Calibration of a Mechanistic-Empirical Fatigue Model Using the PaveLab Heavy Vehicle Simulator

Submitted to the 95th Annual Meeting of the Transportation Research Board
Submitted on July 31st, 2015

Fabricio Leiva-Villacorta, Ph.D., MBA, (corresponding author)
National Laboratory of Materials and Structural Models (LanammeUCR),
University of Costa Rica, P.O.Box 11501-2060, UCR, San José, Costa Rica
fabricio.leiva@ucr.ac.cr

Adriana Vargas-Nordcbeck, Ph.D., MBA
National Laboratory of Materials and Structural Models (LanammeUCR)
University of Costa Rica, San José, Costa Rica
adriana.vargasnordcbeck@ucr.ac.cr

José P Aguiar-Moya, Ph.D.
National Laboratory of Materials and Structural Models (LanammeUCR)
University of Costa Rica, San José, Costa Rica
jose.aguiar@ucr.ac.cr

Luis G Loría-Salazar, Ph.D.
National Laboratory of Materials and Structural Models (LanammeUCR)
University of Costa Rica, San José, Costa Rica
luis.loriasalazar@ucr.ac.cr

Word Count: Abstract (253) + Body (4,140) + Figures and Tables (10x250) = 6,893

ABSTRACT

Traditionally, the design of pavements in Costa Rica has been made using the AASHTO 93 design method, which is related to empirical correlations from results obtained several decades ago in the AASHO road test. In order to return to the use of more fundamental engineering principles, the necessity of a transition from this empirical state to a mechanistic-empirical state has been recognized. The National Laboratory of Materials and Structural Models of the University of Costa Rica (LanammeUCR) has implemented an Accelerated Pavement Testing (APT) program. In this case, a Heavy Vehicle Simulator (HVS) provides a first step in the validation/calibration process of transfer functions. An "Incremental-Recursive" method in which the materials properties are updated in terms of damage for each time increment and used (recursively) as input to the next time increment was first used with laboratory samples and then with instrumented full scale sections. Controlled strain fatigue tests on beams were used to derive the initial damage model parameters to predict a decrease in the modulus for the asphalt concrete. All full scale sections have been instrumented with Multi Depth Deflectometers (MDDs) to measure the decrease in asphalt concrete modulus throughout the duration of tests until failure. By comparing the pavement conditions at any point in time during the pavement life to the laboratory performance, a shifted function for the asphalt concrete damage was obtained. The results indicated that a single shift factor cannot be achieved at this time and further regression analysis and data are required to performed full model calibration.

INTRODUCTION

Significant developments in pavement engineering have been traditionally achieved through real time load (RTL) testing because the technique does not require large specialized equipment for carrying out the tests (*1, 2*). However, the time required to perform the RLT tests (more than 10 years of continuous monitoring of an experimental section) is associated with many difficulties, since most of the experimental sections are located along roadways in operation.

In the case of Costa Rica, because of the large variability in climatic conditions, materials and traffic, the cost of developing a suitable RTL test program covering all these conditions for prolonged periods of time is prohibitive. However, there is a great need to characterize the performance of national pavement structures as the only means of developing and calibrating design methodologies. For this purpose it was considered that the implementation of an Accelerated Pavement Testing (APT) program was a better alternative.

In order to implement a Costa Rican APT program, a technical and economical study was performed and assisted in determining that the Heavy Vehicle Simulator (HVS) was the best fit for the medium and long term pavement performance assessment. Specifically, the HVS met the following: mobility, accelerated pavement evaluation, application of real loads and comparable results from similar equipment (*3, 4, 5*).

APT program deliverables

The HVS will be essential to ensure a breakthrough in the level of research conducted by the LanammeUCR in transportation engineering. In particular, it is expected that a series of products that have already been obtained under similar studies (*6, 7, 8*) be generated:

- Mechanistic-empirical pavement design methodology and software based on material conditions, weather, traffic and actual construction practices.
- Development of new material specifications that are based on actual performance and contribution of structural materials in the field.

- Optimization of pavement structures in use at the national level, based on structural, materials, traffic, and climatic conditions specific to the area where the structure is planned to be built.
- Potential for an improved evaluation methodology of new materials or materials currently in use.
- Capacity to evaluate pavement structures of high importance prior to opening to traffic in order to ensure required performance of the structure or identify possible deficiencies.

TEST TRACKS

For the first stage of accelerated tests in Costa Rica the construction of 4 experimental sections was performed in May 2012 (Figure 1). The objective of this phase was to perform a structural comparison in terms of thickness of the asphalt concrete layer and base material type (granular vs. cement treated). Table 1 shows the characteristics of the 4 sections with their respective layer thicknesses obtained from Ground Penetrating Radar (GPR) measurements and backcalculated layer moduli based on Falling Weight Deflectometer results. The top layer consists of an HMA mixture with nominal maximum aggregate size of 19.0 mm with an optimum binder content of 4.9% by total weight of mixture. The cement treated base (CTB) was designed to withstand a pressure of 35 kg/cm^2 with an optimum cement content of 1.7% by volume of aggregate and with a maximum density of 2013 kg/m^3. The base material and granular sub-base were placed at a maximum density of 2217 kg/m^3 with an optimum moisture content of 8.6%. The sub-base material had a CBR of 95%. Finally, the subgrade material was constructed for a maximum density of 1056 kg/m^3 with an optimum moisture content of 52% and CBR of 6.6%.

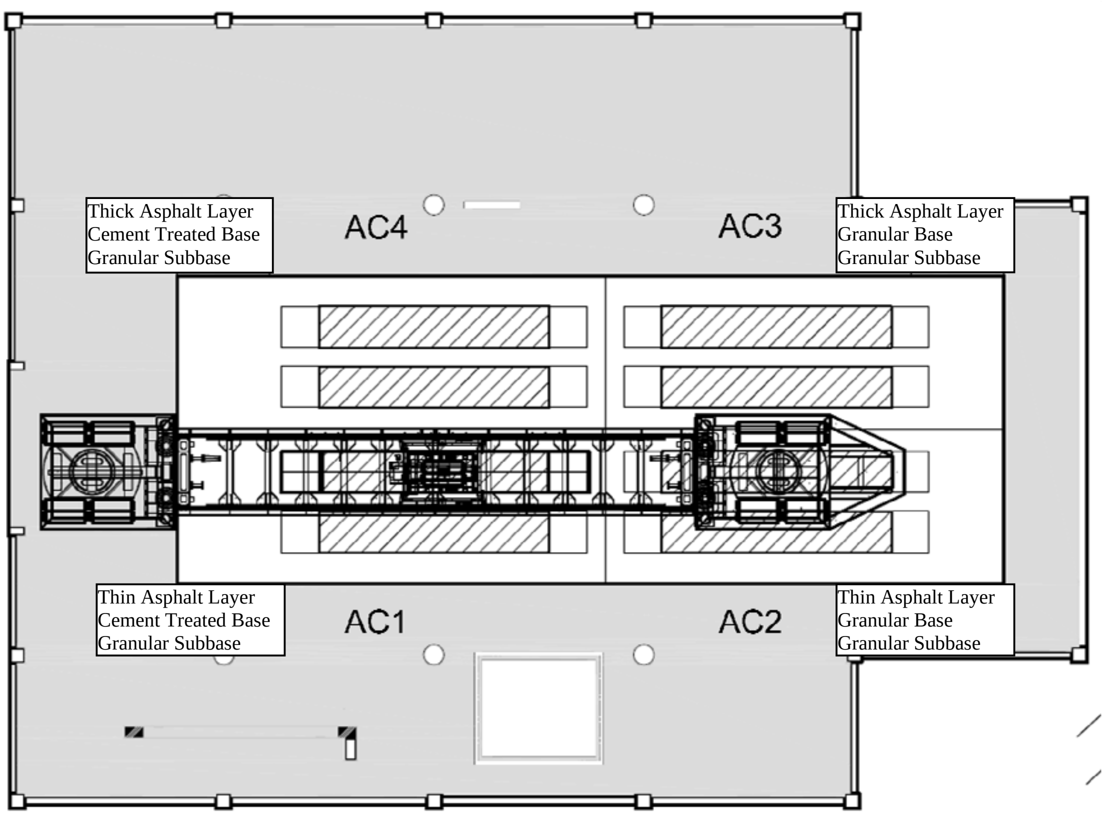


FIGURE 1 Test track distribution.

TABLE 1 Test Tracks in-place Properties

Properties\Section	AC1	AC2	AC3	AC4
HMA Thickness (H1), cm	5.1	6.3	13.2	13.2
Base Thickness (H2), cm	21.9	21.2	31.0	24.9
Subbase Thickness (H3), cm	30.1	30.1	30.1	30.1
HMA Modulus (E1) @ 25 °C, MPa	3800	3800	3800	3800
Base Modulus (E2), MPa	1200	170	170	1200
Subbase Modulus (E3), MPa	140	140	140	140
Subgrade Modulus (E4), MPa	70	70	70	70

Instrumentation

The experiment included not only the instrumentation integrated with the HVS system but also embedded instrumentation in all four test sections. HVS onboard sensors can record the applied load, tire pressure and temperature, position of the load and the velocity of the load carriage. Embedded instrumentation include asphalt strain gauges (PAST model sensors), pressure cells (SOPT model sensors), multi depth deflectometers (MDDs), moisture and temperature probes. These sensors were chosen based on previous HVS owner's experience (*9, 10*). Additionally, the HVS was equipped with a laser profiler that can be used to create a tridimensional profile of the section and a Road Surface Deflectometer is added to the testing equipment to obtain deflection basins at any location along the test section.

Figure 2 shows the instrumentation array used for the first series of experimental sections. The PAST sensors were placed at the base/HMA layer interface in the longitudinal or traffic loading direction and in the transverse direction. MDD sensors were installed at 4 different depths to cover all 4 structural layers. The thermocouples were placed at 4 depths: surface, middle depth of the HMA layer, at the PAST sensors depth and 5 cm into the base layer. In the case of AC1 and AC3 sections the same gauge array was used while excluding PAST sensors.

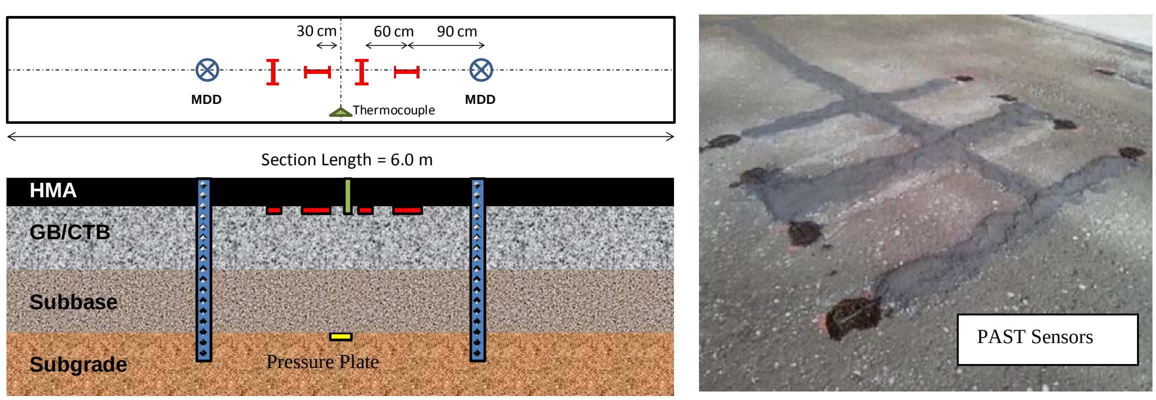


FIGURE 2 Sensor Array.

Data collection of the 3D profile, strain, pressure, temperature and deflection is performed based on load repetitions. At the beginning of the test, data are obtained at short intervals. After 15,000 load repetitions, data are collected on daily basis. Inspection of fatigue and reflective cracking, friction loss, loss of aggregate-asphalt bond and any other surface damage is performed on daily basis during the HVS daily maintenance work.

FATIGUE CRACKING

Fatigue cracking is one of the main distress types for flexible pavements. Cracks generally initiate at the bottom of the asphalt layer due to the large tensile strains produced by repetitive traffic loads and propagate upwards as the loading continues, eventually appearing on the surface. However, load-related fatigue cracking can also initiate at or near the surface of the pavement and propagate from the top down (*11*).

Fatigue cracking resistance of asphalt mixtures depends on material properties as well as pavement structural factors. In the laboratory, fatigue evaluation is focused on factors related to the material properties of the hot mix asphalt (HMA) mixtures. One of the most popular test procedures used to determine susceptibility to fatigue cracking is the beam flexural test. This test was designed to simulate the bending that a HMA layer experiences in a pavement structure. The results are usually interpreted in terms of a relationship between applied stress or strain and number of cycles to failure. There are several models used to predict the fatigue life of asphalt mixtures, the simplest one being the model proposed by Pell (*12*). For a controlled-strain test, the relationship is described by Equation 1:

$$N_f = k_1 \left(\frac{1}{\varepsilon}\right)^{k_2} \qquad \text{Eq. 1}$$

where
N_f = number of cycles to failure
ε = tensile microstrain
k_1, k_2 = mix-dependent regression coefficients

Although this phenomenological approach provides some guidance necessary to understand fatigue performance of HMA pavements, there are limitations that must be considered. It is essentially an empirical approach and does not provide a relationship between loading and any form of damage accumulation in the mixture (*13*). The results are either material dependent, or loading mode dependent, or both, so this approach cannot be applied directly to the complex loading scenarios that are actually common to in-service pavements (*14*). In addition, the strain fatigue life relationship is treated linearly, which has been found to be inappropriate at low strains (*15*).

EVALUATED MODEL

Damage functions are used for cracking of bound material and for permanent deformation and roughness of all layers. The general format of the damage equation can be expressed as (*16*):

$$Damage = A \times MN^{\alpha} \times \left(\frac{resp}{resp_{ref}}\right)^{\beta} \times \left(\frac{E}{E_{ref}}\right)^{\gamma} \qquad \text{Eq. 2}$$

where
MN = the number of load repetitions in millions,
resp = the response (stress or strain),
$resp_{ref}$ = a reference response (can be related to strength),
E = the modulus of the material (adjusted for climate and damage),
E_{ref} = a reference modulus, and
A, α, β, and γ = constants.

For bound materials, (structural) damage may be defined as the relative decrease in modulus, i.e. the decrease in modulus, dE, divided by the initial modulus, Ei. Early in the life of the layer, the decrease in modulus will primarily be due to microcracking which, much later, will develop into macrocracking. The process is complex and using the average modulus of the layer is a simplification.

If a damage level is defined, for instance 50% drop in the initial asphalt concrete modulus, the damage model can be transformed into a classical function such as the one shown in Equation 3. This is an extended form of Equation 1.

$$Permissible\,\mathrm{Re}\,sponse = CA \times MN^{c\alpha} \times \left(\frac{E}{E_{ref}}\right)^{c\gamma} \qquad \text{Eq. 3}$$

where
MN = the number of ESAL applications in millions,
E = the modulus of the material, and
CA, $c\alpha$, $c\gamma$ and $Eref$ = constants.

RESULTS

Laboratory fatigue tests

Tests were conducted in accordance with AASHTO T 321 under constant strain loading for three strain levels 400, 600 and 800 microstrain and three temperatures (10, 20 and 30 °C). Tests were performed on laboratory produced asphalt mix and plant produced mix. Figure 3 shows the number of load repetitions to reach 50% stiffness reduction as function of tensile strain at different temperatures for both asphalt mixes. In general, the plant produced mix exhibited superior number of repetitions to failure. Each point is an average of three samples for the 20 °C temperature and two samples for the remaining temperatures which corresponds to a total of 21 samples or observations to perform a multiple regression analysis to obtain a fatigue model.

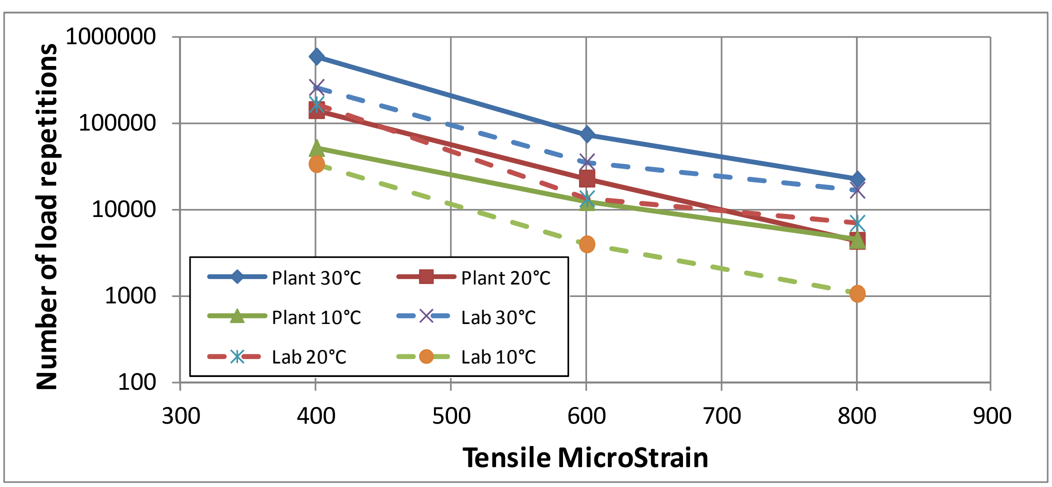


FIGURE 3 Laboratory fatigue test results.

Based on laboratory results a classical-extended transfer function (Equation 4) was developed for each asphalt mixture. Table 2 exhibits all the regression coefficients computed for both laboratory and plant mixes. In both cases, it was determined that the modulus of the mixture was not statistically significant at a 95% confidence level. This could be due to the high correlation (R-value = 0.91) between temperature and modulus (collinearity) and the limited number of observations used to develop each model. Therefore, a new regression analysis was performed and the results are shown in Table 3 and Equations 5 and 6 for plant produced and lab mixes.

$$N_f = k_1(\epsilon)^{k_2}(E)^{k_3}e^{Tk_4} \quad \text{Eq. 4}$$

where
N_f = number of cycles to failure
ε = tensile microstrain
E = the modulus of the material
k_1, k_2, k_3, k_4 = mix-dependent regression coefficients
T = Temperature (°C)

TABLE 2 Initial fatigue models

Mix	Coefficient	Value	SE	T-stat.	P-value
Plant	ln (k_1)	44.022	6.40	6.88	<0.05
	k_2	-4.702	0.38	-12.36	<0.05
	k_3	-0.623	0.56	-1.12	0.28
	k_4	0.061	0.03	1.87	0.08
	R^2	0.926			
	R^2aj.	0.913			
	MSE	0.216			
Laboratory	ln (k_1)	27.250	13.46	2.02	0.06
	k_2	-4.309	0.4056	-10.62	<0.05
	k_3	0.821	1.314	0.62	0.541
	k_4	0.172	0.09291	1.85	0.082
	R^2	0.921			
	R^2aj.	0.906			
	MSE	0.225			

TABLE 3 Final fatigue models

Mix	Coefficient	Value	SE	T-stat.	P-value
Plant	ln (k1)	37.352	2.30	16.24	<0.05
	k2	-4.554	0.36	-12.69	<0.05
	k4	0.094	0.01	6.99	<0.05
	R2	0.921			
	R2aj.	0.912			
	MSE	0.219			
Laboratory	ln (k1)	35.533	2.35	15.13	<0.05
	k2	-4.401	0.37	-11.87	<0.05
	k4	0.115	0.01	8.13	<0.05
	R2	0.919			
	R2aj.	0.909			
	MSE	0.217			

$$N_f = \mathrm{e}^{37.352}(\epsilon)^{-4.554}e^{T0.094} \quad \text{Eq. 5}$$

$$N_f = \mathrm{e}^{35.533}(\epsilon)^{-4.401}e^{T0.115} \quad \text{Eq. 6}$$

In order to obtain the damage model, damage of the asphalt mixture was defined as the relative decrease in modulus dE, divided by the initial modulus, Ei for each sample. All 21 samples of each mixture were combined and non-linear regression analysis utilizing the Gauss-Newton algorithm was performed to obtain fatigue damage models of the form shown Table 4 and in Equations 7 and 8. γ was assume to be equal to $\beta/2$, making damage a function of the internal energy density (*16*). The parameters of Equations 7 and 8 were determined from four point beam controlled strain fatigue testing, by minimizing the Root Mean Square (RMS) of the difference between the measured damage and the damage calculated from Equation 2 with the incorporation of the temperature variable. The minimization was done in Excel using Solver and the statistical parameters were also checked with the software "R" version 3.2.1.

In both cases, it was determined that all variables were statistically significant at a 95% confidence level. Figure 4 shows a comparison between measured and predicted damage. It can be observed that the damage models reasonably predict the measured laboratory damage which in this case was restricted to 50%.

TABLE 4 Damage fatigue model NLR parameters

Mix	Coefficient	Value	SE	T-stat.	P-value
Plant	A	0.093	0.0183	88.31	<0.05
	ALPHA	0.320	0.0011	115.48	<0.05
	BETA	1.595	0.0053	90.90	<0.05
	GAMA	0.798	0.0071	22.84	<0.05
	DELTA	0.041	0.0005	28.23	<0.05
	Residual Standard Error		0.03978 on 22248 degrees of freedom		
Laboratory	A	0.189	0.0079	65.85	<0.05
	ALPHA	0.271	0.0014	162.82	<0.05
	BETA	1.070	0.0084	117.12	<0.05
	GAMA	0.535	0.0081	19.23	<0.05
	DELTA	0.035	0.0005	32.29	<0.05
	Residual Standard Error		0.04897 on 25172 degrees of freedom		

An important key feature regarding fatigue damage equations is that the regression analysis utilizes the entire data set from the initial modulus to the very last one at failure criterion. This compared to the classical fatigue models shown in Table 2 and equations 5 and 6 depend upon the initial modulus which is considered to be constant during the entire life span of the structure.

$$\omega = 0.09295 \times (MN)^{0.31003} \times \left(\frac{\varepsilon}{200}\right)^{1.59529} \times \left(\frac{E}{3000}\right)^{0.79765} \times e^{(0.04121 \times T)} \quad \text{Eq. 7}$$

$$\omega = 0.18941 \times (MN)^{0.2707} \times \left(\frac{\varepsilon}{200}\right)^{1.0696} \times \left(\frac{E}{3000}\right)^{0.53480} \times e^{(0.03517 \times T)} \quad \text{Eq. 8}$$

where
w = damage
MN = the number of load repetitions in millions
ε = tensile microstrain
E = the modulus of the material, MPa

T = Temperature °C

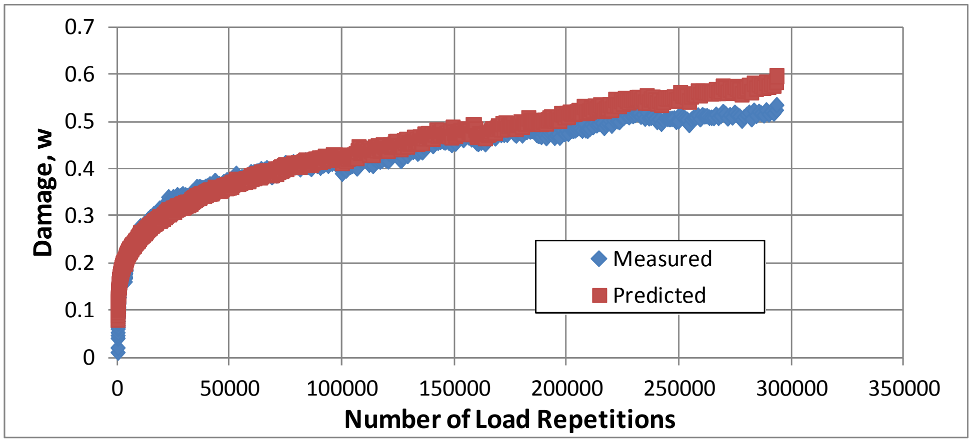


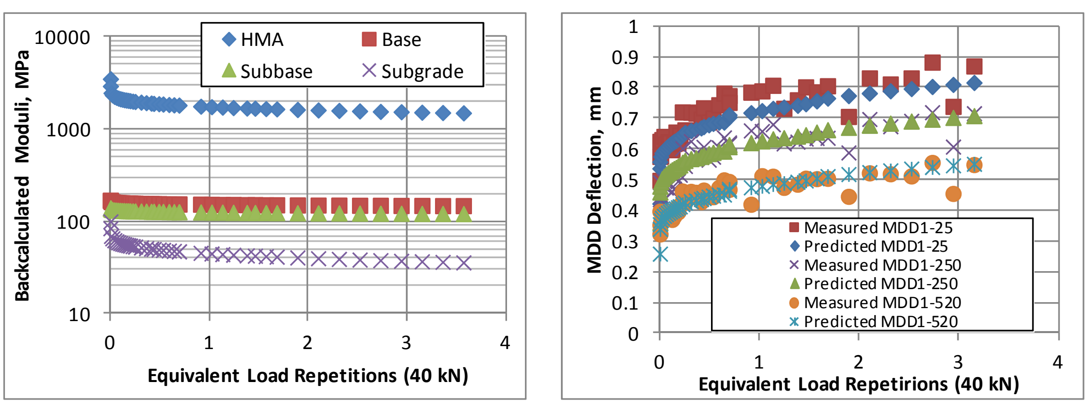
FIGURE 4 Measured and Predicted asphalt concrete damage (Plant produced mix).

If a damage level was defined for instance as a 50% drop in the initial asphalt concrete modulus, the damage model could be transformed into a classical function such as the one shown in Equation 3. This type of model would be more robust over the classical model (Equation 4) because, in this case, the variable "modulus" is statistically significant and can be used to predict the decrease in asphalt concrete modulus, during the full duration of full scale tests carried to failure.

Backcalculated Layer Moduli

MDD deflection data were used to determine the progression of the pavement layer moduli. This was done by applying the method of equivalent thickness (*17, 18*) whereby the thickness of the structure is transformed into a single layer. This transformation is done using Odemark's methodology and calculation of stresses, strains and deflections were performed using Boussinesq theory.

Figure 5 shows an example of the backcalculated layer moduli for the different layers for one of the test tracks as function of equivalent load repetitions in millions. A good correlation was obtained between measured and estimated deflections: a small deviation from equality indicated that the criteria used to perform backcalculation successfully.

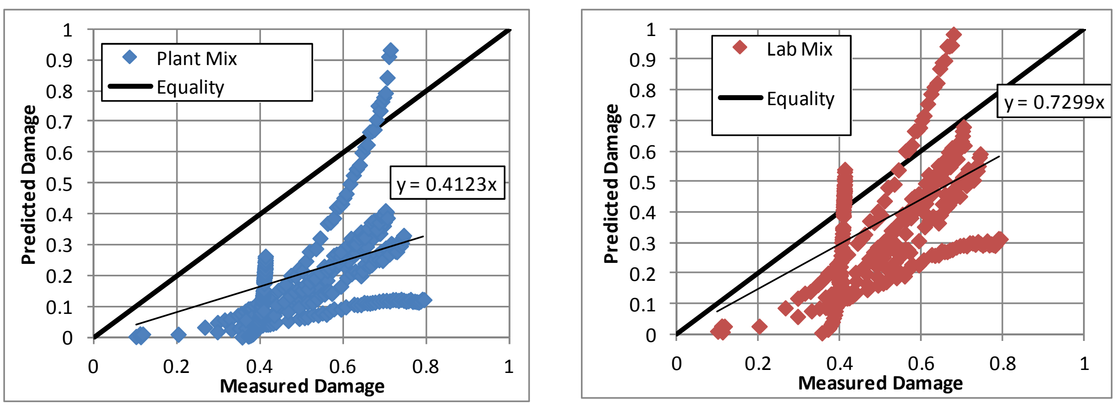


a. b.

FIGURE 5 MDD Backcalculated Layer moduli (a), estimated vs. measured deflections (b).

For the asphalt concrete layer, structural damage was defined as the relative decrease in modulus dE, divided by the initial modulus, Ei. This information was obtained for each section and for two different locations within the same section. With these results and having records of temperature at mid depth of the asphalt layer, thicknesses and the equivalent load repetitions it was possible to determine strain responses from Layered-Elastic Analysis.

Using the coefficients obtained from laboratory samples to develop Equation 5 the damage on the asphalt concrete was estimated. The results indicated that a single shift factor cannot be achieved at this time. Figure 6 shows the comparison between predicted and measured asphalt concrete damage. As can be seen the fatigue damage models developed using plant and lab produced mix tended to underestimate the observed APT damage by an average of 59% and 28% respectively. In order adjust the predicted values close to the line of equality a shift factor of 2.5 could be applied to the plant produced mixture and a shift factor of 1.4 could be applied to the lab produced mixture. Another important fact that can become an issue is the level of variability and how scattered the results are when trying to predict full scale damage conditions. Although this models tend to describe fairly well the damage as a function of load repetitions, tensile strain and stiffness, using parameters from flexural beam testing and may indicate that further calibration of one or several of the remaining coefficients may be necessary to improve its goodness of fit.



a. b.

FIGURE 6 HVS Measured versus model predicted asphalt concrete damage, a. Plant produced mix, b. Laboratory produced mix.

The damage level was defined as a 50% drop in the initial asphalt concrete modulus for the laboratory produced mix and the damage model was transformed into a classical function shown in Equation 9. This damage level was set at 50% since none of the sections have shown visible fatigue cracking. Up to this day the failure criteria used to stop the test has been rutting.

The number of data sets at this point is limited as well as the number of the spots where the information (deflections) is generated. Therefore, further investigation and increasing the number of test sections and test locations within each section are recommended to improve the damage model and make it more reliable.

$$MN = 18.39 \times \left(\frac{\varepsilon}{200}\right)^{-3.951} \times \left(\frac{E}{3000}\right)^{-1.976} \times e^{(-0.129 \times T)} \qquad (9)$$

where

MN = the number of load repetitions in millions

ε = tensile microstrain
E = the modulus of the material, MPa
T = Temperature °C

CONCLUSIONS AND RECOMMENDATIONS

A set of models has been developed for flexible pavement performance prediction operating in an incremental-recursive mode. These models include:

- Asphalt concrete fatigue models that predict damage as a function of load repetitions, tensile strain and stiffness, using parameters from flexural beam testing.
- Initially calibrated concrete fatigue models that predict damage as a function of load repetitions, tensile strain and stiffness from HVS test results.

Incremental mode models which is a standard Miner's Law approach, permitting damage calculation for the axle load spectrum and expected temperature regimes, but with no updating of materials properties through the life of the project can also be derived from damage function models. This is similar to the approach included in the NCHRP 1-37A design guide. This type of approach is calibrated against an end failure state or a damage level, such as 25% cracking of the wheelpath or 75% damage of the material.

Further investigation and increasing the number of test sections under different environmental conditions and test locations within each section are recommended to improve the damage model and make it more reliable. It is expected to calibrate the new damage models to 25% cracking and to develop a reflection cracking model based on tensile strain and use of the same damage parameters developed for asphalt concrete fatigue.

Further investigation may be necessary to improve the predictability of the APT damage results, which may involve a full calibration of laboratory developed equations or even consider a different form of the damage equation. Variability of the input parameters and uncertainty on the damage models also need to be addressed. The use of Monte-Carlo simulation using those critical variables is one possibility that will be explored.

Before any model can be applied to design of new pavements a number of issues need to be addressed. The HVS tests are accelerated and slightly affected by aging. Aging is expected to affect both the stiffness and the fatigue characteristics of the materials. Aging increases stiffness, therefore deflection measurements in the field should reflect a net effect of aging and damage with accumulating traffic loads. Field calibration is required to evaluate the difference in response between the field pavement and the incremental-recursive simulation that should be attributed to aging. The effects of seasonal variations on the unbound materials need to be established from seasonal monitoring sites. Under real traffic there are rest periods of different duration between the loads and the wheel speeds are higher than under HVS loading. This may affect the shift factor for asphalt fatigue. Finally the effects of variability of materials, structure, loads and climate, and of the uncertainty on the models must be established and simulated as it is expected to be done with the construction of the new PaveLab´s environmental chamber.

REFERENCES

1. LTPP. *LTPP Beyond FY 2009: What Needs to Be Done*?. Report FHWA-HRT-09-052. 2009.
2. Metcalf, J. B. *NCHRP Synthesis of Highway Practice 235*. 1996.
3. Coetzee, N et al. *The Heavy Vehicle Simulator in Accelerated Pavement Testing: Historical Overview and New Developments*. 3rd International Conference APT. 2008.
4. Harvey, J. T., Hoover, T., Coetzee, N. F., Nokes, W. A., and Rust, F. C. *Caltrans Accelerated Pavement Test (CAL/APT) Program—Test Results: 1994–1997*. AAPT Symposium on Accelerated Pavement Testing, Boston, MA, March 16-18, 1998.
5. Leiva, Aguiar y Loría. *Ensayos Acelerados De Pavimentos En Costa Rica*. Revista Infraestructura Vial, Vol 15 (#26), 33-41. San José Costa Rica, 2013.
6. Harvey, J. T., L. du Plessis, F. Long, S. Shatnawi, C. Scheffy, B-W. Tsai, I. Guada, D. Hung, N. Coetzee, M. Reimer, and C. L. Monismith. *Initial CAL/APT Program: Site Information, Test Pavement Construction, Pavement Materials Characterizations, Initial CAL/APT Test Results, and Performance Estimates.* Report prepared for the California Department of Transportation. Report No. RTA-65W485-3. Pavement Research Center, CAL/APT Program, Institute of Transportation Studies, University of California Berkeley. 1996.
7. Timm, D. NCAT Report 09-01 *Design, Construction And Instrumentation Of The 2006 Test Track Structural Study*, NCAT. 2009.
8. Leiva-Villacorta, F. and D. Timm, *Analysis of Measured Versus Predicted Critical Pavement Strain Responses*, Proceedings of the 90th Annual Transportation Research Board, Washington, D.C. 2011.
9. Heavy Vehicle Simulator. *Monitoring of test sections and instrumentation*. Documento consultado el 6 de abril del 2010. http://www.gautrans-hvs.co.za/}
10. Baker Harris B., Buth Michael R., Van Deusen David A. Minnesota Road Research Project: *Load response Instrumentation Installation and Testing Procedures*. Minnesota Department of Transportation. 1994.
11. Roberts, F.L. P.S. Kandhal, E.R. Brown, D.Y. Lee, and T.W. Kennedy. *Hot Mix Asphalt Materials, Mixture Design, and Construction.* NAPA Education Foundation, Lanham, MD, Third Edition, 2009.
12. Pell, P.S. Fatigue of Asphalt Pavement Mixes. *Proceedings of the Second International Conference on the Structural Design of Asphalt Pavements*, Ann Arbor, MI, pp. 577-593.
13. Diefenderfer, S. *Investigation of Fatigue Properties of Superpave HMA at the Virginia Smart Road*. Ph.D. dissertation. Virginia Polytechnic Institute and State University, October, 2009.
14. Shen, S. and S. Carpenter. *Dissipated Energy Concepts for HMA Performance: Fatigue and Healing.* Dept. of Civil and Environmental Engineering, Univ. of Illinois at Urbana-Champaign, Advanced Transportation Research and Engineering Laboratory, ATREL, 2007.
15. Carpenter, S.H., K.A. Ghuzlan, and S. Shen. A Fatigue Endurance Limit for Highway and Airport Pavement. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1832*, Transportation Research Board of the National Academies, Washington, D.C., 2003, pp. 131-138.
16. Ullidtz, P., Harvey, J.T., Tsai, B-W. and Monismith, C.L. "Calibration of Mechanistic-Empirical Models for Flexible Pavements Using the WesTrack Experiment", AAPT, 2008.
17. Ullidtz, P., *Pavement Analysis, Development in Civil Engineering*, Vol.19, Amsterdam, the Netherlands. 1987.
18. Leiva, Aguiar y Loría. *Accelerated Pavement Testing First Results At The LanammeUCR APT Facility*. Proceedings of the 94th Annual Transportation Research Board, Washington, D.C. 2015.