Development and Calibration of Permanent Deformation Models

Fabricio Leiva-Villacorta, Adriana Vargas-Nordcbeck, José Pablo Aguiar-Moya and Luis Loría-Salazar

Abstract As part of the first accelerated pavement tests completed in Costa Rica, laboratory permanent deformation tests were performed on asphalt mixture, granular base and soil samples from the test sections. The objective of this study was to calibrate prediction models using measured rutting data collected from instrumented flexible pavements. For the asphalt concrete the laboratory test was performed using unconfined cyclic loading at three different temperatures. Permanent deformation was found to be a function of the resilient strain, temperature and number of loading cycles. For the granular base and soil samples, the test was performed under repeated axial cyclic stress at different magnitudes and different confining stresses. Permanent deformation was found to be a function of the confining stress, deviator stress, moisture content, and number of loading cycles. Four instrumented flexible pavements were subjected to heavy vehicle simulator testing. Backcalculated moduli from multi depth deflectometers were introduced into an elastic multilayer system to obtain pavement responses. The number of equivalent standard axle load repetitions, the effective asphalt concrete mean temperature, and the in-place moisture content along with computed responses were used to calculate permanent deformation for the different layers. By comparing the measured rut depths with the predicted values from laboratory based models, the optimum combination of field calibration factors was determined so that the coefficient of variation was minimal by means of ordinary least squares.

Keywords Permanent deformation · HVS · Instrumentation

F. Leiva-Villacorta (✉) · A. Vargas-Nordcbeck · J.P. Aguiar-Moya · L. Loría-Salazar
LanammeUCR, CP-11501-2060 San José, Costa Rica
e-mail: fabricio.leiva@ucr.ac.cr

A. Vargas-Nordcbeck
e-mail: adriana.vargasnordcbeck@ucr.ac.crk

J.P. Aguiar-Moya
e-mail: jose.aguiar@ucr.ac.cr

L. Loría-Salazar
e-mail: luis.loriasalazar@ucr.ac.cr

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1 Introduction

Calibration of pavement performance models is a systematic process to eliminate the bias and minimize the difference between observed or measured performance from the actual pavements and predicted results from an empirical or mechanistic model (Von Quintus et al. 2007). The implementation of the Mechanistic Empirical Pavement Design Guide (MEPDG) includes one of the largest efforts to calibrate pavement performance models, first taking into consideration the various climatic regions across the United States and more recently local or state calibrations (NCHRP 2004). For its implementation the MEPDG not only focused on the calibration of the different models but also the testing and development of those models at the laboratory level. The idea behind the calibration process is to be able to predict pavement performance from material properties and laboratory developed models in a reliable form. In order to have a reliable model in a short period of time, actual data from full scale accelerated testing facilities are currently being used (Harvey and Kanekanti 2006; Ullidtz et al. 2006; Powell 2006).

Rutting is an undesired distress in a pavement for several reasons. For the road users, it gives an increase of fuel consumption due to increased friction and increased risk of hydroplaning in wet weather conditions. Rutting is caused by several factors such as density of the layer (Lekarp 1997), stress conditions (Werkmeister et al. 2003), and number of load applications, among others, and occurs in different layers of the pavement. A rule of thumb is that the wider the ruts are, the deeper in the pavement the permanent deformation has occurred.

There are numerous models for predicting the permanent deformation. The aim of these models is to make a good forecast of the actual rut development. For asphalt concrete, the NCHRP 1-37A mechanistic-empirical model was used in this study. This model estimates the permanent strain in the asphalt layer as function of the resilient strain, temperature and number of loading cycles. On the other hand, the model developed by Guimaraes (2009) for unbound layers was used in this study for a granular base and subgrade materials. This model estimates the permanent deformation in the unbound layer as function of the confining stress, deviator stress and number of loading cycles.

In order to predict the actual behavior of the unbound material under loading conditions, several testing methods are available. The idea behind these existing testing devices is to simulate the stresses occurring in the field beneath the traffic load. In this study only the repeated load triaxial test has been considered.

Finally, in order to perform the calibration from laboratory to field conditions, four instrumented flexible pavements were subjected to Heavy Vehicle Simulator (HVS) testing. Backcalculated moduli from multi depth deflectometers were embedded into an elastic multilayer system to obtain pavement responses. The number of equivalent standard axle load repetitions, the effective asphalt concrete mean temperature, and the in-place moisture content along with computed responses were used to calculate permanent deformation for the different layers. By comparing the measured rut depths with the predicted values from laboratory based models, the optimum combination of field calibration factors was determined so that the coefficient of variation was minimal by means of ordinary least squares (OLS).

1.1 Objective

The objective of this study was to calibrate permanent deformation prediction models using measured rutting data collected from instrumented flexible pavements under accelerated pavement testing conditions.

1.2 Test Sections

For the first stage of accelerated tests in Costa Rica the construction of 4 experimental sections was performed in May 2013 (Fig. 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) (Aguiar-Moya et al. 2012). 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.

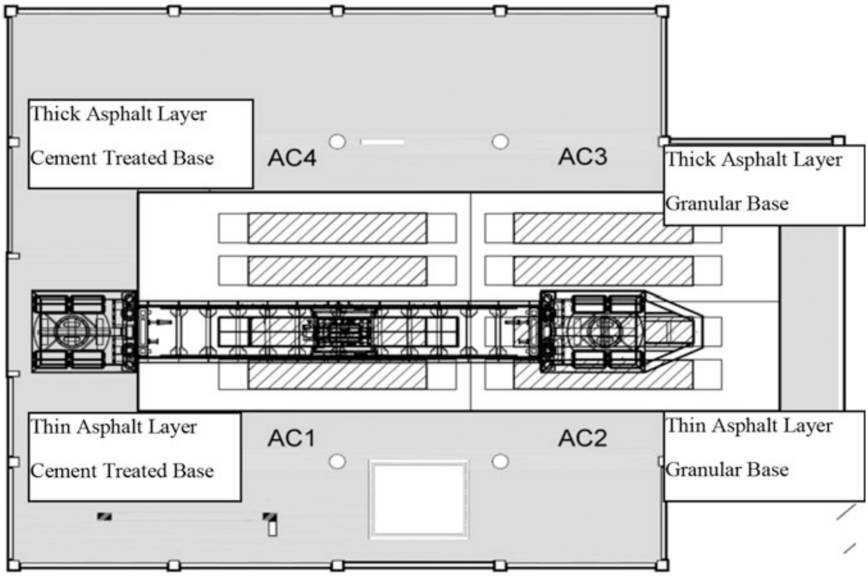


Fig. 1 Test track distribution

Table 1 Test tracks in-place properties

Properties\section	AC1	AC2	AC3	AC4
Asphalt concrete thickness (H1), cm	6.1	6.3	13.2	13.2
Granular base thickness (H2), cm	21.9	21.2	31.0	24.9
Subbase thickness (H3), cm	30.1	30.1	30.1	30.1
AC 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

The top layer consists of an asphalt concrete (AC) 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 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 %.

By September 2015, over a million load repetitions ranging from 40 to 80 kN have been applied on each section. The total estimated equivalent 40 kN load repetitions is over 50 million in a 2 years period of time.

1.3 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 and velocity of the load carriage. Embedded instrumentation include asphalt strain gauges (PAST model sensors), pressure cells (SOPT model sensors), multi depth deflectometers (MDDs), and moisture and temperature probes. These sensors were chosen based on previous HVS owner's experience (HVS 2015; Baker et al. 1994). Additionally, the HVS was equipped with a laser profiler that can be used to create a three-dimensional 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 (Leiva-Villacorta et al. 2013, 2015).

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 four different depths to cover all four structural layers. The thermocouples were placed at four depths: surface, middle depth of the AC layer, at

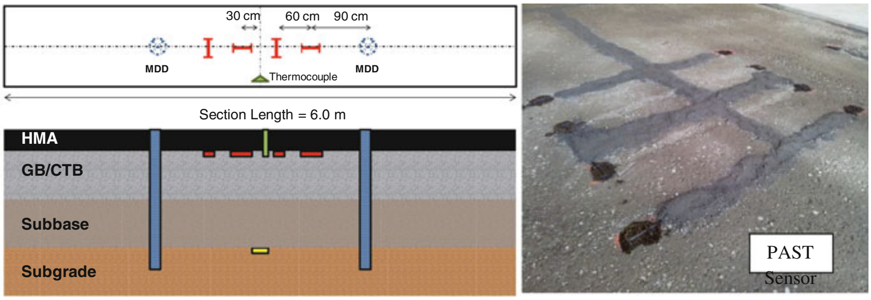


Fig. 2 Sensor array

the PAST sensors depth and 5 cm into the base layer. In the case of AC1 and AC4 sections the same gauge array was used while excluding PAST sensors.

Data collection of the 3D profile, strain, pressure, temperature and deflection is performed based on load repetitions. At the beginning of the test, data is obtained at short intervals. After 20,000 load repetitions, data is 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.

2 Permanent Deformation Models

For the asphalt concrete mixture the test was performed using unconfined cyclic loading at three different temperatures: 46, 52 and 58 °C. Permanent deformation was found to be a function of the resilient strain, temperature and number of loading cycles. The model for permanent deformation of the asphalt concrete layer is given in Eq. 1.

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_1 T^{\beta_2} N^{\beta_3} \tag{1}$$

where,

ε_p: permanent deformation (mm/mm × 10^{-3})
ε_r: resilient deformation (mm/mm × 10^{-3})
N: number of load repetitions
T: Temperature (°C)
β_1, β_2, β_3: regression coefficients

For the granular base and soil samples, the test was performed under repeated axial cyclic stress at different axial loads and different confining stresses. Permanent deformation was found to be a function of the confining stress, deviator stress, moisture content, and number of loading cycles. The model for permanent deformation of the unbound layers is given in Eq. 2. Each test was carried out for up to 5000 load cycles for 27 combinations of axial and confining stresses. The reason for this choice mainly depended on having an analysis period compatible with laboratory work constraints and a practical method to characterize the mechanical behavior of these materials. Moreover, such an analysis period is in accordance with the procedure based on the shakedown concept so the samples would not reach the plastic creep limit (Werkmeister et al. 2003, 2005). The Plastic Creep stage is achieved when the cumulative permanent strain rate is decreasing or constant. Although the deformation is not totally resilient, the permanent deformation is acceptable. In this case, the material could reach the failure after a very large number of load cycles.

$$pd = \beta_1 * N^{\beta_2} * \left(\frac{\sigma_d}{\rho_0}\right)^{\beta_3} * \left(\frac{\sigma_3}{\rho_0}\right)^{\beta_4} * \%w^5 \tag{2}$$

where,

pd: permanent deformation, mm
N: number of load repetitions
σ_d: deviator stress (kg/cm^2)
σ_3: confining stress (kg/cm^2)
ρ_0: reference pressure (1 kg/cm^2)
$\%w$: moisture content (%)
β_1, β_2, β_3, β_4, β_5: regression coefficients

3 Results

3.1 Development of Permanent Deformation Models

The laboratory-based model parameters obtained for the asphalt concrete, granular base and subgrade materials are given in Table 2. It this case it was found that the permanent deformation of the subgrade material was significantly affected by the moisture content and this can be observed with the larger β_5 coefficient compared to the granular base. Regression analysis results are given in Table 3 for each model.

Table 2 Material parameters (laboratory)

Layer	β_1	β_2	β_3	β_4	β_5	R^2
AC	$e^{-10.919}$	2.961	0.355	–	–	0.835
Granular base	$10^{-4.998}$	0.069	1.687	0.077	1.881	0.901
Subgrade	$10^{-32.954}$	0.040	2.041	0.421	16.893	0.856

Table 3 Models regression analyses

Layer	Variable	Coefficient	Standard error	T-stad.	*P* value
AC	$\text{Ln}(\beta_1)$	−10.919	0.098	−110.95	<0.05
	$\text{Ln}(\beta_2)$	2.961	0.024	124.27	<0.05
	$\text{Ln}(\beta_3)$	0.355	0.002	215.74	<0.05
Base	$\text{Log}(\beta_1)$	−4.998	0.0160	−311.4	<0.05
	$\text{Log}(\beta_2)$	0.0685	0.000887	77.2	<0.05
	$\text{Log}(\beta_3)$	1.687	0.00440	383.4	<0.05
	$\text{Log}(\beta_4)$	0.0768	0.00352	21.8	<0.05
	$\text{Log}(\beta_5)$	1.881	0.0213	88.4	<0.05
Subgrade	$\text{Log}(\beta_1)$	−33.0	0.0946	−348	<0.05
	$\text{Log}(\beta_2)$	0.0389	0.00101	38.6	<0.05
	$\text{Log}(\beta_3)$	2.04	0.00498	410	<0.05
	$\text{Log}(\beta_4)$	0.421	0.00741	56.8	<0.05
	$\text{Log}(\beta_5)$	17.0	0.0572	297	<0.05

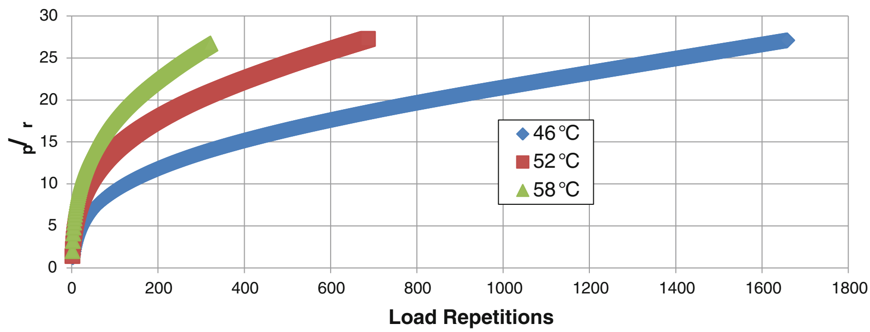


Fig. 3 Asphalt concrete permanent deformation test results

Figure 3 shows an example of the permanent deformation curves for the asphalt concrete. Only the data at which tertiary flow begins were used to fit the model.

Figure 4 shows an example of the permanent deformation curves for the granular base at one stress combination and at different moisture contents. In particular, for each test after the set confining pressure was reached, the axial stress with a haversine wave frequency of 1 Hz was applied.

The test program, in terms of confining pressure σ_3 and cyclic axial stress σ_d, was selected according to European Standards (EN 13286-7 2004). In Table 4 a detailed summary of the testing program is shown.

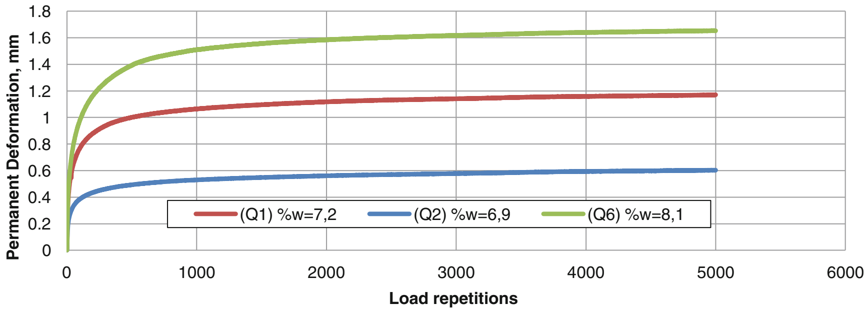


Fig. 4 Granular base permanent deformation for σ_3 = 40 kPa and σ_d = 60 kPa (Araya 2015)

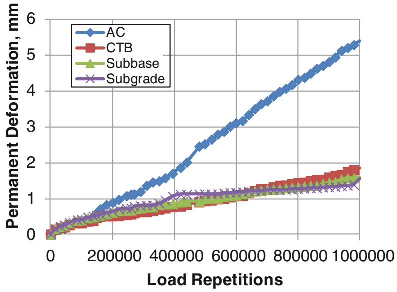
Table 4 Unbound materials test program

Granular material		Subgrade material	
σ_d, kPa	σ_3, kPa	σ_d, kPa	σ_3, kPa
60	40	40	30
80		60	
120		80	
80	69	60	40
138		80	
205		100	
155	103	75	50
206		100	
309		150	

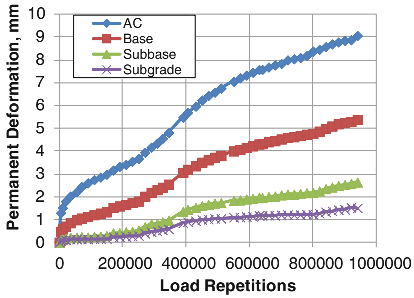
3.2 *Measured Permanent Deformation*

Various types of data are collected during an HVS test. The most important data from the viewpoint of developing permanent deformation models are the data obtained from the Multi-Depth Deflectometer (MDD) system. MDD modules were installed at predetermined depths in the pavement structures with a reference point at the anchor.

Installation was done after pavement construction and the MDD modules were placed at the layer interfaces, at mid depth of the base and subbase and near the road surface. Two kinds of output were obtained from the MDD stack. Firstly, the resilient deflection of each MDD module relative to the reference point at the anchor is measured under the moving wheel load. This results were used to obtain layer moduli by means of a backcalculation process. The second type of data obtained from the MDD stack, is the permanent movement of each MDD module relative to the reference point at the anchor for the duration of the HVS test. An example of the permanent deformation data obtained in this study is illustrated in Fig. 5. For this example, MDD modules were placed at the same depths for each section and both sections are equal in thicknesses but section AC1 has a cement



AC1



AC2

Fig. 5 Example of the measured permanent deformation

treated base (CTB) and AC2 has a granular base. The effect of the CTB layer with respect to deformation can be observed. While section AC1 had lower deformations overall, the layers underneath the CTB had almost the same level of deformation as if the CTB layer was behaving as a concrete slab with equally stress distribution along the test section. On the other hand, it can be observed that each layer of section AC2 had different deformation levels with the highest at the top and the lowest at the bottom.

The average measured deformation at eight million 40 kN equivalent axles for section AC1 was 8.7 mm, 11.7 mm for section AC2, 5.1 mm for section AC3, and 2.1 mm for section AC4 (Fig. 6). These results followed the expected trend since the section with the highest structural capacity was AC4 followed by AC3, then AC1 and finally AC2. The permanent deformation curves exhibited in Fig. 6 were generated from the HVS laser profiler. While this deformation considers the maximum surface rut depth, the MDD rut depth is 25 mm into the asphalt concrete and at the center of the wheel path. Therefore lower deformations were not only expected but also obtained for the MDD stack.

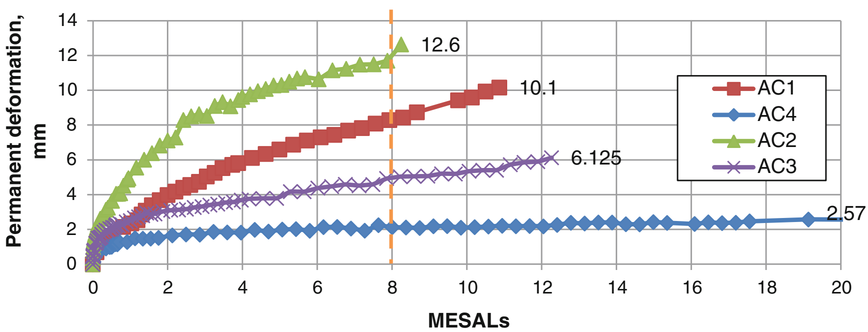


Fig. 6 Average measured permanent deformation

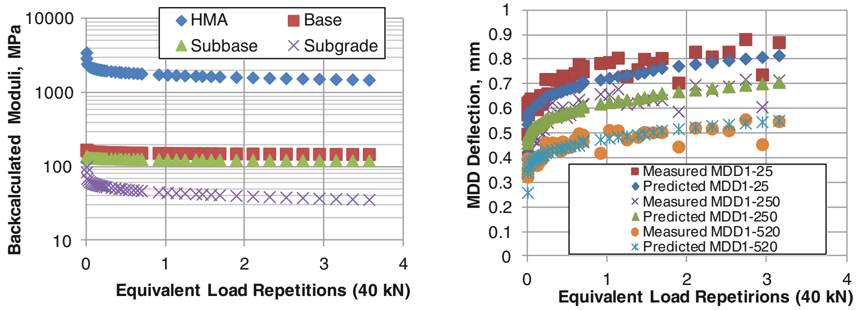


Fig. 7 MDD backcalculated layer moduli

3.3 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 (Ullidtz 1987) 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 7 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 ($R^2 > 0.8$) obtained between measured and estimated deflections along with a small deviation from the equality were the criteria to perform backcalculation and ensure that this methodology was successfully applied to each particular data set.

3.4 Estimated Versus Measured Rutting

An example of the estimated and measured permanent deformation trend for each layer is shown in Fig. 8. While measured rut depths show a typical variable trend during the test, predicted rut depths follow a well defined trend. However, in all cases, the model predictions did not fall within a established confidence interval of the measured results; therefore a calibration process and adjustment of the model coefficients was required. Only one MDD sensor was installed at the surface of the asphalt layer providing four different data sets. For the granular base and subgrade one or more MDD sensors were installed to capture the permanent deformation providing at least six different data sets.

For asphalt concrete, the model underestimated the measured deformation at any stage of the field test. For the granular base and subgrade materials the estimated results were more scattered and the trend was to overestimate at the early stages while throughout the end of the test the tendency was to do the opposite. This

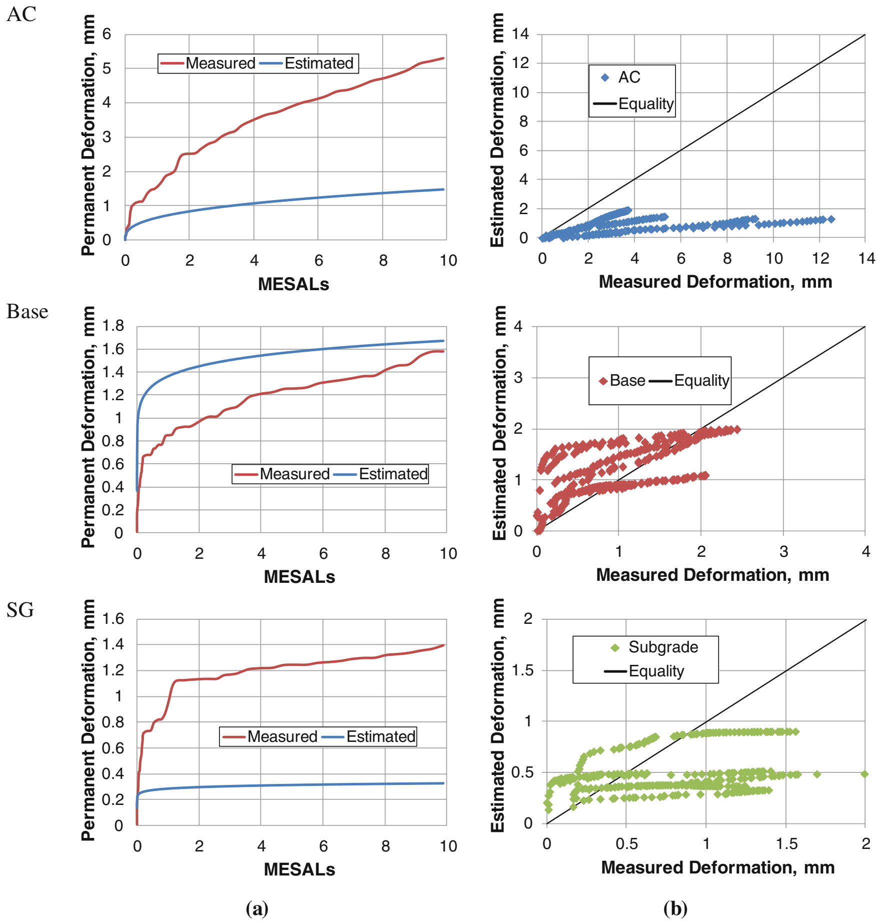


Fig. 8 Estimated versus measured permanent deformation, **a** section AC1, **b** all sections

behavior was an indication that the asphalt concrete model could be calibrated with the adjustment of one coefficient while the remaining models could require a full parameter calibration.

The cumulative distribution plot of the absolute relative errors for all measured/estimated data points is given in Fig. 9. It can be observed that the median relative errors for the AC, base and subgrade were 65 %, 60 % and 28 % respectively. In addition, 90 % of the relative errors for the AC, base and subgrade were below 90 %, 265 % and 180 % respectively. These large errors were expected due to the high deviation of the estimated results with respect to the measured ones (equality).

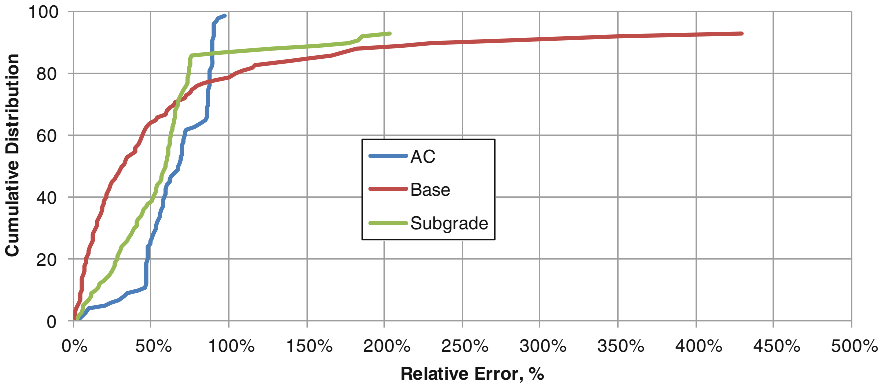


Fig. 9 Cumulative distribution of the absolute relative errors before calibration

3.5 Calibration Process

By comparing the measured rut depths with the predicted values from laboratory based models, the optimum combination of field calibration factors was determined so that the coefficient of variation was minimal by means of OLS. The laboratory-developed model coefficients were used to predict permanent deformation and the results either over-estimated or under-estimated this type of distress. The resulting errors were either always positive or negative, meaning that they will never cancel out, thus, contributing to the bias in the prediction model. To correct these biases (systematic errors) in the model, the bias correction factors were introduced (also known as the calibration coefficients).

The calibration process performed in this study started off by multiplying a single correction factor (k_i) to each individual regression coefficient (β_i) and completely adjust the initial model. As mentioned above, the process ended when the coefficient of variation was minimal by means of OLS. Table 5 shows the resulting calibration factors for each of the evaluated models. In the case of the asphalt concrete model a single factor was applied. For the remaining models a full calibration was required in order to correct the systematic errors in the model.

The cumulative distribution plot of the absolute relative errors for all measured/estimated data points is given in Fig. 10. It can be observed that the median relative errors for the AC, base and subgrade were after calibration 27 %, 9 % and 7 % respectively. In addition, 90 % of the relative errors for the AC, base and subgrade were below 90 %, 100 % and 40 % respectively. These errors are significantly lower than the results obtained before calibration. A further investigation of this errors indicated that large errors were associated with low level rut depths where a small change in the predicted value could result in errors up to 100 %. The final-calibrated models are shown in Eqs. 3–5.

Table 5 Calibration factors

Layer	k_1	k_2	k_3	k_4	k_5
AC	16.256	–	–	–	–
Granular base	1.048	3.841	0.608	1.911	0.873
Subgrade	0.346	5.728	−0.411	−0.631	0.393

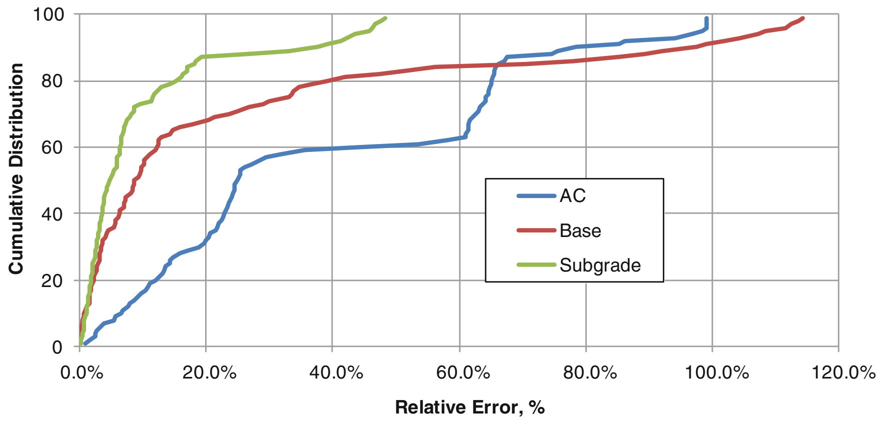


Fig. 10 Cumulative distribution of the absolute relative errors after calibration

$$\varepsilon_p = 16.254 * e^{-10.919} * T^{2.961} * N^{0.355} * \varepsilon_r \quad (3)$$

$$pd_{GB} = 10^{-5.239} * N^{0.265} * \sigma_d^{1.025} * \sigma_3^{0.147} * \%w^{1.642} \quad (4)$$

$$pd_{SG} = 10^{-11.424} * N^{0.229} * \sigma_d^{-0.839} * \sigma_3^{-0.266} * \%w^{6.677} \quad (5)$$

where,

εp: permanent deformation (mm/mm × 10^{-3})
εr: resilient deformation (mm/mm × 10^{-3})
N: number of load repetitions
T: Temperature (°C)
pd: permanent deformation, mm
σ_d: deviator stress (kg/cm^2)
σ_3: confining stress (kg/cm^2)
ρ_0: reference pressure (1 kg/cm^2)
$\%w$: moisture content (%)

4 Conclusions and Recommendations

Basic, conceptual models have been developed for the permanent deformation of pavement layers from repeated triaxial load testing. The application of these models to predict actual pavement deformation resulted in large errors which lead to a calibration process. Calibration of the asphalt concrete permanent deformation model was achieved with the use of a single factor at reasonable low errors. On the other hand, calibration of the unbound materials required a full set of factors to significantly decrease the error. However, this type of calibration provided minimal coefficient of variation and lowest error by means of ordinary least squares (OLS).

The main source of data for developing these design models was the Heavy Vehicle Simulator test data collected over 2 years period of time in Costa Rica for one asphalt mixture type, one granular base and subbase type and one subgrade type. Therefore, the application of these models is limited to the materials evaluated until now.

This is the first attempt ever performed in Costa Rica to calibrate permanent deformation models based on full scale accelerated test results. The process was considered to be successfully applied to the collected data set. Therefore, this process will be utilized and improved for future studies.

Because of the time involved in large scale accelerated pavement testing, it is expected that a combination of laboratory testing and accelerated pavement testing would yield the quickest results. Therefore, it should be possible to generate the type of data needed to develop transfer functions for these materials from laboratory test methods and then be verified by means of a limited number of full scale, accelerated pavement tests.

References

Aguiar-Moya, J. P., Corrales, J. P., Elizondo, F., & Loría-Salazar, L. (2012). PaveLab and heavy vehicle simulator implementation at the National Laboratory of Materials and Testing Models of the University of Costa Rica. *Advances in pavement design through full-scale accelerated pavement testing*, APT 2012.

Araya, Y. (2015). *Desarrollo de modelos de deformación permanente para materiales granulares y suelos*. San José: Universidad de Costa Rica.

Baker, H. B., Buth, M. R., & Van Deusen, D. A. (1994). Minnesota road research project: Load response instrumentation installation and testing procedures. Minnesota Department of Transportation.

EN 13286-7. (2004). Unbound and hydraulically bound mixtures—Part 7: Cyclic load triaxial test for unbound mixtures. European Committee for Standardization.

Guimaraes, A. C. (2009). *Método mecanístico-empírico para la predicción de la deformación permanente en suelos tropicales para pavimentos*. Río de Janeiro: Universidad Federal de Río de Janeiro.

Harvey, J., & Kanekanti, V. (2006). *Calibration of mechanistic-empirical models using the California heavy vehicle simulators. Transportation Association of Canada*. Quebec: International Society for Asphalt Pavements.

Heavy Vehicle Simulator. (2015). Monitoring of test sections and instrumentation. Documento consultado el 6 de abril del. http://www.gautrans-hvs.co.za/.

Leiva-Villacorta, F., Aguiar-Moya, J. P., & Loria-Salazar L. G. (2013). Ensayos acelerados de pavimento en Costa Rica. Infraestructura Vial Vol. 15 Núm. 26.

Leiva-Villacorta, F., Aguiar-Moya, J. P., & Loria-Salazar, L. G. (2015). Accelerated pavement testing first results at the Lanammeucr APT Facility. In *Transportation Research Board 94th annual meeting*. Washington, DC.

Lekarp, F. (1997). Permanent deformation behaviour of unbound granular materials. Licentiate Thesis. Kungliga Tekniska Högskolan.

NCHRP. (2004). Guide for mechanistic-empirical design of new and rehabilitated pavement structures. National Cooperative Highway Research Program, Report 1-37A, March 2004. http://www.trb.org/mepdg/guide.htm.

Powell, R. (2006). *Predicting field performance on the NCAT pavement test track*. Auburn: Auburn University.

Ullidtz, P., et al. (2006). Calibration of incremental-recursive flexible damage models in CalME using HVS experiments. Report prepared for the California Department of Transportation (Caltrans) Division of Research and Innovation. University of California Pavement Research Center, Davis and Berkeley.

Ullidtz, P. (1987). Pavement analysis. *Development in civil engineering* (Vol. 19). Amsterdam: Elsevier.

Von Quintus, H. L., Darter, M. I., & Mallela, J. (2007). Recommended practice for local calibration of the ME pavement design guide. Applied Research Associates, Inc.—Transportation Sector.

Werkmeister, S., Dawson, A., & Wellner, F. (2003). Permanent deformation behavior of granular materials and the shakedown concept. *Journal of the Transportation Research Board, 1757*, 75–81.

Werkmeister, S., Dawson, A., & Wellner, F. (2005). Permanent deformation behavior of granular materials. *Road Materials and Pavement, 6*, 31–51.