Integrating the Dynamic Modulus of Asphalt Mixes in the 1993 AASHTO Design Method

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The *AASHTO Guide for Design of Pavement Structures 1993* (1993 Design Guide) remains the most widely used pavement design manual by highway agencies and design consultants around the world. As defined in the 1993 Design Guide, the structural coefficient of a pavement layer (a_i) is an abstract measure of the relative ability of a unit thickness of a given material to function as a structural component of the pavement. Nevertheless, the assumed a_i values of the asphalt layers and a proposed relationship between a_i and the resilient modulus do not account for the mechanical and physical properties of asphalt materials, traffic volume and speed, layer thicknesses (thin versus thick pavements), climate, and unbound layer properties. The purpose of this research was to enhance the design methodology incorporated in the 1993 Design Guide by integrating asphalt mixture properties in the design process. The objective was to devise a relationship between the structural coefficient (a_i) of the asphalt layer and the effective dynamic modulus ($|E^*|_{eff.}$) of the corresponding asphalt mix to yield a more realistic estimate of the structural capacity of the asphalt layer. The paper illustrates the development of a multilinear relationship between a_i, ($|E^*|_{eff.}$), and the resilient modulus of the aggregate base layer. Pavement structural designs for various asphalt mixes and design inputs using the developed a_i–($|E^*|_{eff.}$) relationship yielded asphalt layer thicknesses that were generally smaller than those obtained using the typical a_i value of 0.44 for the asphalt layer and closer to thicknesses obtained with the AASHTO mechanistic–empirical design method using the Pavement ME software.

In a recent survey on pavement structural design practices in the United States, it was found that 44 states still use empirical design methods, though some have incorporated, to varying extents, mechanistic–empirical methods as well (*1*). Indiana is, to date, the only state that has fully and officially adopted the *Mechanistic–Empirical Pavement Design Guide* (MEPDG) methodology using AASHTOWare Pavement ME. Following a series of implementation studies, and as of January 1, 2009, the Indiana Department of Transportation mandated the use of MEPDG as the design methodology for all new state highway and Interstate pavements (*2–5*).

Apart from Indiana, many states have initiated official or research-driven MEPDG implementation and local calibration studies (*5–12*). However, adopting MEPDG has seen its challenges, including the need for extensive data, proficiency in pavement design, and financial resources, leaving many agencies constrained to empirical design methods, such as the *AASHTO Guide for Design of Pavement Structures 1993* (1993 Design Guide) (*13*).

Some state highway agencies do not envision implementing MEPDG in the near future and will continue to rely on empirical design methods (*1*). Others believe the 1993 Design Guide is sufficient for some low-traffic roads. One such example is the Washington State Department of Transportation, which developed a new pavement design catalogue based on the 1993 Design Guide, MEPDG, and its own historical records (*14*).

Outside the United States, the 1993 Design Guide is still the primary design methodology adopted in many countries, including all those in the Middle East and North Africa region. This observation is based on an extensive survey that the authors conducted through visits or phone interviews (or both) with highway agencies and major consultants in the region. In the United Arab Emirates, Abu Dhabi is working toward establishing a mechanistic–empirical design manual but is shying away from MEPDG due to the extensive level of detail of its inputs as well as its high cost. Saudi Arabia is gearing toward implementing MEPDG, and research has been conducted on its implementation in Qatar (*15, 16*). Both countries, however, still have to tread a lengthy path encompassing tasks on research, data collection, calibration, and training before full adoption. The highway agencies of other countries, provinces, and cities in the Middle East and North Africa region do not plan on parting from the 1993 Design Guide soon.

Although MEPDG implementation in some U.S. states and in countries outside the United States is progressing, full implementation will require significant time and effort (*5–12*). In the meantime, it is of benefit to build on and improve currently used design methods, namely the 1993 Design Guide, especially as many highway agencies do not intend on abandoning it in favor of MEPDG.

The purpose of this research was to enhance the design methodology for asphalt pavements incorporated in the 1993 Design Guide and its predecessors (the 1972 and 1986 guides) by integrating asphalt mixture properties in the design process. The objective was to provide a more accurate estimate of the structural coefficient (a_i) of the asphalt layer by establishing a new relationship between the structural coefficient and dynamic modulus ($|E^*|_{eff.}$) of the asphalt mix.

BACKGROUND AND LITERATURE

The 1993 Design Guide has intrinsic limitations that multiple research initiatives have attempted to address.

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Limitations of the 1993 AASHTO Design Guide

The basic limitation of the 1993 Design Guide is its inherently empirical nature. It fails to account for mechanistic responses (stresses and strains) of the pavement structure to loading and unloading, and it does not incorporate inputs that are essential for design, such as fundamental material properties, traffic characteristics, and climatic factors. In addition, the design methodology is based on a set of strictly limited parameters used at the AASHO Road Test, among which are a limited time frame of 2 years, one truck type and tire pressure, a traffic volume of less than 2 million axle-load repetitions, one type of dense-graded hot-mix asphalt (HMA), one aggregate base type, one subgrade type, a limited number of cross sections with asphalt layer thicknesses that are less than 6 in., and the climate of northern Illinois (*17*).

The empirical method was not calibrated for the numerous new mixes developed since the inception of the road test. It seems reasonable to assume that these new improved asphalt mixes would have a higher structural capacity, which should subsequently be reflected in a higher structural layer coefficient.

Initiatives to Adjust the Structural Layer Coefficients

The 1972 Design Guide states, "Because of widely varying environments, traffic, and construction practices, it is suggested that each design agency establish layer coefficients applicable to its own experience. Careful consideration should be given before adoption of values developed by others." Yet, currently, 38 states in the United States have adopted a structural coefficient for the asphalt layer that is equal to or less than the value of 0.44 originally recommended by AASHTO in 1962 (*1*). Two states, Alabama and Washington, have recently revised their asphalt structural layer coefficients to better reflect observed flexible pavement performance in their states. Alabama (*18*) increased the asphalt layer coefficient value to 0.54, and Washington (*14*) increased its value to 0.50, which translate to 18.5% and 12% thinner cross sections, respectively (*1*). Likewise, the Vermont Agency of Transportation found that layer coefficients estimated for asphalt concrete were generally 25% to 35% higher than AASHTO's implied maximum of 0.44 (*19*). The methodology used by the Vermont Agency of Transportation was based on falling weight deflectometer backcalculations according to recommendations of the 1993 Design Guide, as well as the NCHRP Project 10-48 report (*20*).

Other research initiatives have investigated falling weight deflectometer backcalculations to estimate the structural coefficient of asphalt layers for new asphalt technologies and mix types, as well as those of nonstandard aggregate base types (*21–25*). The main findings of these studies are summarized below:

- Qi et al. in 1995 found that the layer coefficients of polyethylene-modified mixtures were 75% to 85% higher than the layer coefficients of unmodified mixtures (*26*).
- Hossain et al. in 1997 reported that for crumb rubber–modified asphalt overlays, the average surface layer coefficients varied between 0.11 and 0.46, with most values falling around 0.3. For newly constructed crumb rubber–modified pavements, the structural layer coefficients varied from 0.25 to 0.48, with the average being around 0.35 (*27*).
- Marquis et al. in 2003 noted that the layer coefficient of foamed asphalt ranged between 0.22 and 0.35 (*28*).
- Peters-Davis and Timm in 2009 found that the average recalibrated layer coefficient for HMA was found to be 0.54, with a standard deviation of 0.08 (*18*).

These studies were limited to a narrow set of asphalt mix types and climatic and traffic conditions. In addition, the backcalculation algorithm assumed the asphalt layer was undamaged and elastic as opposed to the more accurate viscoelastic nature with the possibility of existing damage.

SCOPE AND METHODOLOGY

This section summarizes the scope and methodology employed to achieve the stated objective of this study.

Research Scope

The scope of this study entailed developing a relationship between the structural layer coefficient of asphalt (a_i) and the effective dynamic modulus ($|E^*|_{\text{eff.}}$) and investigating the sensitivity of the relationship to various design inputs.

The dynamic modulus ($|E^*|$) is the ratio of stress to strain of an asphalt mix obtained at a range of temperature and sinusoidal loading frequency combinations. The effective dynamic modulus ($|E^*|_{\text{eff.}}$) is that which corresponds to a specific combination of frequency ($f_{\text{eff.}}$) and temperature ($T_{\text{eff.}}$) that is prevalent for the roadway pavement under design (*29–32*). The concept of effective $|E^*|$ is commonly used in knowledge-based tools such as the E* SPT Specification Criteria Program (*33*), the Quality Related Specifications Software (*31, 34*), and the Program for Integrated Analysis of HMA Mix and Structural Designs (*35*).

Developing the a_i–$|E^*|_{\text{eff.}}$ Relationship

The design inputs used to develop the a_i–$|E^*|_{\text{eff.}}$ relationship are summarized in Table 1. Those inputs that are not mentioned were kept constant at their default values as given in Pavement ME. Thirty-three asphalt mixes belonging to five main mix type categories were considered, as detailed in Table 2 and Figure 1 (*36–39*). Consequently, the developed relationship was based on 264 data points, each representing a distinct structural design scenario (Equation 1). All statistical analysis of the data was conducted using R, a software environment for statistical computing and graphics (*40*).

$$\text{number of scenarios} = 33 \text{ mixes} \times 4 \text{ climates} \times 2 \text{ traffic volumes} = 264 \tag{1}$$

Research Methodology

The research methodology consisted of three steps: (*a*) establishing the relationship that relates the structural coefficient a_i to $|E^*|_{\text{eff.}}$, as detailed in Figure 2; (*b*) conducting a sensitivity analysis on the a_i–$|E^*|_{\text{eff.}}$ relationship to the design input parameters defined in the scope; and (*c*) comparing the design thicknesses obtained using

TABLE 1 Design Input Values Considered in Study Scope

Category	Input Parameters Considered in Scope	Category	Input Parameters Considered in Scope
Climate	Chicago, Illinois, cold, MAAT ~52°F (11°C) Saint Louis, Missouri, moderate to cold, MAAT ~56°F (13.5°C) Dallas, Texas, moderate to hot, MAAT ~66°F (19°C) Phoenix, Arizona, hot, MAAT ~75°F (24°C)	Aggregate base layer Thickness Subgrade layer modulus Design life Failure criterion[b]	12 in. (~30 cm) for low traffic volume 15 in. (~38 cm) for high traffic volume 25,000 psi (~172 MPa) 10 years 15% fatigue cracking
Traffic volume	Low volume (1,500 AADTT) High volume (15,000 AADTT)		
Traffic speed[a]	60 mph (~96 km/h) 30,000 psi (~200 MPa)		

NOTE: MAAT = mean annual air temperature; AADTT = average annual daily truck traffic.
[a]It is known that temperature and frequency (speed) have similar effects on fundamental asphalt material properties due to the applicability of time–temperature superposition. Therefore, traffic speed was held constant at 60 mph and was not varied, as its effect can be mapped from that of temperature.
[b]No failure criterion was set for asphalt rutting because rutting is essentially a material-related distress that is independent of structural design.

TABLE 2 Summary of Asphalt Mixes Included in Study Scope

Mix Designation	Mix Description	NMAS (mm)	Source
M-HMA-1	Highly modified HMA mix	19	AUB
M-HMA-2	Highly modified HMA mix	19	AUB
HMA-1	Conventional HMA	19	AUB
HMA-Fib	HMA + fibers	19	AUB
M-HMA-3	Polymer-modified HMA	19	AUB
M-WMA-O	Polymer-modified WMA, organic additive	19	AUB
M-WMA-C	Polymer-modified WMA, chemical additive	19	AUB
M-WMA-F	Polymer-modified WMA, foaming agent	19	AUB
HMA-2	Conventional HMA	25	AUB
WMA-O	WMA, organic additive	25	AUB
WMA-C	WMA, chemical additive	25	AUB
WMA-F	WMA, foaming agent	25	AUB
HMA-3	Conventional HMA	12.5	AUB
HMA-RA-1	HMA + 10% recycled concrete aggregate	12.5	(*36*)
HMA-RA-2	HMA + 20% recycled concrete aggregate	12.5	(*36*)
HMA-RA-3	HMA + 30% recycled concrete aggregate	12.5	(*36*)
HMA-4	Conventional HMA	12.5	(*37*, *38*)
HMA-RAP-1	HMA + 10% RAP	12.5	(*37*, *38*)
HMA-RAP-2	HMA + 25% RAP	12.5	(*37*, *38*)
HMA-RAP-3	HMA + 40% RAP	12.5	(*37*, *38*)
SM-1	HMA + 15% RAP	9.5	(*39*)
SM-2	Polymer-modified HMA + 12% RAP	12.5	(*39*)
SM-3	HMA + 25% RAP	12.5	(*39*)
SM-4	Polymer-modified HMA + 15% RAP	12.5	(*39*)
BM-1	HMA + 15% RAP	25	(*39*)
BM-2	HMA + 15% RAP	25	(*39*)
BM-3	HMA + 15% RAP	25	(*39*)
BM-4	HMA + 25% RAP	25	(*39*)
BM-5	HMA + 25% RAP	25	(*39*)
BM-6	HMA + 25% RAP	25	(*39*)
SMA-1	Polymer-modified SMA	12.5	(*39*)
SMA-2	Polymer-modified SMA	12.5	(*39*)
SMA-3	SMA	12.5	(*39*)

NOTE: NMAS = nominal maximum aggregate size. Testing was performed at the American University of Beirut (AUB) Superpave Laboratory.

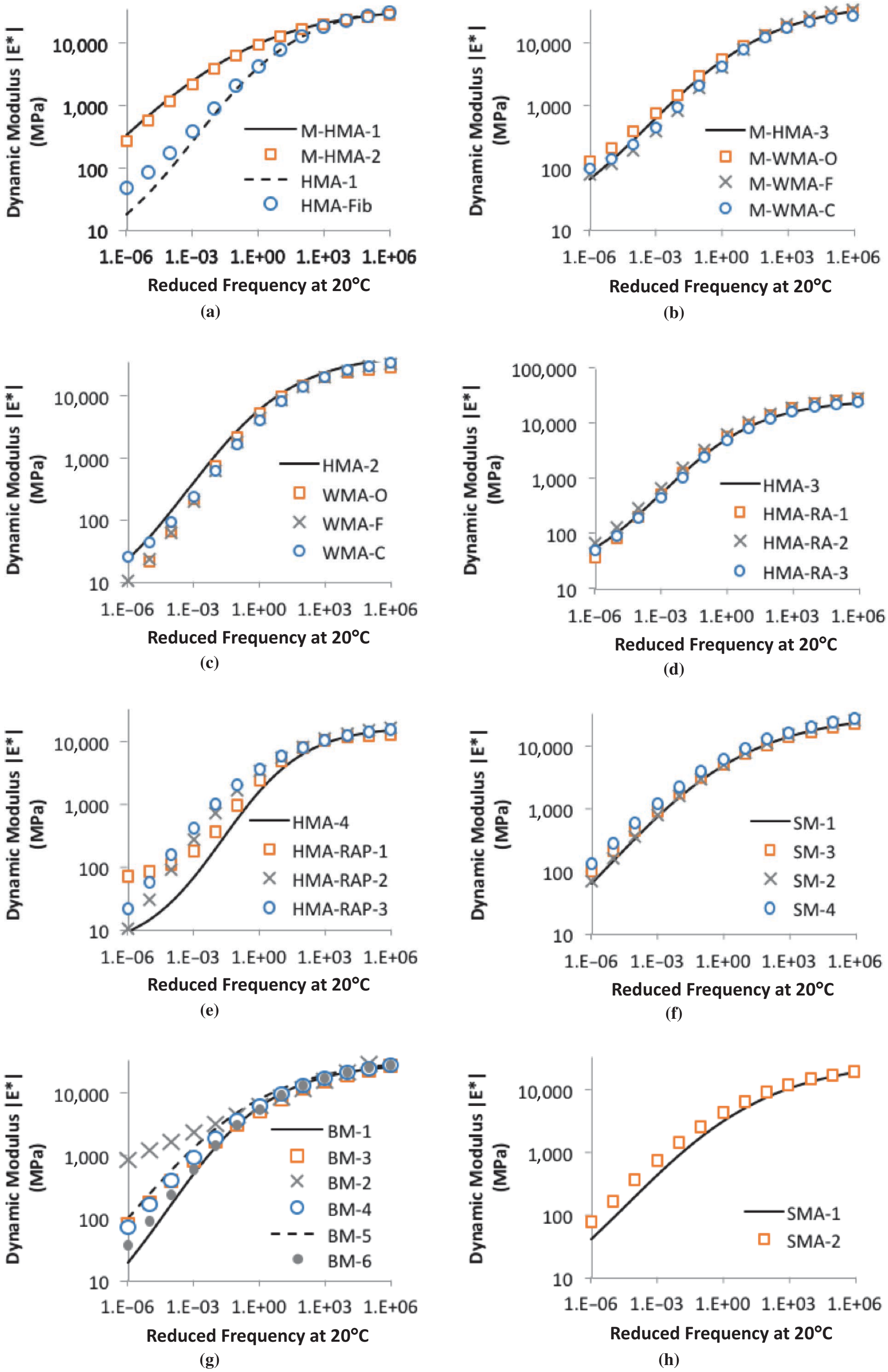

FIGURE 1 Dynamic modulus master curves of mixes included in scope: (*a*) polymer- and fiber-modified mixes, (*b*) WMA with polymer-modified binder, (*c*) WMA with neat binder, (*d*) HMA with recycled aggregate, (*e*) HMA with RAP, (*f*) Virginia Department of Transportation (DOT) surface mixes, (*g*) Virginia DOT base mixes, and (*h*) Virginia DOT SMA mixes.

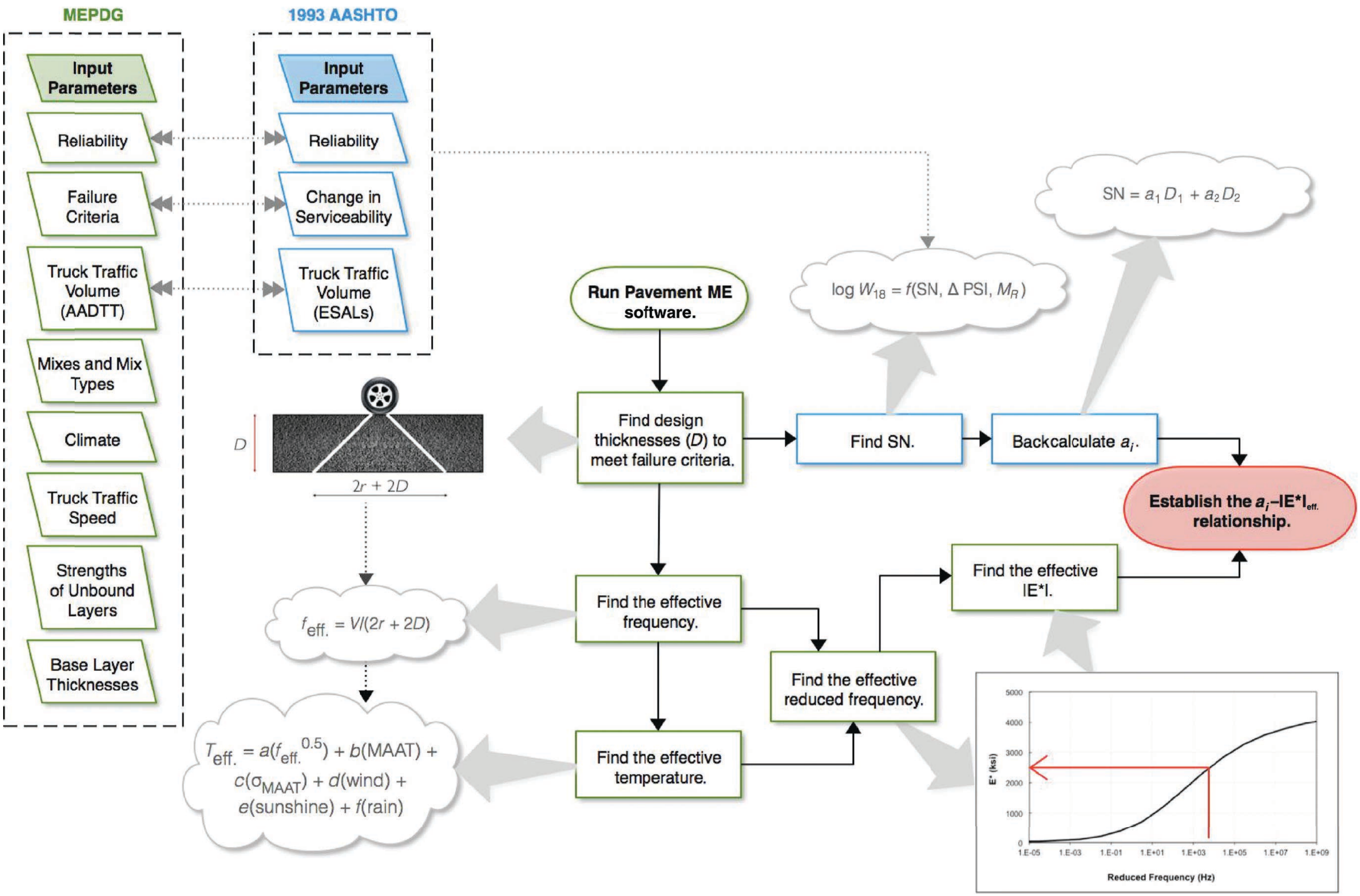

FIGURE 2 Flowchart detailing development of a_i–$|E^*|_{\text{eff.}}$ relationship.

the developed model to those acquired using the 1993 Design Guide and MEPDG (Pavement ME software) for a given fatigue failure limit of 15%.

Assumptions

The methodology was based on an underlying assumption that the MEPDG method using Pavement ME is more reliable than that of the 1993 Design Guide and generates design thicknesses that are closer to optimal. Other assumptions included the following:

- As-built air void content ranges from 6% to 8% depending on the mix type.
- Fatigue cracking of 15% is equivalent to a change in pavement serviceability index (PSI) of 1.2. This equivalence is obtained by matching the MEPDG recommended performance criteria for fatigue cracking (*41*) to the recommended 1993 Design Guide serviceability loss criteria (*13*) for Interstate pavements (*42*).
- Reliability was assumed to be 90% for both design methods. Although the concept of reliability is different in each of the two design guides, change in reliability was assumed to have a similar effect on design thickness (*43*).
- Conversion of traffic from average annual daily truck traffic to equivalent single-axle loads was based on calculations performed in Pavement ME, which assumes a structural number of 5 and a terminal serviceability of 2.5 (*41*).

RESULTS AND ANALYSIS

The structural asphalt layer coefficient (a_i) acquired by using the methodology presented in Figure 2 was found to be primarily dependent on three variables: mix type, climate (mainly temperature), and traffic speed. Figure 3 distinguishes between the a_i of each mix type under the different climatic conditions considered in this study. The effect of traffic speed is not portrayed directly, but it is represented by the effect of climate due to the applicability of time–temperature superposition in the asphalt material's linear viscoelastic range (*44*).

A one-way analysis of variance was conducted on the a_i values to statistically validate the effect of mix type on the layer coefficient. The analysis revealed a significant effect of mix type on the average a_i at the 95% confidence level [$F(9, 228) = 43.87$, p-value < 2E–16], as visualized in the box plots in Figure 3. A post hoc Tukey honest significant difference test was conducted to evaluate the statistical significance of the difference between the average a_i among the mixes for p-value < .05. The results showed no significant difference among HMA and warm-mix asphalt (WMA) (for both conventional and polymer-modified binder), recycled asphalt pavement (RAP) with polymer-modified binder and conventional HMA, and all HMA mixes with unmodified binder. Therefore, the mixes included in the scope could be categorized into three groups when developing the a_i–$|E^*|_{eff.}$ relationship.

Relationship Between Asphalt Layer a_i and $|E^*|_{eff.}$

The research methodology presented in Figure 2 culminated in establishing a linear relationship (Equation 2) between the structural layer coefficient of the asphalt layer (a_i) and the effective dynamic modulus ($|E^*|_{eff.}$) of the asphalt mix.

$$a_i = m|E^*|_{eff.} + \text{int.} + \varepsilon \tag{2}$$

where

m = slope
int. = intercept, and
ε = standard error = $N(0, \sigma^2)$.

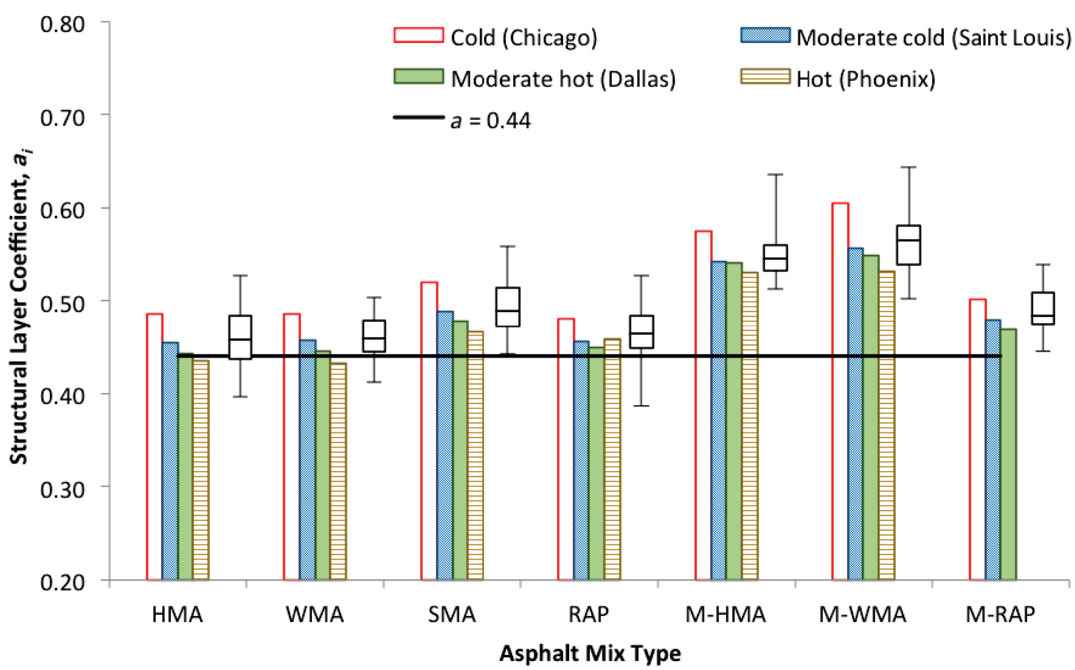

FIGURE 3 Average structural asphalt layer coefficient (a_i) for various asphalt mix categories along with box plots showing the effect of mix type on a_i (M = polymer modified).

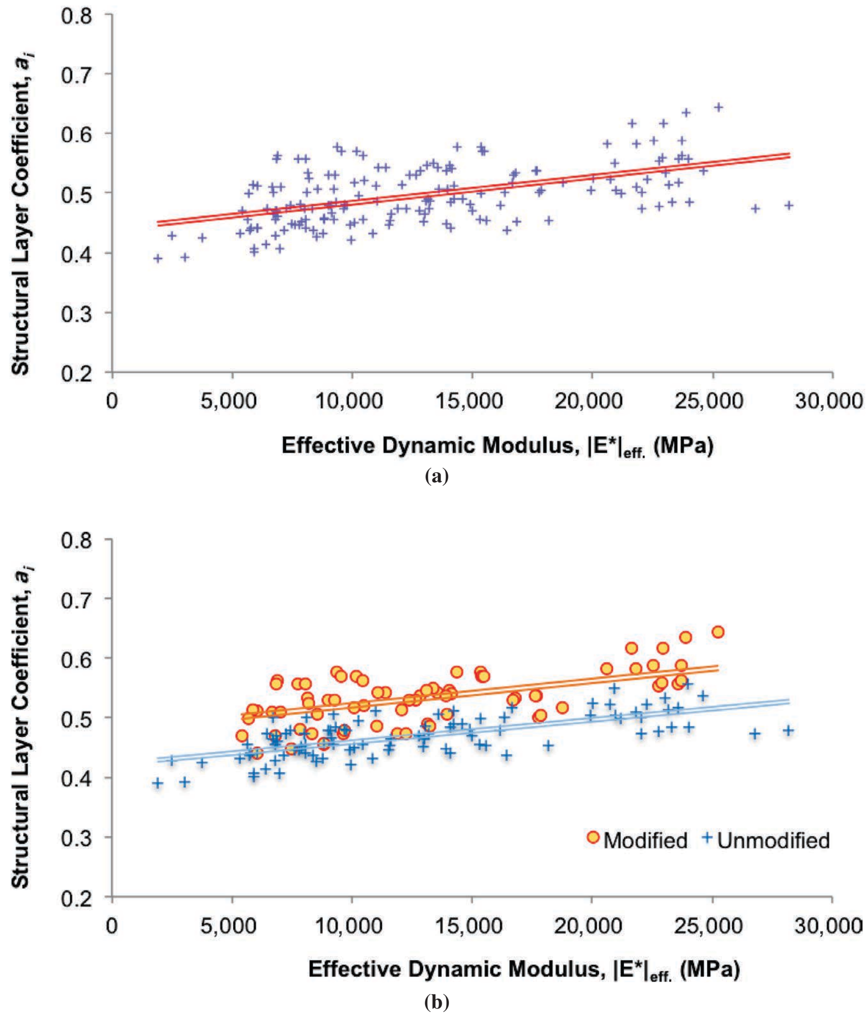

FIGURE 4 (*a*) Universal a_i–$|E^*|_{eff.}$ relationship (based on database considered in scope) and (*b*) a_i–$|E^*|_{eff.}$ relationship for mixes with polymer-modified binder versus unmodified binder.

Figure 4*a* shows the universal a_i–|E*| relationship. The term “universal” is used to refer to the general a_i–|E*| curve established based on the scope of the study, which therefore cannot be regarded as a generic curve. The linear fit shown in Figure 4*a* was significant at the 95% confidence level (*p*-value for the regression = 3.45E–13), and it accounted for more than 27.6% of the variability in the data (R^2). The model indicated that, on average, an increase in asphalt stiffness of 5,000 MPa would lead to an increase of approximately 0.02 in a_i.

Further inspection of the data and the universal a_i–$|E^*|_{eff.}$ relationship revealed that the regression could be improved by segregating the data points according to mix category and fitting a linear model for each separately, with the (virtual) intercept being a function of mix type. The mixes were separated into two broad categories: mixes with polymer-modified binder and those with unmodified binder. As shown in Figure 4*b*, the regression improved with polymer-modified mixes having, on average, higher a_i than mixes with unmodified binders. The new linear models for unmodified and polymer-modified mixes were both statistically significant at the 95% confidence level (*p*-value = 2.97E–14 and 3.25E–8, respectively), and they accounted for greater variability in the data than the universal curve shown in Figure 4*a* (29.8% and 35.6%, respectively).

Statistical Justification

The structural layer coefficients (*y*-axis) were found to be normally distributed about $|E^*|_{eff.}$ (*x*-axis), and the residuals were well scattered for all fits. All linear regressions yielded satisfactory R^2 values, and their slopes and intercepts were found to be significant at the 95% confidence level.

Figure 5 demonstrates a summarized a_i–$|E^*|_{eff.}$ relationship for three main mix categories: (*a*) conventional HMA and RAP; (*b*) stone mastic asphalt (SMA), dense-graded RAP, and polymer-modified HMA; and (*c*) polymer-modified HMA and WMA. The color gradient represents the effect of temperature on $|E^*|_{eff.}$ and, accordingly, on a_i. Hotter temperatures (red, left) yielded lower $|E^*|_{eff.}$, and thus lower a_i, and colder temperatures (blue, right) yielded higher $|E^*|_{eff.}$, and thus higher a_i.

Sensitivity Analyses and Interpretation of Results

Design input values (Table 1) were initially assumed for the back-calculation a_i. These inputs may have affected the output of the MEPDG (Pavement ME) runs, and in turn, may have affected the

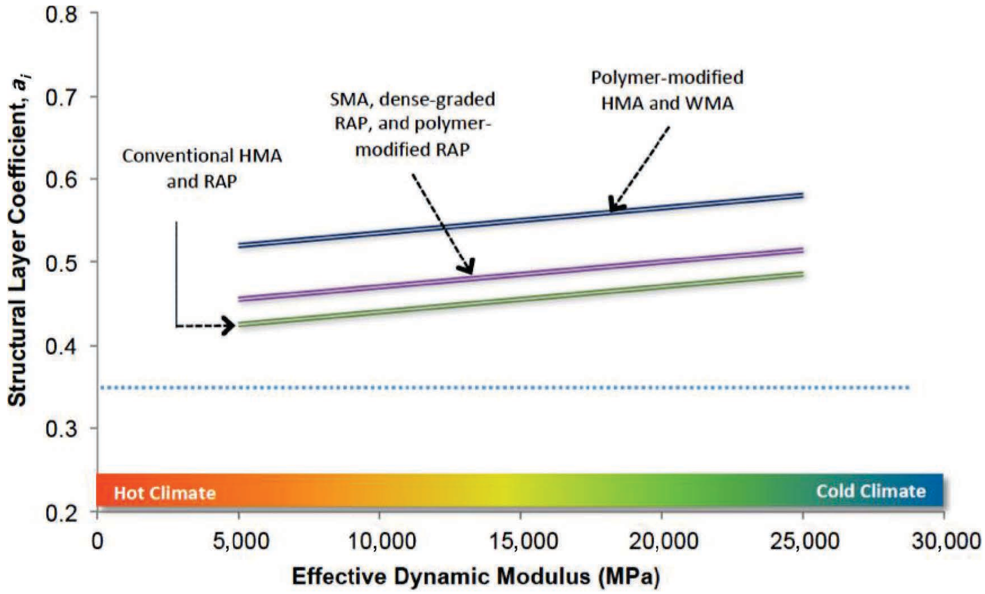

FIGURE 5 a_i–$|E^*|_{eff.}$ relationships for different asphalt mix categories based on the research scope (slope = 3 E–06; R^2 = .791).

backcalculated a_i. Therefore, a sensitivity analysis was conducted to examine the sensitivity of the backcalculated a_i, and subsequently the a_i–$|E^*|_{eff.}$ relationship, to each of the input parameters. Because fatigue cracking was used as the basis for structural design, it was essential to conduct the sensitivity analyses on inputs that have the greatest effect on fatigue-cracking predictions in the MEPDG. Based on the literature review, four main parameters were selected for investigation: traffic volume, modulus of the aggregate base layer (E_{base}), thickness of the aggregate base layer, and modulus of the subgrade (*45, 46*). In addition, it was essential to investigate the assumption that 15% fatigue cracking (MEPDG) is equivalent to a change in serviceability (ΔPSI) of 1.2 (1993 Design Guide). All sensitivity analyses were conducted for two climatic conditions: cold (represented by Chicago, Illinois) and moderate to hot (represented by Dallas, Texas).

Sensitivity to Fatigue–ΔPSI Correlation

An assumption in the backcalculation of the structural layer coefficients was the equivalency of 15% fatigue cracking to a ΔPSI of 1.2. The sensitivity of a_i to that assumption was investigated in two steps. Initially, the change in PSI was varied between 1.1 and 1.3, in increments of 0.1, keeping the fatigue cracking limit at 15%. It was found that the sensitivity of the backcalculated a_i to the assumed ΔPSI was negligible.

Second, both ΔPSI and the failure limit of fatigue cracking were varied simultaneously. Fatigue cracking limits of 10%, 15%, and 20% were considered. The challenge was to associate the fatigue limit with a representative ΔPSI. Although empirical mathematical relationships that relate PSI to international roughness index (IRI) exist, the correlation between PSI and IRI lacks a fundamental comparative basis (*47–49*). PSI includes rideability parameters such as longitudinal and transverse variations in slope (roughness) that are not considered in IRI calculations. Moreover, no consensus exists on the extent of the contribution of fatigue cracking to IRI (*41*).

It was thus assumed that 10% fatigue cracking was equivalent to ΔPSI of 1.1 and 20% fatigue cracking to ΔPSI of 1.7. Accordingly, almost no effect on the backcalculated a_i was observed. More on this topic is available elsewhere (*42*).

Sensitivity to Traffic Volume

The average annual daily truck traffic was varied from 10,000 to 20,000 in increments of 5,000, showing no effect on a_i.

Sensitivity to Resilient Modulus of Subgrade

The resilient modulus of the subgrade was varied from 15,000 to 25,000 pounds per square inch (psi) (~103 to 172 MPa) in increments of 5,000 psi (~34.5 MPa). Results showed that for polymer-modified mixes, a_i was insensitive to the resilient modulus of the subgrade, but for mixes with unmodified binder, a_i was slightly sensitive to it. The effect of the subgrade resilient modulus may be masked by the thickness and modulus of the base layer. The sensitivity of a_i to the resilient modulus of the subgrade must be investigated further.

Sensitivity to Thickness of Aggregate Base Layer

The thickness of the aggregate base layer was varied from 13 to 17 in. (~33 to 43 cm) in increments of 2 in. (~5 cm). Results showed that a_i was relatively insensitive to the thickness of the aggregate base layer.

Sensitivity to Resilient Modulus of Aggregate Base

The modulus of the aggregate base layer (E_{base}) was varied between 20,000, 22,500, 25,000, 27,500, 30,000, and 35,000 psi (138, 155, 172, 190, 207, and 241 MPa, respectively). Results showed that the assumed modulus had a significant effect on the value of a_i.

On average, a_i decreased by 0.0055 for every increment in E_{base} of 1 kip per square inch (ksi) (6.9 MPa). This decrease is reasonable because E_{base} has a central effect on the pavement's structural number in the 1993 Design Guide and on the stress–strain response of the asphalt layer. It is important to distinguish between asphalt layer (structural property) and asphalt material. By definition, the layer coefficient is a combined structural and material indicator. It not only indicates the integrity of the material but also its ability to act as a structural component in the given pavement. Therefore, it was expected that the structural coefficient would not only be dependent on the asphalt material type and material properties but also on the layer's boundary conditions, represented here by the modulus of the base layer.

Relationship and Discussion

On the basis of the sensitivity analysis, in particular the effect of E_{base} on a_i, it can be concluded that the curves developed in Figure 5 were specific to the assumed E_{base} of 30,000 psi. Thus, it is important to account for other values of the aggregate base modulus and the subsequent effect on a_i. To do so, the base moduli included in the sensitivity were considered. By applying linear regression to the three mix types (conventional HMA, polymer-modified HMA, and WMA), the final regression equation was developed and is shown in Equation 3, where a_i is a function of $|E^*|_{eff.}$ and the logarithm of E_{base} (through a Box–Cox transformation analysis).

$$a_i = m|E^*|_{eff.} + n\log(E_{base}) + \text{int.} \tag{3}$$

where m is the slope (unit change in a_i for unit change in $|E^*|_{eff.}$) and n is the slope [unit change in a_i for unit change in $\log(E_{base})$].

On the basis of the results of the three mixes considered, it was found that the effect on E_{base} was fairly independent of mix type; it was represented by an average slope of −0.32. As such, for every unit increase in $\log(E_{base})$ (i.e., as E_{base} increased by 10 times), the layer coefficient a_i decreased by 0.32, given that $|E^*|_{eff.}$ was constant.

To portray the relationship between $|E^*|_{eff.}$, E_{base}, and the structural coefficient of the asphalt layer, a nomograph was developed based on the relationship of a representative conventional HMA mix (Figure 6). For example, for an effective dynamic modulus of 17,000 MPa and an aggregate base with a modulus of 20 ksi (138 MPa), a_i was approximately 0.51.

VALIDATION

To assess the developed a_i–$|E^*|_{eff.}$–E_{base} relationship, 32 scenarios of various asphalt mix types, climatic locations (Saint Louis, Missouri; Houston, Texas; and Muncie, Indiana), traffic volumes and speeds, and aggregate base moduli were considered (Table 3). The design thicknesses acquired by using the 1993 Design Guide method ($a_i = 0.44$) and those acquired by the a_i–$|E^*|_{eff.}$–E_{base} relationship were compared with design thicknesses resulting from analysis using MEPDG (Pavement ME). The results, summarized in Figure 7, revealed that the developed relationship yielded design thicknesses that were notably close to those acquired by using MEPDG (Pavement ME) (Figure 7*b*); in contrast, the 1993 Design Guide tended to consistently overpredict the design thickness compared with MEPDG (Figure 7*a*). It is thus evident that the a_i–$|E^*|_{eff.}$–E_{base} relationship developed in this research efficiently accounted for material properties (asphalt mix

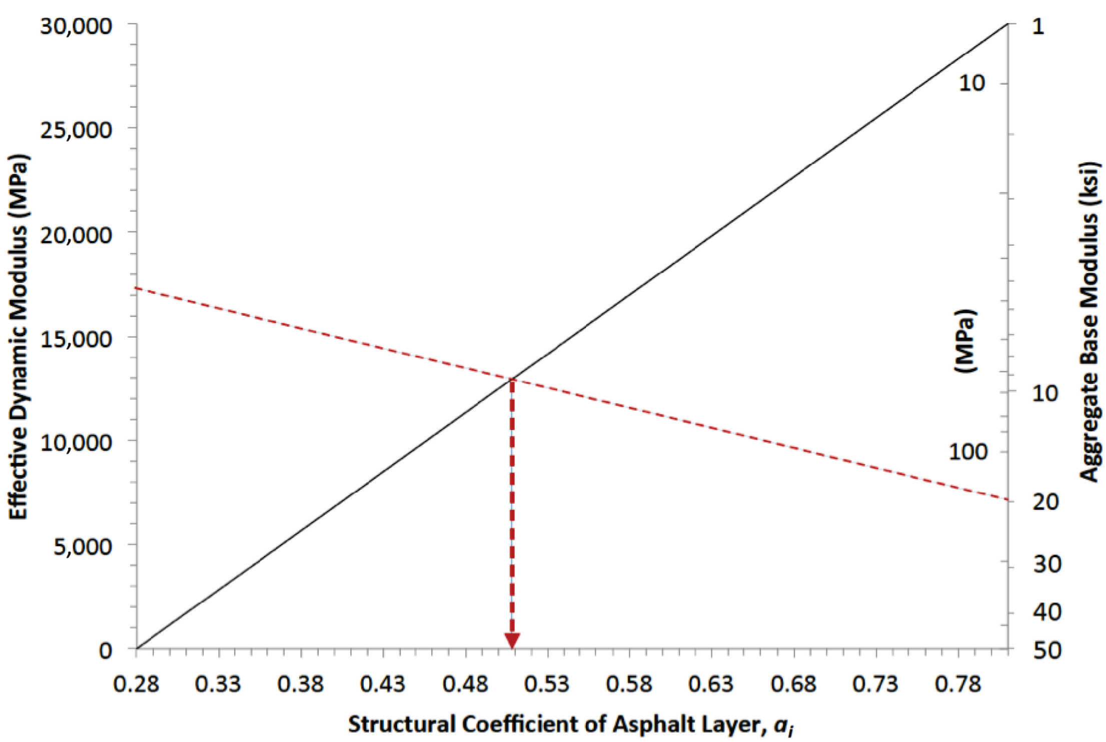

FIGURE 6 Sample nomograph for calculating the structural layer coefficient of an average conventional HMA mix based on $|E^*|_{eff.}$ and E_{base}.

TABLE 3 Comparison of Output of Design Tool to Design Thicknesses Using 1993 AASHTO Design Guide and AASHTOWare Pavement ME

Mix Type	Climate	Traffic Volume (AADTT)	Traffic Speed (mph)	Base Modulus (psi)	Derived a_i	Thickness Based on Derived a_i (in.)	Thickness Based on $a_i = 0.44$ (in.)	Thickness Based on MEPDG (Level 1)
HMA	Saint Louis	5,000	60	25,000	0.5	6	6.9	6.2
	Saint Louis	20,000	60	25,000	0.5	8	9.1	8.2
	Saint Louis	35,000	60	25,000	0.5	9	10.2	9.15
	Saint Louis	20,000	30	25,000	0.48	8.4	9.1	8.6
	Houston	20,000	60	25,000	0.46	8.7	9.1	8.55
	Houston	20,000	30	25,000	0.44	9.1	9.1	9
	Muncie	20,000	60	25,000	0.53	8.2	9.1	8
	Saint Louis	20,000	60	35,000	0.51	7.3	9.1	7.7
HMA-PMB	Saint Louis	5,000	60	25,000	0.57	5.3	6.9	5.35
	Saint Louis	20,000	60	25,000	0.57	7	9.1	7.1
	Saint Louis	35,000	60	25,000	0.57	7.9	10.2	7.9
	Saint Louis	20,000	30	25,000	0.56	7.2	9.1	7.35
	Houston	20,000	60	25,000	0.55	7.2	9.1	7.15
	Houston	20,000	30	25,000	0.54	7.4	9.1	7.4
	Muncie	20,000	60	25,000	0.6	6.7	9.1	6.95
	Saint Louis	20,000	60	35,000	0.54	6.5	9.1	6.7
WMA-PMB	Saint Louis	5,000	60	25,000	0.58	5.3	6.9	5.4
	Saint Louis	20,000	60	25,000	0.57	7	9.1	7.2
	Saint Louis	35,000	60	25,000	0.57	7.8	10.2	8.05
	Saint Louis	20,000	30	25,000	0.56	7.6	9.1	7.5
	Houston	20,000	60	25,000	0.55	7.2	9.1	7.2
	Houston	20,000	30	25,000	0.54	7.4	9.1	7.55
	Muncie	20,000	60	25,000	0.59	6.8	9.1	7.05
	Saint Louis	20,000	60	35,000	0.53	6.5	9.1	6.75
HMA-RAP	Saint Louis	5,000	60	25,000	0.46	6.5	6.9	6.6
	Saint Louis	20,000	60	25,000	0.46	8.7	9.1	8.65
	Saint Louis	35,000	60	25,000	0.46	9.7	10.2	9.65
	Saint Louis	20,000	30	25,000	0.45	8.9	9.1	9
	Houston	20,000	60	25,000	0.45	8.9	9.1	9
	Houston	20,000	30	25,000	0.44	9.1	9.1	9.4
	Muncie	20,000	60	25,000	0.48	8.4	9.1	8.55
	Saint Louis	20,000	60	35,000	0.43	9	9.1	8.15

NOTE: PMB = polymer-modified binder; subgrade modulus = 15,000 psi.

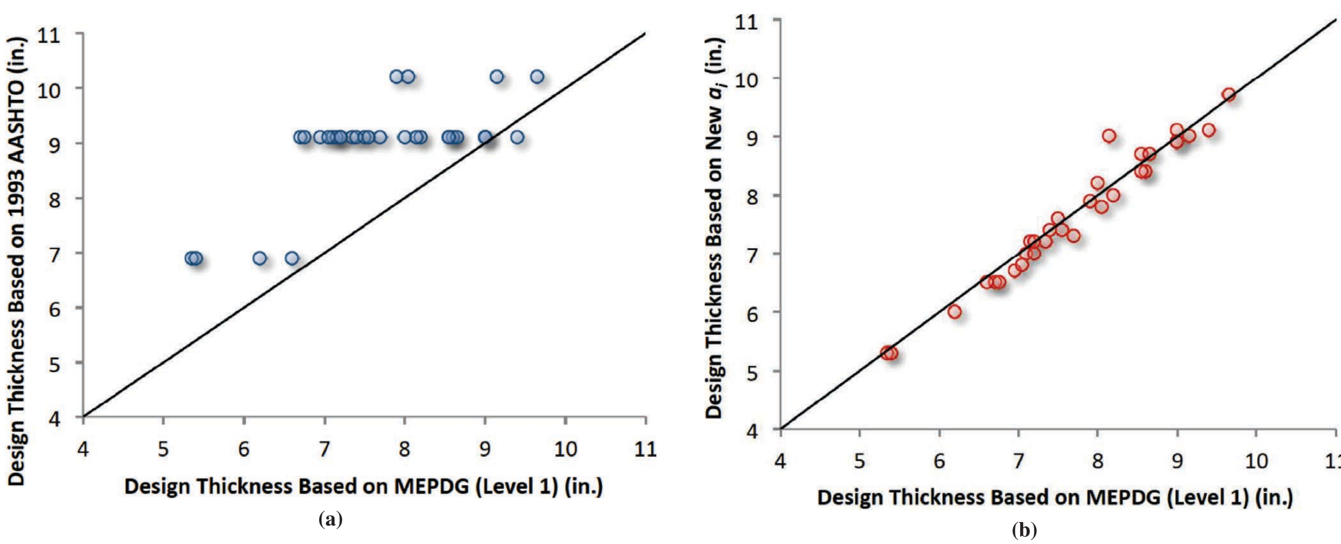

FIGURE 7 Design thicknesses acquired using (*a*) 1993 AASHTO Design Guide ($a_i = 0.44$) and (*b*) new a_i compared with those acquired using MEPDG (Level 1 for material properties) based on the scenarios in Table 3.

type and aggregate base modulus) and climatic and traffic conditions. Further validation against field data is necessary.

CONCLUSIONS

The objective of the research presented in this paper was to improve the estimate of the asphalt layer coefficient in the 1993 Design Guide to accommodate a wider range of mix types and to incorporate the effect of climate and traffic speed. The asphalt layer coefficient was initially found to be dependent on two main factors: the mix type and the temperature–frequency combination to which the pavement was subjected. As a result, a relationship was developed between the layer coefficient and the effective dynamic modulus (the material property chosen represents the two factors). Upon exploring the developed relationship, it was found that the layer coefficient was also dependent on the resilient modulus of the base layer, which resulted in the development of a multilinear relationship between the structural layer coefficient of the asphalt layer, the effective dynamic modulus of the asphalt mix, and the resilient modulus of the aggregate base layers. Acquiring the structural number from the developed relationship yielded design thicknesses that were generally close to those acquired using the Pavement ME.

The main limitations of the research are summarized as follows.

- The research was based on an underlying assumption that using MEPDG for pavement design gives close-to-optimal design thicknesses. This assumption may be debatable. Therefore, the accuracy of the developed a_i–|E*|–E_{base} is directly dependent on the reliability of MEPDG's performance predictions, particularly given that nationally calibrated coefficients were used.
- The a_i–|E*|–E_{base} was limited by the number and types of mixes included within the scope of the study.
- The developed a_i–|E*|–E_{base} relationship did not directly account for pavements that have more than one asphalt layer.
- The developed model has yet to be validated across field performance data.

Future work includes expanding the scope of the research to cover a wider array of mix types and structural design scenarios. Validating the design thicknesses acquired from the developed relationship across measured performance data is also a future objective.

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