7-2017

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Journal of Transportation Engineering, Part A: Systems, Vol. 147, No. 7 (July 2017). DOI.

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Abstract
The *Mechanistic-empirical pavement design guide* (*MEPDG*) provides theoretically superior methodology, as compared with its predecessor, for the design and analysis of pavement structures.

The *mechanistic* part refers to simulating pavement–tire interaction to calculate critical responses within pavement. The *empirical* part means prediction of pavement distress propagation over time using transfer functions that link a critical pavement response to a particular pavement distress. The mechanistic part of *MEPDG* simulates tire–pavement interaction in three steps: subdivision of pavement layers; complex modulus calculation at the middepth of each sublayer, considering velocity and temperature; and running the multilayered elastic theory (MLET) software, *JULEA*.
Although *MEDPG* has a grounded methodology for pavement analysis, it has a number of limitations and unrealistic simplifications that result in inaccurate response predictions. These limitations are primarily related to the pavement analysis approach used in the *MEPDG* framework, *MLET*. By contrast, finite-element (FE) analysis has proven to be a promising numerical approach for overcoming these limitations and simulating pavement more accurately and realistically. Although comparison of MLET with FE analysis has been studied, the difference between FE and *MEPDG* simulations has not been quantified. This study fills that gap by developing linear equations that connect pavement responses produced by these two approaches to pavement analysis. The equations are developed for ten different pavement responses, using a total of 336 cases simulated using FE and *MEPDG* analyses. The cases modeled in simulations were selected to capture extreme conditions, i.e., thick and thin pavement structures with strong and weak material properties. The equations developed can help pavement researchers understand quantitatively the effect of *MEPDG* limitations. In addition, the equations may be used as adjustment factors for *MEPDG* to compute pavement responses more realistically without using computationally expensive approaches, such as FE analysis.

Introduction

All American Association of State Highway and Transportation Officials (AASHTO) pavement design guides issued between the early 1960s and 1993 are based on empirical equations that rely heavily on the results of the AASHTO road test conducted in Ottawa, Illinois, in the late 1950s (AASHTO 2008). For empirical design guides to deliver accurate performance predictions, design inputs for new pavement structures should be similar to the ones used in the AASHTO road test. However, tire type, truck type, axle load limits, and materials have significantly changed since the AASHTO road test.

In 1986, researchers, engineers, and transportation institutions clearly recognized the need to have a pavement design guide that incorporates changes in materials and loadings and that considers direct climate effects on pavement performance (AASHTO 2008). Consequently, NCHRP Project 1-37A was launched in 1998 under the sponsorship of the AASHTO, National Cooperative Highway Research Program (NCHRP), and Federal Highway Administration (FHWA) for the development of an advanced and comprehensive design guide. The *Mechanistic-empirical pavement design guide* (MEPDG Interim Guide 2008) was released in 2004. After that, *MEPDG* was reviewed and revised under NCHRP 1-40A, 1-40B, and 1-40D, which resulted in the development of *MEDPG* design software in 2007 (later known as *DARWin-ME*) and *MEPDG—A manual of practice, interim edition*, in 2008. In August 2013, the current software version, *AASHTOWare Pavement ME Design* (*AASHTOWare Pavement*) was released.

In *MEPDG* design approach, the user assumes a pavement structure as a trial design and provides all other inputs to the software, such as traffic, material properties, and environmental conditions. Structural responses (strain, stress, and/or deflections) are then calculated within the pavement, which refers to the mechanistic part of the guide. By exploiting empirical models, these responses are linked to distress propagations over a design period and are consequently used for an international roughness index assessment. Finally, the user checks the design criteria against predicted ones. If the design requirements are not satisfied, the trial design should be modified and the steps repeated until they are met. Fig. 1 illustrates the *MEPDG* procedure.

Accurate prediction of pavement responses is key for the realistic simulation of distress propagation over time. Although *MEDPG* has a grounded methodology for pavement analysis, it has a number of limitations that result in inaccurate response predictions. Vertical uniform tire pressure, circular contact area, linear elastic analysis of asphalt concrete (AC) and base materials, and the spring model assumption for the layer interface can be given as examples of unrealistic simplifications in *MEPDG*. By contrast, finite-element (FE) analysis simulates pavement responses more realistically in terms of loading conditions and material characterization. However, FE analysis is computationally too costly to adopt into the *MEPDG* framework.

This paper presents the development of linear equations that connect pavement responses obtained from *MEPDG* to FE analysis. These equations can help pavement researchers quantitatively understand the effect of limitations and simplifications of *MEPDG* on pavement responses. Additionally, this study provides *MEPDG* the opportunity to obtain more realistic pavement responses without having to implement advanced structural analysis methods like FE analysis.

The paper is organized as follows. The next section presents the three-dimensional (3D), flexible pavement FE model with detailed explanations. Then the mechanistic part of the *MEPDG* is introduced, along with its limitations. The research methodology that was followed to develop the linear equations is explained. The following sections successively give the results, main findings, and conclusion.

3D Finite-Element Model

Simulating a flexible pavement is a challenging task in terms of material characterization and loading conditions. AC exhibits viscoelastic behaviors, meaning that its behavior depends on temperature, frequency of loading, and time. Granular materials, by contrast, are characterized as nonlinear, stress-dependent anisotropic materials. Their stiffness not only increases with an increasing stress level but also changes in each principal direction. Moreover, pavement is exposed to three-dimensional, nonuniform, and moving tire contact stresses. The literature clarifies the significant effects of these conditions on pavement behavior (Al-Qadi et al. 2008; Siddharthan et al. 1998; Bayat and Knight 2012; Kim et al. 2009; Maina et al. 2012; Myers et al. 1999; Ziyadi et al. 2016). Therefore, it is important to capture them while simulating pavement behavior under the tire load for accurate computation of pavement responses.

The FE analysis has proven to be a promising numerical method for simulating tire–pavement interactions more realistically and accounting for nonlinearity in material characterization. The three-dimensional, flexible-pavement FE model presented in this paper is the ultimate version of more than 10 years of ongoing research by Al-Qadi and coworkers (Al-Qadi and Yoo 2007; Elseifi et al. 2006; Wang et al. 2010; Yoo et al. 2006; Yoo and Al-Qadi 2007; Hernandez et al. 2016). Moreover, the model developed has been successfully validated using experimental field data from various pavement sections (Gungor et al. 2016). The key features of the FE model developed can be categorized into three different groups: (1) model geometry and boundary conditions, (2) material characterization, and

(3) loading conditions and analysis method. A brief explanation of each key feature is given in the following sections.

Model Geometry and Boundary Conditions

The three-dimensional, flexible-pavement FE model was developed using a commercial FE software called *Abaqus v. 6.13*(Simulia 2013). FE is a numerical method that approximates the solution by dividing the model into smaller elements. Consequently, it generates more accurate results as the size of the elements gets smaller. By contrast, computational time increases as the number of elements in the model increases. Additionally, an optimum finite thickness for the subgrade, which is assumed to be infinite in traditional approaches to pavement analysis, should be determined. Therefore, mesh sensitivity analysis was performed to find the element size and subgrade thickness that optimize computational time. Because pavement analysis lacks a closed-form solution, *BISAR*—a multilayer, linear elastic software—was used for mesh sensitivity.

The first step of mesh sensitivity analysis was to create an axisymmetric model in FE that matches the assumptions existing in *BISAR* such as linear elastic material characterization, two-dimensional uniform pressure, and circular contact area. Afterward, the axisymmetric FE model was compared with *BISAR* for six different critical pavement responses, including maximum transverse and longitudinal tensile strain at the bottom of the AC; maximum compressive strain within the subgrade; and maximum vertical shear strain within the AC, base, and subgrade. The model was refined until the difference in the results between the FE model and *BISAR* was about 5%. As a final step, the 3D model was created based on the mesh configuration obtained from the axisymmetric model under the same assumptions and then compared with *BISAR*. The mesh was modified until the 3D model matched with *BISAR* within 5% error tolerance for the same pavement responses. Consequently, the final mesh configuration was obtained to develop a 3D model that considers nonlinearity in material characterization and tire-contact stresses.

Mesh sensitivity analysis should be repeated for each pavement structure. Table 1 presents a sample mesh sensitivity result for three sections used in this study. In Table 1, L1/B1 and L2/B2 stand for the first and second transition zones that provide a smooth transition to the infinite boundary from a densely meshed wheel path, while still maintaining accuracy and reducing computational time (Fig. 2). Apart from infinite elements used at the boundaries, eight node-brick elements were employed elsewhere.

Table 1. Sample Mesh Sensitivity Results for Three Sections

Pavement layers	Mesh properties	Model 1	Model 2	Model 3
		AC: 75 mm base: 150 mm	AC: 125 mm base: 150 mm	AC: 125 mm base: 600 mm
All layers	Length (mm)	4,300	4,800	5,300
	Width (mm)	4,300	4,800	5,300
	Depth (mm)	4,500	4,500	4,500
	L1 = B1 (mm)	1,200	1,450	1,700
	L2 = B2 (mm)	300	300	300

AC	Number of elements	12	15	15
	Bias	1.0	1.2	1.2
Base	Number of elements	12	12	25
	Bias	1.7	1.7	1.0
Subgrade	Number of elements	15	15	15
	Bias	70.0	50.0	30.0
L1 = B1	Number of elements	25	30	25
	Bias	10.0	10.0	15.0
L2 = B2	Number of elements	1	1	1
	Bias	1.0	1.0	1.0

The wheel path length in the model is another geometric property that needs to be optimized. It has been shown that modeling the entire wheel path from any accelerated pavement testing to capture the whole pulse of tire loading is computationally expensive. Therefore, the intention was to make the model wide enough to capture maximum pavement response.

Fig. 3 illustrates the variation of the vertical compressive stress over the total simulation time. Varying wheel path length in the pavement model from 1,000 to 2,000 mm did not show significant difference. Therefore, a 1,000-mm wheel path was considered sufficient to capture the required maximum pavement responses.

Material Characterization

Asphalt concrete was characterized as a linear viscoelastic material in the developed model. Prony series obtained from the complex modulus test were used to capture the linear viscoelastic behavior of the AC. Shear and bulk moduli were then calculated by assuming a constant Poisson's ratio and Prony coefficients [Eqs. (1) and (2)]. Time and temperature dependency of the asphalt concrete was modeled by the William-Landell-Ferry function given in Eq. (3)

(1)

$$G(t) = G_0[1 - \sum_{i=1}^{n} G_i(1 - e^{-t/\tau_i})]$$

(2)

$$K(t) = K_0[1 - \sum_{i=1}^{n} K_i(1 - e^{-t/\tau_i})]$$

where G = shear modulus; K = bulk modulus; t = reduced relaxation time; G_0 and K_0 = instantenous shear and volumetric modulus; and G_i, K_i, and τ_i = Prony series parameters.

In addition

(3)

$$\log(\alpha_t) = \frac{-C_1(T - T_r)}{C_2 + (T - T_r)}$$

where α_t = shift factor; C_1, C_2 = regression coefficients; T = analysis temperature; and T_r = reference temperature.

Traditionally, granular materials in both base and subgrade layers are characterized as linear elastic materials. However, literature (Xiao et al. 2011) has clearly shown that, under high stress levels, granular materials exhibit nonlinear, stress-dependent, cross-anisotropic behavior whose effects on pavement responses are significant.

Granular base stress dependency and nonlinearity diminish for thick pavement sections because of the low stress value below the AC layer. Therefore, the base layer for thick pavement sections was characterized as linear elastic material to reduce computational time while anisotropic stress dependency for granular materials was considered only for thin pavements.

The *MEPDG* model (NCHRP 2004) was used to characterize nonlinear, stress-dependent, cross-anisotropic behavior of the base materials [Eqs. (4)–(6)]

(4)

$$M_{rv} = k_1 \left(\frac{\theta}{p_o}\right)^{k_2} \left(\frac{\sigma_d}{p_o}\right)^{k_3}$$

(5)

$$M_{rh} = k_4 \left(\frac{\theta}{p_o}\right)^{k_5} \left(\frac{\sigma_d}{p_o}\right)^{k_6}$$

(6)

$$M_{rs} = k_7 \left(\frac{\theta}{p_o}\right)^{k_8} \left(\frac{\sigma_d}{p_o}\right)^{k_9}$$

where M_{rv}, M_{rh}, M_{rs} = vertical, horizontal, and shear-resilient moduli, respectively; $\theta = \sigma_1 + \sigma_2 + \sigma_3$ = bulk stresses; σ_d = deviatoric stress; p_o = unit reference pressure;
and k_1, k_2, k_3, k_4, k_5, k_6, k_7, k_8, k_9 = regression coefficients.

Loading Conditions and Analysis Method

One key condition omitted by traditional approaches to pavement analysis is nonuniformity and three-dimensionality of tire loading. In the model developed, contact stresses experimentally measured by the Council for Scientific and Industrial Research in South Africa were applied on element nodes to simulate tire loading, along with a realistic tire footprint (Fig. 4). Details about tire contact stress measurements can be found elsewhere (Hernandez et al. 2013). Additionally, inertial and damping effects of the moving tire were considered in the model. Therefore, the dynamic analysis approach

[Eq. (7)] was exploited to capture both nonlinearity in material characterization and the dynamic effect of the moving tire load

(7)

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=\{P\}$$

where $[M]$ = mass matrix; $[C]$ = damping matrix; $[K]$ = stiffness matrix; $\{P\}$ = external force vector; $\{\ddot{U}\}$ = acceleration vector; $\{\dot{U}\}$ = velocity vector; and $\{U\}$ = displacement vector.

Mechanistic Part of the Design Guide and Its Limitations

The mechanistic part of the *MEPDG* (or *MEPDG* analysis) refers to pavement analysis conducted for obtaining critical responses. *MEPDG* analysis exploits the multilayered elastic theory (MLET) to compute pavement responses under tire loading. Several types of software implement MLET, such as *MnLayer*, *KenLayer*, *BISAR*, and *JULEA*; *MEPDG* uses *JULEA* in its framework.

MEPDG recognizes the fact that AC exhibits viscoelastic behavior and it implicitly considers the effect of time (aging), temperature, and frequency of loading. *MEPDG* incorporates the aging, i.e., stiffening of the AC layer with time through a global aging model. Moreover, temperature within the pavement is determined using the integrated climatic model (ICM). Frequency of loading is calculated as a function of vehicle speed; axle type (single, tandem, or tridem); and pavement structure. In addition, the pavement is divided into sublayers to account for temperature and frequency changes along the AC layer depth. The dynamic modulus (E^*) is computed at the middepth of each sublayer by considering aging, temperature, and frequency, and it is inserted into *JULEA* along with other inputs, such as layer thickness, load, and tire pressure.

In summary, the *MEPDG* analysis consists of a three-step procedure: (1) subdivision of the pavement structure; (2) calculation of the modulus at the middepth of each sublayer, considering aging, temperature, and frequency of loading; and (3) running *JULEA* with the calculated dynamic modulus and other inputs such as thickness and load. Fig. 5 shows the *MEDPG* analysis scheme for computing pavement responses.

Although the mechanistic part of the guide provides a theoretically sound procedure for computing critical pavement responses, it has a number of limitations and simplifications, which may lead to unrealistic response prediction. These limitations and simplifications are mostly caused by the assumptions behind the MLET used in *MEPDG*'s framework. By contrast, the FE method can simulate tire–pavement interaction more realistically, thereby overcoming most of the *MEPDG* analysis limitations.

Limitations of MEPDG Pavement Analysis

Tire–pavement interaction is unrealistically simulated because of the assumptions behind the MLET, such as uniform, two-dimensional (2D) vertical tire pressure and a circular contact area.
Table 2 demonstrates the limitations of *MEPDG* by comparing it with FE analysis.

Table 2. Limitations of the MEPDG Analysis by Comparing It with FEA

Variable	FEA	*MEPDG* analysis
Analysis type	Dynamic analysis, considering motion of the tire and viscoelasticity of the AC	Linear elastic analysis
Tire type	Both WBT and DTA can be simulated	Only DTA can be considered
Contact stress	Nonuniform, realistically measured, 3D contact stresses	2D uniform vertical pressure
Contact area	True measured tire contact area	Circular contact area
Speed and temperature	Directly considered in viscoelastic dynamic analysis	Implicitly considered in dynamic modulus calculations
Friction between layers	Elastic stick model, defined by τ_{max} and d_{max}	Distributed spring model
AC layer material properties	Viscoelastic characterization using Prony series	Dynamic modulus obtained from master curve (*MEPDG* procedure)
Base layer	Stress-dependent, nonlinear model for base—especially important for thin pavement	Linear elastic

In addition to the limitations given in Table 2, Al-Qadi et al. (2008a, b) proved that additional errors were introduced by the *MEPDG* procedure for calculating loading frequency, which translated into inaccurate dynamic modulus calculation. *MEPDG* analysis calculates loading frequency using Eq. (8). Al-Qadi et al. (2008b) proved that this conversion does not realistically simulate loading frequency and is, therefore, the first source of error. In the same study, a novel approach was suggested, based on fast Fourier transformation, and validated by FE simulations

(8)

$$f = \frac{1}{t}$$

where t = time of loading (s) and f = frequency of loading (Hz).

Time of loading is calculated as follows:

(9)

$$t = \frac{L_{\mathrm{eff}}}{17.6 v_s}$$

where v_s = vehicle speed and L_{eff} = effective length.

To calculate effective length, all layer thicknesses are transformed into their equivalent thicknesses based on the stiffness of the subgrade layer. This process is known as Odemark's method of thickness equivalency (Fig. 6).

After all layer thicknesses are transformed, the effective length [Eq. (10)] is computed by assuming that stress is distributed at 45° through the soil depth (Fig. 7). This assumption is considered the second source of error in frequency calculation. The assumption especially fails to capture the far-field effect of the approaching–leaving rolling wheel (Al-Qadi et al. 2008). The detailed procedure for calculating the frequency of loading can be found in NCHRP (2004)

(10)

$$L_{\text{eff}} = 2 \times (a_c + Z_{\text{eff}})$$

where Z_eff = effective length and a_c = radius of contact area.

As explained by Al-Qadi et al. (2008a, b), the two aforementioned errors could result in a discrepancy of up to 140% in loading frequency, depending on vehicle speed and the depth at which the calculation was made.

Research Methodology

In this section, the methodology followed in developing the regression-based equations to quantify the relation between *MEDPG* and FE analysis is explained. The methodology consists of three main parts: determining the simulation matrix, input conversion from FE analysis to *MEPDG* analysis, and implementation of the mechanistic part of *MEPDG*.

Simulation Matrix Selection

Three different sets of inputs are required to conduct pavement analysis: pavement structure (i.e., layer thicknesses); loading parameters; and material characterization parameters. These input parameters can produce values over a very wide range. Hence, an attempt to simulate all possible pavement sections that combines all possible values for each inputs is an impossible task. The study of case selection (i.e., selection of layer thickness, axle loads, and tire pressures), therefore, was needed to determine parametric values required for the pavement simulation.

Linear equations were developed based on regression analysis. As a general rule, to increase reliability, it is important to stay in the range of inputs of the regression-based functions. Therefore, it was decided to cover extreme values for each input so that extrapolation could be avoided during implementation of those equations.

The selection of pavement structure was based on two extreme conditions: low-volume and interstate highways, which could be interpreted as thin and thick pavement, respectively. The selected thicknesses are given in Tables 3 and 4. Loading conditions were selected to cover extreme conditions as well, as indicated in Table 5.

Table 3. Thin (Low-Volume) Pavement Structure Factorial

Pavement layers	Thickness (mm)
AC	75 and 125
Granular base	150 and 600

Table 4. Thick (Interstate Highway) Pavement Structure Factorial

Pavement layers	Thickness (mm)
Wearing surface	25 and 62.5
Intermediate	37.5 and 100

Binder	62.5 and 250
Granular base	150 and 600

Table 5. Selected Tire Loading Cases

Tire type	Axle load (kN)	Tire pressure (kPa)
NG-WBT	26.7	552
NG-WBT	26.7	862
NG-WBT	79.9	552
NG-WBT	79.9	862
NG-WBT	44.4	758
DTA	26.7	552
DTA	26.7	862
DTA	79.9	552
DTA	79.9	862
DTA	44.4	758

To extract material properties for the AC layer, approximately 1,000 complex modulus data were exploited from the Long-Term Pavement Performance database (FHWA 2017). First, the suitable nominal maximum aggregate size (NMAS) was assigned to each AC layer. While 9.5–12.5 mm sizes were selected as the NMAS for the wearing surface, 19.5–25.0 and 25.0–37.5 mm were considered to be typical NMAS for the intermediate and binder layers, respectively. Then, the data were classified based on NMAS and filtered through statistical analysis. Finally, the remaining data were plotted, and one strong and one weak complex modulus data were visually chosen among them for each AC layer.

The database collected by Tutumluer (2008) was used to select appropriate granular material parameters for the base and subgrade layers. First, the estimated stress levels were obtained from Xiao et al. (2011) to calculate the resilient modulus of each material in the database. Afterward, the mean (μ) and standard (σ) deviations of the resilient modulus for all granular materials were computed. Finally, weak and strong resilient test data were determined by setting lower and upper limits as $\mu \pm 2\sigma$. This procedure resulted in selecting representative weak and strong resilient moduli as 140 and 415 MPa, respectively. Details about material selection were provided by Hernandez et al. (2016).

Input Conversion from FE Model to MEPDG

It is critical to convert all inputs used in the FE analysis to suitable parameters for the *MEPDG* analysis to ensure a fair comparison between the two methods. Table 6 compares all inputs from finite-element analysis (FEA) with those of the *MEPDG* analysis.

Table 6. FEA and MEPDG Input Comparison

Variable	FEA (reference)	*MEPDG* analysis
Axle load (P)	Not applicable because contact stress is used in FEA	The axle load applied in contact stress experiment

Contact stress (p)	Nonuniform, 3D stresses (pressure + traction) measured for each known axle load	2D uniform vertical stresses—applied inflation pressure in the experiment
Contact area (A)	True contact area measured for each axle load	Circular (P/p)
Motion of tire (speed)	Tire is moved at a given velocity in simulation	Implicitly considered in dynamic modulus calculations
Temperature	Directly considered in viscoelastic analysis	Implicitly considered in dynamic modulus calculations
Friction between layers	Elastic stick model, defined by $\tau_{\max}$ and $d_{\max}$	Friction coefficient (user input)
AC layer material properties	Linear viscoelastic characterization by Prony series	Dynamic modulus obtained from master curve
Base layer	Thick = elastic modulus	Elastic modulus
	Thin = stress-dependent nonlinear model	
Subgrade	Elastic modulus	Elastic modulus

The same axle load and tire inflation pressure, applied during experiments to measure contact stresses (see the section "Loading Conditions and Analysis Method"), were used as loading inputs for *MEPDG*. The circular contact area was calculated by dividing the axle load by the tire pressure. While speed was used to calculate the loading frequency using Eq. (9), temperature was embedded into the shift factor calculation. The same material parameters (e.g., elastic modulus and master curve) were used as inputs to both the FE and *MEPDG* analyses.

Converting the input parameters used in FE into *MEPDG* analysis form was not complicated except for the pavement interface model parameters. In FE analysis, interaction between layers is simulated by a model called the elastic stick model (ESM). ESM is an improved version of the well-known Coulomb friction model, presented in Eq. (11)

(11)

$$\mu = \frac{\tau_{\max}}{\sigma}$$

where μ = friction coefficient; $\tau_{\max}$ = maximum shear stress; and σ = normal stress at the interface.

The improvement supplied by the ESM is that it allows tangential stress and a certain amount of elastic slip before the surfaces defining the interface start to slip, in contrast to the Coulomb model. Romanoschi and Metcalf (2001) suggested that $\tau_{\max}$ and $d_{\max}$ are 1.415 MPa and 1.6 mm, respectively, for pavement modeling, based on direct shear test results.

By contrast, *MEPDG* analysis assumes uniformly distributed shear spring to connect the interfaces and allow relative horizontal movement between two layers. The spring works in the radial direction and follows the relationship in Eq. (12)

(12)

$$\tau_i = k_i \times (u_i - u_{i+1})$$

where τ_i = radial shear stress at the interface between layers i and $i+1$; $u_i - u_{i+1}$ = relative radial displacement across the interface; and k_i = interface spring stiffness.

To reduce numerical complications, *MEPDG* converts Eqs. (12) and (13) by using the variable l given in Eq. (14)

(13)

$$(1 - l_i) \cdot \tau_i = l_i \cdot (u_i - u_{i+1})$$

(14)

$$k_i = \frac{l_i}{1 - l_i}$$

The variable l is computed using the user-defined parameter mm

(15)

$$l = \begin{cases} 0 & \text{for } m \geq 100{,}000 \\ 10^{-m/E_2} & \text{for } m < 100{,}000 \end{cases}$$

where E_2 = modulus of Layer 2 (below the surface layer).

The spring stiffness is basically the slope of τ/d, i.e., the ratio of $\tau_{\max}$ and $d_{\max}$. After spring stiffness is calculated, the user parameter mm is calculated using Eq. (15).

Implementation of the Mechanistic Part of MEPDG

Initially, the *AASHTOWare* software was considered to obtain responses for 336 cases. However, implementation of the *MEPDG* analysis as a separate numerical tool was needed for two reasons. First, it is time consuming and cumbersome to run the *AASHTOWare* software for 336 cases because the software uses a significant amount of inputs that make comparison to FE results impossible. For instance, the software uses an axle load spectrum; however, only one set of contact stresses belonging to the specific axle-load/tire-pressure combination is considered in each FE simulation. In addition, *AASHTOWare* has temperature-based models for material characterization of the base and subgrade. Conversely, in FE analysis, the base and subgrade are characterized without taking temperature into account, as it would take tremendous effort and time to adopt ICM into the FE model. Second and more importantly, the *AASHTOWare* software gives only critical pavement responses (e.g., tensile strain at the bottom of the AC or compressive strain within the base layer). Comparing shear strain within the pavement is of interest in this study; however, it is not provided as an output in the software. It is believed that shear strain within the AC is relevant to near-surface cracking (Yoo and Al-Qadi 2008).

Therefore, the *MEPDG* analysis was implemented by using the computer languages *MATLAB* and *AutoHotkey*. The main steps to implement and automate the *MEPDG* procedure are listed below:

1. Subdivision of the pavement structure in sublayers.

2. Calculation of the dynamic modulus at the middepth of each sublayer
3. Creation of the input file of *JULEA*.
4. Running *JULEA* (the linear elastic computer program used by *MEPDG*).
5. Postprocessing to obtain pavement responses.

Pavement subdivision and complex modulus calculation were implemented by following the corresponding guidance in *MEPDG*.

Results

The objective was to identify a relationship for converting the pavement responses resulting from FE analysis into *MEPDG*. In total, 336 cases were simulated by *MEPDG* and FE analyses, using compatible input parameters. Because wide-base tires cannot be simulated in *MEPDG* (Gungor et al. 2016), only dual-tire assembly loading cases were selected. After plotting the simulation results, it was observed that the relationship between the two approaches could be represented by linear equations. Developing the equations in complex forms such as using high-degree polynomials or machine-learning regression techniques was avoided, as they would significantly increase the computation effort to implement the equations within the *AASHTOWare Pavement ME Design* software.

However, differences in loading conditions (three-dimensionality and nonuniformity of the contact stresses), material characterization, and layer interaction between FE analysis and *MEPDG* introduce serious challenges that complicate development of the linear equations. Therefore, in order to obtain statistically acceptable correlations, the cases were divided into three groups: thick pavement, thin pavement with strong base material, and thin pavement with weak base material.

Fig. 8 shows the linear equations developed for maximum tensile strain along the traffic and transverse directions at the AC surface. The plots show two lines: an equality line ($y = x$) and a fitted linear function. The equality line is dashed, and the fitted line is solid. The purpose of the equality line is to demonstrate the significance of applying an adjustment factor to each particular response.

The linear equations were developed for ten different pavement responses; two plots are presented herein. The results for all pavement responses are presented in Tables 7–9, with the corresponding coefficients of determination.

Table 7. MEPDG to FEA for Thick Pavement

Response	Location	Linear equation	R^2
Maximum tensile strain in traffic direction	AC surface	$4.63 \times MEPDG + 37.57$	0.933
Maximum tensile strain in transverse direction	AC surface	$3.55 \times MEPDG + 42.15$	0.902
Maximum tensile strain in traffic direction	Bottom of AC	$0.85 \times MEPDG + 0.05$	0.982
Maximum tensile strain in transverse direction	Bottom of AC	$0.99 \times MEPDG - 2.94$	0.969
Maximum vertical compressive strain	Within AC	$0.95 \times MEPDG - 9.46$	0.969
Maximum vertical compressive strain	Within base	$0.65 \times MEPDG - 6.69$	0.947
Maximum vertical compressive strain	Within subgrade	$0.74 \times MEPDG - 10.16$	0.981
Maximum vertical shear strain	Within AC	$0.55 \times MEPDG + 3.21$	0.324
Maximum vertical shear strain	Within base	$0.57 \times MEPDG - 7.03$	0.929
Maximum vertical shear strain	Within subgrade	$0.52 \times MEPDG + 10.71$	0.954

Table 8. MEPDG to FEA for Thin Pavement with Weak Base

Response	Location	Linear equation	R^2
Maximum tensile strain in traffic direction	AC surface	$1.71 \times MEPDG + 8.69$	0.743
Maximum tensile strain in transverse direction	AC surface	$1.16 \times MEPDG + 5.88$	0.891
Maximum tensile strain in traffic direction	Bottom of AC	$0.93 \times MEPDG + 8.08$	0.930
Maximum tensile strain in transverse direction	Bottom of AC	$1.0906 \times MEPDG + 2.43$	0.873
Maximum vertical compressive strain	Within AC	$1.22 \times MEPDG + 5.30$	0.919
Maximum vertical compressive strain	Within base	$2.23 \times MEPDG + 140.1$	0.918
Maximum vertical compressive strain	Within subgrade	$0.73 \times MEPDG + 45.89$	0.817
Maximum vertical shear strain	Within AC	$0.38 \times MEPDG + 21.17$	0.323
Maximum vertical shear strain	Within base	$1.06 \times MEPDG + 6.37$	0.864
Maximum vertical shear strain	Within subgrade	$0.52 \times MEPDG + 45.37$	0.581

Table 9. MEPDG to FEA for Thin Pavement with Strong Base

Response	Location	Linear equation	R^2
Maximum tensile strain in traffic direction	AC surface	$2.51 \times MEPDG + 10.64$	0.607
Maximum tensile strain in transverse direction	AC surface	$1.57 \times MEPDG + 6.54$	0.797
Maximum tensile strain in traffic direction	Bottom of AC	$1.23 \times MEPDG + 11.49$	0.835
Maximum tensile strain in transverse direction	Bottom of AC	$1.33 \times MEPDG + 12.54$	0.739
Maximum vertical compressive strain	Within AC	$1.52 \times MEPDG + 7.93$	0.849
Maximum vertical compressive strain	Within base	$3.64 \times MEPDG + 118.59$	0.894
Maximum vertical compressive strain	Within subgrade	$0.80 \times MEPDG + 101.53$	0.725
Maximum vertical shear strain	Within AC	$0.37 \times MEPDG + 20.48$	0.325
Maximum vertical shear strain	Within base	$1.49 \times MEPDG + 12.78$	0.669
Maximum vertical shear strain	Within subgrade	$0.59 \times MEPDG + 56.05$	0.556

Discussion of Results and Main Findings

As discussed earlier, FE and *MEPDG* procedures have significant differences regarding simulating tire–pavement interaction. Among other factors, 3D nonuniform contact stress and nonlinear material characterization for the base layer (in the case of thin pavement only) seem to result in the highest differences in pavement responses between the two methods. Observations and comments on the results follow:

1. After analyses were performed on all cases (*MEPDG* versus FE analysis), two distinct trends were clearly observed based on AC thickness (thick or thin pavement). Hence, during the correlation analyses, thin and thick pavements were independently investigated.
2. Thin pavements were separated into two groups depending on the base material characterization (i.e., strong or weak) because of its nonlinear and stress-dependent behavior.
3. Higher R^2 values were obtained for thick pavement than for thin pavement because thick pavement responses were less affected by nonuniform contact stresses. Also, stress-dependent and nonlinear characterization complicates the comparison between FE and *MEPDG* for thin pavement cases.

4. The coefficients of the independent variable in the fitted equations for thick pavement are smaller than 1 for all the responses except for the tensile strain at the AC surface. Consequently, MEPDG overestimates the other nine pavement responses
5. There is no regular trend for thin pavements in terms of the coefficients of the independent variable in the fitted equations. Although the *MEPDG* procedure yielded higher values for maximum compressive strain within subgrade, FE resulted in higher values for other types of responses, such as tensile strain at the bottom of the AC and compressive strain within the AC and base layers.
6. FE analysis provided approximately three times higher compressive strain within the base than the *MEPDG* procedure for thin pavement. This observation emphasizes the importance of considering stress-dependent, nonlinear characterization of the base material.
7. The maximum shear strain within AC occurs at shallow depths (around 2.54 cm (1 in.) below the AC surface), so it is governed by the nonuniform, 3D contact stresses, which are not considered in the *MEPDG* analysis. Hence, as shown in Tables 7–9 low R^2 (between 0.3 and 0.4) was obtained for maximum shear within the AC. Low R^2 values illustrate the inability of *MEPDG* analysis to capture nonuniformity and three-dimensionality of tire–pavement contact stresses.
8. Maximum tensile strains at the AC surface occurred far away from the loaded area, where the axle load was the dominant factor on pavement responses. Therefore, the R^2 value was generally high for maximum tensile strain at the surface.
9. The *MEPDG* procedure underestimates the maximum tensile strain at the AC surface for both thin and thick pavement cases, which conforms to the literature.

Conclusion

In the last decade, more states have considered adopting the *MEPDG* for design and rehabilitation of pavement structures. Although *MEPDG* has a more theoretically grounded methodology for pavement analysis, as compared with traditional pavement design guides (e.g., 1972, 1986, and 1993 AASHTO), it has a number of limitations and unrealistic simplifications that may result in inaccurate response predictions. Finite-element analysis is capable of overcoming these limitations and simulating pavement more accurately and realistically; however, it is computationally too expensive to adapt FE within the *MEPDG* framework. In total, 336 cases were simulated, using both FE and *MEPDG* analyses. All input parameters used in the FE analysis were converted into suitable parameters for the *MEPDG* analysis to perform valid comparisons. In addition, wide-base tires (WBTs) could not be simulated in *MEPDG* analysis; hence, only dual-tire assembly (DTA) loading was considered in the simulations. Linear equations were developed to quantify the effect of the limitations of the *MEPDG's* pavement simulation approach by comparing the results from the FE analysis.

The developed equations showed that *MEPDG* fails to capture the effect of nonuniformity and three-dimensionality of contact stresses. The discrepancy becomes significant for pavement responses such as the vertical shear strain within AC and tensile strain at the AC surface, which are considered the cause of near-surface cracking within AC pavements. By contrast, the differences in pavement responses obtained from *MEPDG* and FE analyses are reduced as the pavement response depth increases because the effect of longitudinal and transverse contact stresses diminishes and vertical contact stress becomes the dominant factor in the pavement response. The importance of characterizing granular material as stress dependent was highlighted. Results clearly showed that linear elastic characterization of granular material results in stiffer pavement behavior.

Use of the developed equations to modify *MEPDG* output responses allows for a more realistic computation of pavement responses without using computationally expensive pavement analysis methods. It is recognized that implementation of these equations in *MEPDG* may require recalibration of the *MEPDG* transfer functions.

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