Concrete Pavement Design in Kansas Following the Mechanistic-Empirical Pavement Design Guide

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ABSTRACT

The AASHTO Guide for Design of Pavement Structures uses empirical performance equations for concrete pavement design. The new Mechanistic-Empirical Pavement Design Guide (MEPDG) provides methodologies for mechanistic-empirical pavement design. Five roadway sections, designed by the Kansas Department of Transportation (KDOT) using 1986 and 1993 AASHTO pavement design guides, and three long-term pavement performance (LTPP) sections in Kansas were analyzed using MEPDG. Project-specific construction and materials data and MEPDG default traffic data were used in the analysis. The predicted output variables, IRI and faulting, were compared with those obtained during KDOT annual pavement condition survey or from the LTPP database. The results show that the predicted IRI values are similar to the measured values. MEPDG analysis showed minimal or no faulting, although both predicted and measured faulting values were insignificant for all practical purposes. The sensitivity analysis results show that IRI is the most sensitive output with respect to the traffic inputs. Percentage of slabs cracked increases significantly with increasing truck traffic and decreases with increasing slab thickness. Faulting is the least sensitive parameter.

Key words: concrete pavement design—Mechanistic-Empirical Pavement Design Guide

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INTRODUCTION

The most widely used procedure for design of concrete pavements is specified in the *Guide for Design of Pavement Structures*, published in 1986 and 1993, by the American Association of State Highway and Transportation Officials (AASHTO 1986; AASHTO 1993). A few states use the 1972 American Association of State Highway Officials (AASHO) Interim Guide procedure, the Portland Cement Association (PCA) procedure, their own empirical or mechanistic-empirical procedures, or a design catalog (Hall 2003). The design methodologies in all versions of the AASHTO Guide are based on the empirical performance equations developed using the AASHO Road Test data from the late 1950's. The 1986 and 1993 guides contain some state-of-the-practice refinements in material input parameters and design procedures for rehabilitation design. Due to the limitations of earlier guides, a design guide, based as fully as possible on mechanistic principles, was developed under the National Cooperative Highway Research Program (NCHRP) (NCHRP 2004). The goal was to provide a user-friendly mechanistic-empirical design procedure that would account for local environment conditions, local materials, and actual highway traffic distribution by means of axle load spectra. Since the resulting procedure is very sound and flexible and it considerably surpasses the capabilities of any currently available pavement design and analysis tools, it is expected that it will be adopted by AASHTO as the new AASHTO design method for pavements structures.

BACKGROUND OF THE MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE

Basic Design Concept

Mechanistic-empirical (M-E) design combines the elements of mechanical modeling and performance observations in determining required pavement thickness for a given set of design inputs. The mechanical model is based on elementary physics and determines pavement response to wheel loads or environmental condition in terms of stress, strain, and displacement. The empirical part of the design uses the pavement response to predict the life of the pavement on the basis of actual field performance (Timm, Birgisson, and Newcomb 1998). Yoder and Witczak (1975) pointed out that for any pavement design procedure to be completely rational in nature, three elements must be fully considered: (1) the theory used to predict the assumed failure or distress parameter, (2) the evaluation of the materials properties applicable to the selected theory, and (3) the determination of the relationship between the magnitude of the parameter in question to the performance level desired. The newly developed M-E design guide considered all three elements. While the mechanistic approach to pavement design and analysis is much more rational than the empirical approach, it also is much more technically demanding. However, there are some specific advantages of M-E design over traditional empirical procedures: consideration of changing load types, better utilization and characterization of available materials, improved performance predictions, better definition of the role of construction by identifying parameters that have the most influence over pavement performance, relation of material properties to actual pavement performance, better definition of existing pavement layer properties, and accommodation of environmental and aging effects on materials. In essence, M-E design has the capability of changing and adapting to new developments in pavement design by relying primarily on the mechanics of materials. For example, M-E design can accurately examine the effect of new load configurations on a particular pavement. Empirical design, on the other hand, is limited to the observations on which the procedure was based (e.g., single axle load). Additionally, since the process is modular, new advances in pavement design may be incorporated without disrupting the overall procedure (Timm, Birgisson, and Newcomb 1998).

Design Approach

The design approach followed in MEPDG is summarized in Figure 1. The format provides a framework for future continuous improvement to keep up with the changes in truck traffic technology, materials, construction, design concepts, computers, and so on. As shown in the figure, in this guide, the designer first considers site conditions (traffic, climate, material, and existing pavement condition, in case of rehabilitation) and construction conditions in proposing a trial design for a new pavement or rehabilitation. The trial design is then evaluated for adequacy against some predetermined failure criteria. Key distresses and smoothness are predicted from the computed structural responses of stress, strain, and deflection due to given traffic and environmental loads, such as temperature gradient across the PCC slab. If the design does not meet desired performance criteria at a preselected level of reliability, it is revised and the evaluation process is repeated as necessary (NCHRP 2004). Thus, the designer is fully involved in the design process and has the flexibility to consider different design features, climatic conditions, and materials for the prevailing site condition. This approach makes it possible to optimize the design and to more fully insure that specific distress types will not develop.

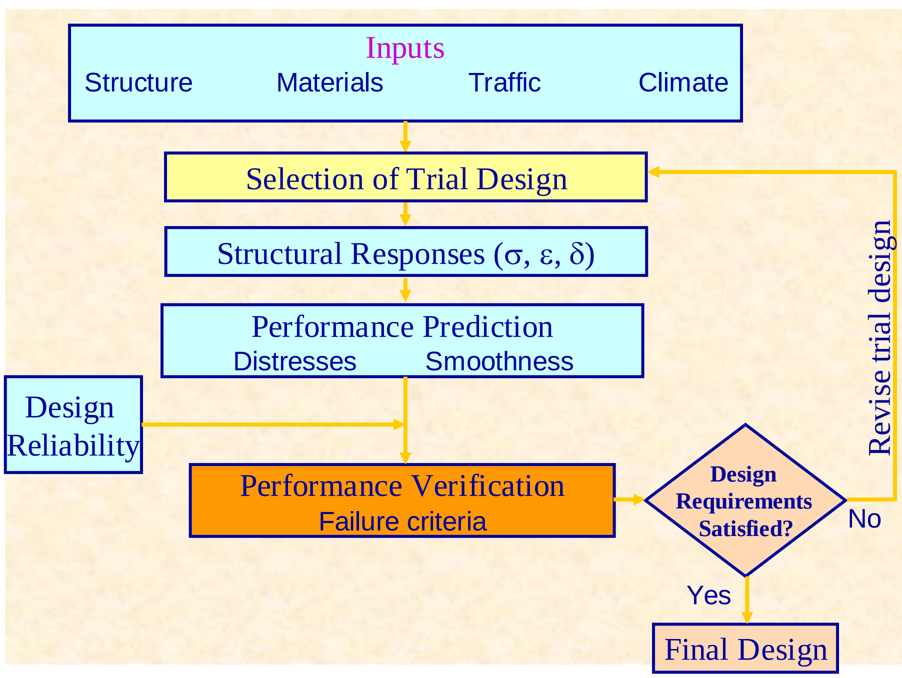

Figure 1. PCC mechanistic-empirical design framework (NCHRP 2004)

OBJECTIVE

The overall objective of this paper is to present the MEPDG design analysis results for selected in-service Portland cement concrete (PCC) projects in Kansas. Sensitivity analysis results for determining the sensitivity of the output variables due to variations in the key input parameters used in the design process are also presented.

TEST SECTIONS

Eight in-service PCC projects were selected for the MEPDG analysis. Three of these projects were the experimental sections chosen from the Kansas SPS-2 project located on Interstate route 70. Two other projects are located on Interstate Route 70, two on US-50, and one on route K-7. Table 1 tabulates the features of these sections. The SPS-2 sections were 500 ft long each, and the rest are one mile to several miles long.

All sections have 15 ft joint spacings with dowelled joints. Most have 1.5 in diameter steel dowels. Sections, where the PCC slab thickness is less than 10 inches, have 1.25 in diameter dowels. All sections have 12 ft lanes with tied concrete shoulders except the SPS-2 section 6. That section has a widened lane of 14 ft with tied PCC shoulder. The sections were constructed on stabilized base and on treated subgrade. Base stabilization was done with Portland cement. Depending upon the cement content and gradation, the bases were designated as Portland cement treated base (PCTB), bound drainable base (BDB), or lean concrete base (LCB). Base thickness ranged from 4 to 6 inches. The projects have primarily silty clay as subgrade. The top six inches of the natural subgrade were treated with lime or fly ash to reduce the plasticity and/or control the moisture during construction. PCC slab thickness, designed according to the 1986 or 1993 AASHTO design guide, ranged from 9 to 12 inches. The strength (modulus of rupture or compressive strength) values shown in Table 1 are the actual average values obtained during construction. The as-constructed International Roughness Index (IRI) values on these projects varied from 59 in/mile to 122 in/mile.

The annual average daily traffic (AADT) on these sections ranged from 2,080 for a US-50 Chase County project to 36,000 for the I-70 Shawnee County project. Very high percentage of truck traffic was observed on the US-50 projects, and the lowest percentage was on the I-70 Shawnee County project.

Table 1. Project features of the study sections

Project ID	Route	County	Year Built	Mile post Limit	PCC Thickness (in)	PCC 28-day Strength (psi)	Subgrade Soil Type	Initial AADT	% Truck	Initial IRI (in/mi)
K-2611-01*	I-70	Geary	1990	0 – 7	11	690	A-6	9,200	18	60
K-3344-01**	I-70	Shawnee	1993	9 – 10	10.5	473	A-7-6	36,000	5	96
SPS-2 (Sec-5)†	I-70	Dickinson	1992	20 – 22.61	11	945	A-6	11,970	22.3	122
SPS-2 (Sec-6)†	I-70	Dickinson	1992	20 –22.61	11	617	A-6	11,970	22.3	98
SPS-2 (Control)*	I-70	Dickinson	1992	20 – 22.61	12	647	A-6	11,970	22.3	95
K-3216-02***	US-50	Chase	1997	0 – 9	10	5,569	A-7-6	2,080	45.5	59
K-3217-02***	US-50	Chase	1997	9 – 19	10	4,362	A-7-6	3,480	40.5	68
K-3382-01**	K-7	Johnson	1995	12 – 15	9	537	A-7-6	13,825	7	81

*6” Portland Cement-Treated Base (PCTB)
** 4” Portland Cement-Treated Base (PCTB)
*** 4” Bound Drainable Base (BDB)
† 6” Lean Concrete Base (LCB)

MEPDG DESIGN INPUTS

Hierarchical Design Inputs

The hierarchical approach is used for the design inputs in MEPDG. This approach provides the designer with several levels of "design efficacy" that can be related to the class of highway under consideration or to the level of reliability of design desired. The hierarchical approach is primarily employed for traffic, materials, and environmental inputs (NCHRP 2004). In general, three levels of inputs are provided:

Level 1 - Level 1 is a "first class" or advanced design procedure. It provides for the highest practically achievable level of reliability and recommended for design in the heaviest traffic corridors or wherever there are dire safety or economic consequences of early failure. The design inputs also are of the highest practically achievable level and generally require site-specific data collection and/or testing. Example is the site-specific axle load spectra for traffic input.

Level 2 - Level 2 is the input level expected to be used in routine design. Level 2 inputs are typically user-selected, possibly from an agency database. The data can be derived from a less than optimum testing program or can be estimated empirically. Estimated Portland cement concrete elastic modulus from the compressive strength test results is an example of Level 2 input in the material input data category.

Level 3 - Level 3 typically is the lowest class of design and should be used where there are minimal consequences of early failure. Inputs typically are user-selected default values or typical averages for the region. An example would be the default value for the Portland cement concrete coefficient of thermal expansion for a given mix class and aggregates used by an agency.

For a given design, it is permissible to mix different levels of input.

MEPDG Design Features

Due to climatic variation and repeated traffic loads over the design life of a pavement, very high amount of uncertainty and variability exists in the pavement design and construction processes (NCHRP 2004). In the mechanistic-empirical design, the key outputs are the individual distress quantities. For jointed plain concrete pavements, MEPDG analysis predicts distresses, such as faulting, transverse cracking, and smoothness or IRI. Therefore, the reliability term has been incorporated in MEPDG to come up with an analytical solution, which allows the designer to design a pavement with an acceptable level of distress at the end of design life. Design reliability is defined as the probability that each of the key distress types and smoothness will be less than a critical level over the design period. Therefore, failure criteria are associated with this design reliability. The failure criteria and design reliability are also required inputs for the MEPDG analysis; although, the designer and the agency have the control over these values. The design can fail if the predicted distress is greater than the allowable amount or if the predicted distresses are unacceptable. In this study, the design reliability used for all projects was 90% and the corresponding failure criteria was 164 in/mile for IRI, 0.12 in for faulting, and 15% for slab cracking.

Inputs for Concrete Pavement Design Analysis

Input data used for the MEPDG analysis of concrete pavements are categorized as (a) general information, (b) traffic, (c) climate, (d) pavement structures, and (e) miscellaneous.

General Information

The general information inputs include design life, construction month, traffic opening month, pavement type (JPCP/CRCP), initial smoothness (IRI), etc. All pavement sections in this study were jointed plain concrete pavements (JPCP) and were analyzed for 20-year design life. Inputs, such as construction and traffic opening months, were required to compute the drying shrinkage and concrete strength gain.

Traffic

Traffic data is one of the key elements required for the MEPDG analysis. The basic required information is AADT for the year of construction, percentage of trucks in the design direction and on the design lane, operational speed, and traffic growth rate. The AADT in this study ranged from 2,080 to 36,000. The traffic growth in MEPDG can be linear or compound. Project-specific linear traffic growth rates varying from 2% to about 7% were used in this study. Truck percentages in the design direction varied from 47% (provided by LTPP for the SPS-2 sections) to 50%. For the 4-lane divided highways, 95% percent of trucks were assigned in the design lane based on the default level 3 input.

For this study, all other required traffic inputs, such as monthly and hourly truck distribution, truck class distribution, axle load distributions, and some other general traffic inputs, were derived from the design guide level 3 or default values.

Climate

Environmental conditions have significant effects on the performance of rigid pavements. The seasonal damage and distress accumulation algorithms in the MEPDG design methodology require hourly data for five weather parameters, such as air temperature, precipitation, wind speed, percentage sunshine, and relative humidity (NCHRP 2004). The design guide recommends that the weather inputs be obtained from weather stations located near the project site. At least 24 months of actual weather station data are required for computation. The design guide software includes a database of appropriate weather histories from nearly 800 weather stations throughout the United States. This database can be accessed by specifying the latitude, longitude, and elevation of the project location. The MEPDG software locates six weather stations closest to the site. In this study, project specific virtual weather stations were created by interpolation of climatic data from the selected physical weather stations. Specification of the weather inputs is identical at all three hierarchical input levels in MEPDG.

Pavement Structures

In this study, the baseline rigid pavement structure for design analysis is a four-layer JPCP consisting of a Portland cement concrete (PCC) slab over a stabilized base and a treated subgrade. The bottom most layer is the compacted natural subgrade. The inputs required for the PCC layer were layer thickness, material unit weight, Poisson’s ratio, coefficient of thermal expansion, cement type, cement content, water-cement ratio, aggregate type, modulus of rupture, modulus of elasticity, compressive strength, etc. PCC slab thickness in this study ranged from 9 to 12 inches.

All projects in this study have stabilized bases. The inputs required for these bases were layer thickness, mean modulus of elasticity, unit weight of the material, Poisson’s ratio, etc. Layer thickness ranged from 4 to 6 inches. The modulus of elasticity for the cement-treated bases was 500,000 psi. The lean concrete base modulus was taken as 2 million psi. All projects have 6-inch lime or fly-ash treated subgrade (LTSG/FASG) with an input modulus of 50,000 psi. Natural subgrade modulus was calculated by the

MEPDG software from a correlation equation involving the plasticity index and soil gradation. These properties were the inputs in this analysis process.

Miscellaneous

The thermo-hydraulic properties required as inputs into MEPDG are groundwater depth, infiltration and drainage properties (pavement cross slope (%) and drainage path length), physical/index properties (specific gravity of soil solids, maximum dry unit weight, etc.), hydraulic conductivity, thermal conductivity, heat capacity, etc. (Barry and Schwartz 2005). All projects in this study have 1.6% of pavement cross slope. Other parameters were derived from the design guide level 3 values or determined based on correlations. The recommended calibrated values of 1.25 BTU/hr-ft-°F and 0.28 BTU/lb-°F were used for thermal conductivity and heat capacity, respectively. Physical and index properties were derived based on the gradation of the unbound materials. Surface shortwave absorptivity and drainage path length were chosen based on the default inputs, and were 0.85 and 12 ft, respectively. County soil reports were used to estimate the ground water table depth for all sections. Project specific input parameters for the analysis are summarized in Table 2.

RESULTS AND DISCUSSIONS

Design Analysis

As mentioned earlier, key rigid pavement distresses predicted for JPCP from the MEPDG analysis are IRI, faulting, and percentage of slabs cracked.

Smoothness or IRI

In this study, MEPDG-predicted IRI values for the sections were compared with the KDOT-measured and LTPP DataPave (online database) values for 2003. The average IRI, obtained from the left and right wheel path measurements on the travel lane, was used in the comparison. The profile survey for the KDOT Pavement Management System (commonly known as Network Optimization System or NOS) was done on both eastbound and westbound directions for the I-70 Geary and I-70 Shawnee county projects and on northbound and southbound directions for the K-7 Johnson County project. For the SPS-2 sections, measured values were obtained from the LTPP database. Figure 2 shows the comparison between the predicted and measured IRI values for 2003 for all projects. The predicted IRI values for all Interstate sections are fairly similar to the measured values except for the I-70 Geary county project. SPS Section-5 has the highest predicted IRI of 130 in/mi and the I-70 Geary county project has the lowest. The predicted IRI for the Geary county project is 68 in/mi compared to the measured values of 103 in/mi and 102 in/mi for the eastbound and westbound direction, respectively. For the non-interstate sections, NOS-measured IRI values are much higher than the predicted values for both Chase county projects. The MEPDG predicted IRI of 88 in/mi for the K-7 Johnson county project is very similar to the measured values of 86 in/mi and 81 in/mi for the northbound and southbound directions, respectively.

Table 2. Input parameters for MEPDG rigid pavement design analysis

INPUT PARAMETERS	Input Value							
	(I-70 GE)	(I-70 SN)	SPS (Sec 5)	SPS (Sec 6)	SPS (Control)	(US-50-CS1)	(US-50-CS2)	(K-7 JO)
General Information								
Pavement construction Date	Oct, 93	Nov,90	July, 92	July, 92	July, 92	Dec, 97	Dec, 97	Sep, 95
Traffic								
Initial two-way AADTT	1656	1800	2670	2670	2670	946	1409	968
No. of lanes in design direction	2	2	2	2	2	1	1	2
Traffic growth factor (%)	1.2	3	3.5	3.5	3.5	2	2	6.7
Design lane width (ft)	12	12	12	14	12	12	12	12
PCC Layer								
PCC Layer thickness (in)	11	10.5	11	11	12	10	10	9
Material Unit Weight (pcf)	140	142	143	139.2	146	136.9	138.5	142
Cement Type	II	I	II	II	II	II	II	II
Cement Content (Lb/yd^3)	653.4	630	862	532	600	622.1	626.3	622.9
Poisson’s ratio	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
Co-eff of thermal. expansion (in./in./°F X 10^{-6})	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5
Water-cement ratio (w/c)	0.44	0.411	0.35	0.35	0.42	0.42	0.43	0.46
Base Material								
Base Type	PCTB	PCTB	LCB	LCB	PCTB	BDB	BDB	PCTB
Base Thickness (in)	6	4	6	6	6	4	4	4
Base material unit wt. (pcf)	135	135	135.4	135.4	135	135	135	135
Base Modulus (psi)	500,000	500,000	2,000,000	2,000,000	500,000	500,000	500,000	500,000
Treated Subgrade								
Subgrade type	N/A	LTSG	FASG	FASG	FASG	LTSG	LTSG	LTSG
Subgrade modulus (psi)	N/A	50,000	50,000	50,000	50,000	50,000	50,000	50,000
Unit weight (pcf)	N/A	125	125	125	125	125	125	125
Poisson’s ratio	N/A	0.20	0.15	0.15	0.15	0.20	0.20	0.20
Compacted Subgrade								
Subgrade soil type	A-6	A-7-6	A-6	A-6	A-6	A-7-6	A-7-6	A-7-6
Subgrade Modulus (psi) *	9,746	6,268	6,928	6,928	7,523	6,300	6,098	7,262
Plasticity index, PI	15.8	25.7	26	26	23	25.7	27.3	19.9
Percent passing # 200 sieve	71.8	93.3	78.1	78.1	76.9	92.5	91.9	94.3
% passing # 4 sieve	100	100	100	100	98	100	100	100
D_{60} (mm)	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001

* computed by MEPDG

Faulting

Figure 3 shows the comparison between the predicted faulting and the NOS- or LTPP-measured values. No faulting was observed for the SPS-2 Section-6 which has a widened lane of 14 ft. Good agreement was observed for the other SPS-2 section. On other projects, some discrepancies were observed between the predicted and measured faulting at few locations. However, both measured and predicted values in 2003 were negligible for all practical purposes. For example, the Shawnee county project was projected to show faulting of 0.009 inch in 2003. The discrepancies between the NOS-measured and MEPDG-predicted faulting at a few locations were partly due to the way faulting is interpreted in the NOS survey. During NOS reporting, measured faulting is coded as F1, F2, or F3 depending upon the severity of faulting. F1 describes the faulting of greater than 0.125 in but less than 0.25 in, and this is the only severity observed at few locations on some projects. Also, in NOS faulting is rated on a per mile basis and computed from the profile elevation data. No numerical value of faulting is reported by NOS. Thus, the MEPDG analysis showed minimal faulting and it was confirmed by actual observation.

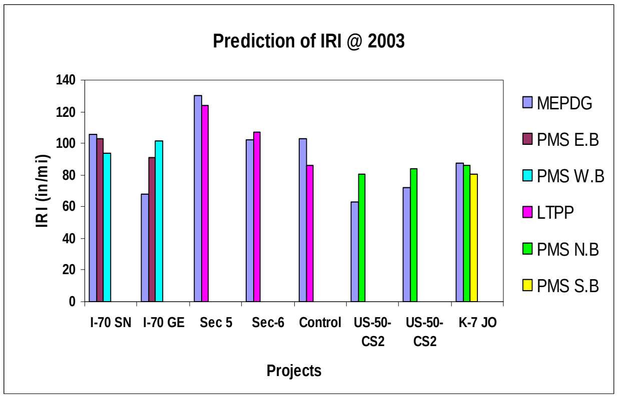

Figure 2. Predicted and measured IRI

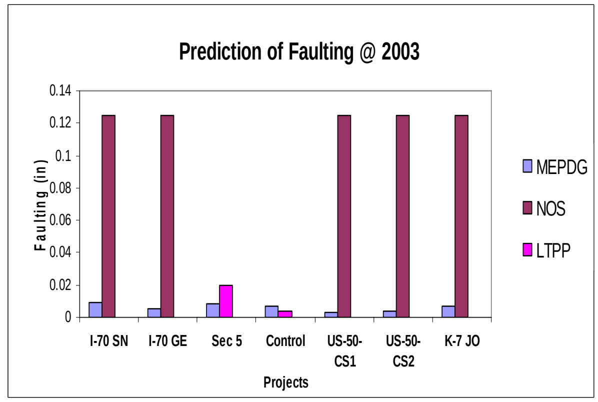

Figure 3. Predicted and measured faulting

Percentage of Slabs Cracked

One of the structural distresses considered for JPCP design in MEPDG is fatigue-related transverse cracking of the PCC slabs. Transverse cracking can initiate either at the top surface of the PCC slab and propagate downward (top-down cracking) or vice versa (bottom-up cracking), depending on the loading and environmental conditions at the project site, material properties, design features, and conditions during construction. The parameter indicates the percentage of total slabs that showed transverse cracking. Figure 4 shows the MEPDG-predicted percentage of slabs cracked values. With the project specific inputs, only Shawnee and Johnson county sections are showing some insignificant amounts of percentage of slabs cracked values of 1.3% and 0.6%, respectively. No measured values were available from the Kansas NOS condition survey report and LTPP database for comparison with the MEPDG predicted cracking. In NOS, no cracking survey is done on rigid pavements. In the LTPP survey, cracking is measured in terms of longitudinal and transverse crack lengths which can not be interpreted as percentage of slabs cracked. Of course, an average value can be computed. It is to be noted that none of the SPS-2 sections in this study showed any cracking up to 2003 in the MEPDG analysis.

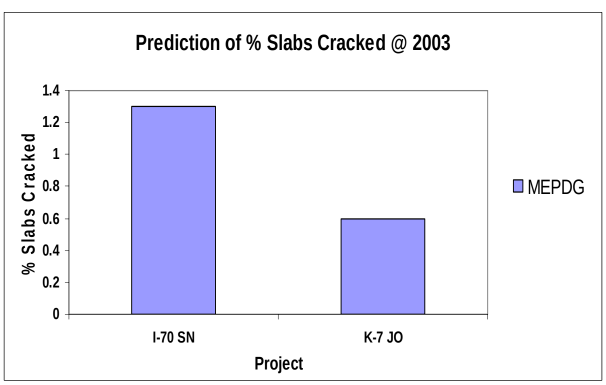

Figure 4. Predicted cracking

Sensitivity Analysis

In this part of the study, the sensitivity of the predicted performance parameters in the MEPDG analysis toward various input parameters has been determined for the I-70 Shawnee County project. The analysis was done for 20 years. The following input parameters were varied at the levels shown, and the predicted IRI, faulting, and percentage of slabs cracked were calculated. All other input parameters were project specific.

1. PCC Thickness (8 to 12 inch, at 0.5-inch increments)
2. AADT (2,080[Low], 12,526 [Medium], and 36,000 [High])
3. Truck (%) (5 [Low], 23.2 [Medium], and 47 [High])
4. Interaction of PCC Thickness and Truck Traffic (10.5 to 12 inch, 5% to 47%, respectively)

Thickness

PCC slab thickness is obviously the basic parameter in the design of rigid pavements. For the I-70 Shawnee county project, the design thickness was 10.5 inch. Thickness was varied from 8 to 12 inches at an increment of 0.5 inch. Figures 5, 6, and 7 show the variation of IRI, faulting, and cracking with thickness, respectively. Figure 5 shows that with increasing thickness, the predicted IRI after 20 years decreases. The predicted IRI decreased from 194 to 115 in/mi when the PCC slab thickness was increased from 8 to 12 inches. However, beyond 10 inches, the predicted IRI remained fairly the same irrespectively of increased thickness. Figure 6 shows that thicker pavement results in reduced faulting. Faulting decreased from 0.018 to 0.014 inches when the thickness was increased from 8 to 12 inches. However, this change in faulting is insignificant for all practical purposes. Figure 7 shows the change in percentage of slabs cracked with thickness. Cracking decreased from 90% to 0.3% when the thickness was changed from 8 to 12 inches. It appears that this parameter is extremely sensitive to thickness. Again, the results show that almost no benefit in terms of fatigue crack resistance can be obtained if the slab thickness is increased beyond 10 inches. The 1986 AASHTO design guide-derived thickness for this section was 10.5 inches.

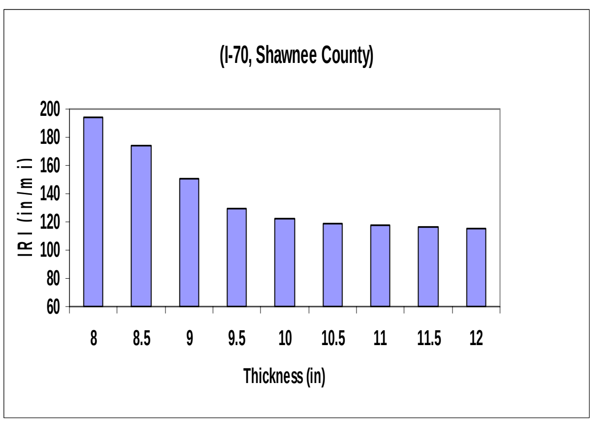

Figure 5. Change in IRI with thickness

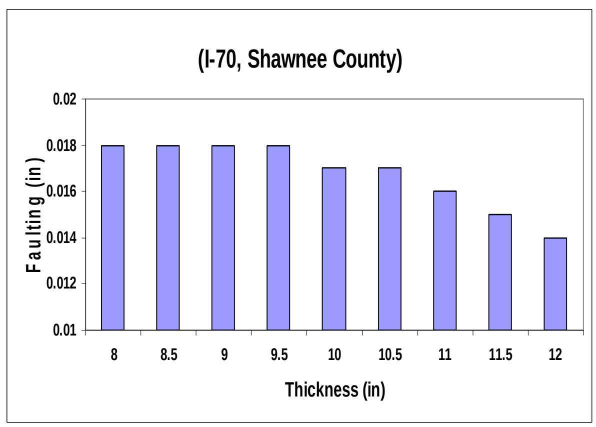

Figure 6. Change in faulting with thickness

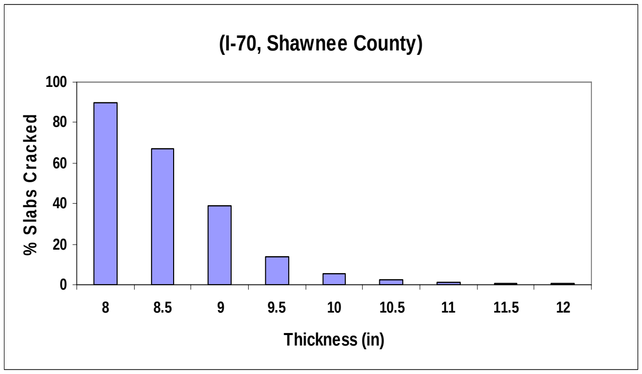

Figure 7. Change in cracking with thickness

AADT

The Annual Average Daily Traffic (AADT) was varied at three levels with a constant truck percentage (23.2%). Figures 8, 9, and 10 show that with increasing AADT, distresses increase significantly. For the three levels of AADT chosen, the predicted IRI ranged from 159 to 111 in/mi. Faulting and percentage of slabs cracked varied from 0.06 to 0.004 in and 23% to 0.2%, respectively.

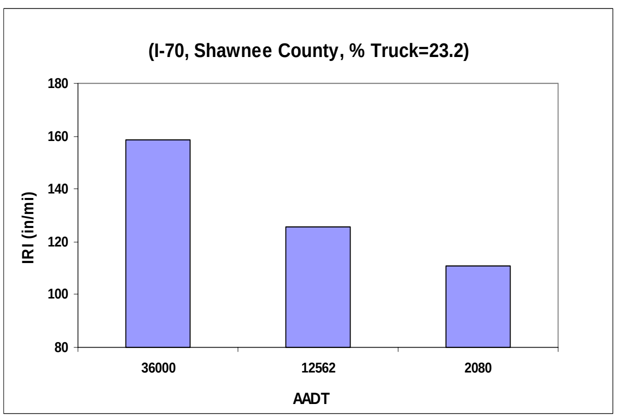

Figure 8. Change in predicted IRI with AADT

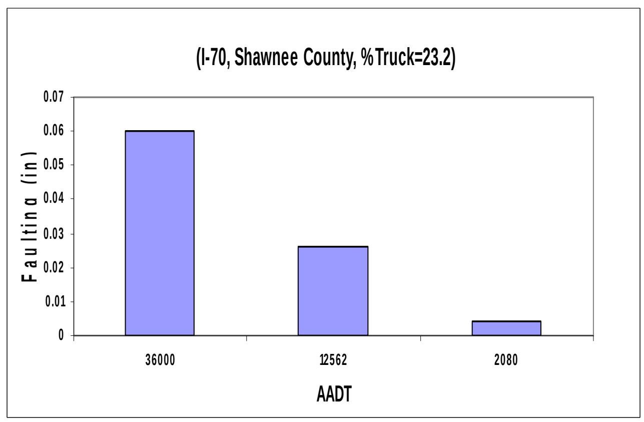

Figure 9. Change in faulting with AADT

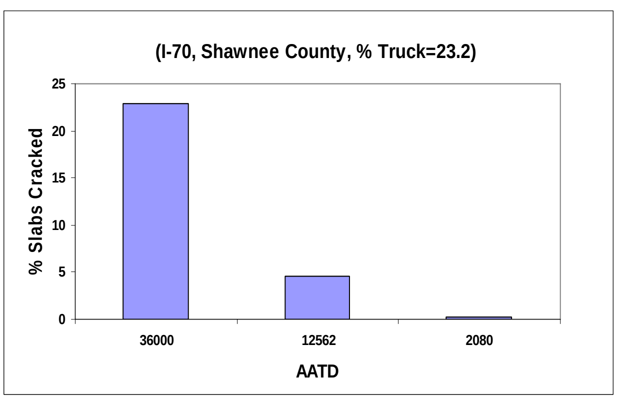

Figure 10. Change in cracking with AADT

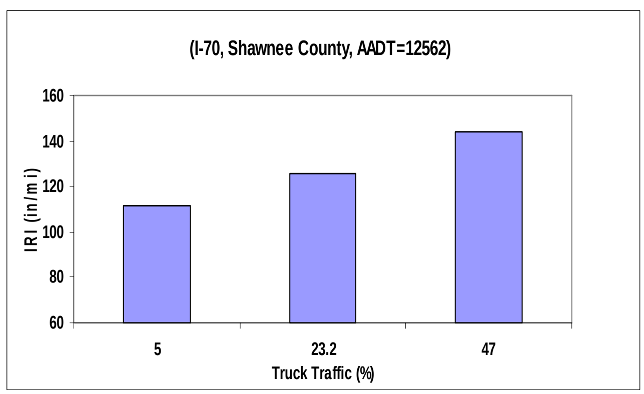

Figure 11. Change in IRI with % trucks

Truck Traffic

Figures 11, 12, and 13 show the variation of predicted distresses with different truck percentage at a constant AADT of 12,562, which is the average AADT level in this study. Figure 11 shows that with increasing percentage of trucks, IRI also increases significantly and ranges from 112 to 145 in/mi. Trend is similar for faulting and percentage of slabs cracked, as shown in Figures 12 and 13, respectively. Faulting varied from 0.006 to 0.047 in, whereas percentage of slabs cracked increased from 0.4% to 14%.

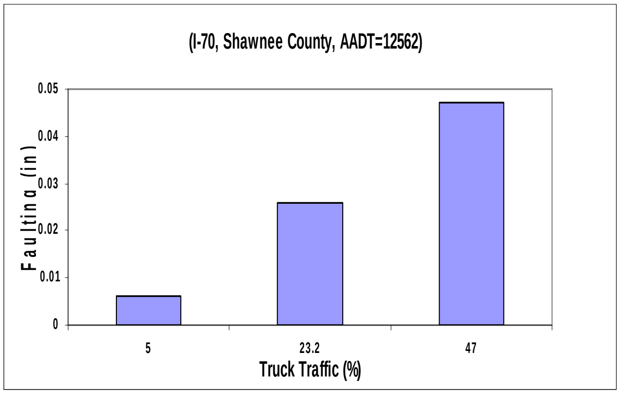

Figure 12. Change in faulting with truck traffic

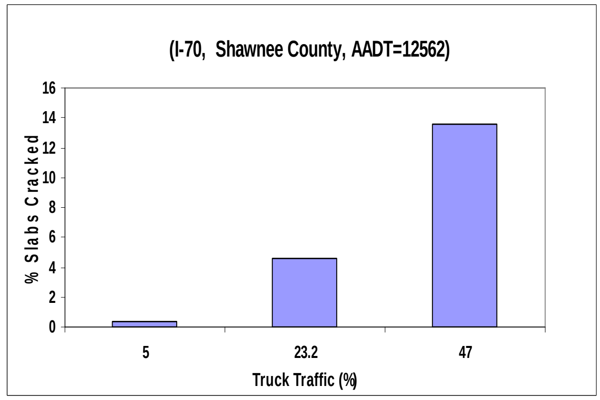

Figure 13. Change in cracking with truck traffic

Interaction of PCC Thickness and Truck Traffic

Truck traffic and PCC slab thickness were varied at three (5%, 20%, and 40%) and four (10.5 to 12 inch at an increment of 0.5 inch) levels, respectively. Thickness variation was done at each level of truck traffic. Figure 14 shows that if the thickness is changed for a lower truck percentage, variation in IRI is not significant compared to the higher percentages of truck traffic. It can be seen that for higher truck percentage, IRI decreases quite significantly with increasing PCC slab thickness. For 40% truck traffic, IRI decreased from 190 to 154 in/mi when the thickness was increased from 10.5 to 12 inches. Figure 15 shows that faulting is not very sensitive to the changes in truck traffic for the given thickness range. However, percentage of slabs cracked is quite sensitive, as shown in Figure 16. For 40% trucks, cracking decreased from 45% to 8% when the thickness was increased from 10.5 to 12 inches.

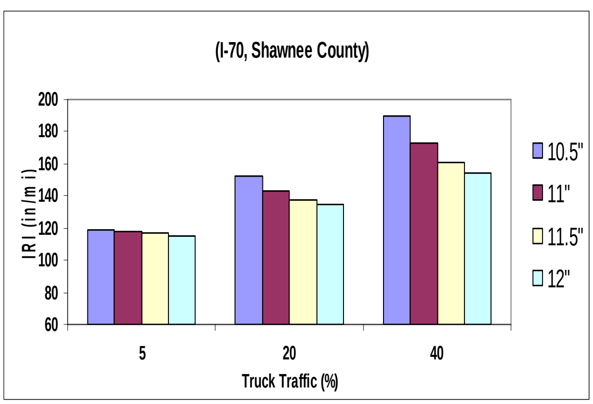

Figure 14. Change in IRI for % truck & thickness

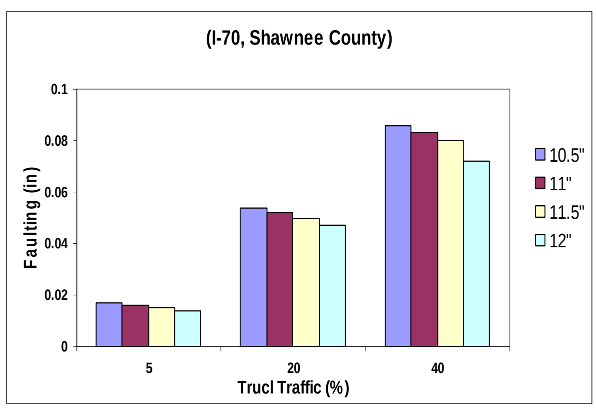

Figure 15. Change in faulting for % truck & thickness

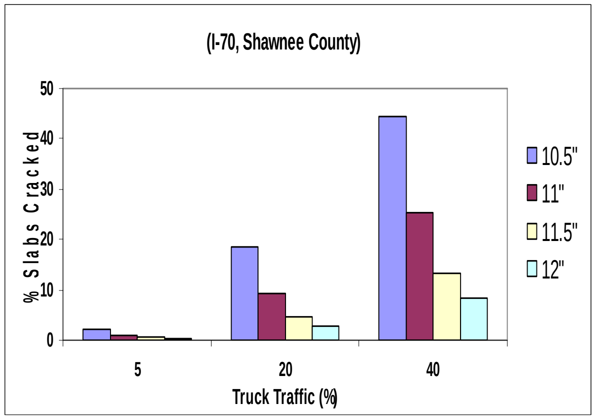

Figure 16. Change in cracking with % truck & thickness

CONCLUSION

This study presents the results of design analysis following the new mechanistic-empirical pavement design guide (MEPDG) for eight in-service concrete pavements in Kansas. The predicted distresses were compared with the measured values. Sensitivity analysis was also done for a project. Based on the study, the following conclusions can be made:

- For most projects in this study, the predicted IRI was similar to the measured values. MEPDG analysis showed minimal or no faulting, and it was confirmed by visual observation.
- IRI is the most sensitive output with respect to traffic.
- Percentage of slabs cracked increases significantly with increasing truck traffic and decreases significantly with increasing slab thickness. Faulting is the least sensitive parameter.

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