASHTO 1993 design.

Through the process of data collection, several problems such as missing data and inconsistent records were reported. Further actions are recommended to address these issues. More calibration sections with severe distress are needed to increase the dataset for a more reliable calibration. Project-level data analysis is suggested to reduce the error from converting distress between PMS and MEPDG. Improvements on current PMS data collection and analysis should be made to better support MEPDG-related studies. Conducting a comprehensive calibration by adjusting all coefficients using advanced optimization methods is also recommended.

Acknowledgements

This study was supported by the Louisiana Transportation Research Centre (LTRC) and the Louisiana Department of Transportation and Development (LADOTD). Comments and advice from the project review committee are gratefully acknowledged. The authors also appreciate the help from engineers in many divisions at LADOTD, in particular, Christopher Fillastre, Xingwei Chen and Partick Icenolge. Mitchell Terrell, Shawn Elisar, and Terrell Gorham from LTRC collected the additional pavement performance data and Xiaoming Yang and Ferdous Intaj helped in data analysis.

References

1. Wu, Z., Yang, X. and Zhang, Z. (2013). Evaluation of MEPDG flexible pavement design using pavement management system data: Louisiana experience. *International Journal of Pavement Engineering*, 14(7), pp. 674-685.
2. AASHTO (2010). Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide. American Association of State Highway and Transportation Officials. Washington, DC, USA.
3. Federal Highway Administration Office of Asset Management. (2010). Local Calibration of the MEPDG Using Pavement Management Systems, Final Report. Washington, DC, USA.
4. Kim, S., Ceylan, H., Ma, D., and Gopalakrishnan. K. (2014). Calibration of Pavement ME Design and Mechanistic-Empirical Pavement Design Guide Performance Prediction Models for Iowa Pavement Systems, Journal of Transportation Engineering, 140(10).
5. Li, J., Muench, S.T., Mahoney, J.P., Sivaneswaran, N., and Pierce, L.M. (2006). Calibration of NCHRP 1-37A Software for the Washington State Department of Transportation: Rigid Pavement Portion. *Transportation Research Record*, No.1949, pp. 43-53.
6. Li, J., Uhlmeyer, J.S., Mahoney, J.P. and Muench, S.T. (2010). Updating the Pavement Design Catalog for the Washington State Department of Transportation: Using 1993 AASHTO Guide, Mechanistic-Empirical Pavement Design Guide, and Historical Performance. *Transportation Research Record*, No. 2153, pp. 124-129.
7. Gedafa, D., Mulandi, J., Hossain, M. and Schieber, G. (2011). Comparison of Pavement Design Using AASHTO 1993 and NCHRP Mechanistic-Empirical Pavement Design Guides. *American Society of Civil Engineers Transportation and Development Institute Congress*. Chicago, Illinois, USA. pp. 538-547.
8. Mallela, J., Titus-Glover, L., Sadasivam, S., Bhattacharya, B., Darter, M, and Von Quintus, H. (2013). Implementation of the AASHTO Mechanistic-Empirical Pavement Design Guide for Colorado. Colorado Department of Transportation, Denver, CO, USA.
9. Timm, D. (2006). Design Comparisons Using the New Mechanistic-Empirical Rigid Pavement Design Guide. *American Society of Civil Engineers Airfield and Highway Pavement*, Atlanta, Georgia, USA. pp. 236-247.
10. Indiana DOT (2013). Indiana Design Manual. Access on March 31, 2014. http://www.in.gov/indot/design_manual/files/Ch52_2013.pdf
11. Nantung, T. (2010). Implementing the Mechanistic–Empirical Pavement Design Guide for Cost Savings in Indiana. TR News 271. Transportation Research Board. Washington, DC, USA. http://onlinepubs.trb.org/onlinepubs/trnews/trnews271rpo.pdf.
12. ARA, Inc. (2004). Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures. ARA, Inc, ERES Consultants Division, Champaign, IL, USA.
13. Darter, M., Titus-Glover, L. and Von Quintus, H. (2009). Implementation of the Mechanistic-Empirical Pavement Design Guide in Utah: Validation, Calibration and Development of the UDOT MEPDG User's Guide. Utah Department of Transportation, Salt Lake City, UT, USA.
14. Bustosl, M., Cordo, O., Girardi, P. and Pereyra, M. (2011). Calibration of Distress Models from the Mechanistic-Empirical Pavement Design Guide for Rigid Pavement Design in Argentina. *Transportation Research Record*, No. 2226, pp. 3-12.
15. Medina-Chavez, C.I. and Won, M. (2006). Mechanistic-Empirical Data Collection Approach for Rigid Pavements. Texas Department of Transportation, Austin, TX, USA.
16. Mamlouk, M., and Zapata, C.E. (2010). Necessary Assessment of Use of State Pavement Management System Data in Mechanistic-Empirical Pavement Design Guide Calibration Process. *Transportation Research Record*, No. 2153, pp. 58-66.
17. Ishak, S., Shin, H.-C., Sridhar, B.K. and Zhang, Z. (2010). Characterization and Development of Truck Axle Load Spectra for Implementation of New Pavement Design Practices in Louisiana. *Transportation Research Record*, No. 2153, pp. 121-129.
18. Schwartz, C.W., Li, R., Kim, S., Ceylan, H. and Gopalakrishnan, K. (2011). Effect of Concrete Strength and Stiffness Characterization on Predictions of Mechanistic–Empirical Performance for Rigid Pavements. *Transportation Research Record*, No. 2226, pp. 41-50.
19. AASHTO (2008). Mechanistic-Empirical Pavement Design Guide, Interim Edition: A Manual of Practice. American Association of State Highway and Transportation Officials, Washington, DC, USA.
20. Khattak, M.J., Baladi, G.Y., Zhang, Z., and Ismail, S. (2008). Review of Louisiana's Pavement Management System: Phase I. *Transportation Research Record*, No. 2084, pp. 18-27.
21. Louisiana Department of Transportation and Development (2008). Louisiana Cracking and Patching Protocol for Concrete Pavements, Version 11. Baton Rouge, LA, USA.
22. Kim, S., Ceylan, H., Gopalakrishnan, K., and Smadi, O. (2010). Use of Pavement Management Information System for Verification of Mechanistic-Empirical Pavement Design Guide Performance Predictions. *Transportation Research Record*, No. 2153, pp. 30-39.