EXPERIENCES IN THE CHARACTERIZATION OF MATERIALS USED IN THE CALIBRATION OF THE AASHTO MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE (MEPDG) FOR FLEXIBLE PAVEMENT FOR COSTA RICA

By

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ABSTRACT

The AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) includes empirical distress models that have been calibrated using the North American conditions. But, the differences of material properties, traffic information, and environmental conditions for Latin American countries make necessary to calibrate these models using local conditions.

This paper presents an overview of Costa Rica's experience in the characterization of materials used in the calibration of the flexible pavement components of the AASHTO MEPDG performed by the National Laboratory of Materials and Structural Models at the University of Costa Rica (In Spanish, LanammeUCR). First, the paper deals with the importance of using mechanistic-empirical (ME) analysis and design models, as opposed to the purely empirical models that have been traditionally used in Latin America and the world. In second place, it discusses the dynamic modulus (E*) model developed in order to assess the improvement in accuracy provided by the local calibration (Witczak-Lanamme Model). Finally, this gives rise to future work in calibration of other performance models. This paper also serves as a guide to identify potential problems to highway agencies in their MEPDG calibrations.

INTRODUCTION

The 1993 *AASHTO Guide for Design of Pavement Structures* was based on empirical equations derived from the AASHO Road Test. The test was conducted between 1958 and 1960, with limited structural sections at one location (Ottawa, Illinois) and with modest traffic levels compared with those of the present day.

The 1993 AASHTO guide has served well for several decades; nevertheless, many limitations exist for its continued use *(1)*:

- Traffic loading deficiencies: Heavy truck traffic design volume levels have increased since the 1960's. Thus, applications of the procedure to modern traffic flows means the designer must often extrapolate outside of the design models. This may result in either "under-designing" or "over-designing" the pavement structure.
- Pavement rehabilitation design procedures were not considered at the AASHO Road Test.
- Climatic effects were not captured, because the AASHO Road Test was conducted at one specific geographic location and the effects of different climatic conditions on pavement performance were not considered.
- One type of subgrade and only two unbound dense granular base/subbase materials were included in the main flexible and rigid pavement sections of the Road Test.
- The vehicle suspension, axle configurations, tire types and pressures were representative of the types of truck used in the late 1950's. Many of these are outdated (tire pressures of 80 psi versus 120 psi today).
- The long-term effect of climate and aging of material were not addressed because of the short duration of the Road Test (over 2 years).
- The inability to incorporate significant materials properties into the design procedure is one of the major limitations.

In order to address the previous limitations, the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) was developed as part of the National Cooperative Highway Research Program (NCHRP) Project 1-37A *(2)*. The AASHTO MEPDG provides significant benefits over the 1993 AASHTO guide in designing cost-effective pavement structures and rehabilitation strategies. The AASHTO MEPDG uses mechanistic-empirical models to analyse input data related to traffic, climate, materials, and proposed structure and estimates damage accumulation for predicting pavement condition over time. Performance predictions are made in terms of different distress types and smoothness. Additionally, the AASHTO MEPDG can account for special loading configurations with multiple tires or axles, and can evaluate the cost-effectiveness of new materials and technologies. Thus, the pavement designer is fully involved in the design process and can make better-informed decisions based on different design features and materials for any particular site. This approach makes possible to optimize the design and to insure that specific distress types will not develop.

Another feature not included in previous design methodologies is the hierarchical approach to the design inputs. This approach provides the designer with a lot of flexibility in obtaining the design inputs for a design project based on the relative importance of the project and the availability of resources. In general, three levels of inputs are provided:

- **Level 1** inputs provide for the highest level of accuracy and, thus, have the lowest level of uncertainty or error associated. Level 1 material inputs require laboratory or field testing, such as dynamic modulus testing of hot-mix asphalt concrete; as well as, site-specific axle load spectra, and nondestructive deflection testing.
- **Level 2** inputs provide an intermediate level of accuracy and more closely resemble the typical requirements used with earlier editions of the AASHTO Guide. Level 2 inputs are user-selected, possibly from an agency database, can be derived from limited testing, or can be estimated through correlations.
- **Level 3** inputs provide the lowest level of accuracy. This level might be used for design where there are minimal consequences for early failures.

DIAGNOSIS OF AASHTO MEPDG´S DATA REQUIREMENTS AND NECESSARY TESTING: WHERE IS COSTA RICA

Any agency interested in adopting the MEPDG should prepare a practical implementation plan that includes training of staff, equipment acquisition, computer hardware acquisition, and calibration/validation to local conditions. Well-calibrated prediction models result in reliable pavement designs and enable precise maintenance plans. Local pavement performance data can be used to validate and adjust calibration coefficients integrated in the MEPDG. The local calibration guide developed during the NCHRP 1-40B *(3)* project provides necessary recommendations and guidelines to ensure proper recalibration and validation to local conditions. Some recommendations are stated below:

- Design input data needed.
- Performance and reliability design criteria.
- Local calibration and validation of distress models:
  - Establishing a database of projects.
  - Input guidelines for local conditions, materials, and traffic.
  - Adjusting distress and IRI models to fit performance.

The following analysis will provide an idea of where is Costa Rica in order to calibrate the MEPDG.

Pavement Foundation

The pavement foundation must be characterized, regardless of whether the design procedure is to be applied to an existing pavement or a new pavement. Different methods for subgrade or foundation characterization are available, including laboratory, nondestructive or intrusive testing (such as the Dynamic Cone Penetrometer) Additionally, experience with the subgrade type can be a valuable tool.

More specifically, the resilient modulus is the property that is needed for pavement design and analysis. In the case of Costa Rica, the designer can obtain the resilient modulus by three basic methods :

- *Laboratory repeated load resilient modulus tests:* AASHTO T 307 (*Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils*) *(4)*.

- *Analysis or backcalculation of non destructive testing data*: ASTM D4694 (*Deflections with a Falling Weight Type Impulse Load Device*) *(5)*.
- Correlations with other physical properties of the materials.

Later, some resilient modulus models for Costa Rican subgrades developed by LanammeUCR will be shown.

Material Characterization

Many combinations of material types and quality are used in flexible and rigid pavement systems. Six major material groups have been developed: asphalt materials, Portland Cement Concrete (PCC) materials, chemically stabilized materials, non-stabilized granular materials, subgrade soils, and bedrock.

In the case of asphalt materials, the response and behavior are heavily influenced by temperature, loading rate, mixing method, the mixing process, and the degree of damage of the material.

In the case of the hot mix asphalt (HMA), LanammeUCR has been doing research to assess the effects of the temperature and rate of loading on the modulus of the asphalt concrete. These studies have resulted in the development of master curves for different types of local mixtures based in the NCHRP 1-28A *(6)* report and ASTM D3497 "*Standard Test Method for Dynamic Modulus of Asphalt Mixtures"(7)*. Also, a general fatigue model for Costa Rican HMA mixtures and the resilient modulus of 5 granular materials have been developed. Such models will be shown for the sake of presenting other performed efforts that LanammeUCR has done to calibrate the AASHTO MEPDG. However, it is important to mention that the focus of the paper is the dynamic modulus (E*) model.

The HMA fatigue model was calibrated using the same 10 different mixtures that were used to calibrate the E* and are explained later. The resistance of the HMA mixtures to fatigue cracking was evaluated at 4.4°C, 21°C and 40°F using the flexural beam fatigue test (AASHTO T321-03(8)) under strain controlled mode of testing. Equation 1 shows the calibrated HMA fatigue model *(9)*.

$$N_f = 10^{27.794} c(\epsilon_t)^{-5.477}(E)^{-2.311} \quad [1]$$

Where,

N_f: Number of load cycles to faliure,

C: Shift factor for local conditions, estimated in 18.4 for Costa Rica,

ε_t: Tensile strain at the bottom of the HMA layer, and,

E: Dynamic modulus for certain conditions of temperature and frequency.

The resilient modulus was determined through AASHTO T 307 (*Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils(4))*. The 5 granular materials were classified by the AASHTO method as A-3 and according to their gradation as Type 1. The granular materials were compacted in a mold of 150 mm diameter by 300 mm height and compaction effort was applied though 56 drops of the modified Proctor hammer. The calibrated model is shown in equation 2 and Table 1shows the various model coefficients for the 5 studied materials *(10).*

$$M_r = k_1 \left(\frac{\theta}{Pa}\right)^{k_2} \quad [2]$$

Where,

Mr: Resilient Modulus of the unbound material,
k_1, k_2 = Material constants,
θ = Bulk modulus of the material = $\sigma_1 + \sigma_2 + \sigma_3$
Pa = Atmospheric pressure, for Costa Rica: 88.38 KPa

TABLE 1 Coefficients of the Developed Resilient Modulus Model for Five Costa Rican Granular Materials.

Material	Estimate parameters		R^2
	K1 (MPa)	K2	
M1	62	0.549	0.88
M2	82	0.557	0.82
M3	99	0.489	0.80
M4	108	0.487	0.84
M5	101	0.545	0.87

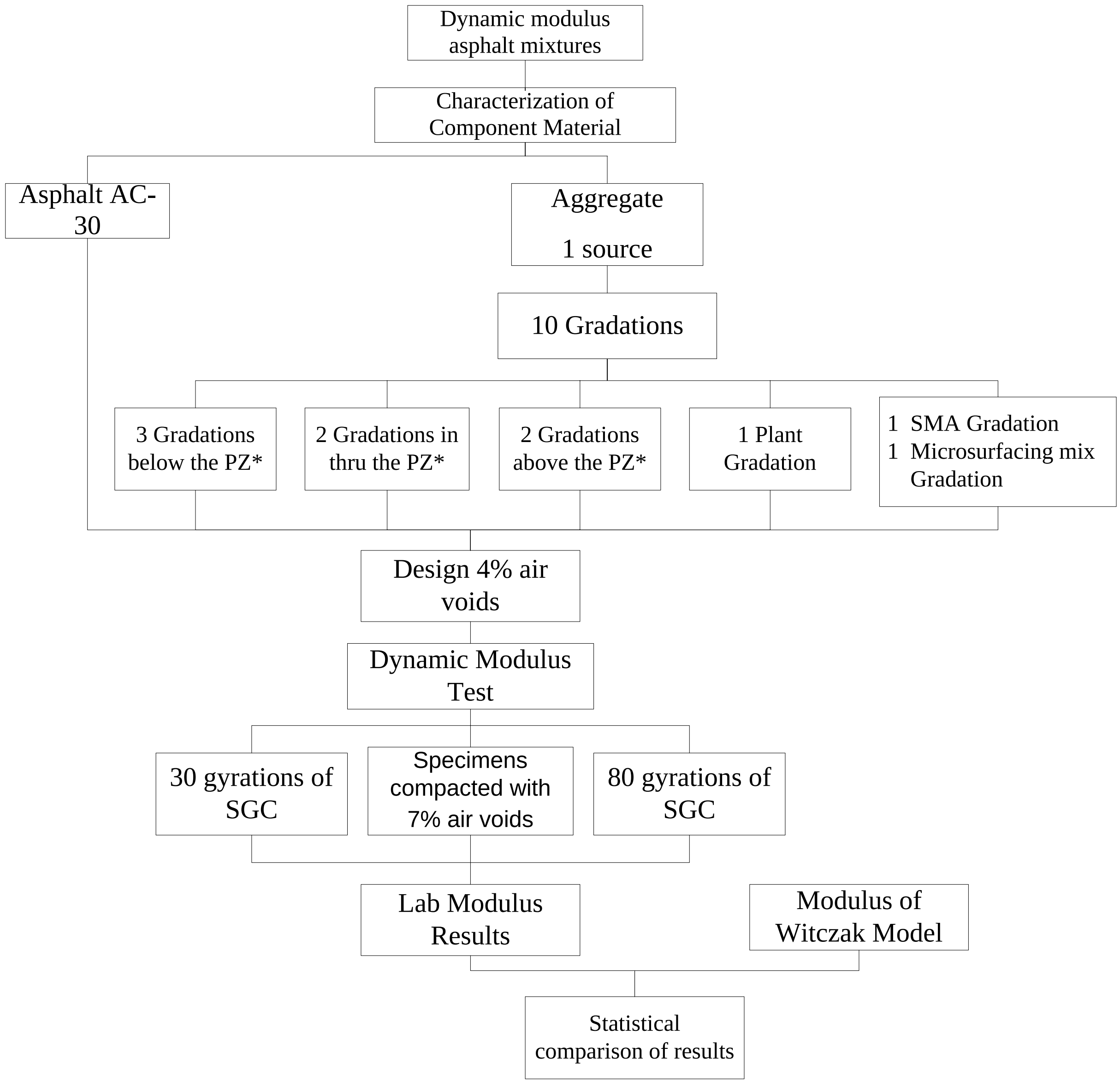
CALIBRATION OF A DYNAMIC MODULUS MODEL FOR COSTA RICA

LanammeUCR has made a significant effort to calibrate some of the MEPDG models. Here, the discussion will be focus on the calibration of the Dynamic Modulus Witczak model. LanammeUCR´s research team call the calibrated model, the Witczak-LanammeUCR model. As it was mention, an HMA fatigue and 5 resilient modulus for unbound materials have been also developed in order to provide calibration to the MEPDG.

Development of the Witczak-LanammeUCR Model

Starting in 2007, LanammeUCR has conducted a laboratory evaluation of the applicability of Witczack Model to a typical aggregate source and one type of asphalt binder produced in Costa Rica *(11)*.

The flow chart presented in Figure 1 summarizes the experimental plan of the study.



PZ*: "prevention zone" or SUPERPAVE's restricted zone

FIGURE 1 Flow Chart for the Experimental Plan

Aggregate Characterization

The study involved one aggregate source (from the northeast region of the country called Guápiles). The aggregate is extruded from igneous deposits along a river. The aggregate properties are shown in Table 2.

TABLE 2 Physical Properties of the Aggregates Used in the Study.

Property	Test Method	Value	Unit	Specifications
Coarse Aggregate				
L.A. Abrasion	AASHTO T 96 *(12)*	21.21	%	37% max.[1]
Specific Gravity	AASHTO T 85 *(13)*	2.652		2.85 max.[1]
Absorption	AASHTO T 85 *(13)*	1.69	%	4% max.[1]
Faces Fractured	ASTM D 5821 *(14)*			
1 face		100	%	90% min.[2]
2 or more		99.8	%	75% min.[2]
Fine Aggregate				
Plasticity index	AASHTO T 90 *(15)*	NP		10% max.[1]
Sand equivalent	AASHTO T 176 *(16)*	78		-
Angularity	AASHTO TP 304 *(17)*	37.2	%	-
Specific Gravity	AASHTO T 84 *(18)*	2.549		2.85% max.[1]
Absorption	AASHTO T 84 *(18)*	3.283	%	-

[1] Nevada DOT Standard Specifications for Road and Bridge Construction, 2001.
[2] Standard Specifications for Constructions of Roads and Bridges on Federal Highways Projects, FP-03

Asphalt Binder Properties

In Costa Rica only one type of asphalt is produced. The binder viscosity classification corresponds to an unmodified AC-30. The properties for the asphalt binder are shown in Table 3.

TABLE 3 Physical Properties of the Used Asphalt Binder.

Aging State	Property	Unit	Asphalt Binder AC-30
Original	Density at 25ºC	g/cm^3	1.030
	Absolute viscosity at 60ºC	Poise	3330
	Kinematic viscosity at 125ºC	centiPoise	961
	Kinematic viscosity at 135ºC	centiPoise	565
	Kinematic viscosity at 145ºC	centiPoise	347
	VTS, regression slope of viscosity temperature susceptibility	-	3.43
	Regression intercept	-	10.26
RTFOT	Absolute viscosity at 60ºC	Poise	11512
	Kinematic viscosity at 125ºC	centiPoise	1712
	Kinematic viscosity at 135ºC	centiPoise	938
	Kinematic viscosity at 145ºC	centiPoise	550

Specimen Preparation

Ten different types of asphalt mixtures were designed in the laboratory. Three dense graded mixtures (G1, G2 and G3) below the "*prevention zone*"(also called SUPERPAVE's restricted zone); two dense graded mixtures (G6 and G7) above the "*prevention zone*"; two dense graded mixtures (G4 and G5) thru the "*prevention zone*"; one Stone Mastic Asphalt (SMA) mixture (G9); one micro surfacing mix (G8) and a typical plant dense graded mixture (G10). The gradations are presented in Table 4 and Figure 2.

TABLE 4 Studied Aggregate Gradations.

ASTM Sieve	Sieve (mm)	Studied Gradation									
		Below the prevention zone			Thru the prevention zone		Above the prevention zone		Micro (!)	SMA	Plant
		G1	G2	G3	G4	G5	G6	G7	G8	G9	G10
3/4	19.0	100	100	100	100	100	100	100	100	100	100
1/2	12.5	95	100	90	95	95	98	90	100	90	95
3/8	9.5	88	95	78	90	90	92	65	81	45	79
N°4	4.75	37	62	40	45	70	67	45	32	28	48
N°8	2.36	28	33	32	37	50	47	42	27	23	32
N°16	1.18	20	23	20	29	27	32	37	22	22	22
N°30	0.60	13	16	14	22	15	23	30	18	19	16
N°50	0.30	9	12	9	14	8	17	20	14	16	12
N°100	0.15	7	9	7	9	6	12	12	10	13	8
N°200	0.075	5	7	6	6	5	8	5	8	10	5

(!) Microsurfacing.

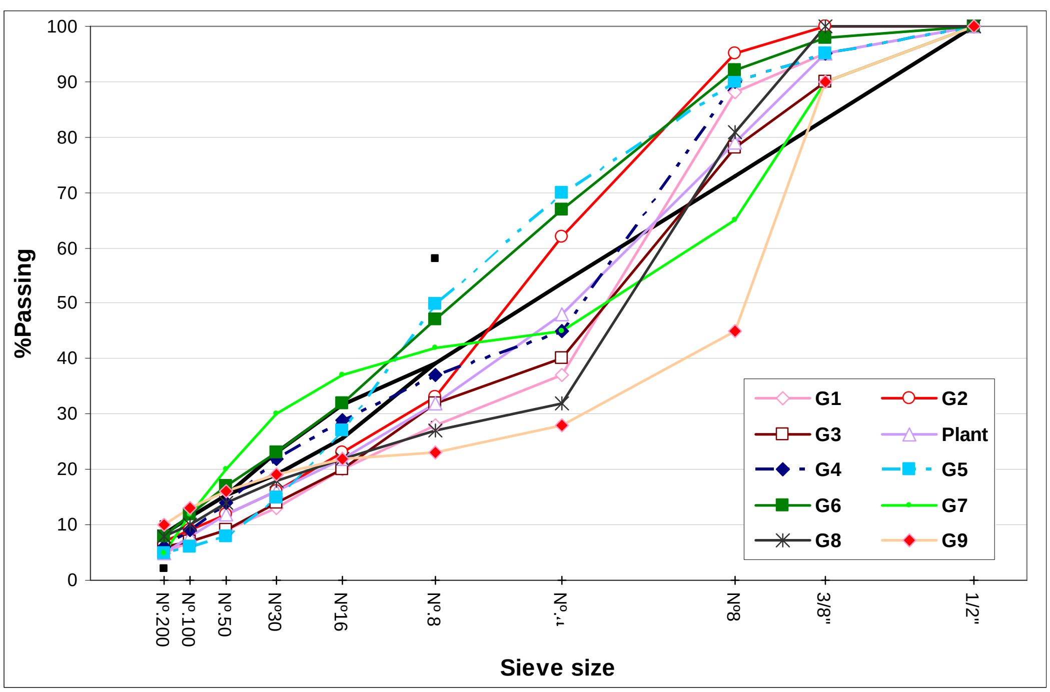


FIGURE 2 Aggregate Gradations Used in the Study

The design air void content was fixed to 4%. Two mixture design methodologies were used: Marshall Methodology and SUPERPAVE. The optimum asphalt content by dry weight of aggregate (DWA) and by total weight of mixture (TWM), voids in the mineral aggregate (VMA), the voids filled with asphalt (VFA), and the effective asphalt content (Pbe) based on both methodologies are shown in Table 5.

TABLE 5 Summary Volumetric Properties of the Mix for All the Aggregate Gradations Studied.

Description	Gradation	Mix design	Va	Pb (DWA)	Pb (TWM)	Pbe	VMA	VFA
Below the prevention zone	G1	SUPERPAVE	4.0%	7.20	6.80	5.69	17.32	77.66
		Marshall	4.0%	6.41	6.02	5.18	15.74	74.66
	G2	SUPERPAVE	4.0%	7.40	6.90	6.06	17.44	76.12
		Marshall	4.0%	6.84	6.40	5.49	16.51	75.78
	G3	SUPERPAVE	4.0%	6.40	6.00	5.25	15.68	73.40
		Marshall	4.0%	6.01	5.67	4.83	15.15	71.93
Thru the prevention zone	G4	SUPERPAVE	4.0%	5.50	5.30	4.31	12.14	73.20
		Marshall	4.0%	5.44	5.16	4.17	13.90	69.53
	G5	SUPERPAVE	8.0%	7.50	7.00	6.00	20.90	61.60
		Marshall	8.8%	6.50	6.10	5.12	20.08	55.50
Above the prevention zone	G6	SUPERPAVE	4.0%	5.50	5.20	4.35	14.10	72.10
		Marshall	4.0%	5.84	5.52	4.41	14.52	70.50
	G7	SUPERPAVE	4.0%	5.00	4.80	3.32	12.32	63.20
		Marshall	4.0%	5.50	5.21	4.13	13.74	70.50
Micro surfacing	G8	SUPERPAVE	4.0%	5.60	5.30	4.29	14.06	78.68
		Marshall	4.0%	5.99	5.65	4.51	14.82	71.00
SMA	G9	SUPERPAVE	4.0%	4.90	4.70	3.74	12.44	68.86
		Marshall	4.0%	5.19	4.93	4.01	13.34	71.00
Plant	G10	SUPERPAVE	4.0%	6.00	5.70	4.76	15.00	73.00
		Marshall	4.0%	5.65	5.35	4.46	14.50	71.10

Dynamic Modulus of Asphalt Mixtures

In order to evaluate the dynamic modulus of the different mixes, all specimens were prepared following the standard method ASTM D3496 "*Practice for Preparation of Bituminous Specimens for Dynamic Modulus Testing"(19)*. The testing was performed according to ASTM D3497 "*Standard Test Method for Dynamic Modulus of Asphalt Mixtures" (7)* and AASHTO T 62 *"Determining Dynamic Modulus of. Hot Mix Asphalt" (20)*.

The experimental design included four factors; the first factor was the gradation with the ten levels (G1, G2, G3, G4, G5, G6, G7, G8, G9 and G10), the second factor was the temperature with five levels (-5, 5, 20, 40 and 55°C), the third factor was the load frequency with six levels

(0.1, 0.5, 1, 5, 10 and 25 Hz), and the fourth level was the grade of compaction with three levels (30 gyrations of the Superpave gyratory compactor(SGC), 80 gyrations of SGC, and specimens compacted with 7% air voids)

Master curves

The master curves and the corresponding shift factors were developed directly from the dynamic modulus tests. The Microsoft Excel Solver was used to optimize calibration coefficients. It involved nonlinear optimization using the sigmoidal function shown in Equations 3 and 4. Both equations describe the time dependency of the modulus (The results are presented in Table 6 and Figure 3):

$$Log\left|E^*\right| = \delta + \frac{\alpha}{1 + e^{\beta + \gamma(\log t_r)}} \qquad [3]$$

where,
E^* = dynamic modulus.
t_r = time of loading at the reference temperature.
δ, α = estimated parameters; for a given set of data, δ represents the minimum value of E* and $\delta+\alpha$ represents the maximum value of E*.
β, γ = parameters describing the shape of the sigmoidal function.

$$a(T) = \frac{t}{t_r} \qquad [4]$$

$$\log(t_r) = \log(t) - \log[a(T)]$$

where,
t_r = time of loading at the reference temperature.
t = time of loading at a given temperature of interest.
$a(T)$ = Shift factor as a function of temperature.
T = temperature of interest.

TABLE 6 Summary of the Fitting Parameters α, β, δ and γ for the Construction of the E* Master Curves

Gradation	Parameter			
	δ	α	β	γ
G1	1.8155	2.3618	-0.5631	0.4766
G2	1.8647	2.4533	-0.3800	0.5018
G3	1.8542	2.3952	-0.3458	0.4784
G4	1.8013	2.5136	-0.7055	0.4589
G5	2.1775	1.8860	-0.1475	0.5982
G6	1.7743	2.7039	-0.5207	0.4182
G7	2.1687	2.3301	-0.5388	0.4960
G8	2.0420	2.0748	-0.6264	0.5309
G9	2.0682	2.3802	-0.6617	0.5529
G10	1.5471	2.7260	-0.7342	0.4276

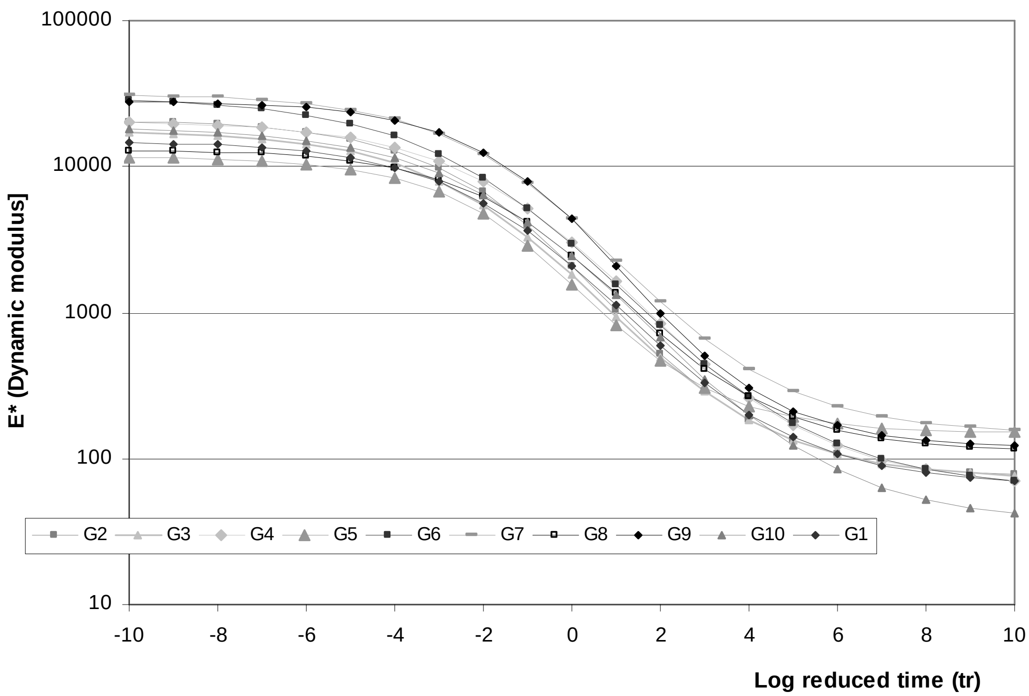


FIGURE 3 Master Curves of Dyanmic Modulus fir the Various Gradations Used in the Study

Witczak Model

For Level 2 and Level 3 analysis, the master curves will be developed directly from the dynamic modulus predictive equation shown in equation 5. This equation is intended to predict the dynamic modulus of asphalt mixtures over a wide range of temperatures, rates of loading, and aging conditions based on information that is readily available from material specifications or volumetric design of the mixture *(2)*.

$$\log E^* = 3{,}750063 + 0{,}02932\rho_{200} - 0{,}001767(\rho_{200})^2 - 0{,}002841\rho_4 - 0{,}058097V_a$$
$$-0{,}802208\left(\frac{V_{beff}}{V_{beff}+V_a}\right) + \frac{3{,}871977 - 0{,}0021\rho_4 + 0{,}003958\rho_{38} - 0{,}000017(\rho_{38})^2 + 0{,}005470\rho_{34}}{1+e^{(-0{,}603313-0{,}31335\log(f)-0{,}393532\log(\eta))}} \quad [5]$$

where:
E^* = dynamic modulus, psi.
η = bitumen viscosity, 10^6 Poise.
f = loading frequency, Hz.
V_a = air void content, %.
V_{beff} = effective bitumen content, % by volume.
ρ_{34} = cumulative % retained on the ¾ in sieve.
ρ_{38} = cumulative % retained on the 3/8 in sieve.
ρ_4 = cumulative % retained on the No. 4 sieve.
ρ_{200} = % passing the No. 200 sieve.

A statistical analysis of the dynamic modulus was performed in order to compare the Witczak prediction model with the local test results. The statistical correlation between the model and the observed data was evaluated. The results are presented in Figure 4

The mean of the 894 observations was 5547.7 MPa with a standard deviation of 1796.7. The model explained the variance of the data by 90.68% (R^2=0.9068). The sum of squared errors (SSE), between predicted and measured data was 43.5940. The results show a high correlation between the laboratory tests and the Witczak model. However, it was necessary to calibrate the model to local conditions. The calibration (optimization) was performed by varying the local calibration coefficients in the model in order to reduce the sum of squared errors (SSE), between predicted and measured data.

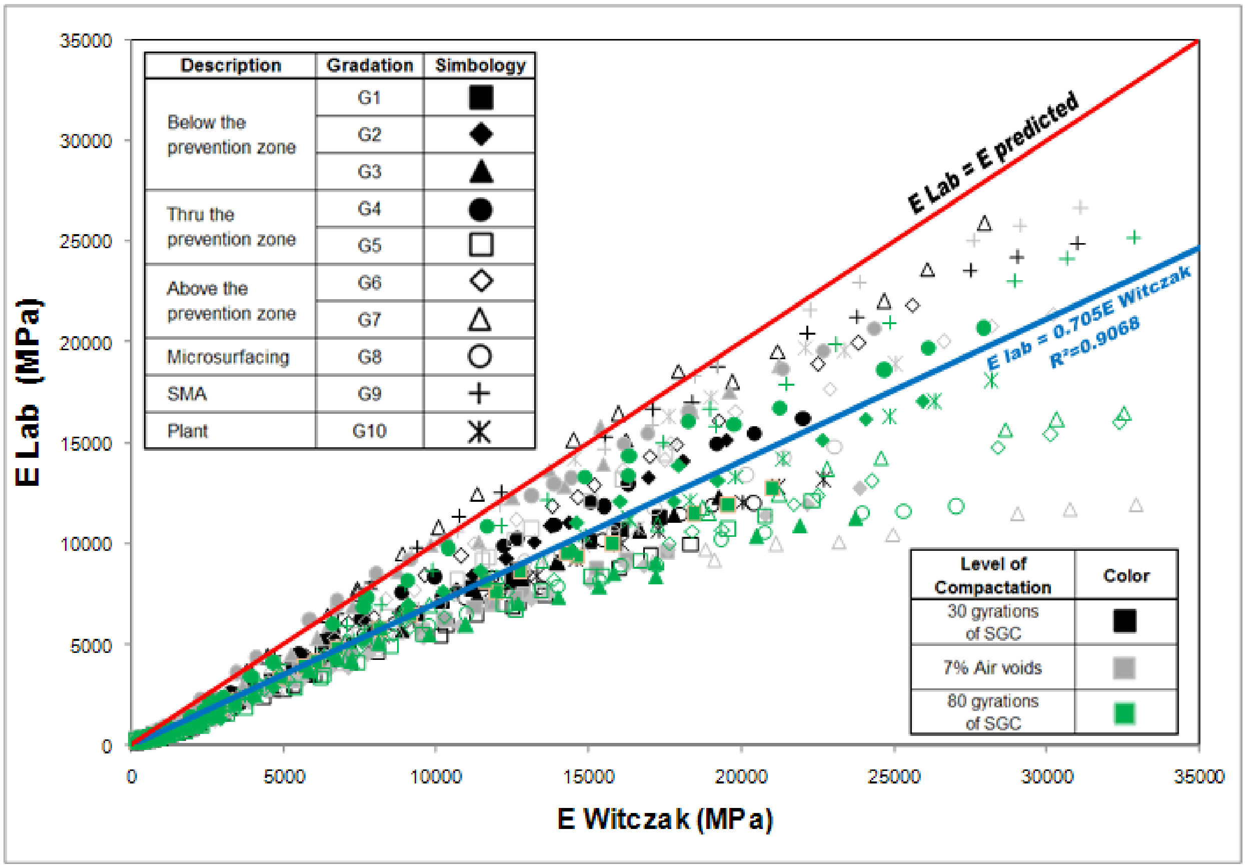


FIGURE 4 Comparison of Results Obtained in the Laboratory Test versus the Results Obtained with the Application of the Witczak Model

Witczak model calibration: Witczak-Lanamme Model

In order to calibrate the Witczak model a nonlinear approach was used. This technique consists in fitting models whose parameter are non linear, using iterative methods. Specifically, the Gauss-Newton method was used to reduce the sum of squared errors (SSE) between the predicted and the measured data and to minimize the standard errors. A comparison was made between the SSE before calibration (SSE=43.5940) and after calibration (SSE=5.1997) in order to assess the improvement in accuracy provided by the local calibration. The steps used in the calibration can be summarized as follow:

- Optimization runs were made using the Witczak model calibration coefficients in order to select initial values for these coefficients.
- The JMP software of SAS Institute Inc. was used to minimize the sum of squared errors by optimizing the coefficients in Witczak Model.

The new set of coefficients is shown in equation 6:

$$\log E^* = 5,535833 + 0,002087\rho_{200} - 0,000566(\rho_{200})^2 - 0,002590\rho_4 - 0,078763V_a$$
$$-1,865947\left(\frac{V_{beff}}{V_{beff}+V_a}\right) + \frac{2,399557 + 0,000820\rho_4 - 0,013420\rho_{38} + 0,000261(\rho_{38})^2 + 0,005470\rho_{34}}{1+e^{(0,052941-0,498163\log(f)-0,691856\log(\eta))}} \quad [6]$$

where the variables were previously defined. Again, a statistical investigation of the dynamic modulus was made. The results are shown in Figure 5. The model explains variance in the data 93.55% (R^2=0.9355) and an estimate of standard deviation of error of 1,494.4.

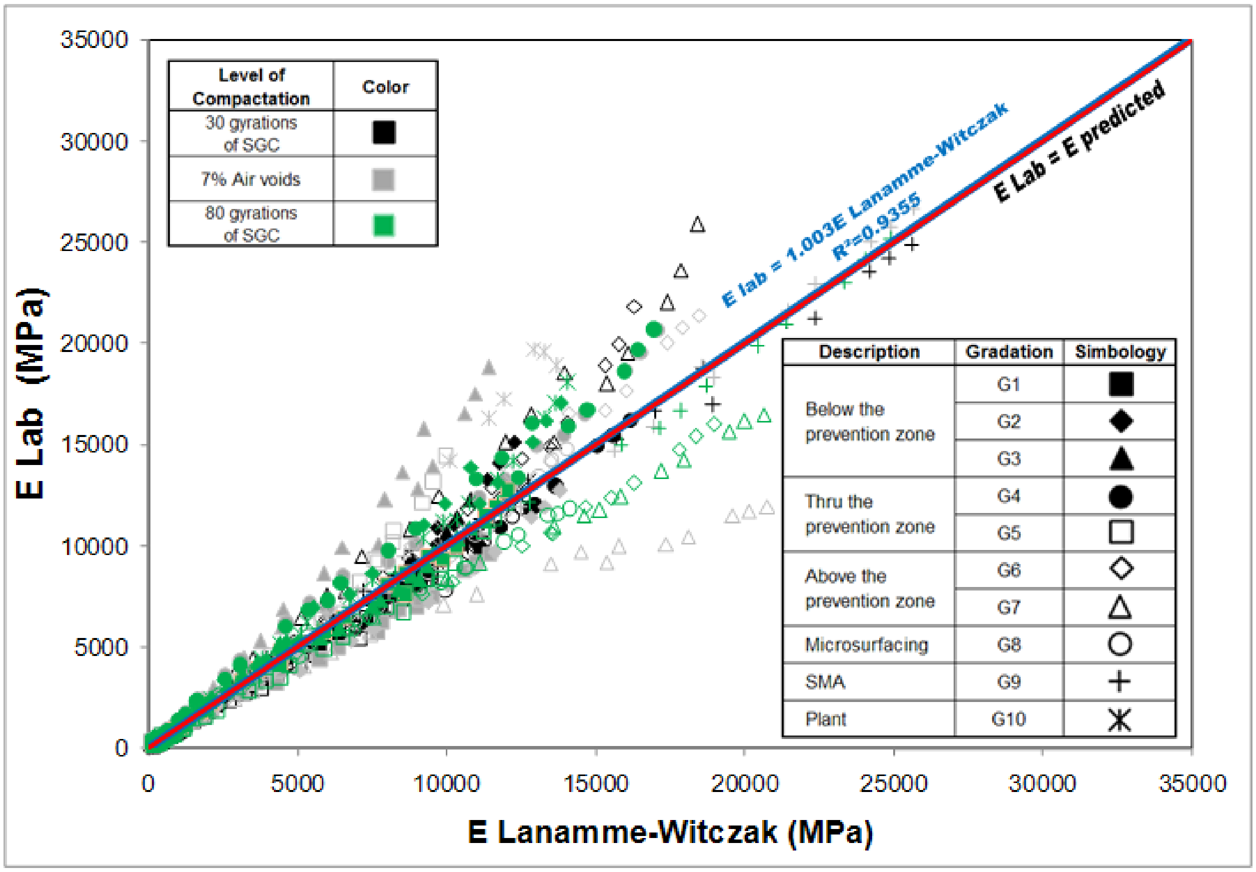


FIGURE 5 Comparison of Results Obtained from the Laboratory Test versus the Results Obtained with the Application of the Witczak-Lanamme Model

The results before and after calibration were compared to evaluate the impact of the local calibration on dynamic modulus. Table 7 shows the value of dynamic modulus before and after Witczak model calibration. It is clear from the table that the Witczak model prior to calibration was over predicting the dynamic modulus when the reduced time is low and under predicting the dynamic modulus for high reduced time, based on Costa Rican materials. Based on these results, it can be concluded that calibration coefficients for the MEPDG prediction models are required.

TABLE 7 Effect of Local Calibration on Dynamic Modulus Models Prediction for 30 Gyrations of SGC.

Gradation	Dynamic Modulus obtained by	Reduced time												
		-10	-8	-6	-4	-2	-1	0	1	2	4	6	8	10
		E*, Dynamic Modulus for gradations studied (MPa)												
G1	Laboratory	14,655	14,062	12,670	9,843	5,653	3,635	2,091	1,116	587	202	108	80	71
	Witczak	28,709	25,706	21,001	14,680	8,060	5,318	3,228	1,812	956	247	72	28	15
	Lanamme-Witczak	14,431	13,984	12,846	10,304	6,116	3,950	2,247	1,167	595	197	106	81	73
G2	Laboratory	20,273	19,411	17,299	12,927	6,703	3,978	2,097	1,039	526	185	107	85	77
	Witczak	33,084	29,381	23,773	16,480	9,032	5,978	3,648	2,061	1,093	280	78	28	14
	Lanamme-Witczak	15,562	15,084	13,870	11,157	6,660	4,313	2,453	1,267	637	202	106	79	71
G3	Laboratory	17,190	16,331	14,355	10,542	5,463	3,305	1,806	9,39	498	185	108	84	76
	Witczak	33,707	29,806	23,858	16,163	8,505	5,480	3,249	1,786	928	237	69	28	15
	Lanamme-Witczak	13,964	13,514	12,359	9,784	5,636	3,569	1,998	1,034	535	189	109	86	78
G4	Laboratory	20,061	19,218	17,307	13,531	7,958	5,212	3,047	1,630	842	260	123	84	71
	Witczak	38,135	33,853	27,273	18,667	9,967	6,477	3,875	2,150	1,125	290	86	34	19
	Lanamme-Witczak	20,856	20,204	18,527	14,763	8,599	5,467	3,055	1,563	791	264	145	112	102
G5	Laboratory	11,465	11,219	10,460	8,426	4,713	2,862	1,548	816	464	228	172	157	152
	Witczak	32,684	28,873	23,045	15,513	8,067	5,156	3,030	1,651	851	216	64	26	14
	Lanamme-Witczak	13,213	12,794	11,714	9,284	5,335	3,360	1,862	950	483	166	94	74	67
G6	Laboratory	28,436	26,467	22,569	16,100	8,407	5,224	2,954	1,569	822	265	126	84	70
	Witczak	46,868	41,298	32,751	21,747	11,043	6,953	4,024	2,165	1,107	283	87	37	22
	Lanamme-Witczak	21,505	20,820	19,031	14,973	8,416	5,200	2,820	1,412	711	246	142	113	104
G7	Laboratory	30,856	29,744	27,009	21,173	12,145	7,727	4,368	2,284	1,186	410	227	175	157
	Witczak	55,227	47,358	36,407	23,561	11,914	7,578	4,463	2,450	1,272	320	88	31	15
	Lanamme-Witczak	23,979	23,253	21,365	17,073	9,968	6,340	3,543	1,816	923	314	176	138	126
G8	Laboratory	12,922	12,620	11,802	9,821	6,211	4,175	2,478	1,345	719	264	155	125	115
	Witczak	36,351	32,172	25,764	17,442	9,152	5,885	3,483	1,913	995	257	77	32	18
	Lanamme-Witczak	16,784	16,273	14,953	11,965	7,022	4,488	2,523	1,299	662	224	124	97	88
G9	Laboratory	27,771	27,153	25,396	20,948	12,627	8,010	4,348	2,104	996	305	168	133	122
	Witczak	58,394	51,262	40,403	26,573	13,313	8,314	4,769	2,542	1,288	324	98	41	24
	Lanamme-Witczak	28,893	27,885	25,384	19,976	11,446	7,209	3,979	1,995	977	297	150	111	98
G10	Laboratory	17,994	17,025	15,004	11,365	6,474	4,202	2,450	1,311	673	196	84	53	42
	Witczak	29,329	26,338	21,651	15,309	8,558	5,706	3,497	1,976	1,044	264	73	27	13
	Lanamme-Witczak	13,993	13,599	12,573	10,214	6,189	4,048	2,334	1,231	638	220	123	96	87

FUTURE WORK

As previously highlighted, the E* model requires calibration if the MEPDG is intended to be used in regions other than the United States. However, the estimation of material response is but one component of the models developed as part of the MEPDG.

The final objective of the MEPDG is to accurately estimate pavement deterioration. However, as in the case of the E* model, the models used to predict the different types of pavement distress have been developed based on material, climatic, structural, and traffic conditions from specific pavement sections throughout the United States. Nonetheless, calibration coefficients which have been originally set to 1.0 have been included in all of the models to facilitate their calibration. However, the estimation of accurate calibration coefficients also requires detailed long term pavement performance data. For this purpose, LanammeUCR has been collecting field performance data for several years, as part of a network evaluation effort, and to evaluate specific projects.

Environmental Effects

Environmental conditions have a significant effect on the performance of both flexible and rigid pavements. External factors such as precipitation, temperature and depth to water table play a key role in defining the impact the environment can have on the pavement performance. Internal factors such as the susceptibility of the pavement materials to moisture, ability to drain of the different layers, infiltration potential of the pavement, and so on define the extent to which the pavement will react to the applied external environmental conditions. However, a sophisticated climatic modeling tool as the Enhanced Integrated Climatic Model (EICM) is still unavailable in Costa Rica. On account of this condition the LanammeUCR has carried out some researches related to this subject. Recently, Orozco (21) presented a division into climatic zone for the road network in Costa Rica which is a tool capable of offering support for the eventual creation of an Asset Management System in Costa Rica.

Traffic

Traffic data is one of the key data elements required for the structural design/analysis of pavements. It is required for estimating the loads that are applied to a pavement and the frequency with which those given loads are applied throughout the pavement's design life. Some of the required traffic inputs are the following:

- Base year truck-traffic volume.
- Vehicle (truck) operational speed.
- Truck-traffic directional and lane distribution factors.
- Vehicle (truck) class distribution.
- Axle load distribution factors.
- Axle and wheel base configurations.
- Tire characteristics and inflation pressure.
- Truck lateral distribution factor.
- Truck growth factors.

Again, the majority of this information is not readily available or is none too reliable. For this reason, procedures to overcome this issue are being developed in Costa Rica. For example, the LanammeUCR has carried out some researches related to axle load distribution factor and average daily traffic in the main road network in Costa Rica (22) y (23).

Evaluation of existing pavements for rehabilitation

Recently, the LanammeUCR acquired a georeferenced digital image capture and extraction system, designed for the purpose of performing the inventory of road infrastructure assets and their geometry that utilizes positioning sensors (GPS, Distance Measuring Instrument (DMI)) and high-resolution digital cameras to create an advanced tool for large-scale data collection and asset management. Using this equipment, it is intended to identify specific details of the projects that may have a significant effect in the repair strategy or rehabilitation design. The use of automated techniques could significantly reduce the time of data collection and the time from data collection to the decision making process. This pavement performance database will allow for the calibration or the development of local pavement deterioration models to be used in ME design.

When pavement condition surveys are conducted, certain information should be available if the engineer is going to make knowledgeable decisions regarding pavement condition assessment and problem definition and, hence, rehabilitation needs and strategies. The following data are required for pavement evaluation:

1. Type—Identify types of physical distress existing in the pavement. The distress types should be placed in categories according to their causal mechanisms.
2. Severity—Note level of severity for each distress type present to assess degree of deterioration.
3. Quantity—Denote relative area (percentage of the lane area or length) affected by each combination of distress type and severity.

In order to perform structural design for pavement rehabilitation, the following are the current Costa Rican possibilities:

- The structural capacity (load related) is determined through the 2 Falling Weight Deflectometers.
- The functional adequacy (user related) is determined using an inertial profiler to calculate the International Roughness Index (IRI). Also, surface texture is determined through a Griptester.
- A database with the history of pavement works has not been developed yet neither by LanammeUCR or the Costa Rican´DOT.

CONCLUSIONS

The dynamic modulus model, included in the current MEPDG, was calibrated for Costa Rican conditions. This calibration was performed using ten gradations of a typical aggregate source and one type of asphalt binder produced in Costa Rica.

The results of dynamic modulus before and after Witczak model calibration were compared to evaluate the impact of the local calibration factors on dynamic modulus. The results showed that the Witczak model prior to calibration was over predicting the dynamic modulus when the reduced time is low and under predicting the dynamic modulus for high reduced time, based on Costa Rican materials. Based on these results, it was concluded that calibration coefficients for the MEPDG prediction models are required. Finally, the Witczak-LanammeUCR model was developed to predict the dynamic models for ten Costa Rican asphalt mixtures.

In order to further improve the prediction models for Costa Rica in future calibration and verification efforts, it is necessary to increase the number of tests performed. Currently, Costa Rica does not have an adequate distresses database and therefore, it is recommend that the country reevaluate distress data collection practices which will lead to the calibration or development of local pavement deterioration models to be used in ME design in the future.

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