REVISION OF DAMAGE MODELS FOR ASPHALT PAVEMENTS

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Abstract

The objective of this paper is to present damage models that were calibrated against laboratory data for six South African hot-mix asphalt (HMA) designs. The work forms part of the effort to revise the South African pavement design method. The permanent deformation and fatigue models development were performed in-line with the international state of the art. The damage functions used in the regression analysis allow a suitable fit to the laboratory data for both permanent deformation and fatigue. The next phase of the project will focus on the further calibration of the damage models presented in this paper against local accelerated pavement testing and long term pavement performance data.

1. INTRODUCTION

The damage models for hot-mix asphalt (HMA) materials in the current South African mechanistic empirical design method have a limited scope. The method only includes basic fatigue models for standard mix types and does not allow the prediction of permanent deformation performance. As part of the SANRAL funded research effort to revise the South African pavement design method (SAPDM), the design models for asphalt will be enhanced in line with the international state of the art. The point of departure for the project is that models should be based on the Mechanistic Empirical Design Guide (MEPDG) (NCHRP, 2004), recently released in the United States. Calibrated damage models are to be developed that allow reliable prediction of rutting and fatigue damage to asphalt layers. Importantly, the models should be sensitive to changes in the material behaviour at different loading speeds, daily temperature fluctuations and ageing of the bituminous binder over time. A need exists to validate and calibrate modern analysis methods for South African mix types, pavement structures, and environmental conditions.

An extensive laboratory study was undertaken to characterize damage accumulation in South African HMA materials. The first step in the process was to select appropriate test methods and develop detailed test protocols to ensure consistent and reliable results throughout the experimental programme. The final test methods and protocols according to which the experiments were performed were presented by Anochie-Boateng, et al, (2010a). Besides the development of damage models, the data from the experimental programme was used to develop resilient response prediction models for the mixes as discussed in a separate paper submitted to this conference by Anochie-Boateng et al (2011). The experiments included advanced binder testing, dynamic modulus testing of cylindrical specimens, Four Point Bending (FPB) flexural fatigue testing and permanent deformation tests. The test results for the selected mixes were published per mix in separate reports (Anochie-Boateng, et al, 2010b-f). Apart from the five mixes selected by the technical committee for the SAPDM, additional testing funded by the Southern African bitumen association

(Sabita) was performed on a High Modulus Asphalt (HiMA) mix (Anochie-Boateng, et al, 2010g). The following mix types were characterized in the laboratory study:

- Mix 1: a Bitumen Treated Base (BTB) with 40/50 penetration grade binder,
- Mix 2: a coarse graded medium continuous mix with a modified binder of AE2 grade,
- Mix 3: a medium continuous mix with a modified binder of AE2 grade,
- Mix 4: a medium continuous mix with 60/70 penetration grade binder,
- Mix 5: a Bitumen Rubber Asphalt Semi Open (BRASO), and
- A HiMA mix design with 20/30 penetration grade binder.

The objective of this paper is to present damage models that were calibrated against a set of laboratory data for six South African asphalt mix designs listed above. These models will be used in static and dynamic pavement analysis to predict the structural response of the pavement system and the service life of HMA surfacings and base courses in the revised SAPDM. The models are developed based on laboratory results and will be further calibrated against field data in the next phase of the project. This paper represents a refinement of the work on damage model development for the SAPDM by Denneman et al (2011).

The models developed as part of this study are based on the international state of the art. Over the years a lot of fatigue and permanent deformation models in particular, have evolved from the USA models. As per the brief for the project, the point of departure was to calibrate models available from literature, notably the new MEPDG (NCHRP, 2004) and the Californian Mechanistic Empirical design method (CalME) (Ullidtz et al 2006a,b), rather than to develop new models.
The proposed damage models have different levels of analysis, which require different levels of input. The level of analysis will be selected based on client requirements and the risk involved with a design. The structure of the analysis levels are discussed in Section 2. The fatigue models developed based on FPB testing are discussed in Section 3. The laboratory results from repeated simple shear tests at constant height (RSST-CH) were used to calibrate damage models for permanent deformation as discussed in Section 4.

2. PROPOSED LEVELS OF ANALYSIS

Extensive testing is required to develop mix specific damage models. Therefore it will probably not be cost-effective to calibrate damage models for every project and mix design. The SAPDM will allow for different levels analysis. Although the exact definition of the different levels may still change, the envisaged hierarchy involves the following three levels:

- Level 1; the highest level of analysis, requiring the most advanced input resulting in the most accurate and reliable damage predictions. The test matrix at this level will involve the calibration of project specific damage models for the HMA materials, based on dynamic modulus, fatigue and permanent deformation testing.
- Level 2; at the intermediate level, project specific testing will still be required, but this will involve only dynamic modulus tests to develop a stiffness master curve for the material (refer Anochie-Boateng et al 2011). Damage

accumulation in the material is predicted using calibrated damage models that require the resilient response of the material as input.

- Level 3; at the lowest level of analysis the resilient response master curve is constructed for an HMA design based on binder and volumetric properties using a predictive equation. The master curve is then used in the damage prediction as discussed under level 2.

3. FATIGUE PREDICTION MODELS

3.1 Background

Load associated fatigue cracking is one of the major distress types occurring in asphalt layers as a result of action of repeated loading caused by traffic induced tensile and shear stresses in the pavement system. Fatigue cracks are initiated at points where critical tensile strains and stresses occur. Once the damage initiates at the critical location, the action of traffic eventually causes these cracks to propagate through the entire bound layer.

Additionally, the critical strain is also a function of the stiffness of the mix. Since the stiffness of an asphalt mix in a pavement layered system varies with depth; these changes will eventually effect the location of the critical strain that varies with depth; these changes will eventually affect the location of the critical strain that causes fatigue damage.

For several decades, it has been common to assume that fatigue cracking normally initiates at the bottom of the asphalt layer and propagates to the surface (bottom-up cracking). This is due to the bending action of the pavement layer that results in flexural stresses to develop at the bottom of the bound layer. However, recent studies have indicated that fatigue cracking may also be initiated from the top and propagates down (top-down cracking). This type of fatigue is not as well defined from a mechanistic viewpoint as the more classical “bottom-up” fatigue.

There are several models that have been developed around the world to predict fatigue cracking. Fatigue models are typically divided into two main types, the strain-based models and the strain-modulus based models. These models are termed phenomenological models as they present observed relationships that have not been derived from a theoretical analysis of pavement mechanics. Recent pavement design methods, such as MEPDG and CalME however, recommend the use of fatigue algorithms or models that predict the number of load repetitions to fatigue cracking as a function of the tensile strain and mix stiffness modulus

For the most part, fatigue life models or performance equations are developed in the laboratory using some form of a fatigue testing apparatus. Typically, HMA samples are cut into prismatic specimens (beams), and are subjected to a repeated flexural loading either in a controlled strain or controlled stress mode. The commonly used apparatus for testing is simple flexure with third-point loading.

In the fatigue test performed under controlled strain conditions, the tensile strains (ε_t) are induced at the bottom of the beam sample using usually a FPB loading system. The repeated loading necessary to induce the specified tensile strains will decrease during the test due to the viscoelastic properties of the HMA until failure. Generally, failure is defined by the number of load cycles applied that reduces the stiffness of the specimen to 50% of its initial value (AASHTO T 321-07, 2007). For the SAPDM project the loading was extended to reach a final stiffness reduction of 70% ($N_{70\%}$).

The results of fatigue life tests are expressed by the fatigue laws as illustrated in Figure 1. The fatigue laws represent the relation between the number of repeated load applied to the sample to reach failure (N_f) and the level of or strain/stress induced.

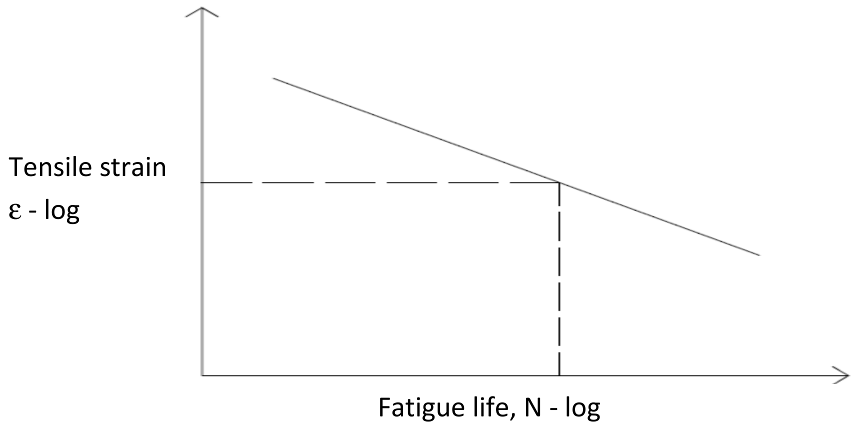

Figure 1 Fatigue life curve from beam fatigue testing

3.2 Fatigue modelling for SAPDM

The current road design methods used in the Unites States recommend the four point flexural beam fatigue test to model fatigue cracking of HMA (NCHRP 1-37A 2004). The four point beam fatigue test protocol as discussed in Anochie-Boateng et al (2010a) was used to establish a database for the mixes tested as part of the SAPDM project. The fatigue tests were performed at three test temperatures of 5, 10, and 20℃, and with a minimum of four strain levels varying between 200 and 1000 micro strain (με). A continuous sinusoidal displacement waveform on top of the sample with frequency of 10 Hz was applied on prismatic beams of 400 x 63 x 50 mm. At least three replicate samples were tested to generate a fatigue curve for each asphalt mix. The laboratory test protocol and test data collected are presented elsewhere in separate reports (Anochie-Boateng et al., 2010b-g). Figure 2 shows the fatigue curves for all five mixes compacted to design density.

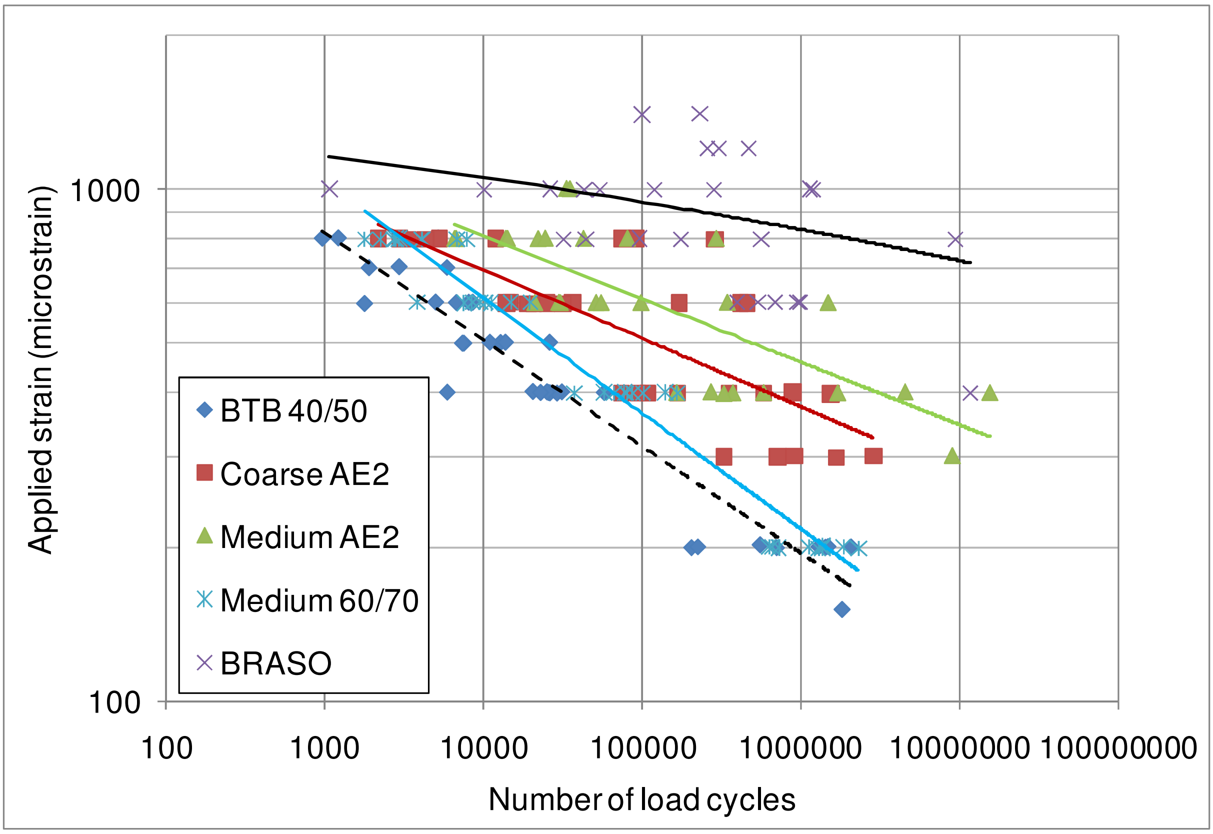

Figure 2 Fatigue curves for different mixes

3.2 MEPDG fatigue prediction approach

The MEPDG uses a fatigue damage model with a structure as expressed by Pell (1987), shown as Equation 1.

$$N_f = k_1 \left(\frac{1}{\varepsilon_t}\right)^{k_2} \left(\frac{1}{E}\right)^{k_3} \tag{1}$$

where,

N_f = Number of load applications until fatigue failure

ε_t = Horizontal tensile strain at the bottom of the HMA layer (mm/mm)

E = Stiffness modulus of HMA (MPa)

k_i = constants determined from four point beam controlled strain fatigue tests

This function is often developed based on laboratory data and then shifted or calibrated to field performance with correction factors. The work for the SAPDM follows the same approach.

The MEPDG provides calibrated values and mix related functions to determine the constants $k_{(1\text{-}3)}$ for a design. The MEPDG relations calibrated based on an extensive laboratory study and the results for 82 LTPP sections located in 24 US States. Damage accumulation in MEPDG is determined using Miner's Law, which states that damage is given by the following relationship:

$$D = \sum_i \frac{n_i}{N_i} \tag{2}$$

where,

n_i = Number of load applications (cycles) at condition, *i*

N_i = Number of load applications (cycles) at failure for condition, *i*

The RStat statistical analysis software package was used to perform multiple regression analyses to establish the model parameters for the individual asphalt mixes. The regression analysis for the SAPDM project is performed using a standard RStat template developed by Kimmie et al. (2009). Table 1 lists the generalized model parameters (k_1, k_2 and k_3) established for the five mixes.

In the next phase of the project the MEPDG fatigue damage models thus developed will be further calibrated and validated using field data to create the final transfer functions.

Table 1: MEPDG fatigue regression parameters for different mixes

Mix type	k_1	k_2	k_3	R^2
BTB 40/50	8.951E-10	4.496	0.478	0.928
Coarse AE2	4.867E-05	5.389	2.454	0.865
Medium AE2	3.869E-03	5.672	3.187	0.936
Medium 60/70	6.604E-08	4.364	0.788	0.976
BRASO	4.096E-08	7.464	3.311	0.829

3.3 CalME approach

In CalME fatigue damage is expressed as a reduction in stiffness of the material. This is done by introducing a damage parameter (ω) to the dynamic modulus relation used in the MEPDG:

$$\log(E) = \delta + \frac{\alpha \times (1-\omega)}{1 + exp(\beta + \gamma \log(tr))} \tag{3}$$

where

E = the stiffness modulus of the damaged material [MPa];

tr = reduced time [s];

α, β, γ, and δ = experimentally determined constants, and

log = logarithm with base 10.

ω = the damage, which is calculated from:

$$\omega = A \times MN^{\alpha}\left(\frac{\mu\varepsilon}{200\mu strain}\right)^{\beta} \times \left(\frac{E}{3000MPa}\right)^{\gamma} \times \exp\left(\delta \times t\right) \tag{4}$$

where

E = modulus of the damaged material;

MN = number of load repetitions in millions ($N/10^6$);

$\mu\varepsilon$ = strain at the bottom of the asphalt layer in micro strain;

t = temperature in °C

A, α, β, γ, and δ = constants determined from four point beam controlled strain fatigue tests. Note that these constants have different values than the constants in the dynamic modulus function in Equation 3.

Eq. 5 represents damage in terms of relative decrease in modulus.

$$\omega = \frac{E_i - E}{E_i} \tag{5}$$

where

E_i = initial modulus

Using Eqs. 4 and 5, an optimization analysis was performed on the fatigue data to determine parameters A, α, β, γ, δ. The CalME damage models thus developed will be further calibrated and validated using field data to create the final transfer functions during the next phase of the SAPDM project.

Table 2: CalME fatigue regression parameters for different mixes

Mix type	A	α	β	γ	δ	R^2
BTB 40/50	0.85259	0.353	1.546	-0.211	-0.017	0.840
Coarse AE2	0.44600	0.251	1.469	-0.197	-0.054	0.814
Medium AE2	0.61084	0.164	0.995	-0.609	-0.063	0.893
Medium 60/70	1.12164	0.395	1.703	-0.415	-0.044	0.908
BRASO	0.35582	0.136	0.824	-0.297	-0.052	0.846

3.4 Implementation of fatigue models into SAPDM

Different levels of input will be required for the fatigue models, depending on the risk involved with the project as discussed in Section 2. In each level of analysis, the strain and stiffness properties will be used in conjunction with the calibrated/validated laboratory parameters obtained from Eq. 1 and Table 1 (k_1 to k_3) or Eq.4 and Table 2 ($A, \alpha, \beta, \gamma, \delta$, for implementation in the SAPDM. Strains will be calculated at critical positions in the asphalt layer, which will be used to predict the occurrence of top-down and bottom-up cracking in the material.

4 PERMANENT DEFORMATION PREDICTION MODELS

4.1 Background

Materials subjected to constant or repeated loading deform over time to relieve stress. This permanent deformation phenomenon is known as creep. Creep occurs as a result of long term exposure to levels of stress that are below the yield or ultimate strength of the material, but above the flow strength of the material at that temperature. The rate of deformation is a function of the material properties, the exposure time, exposure temperature and the applied stress. Like many materials, HMA exhibits three distinct phases in creep response. Figure 3 provides an impression of the development of the plastic strain in HMA under repeated loading. In the primary creep phase, after construction, traffic load leads to densification of the material. The void content of dense graded mixes the void content reduces from approximately 7 percent to around 4 percent during this phase. The traffic action results in aggregate particles moving into their preferred orientation.

In secondary creep phase, the rate of deformation slows down considerably. The deformation is caused by shear stress overcoming flow strength of the material. The flow strength consists of two components, i.e. friction and cohesion. The deformation takes place in small iterative steps with every load application.

Eventually the void condition and the level of permanent strain will cause the HMA to enter the third phase and rapid unstable shear failure occurs.

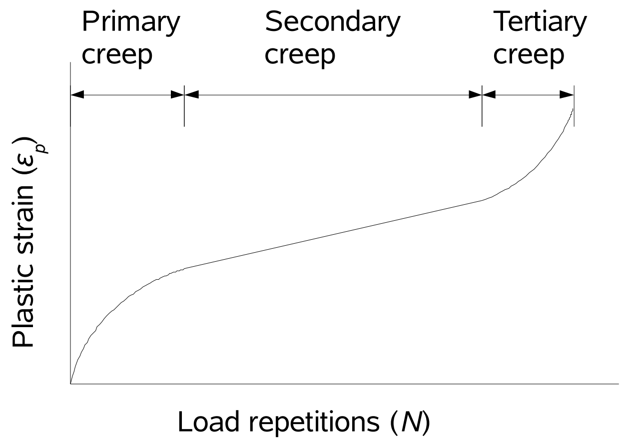

Figure 3 Three stages of creep

A variety of mathematical functions have been used to model permanent deformation in HMA by different researchers. In general, only the first two creep phases are considered in modelling. An often used shape (e.g. Monismith et al, 1975) is a power function. Both the MEPDG and CalME methods use the power function shape in the permanent deformation although CalME also applies a Gamma function shape as an alternative

4.2 Permanent deformation model development

The permanent deformation models in MEPDG are based on uni-axial repeated loading tests performed under laboratory conditions. The relations were then calibrated against field data. The approach is based on incremental damage, wherein rutting at the centre of a pavement layer is estimated for every sub-season. Elastic vertical compressive strains in the asphalt layer are calculated from the stress state at mid depth of the asphalt layer. In the final MEPDG deformation damage model the mix properties are represented by the dynamic modulus only. There is no option to use mix specific experimental permanent deformation characteristics in the design method. The MEPDG model thus assumes the main predictor of permanent deformation to be the vertical strain. The performance of the MEPDG models for South African pavements will be assessed as part of the next phase of the project.

The permanent deformation models developed for SAPDM are based on shear strain, rather than the vertical strain approach used in MEPDG. This is based on the theory that rutting in HMA is mainly due to shear deformations, which is in-line with the assumptions behind the CalME deformation models. RSST-CH tests were performed on six mixes, the results of which are contained in Anochie-Boateng et al. (2010b-g). For five of these mixes specimens were tested at two density levels, i.e. the design density and the construction density (7% air voids). For one mix (HiMA) tests were only performed at design density. For all mixes tests were performed at three different temperatures. Five specimens were tested at each combination of temperature and density. Figure 4 shows the average permanent deformation curves for the specimens compacted to design density.

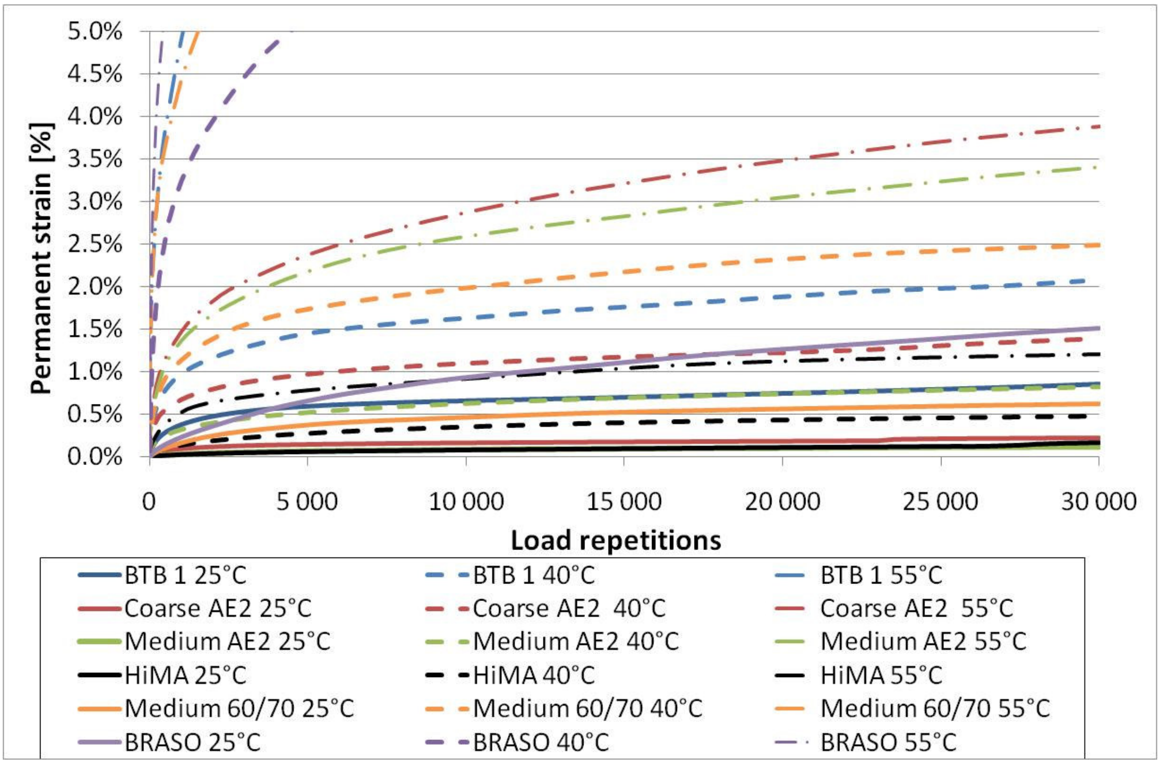

Figure 4 Permanent deformation curves for different mixes

Three different non-linear functions were fitted to the permanent deformation laboratory data. The first two, shown as Equation 5 and 6, are Gamma and power functions shapes based on the functions used in CalME. The functions differ from the original CalME models in that a shear stress term was omitted for the SAPDM analysis. Since all RSST-CH tests were run at the constant stress level of 69 kPa, the shear stress term becomes a constant. Initial explorative analysis of the use of the models to predict permanent deformation of relatively thin South African road surfacing under accelerated pavement test (APT) showed that keeping the uncalibrated shear stress factor in the equations seriously compromised the accuracy of the models.

The third function has the shape of the MEPDG damage model for permanent deformation, with the important difference that the function, shown as Equation 7, uses the shear strain instead of vertical strain as an input. The reason for including the MEPDG function is that it includes temperature as a parameter, which was found to allow better fits for certain mix types.

$$\gamma_p = \exp\left(A + \alpha\left(1 - \exp\left(\frac{-\ln(N)}{\gamma}\right)\left(1 + \frac{\ln(N)}{\gamma}\right)\right)\right)\gamma_e \tag{5}$$

$$\gamma_p = AN^{\alpha}\gamma_e^{\gamma} \tag{6}$$

$$\frac{\gamma_p}{\gamma_e} = \alpha_1 T_C{}^{\alpha_2} N^{\alpha_3} \tag{7}$$

γ_p	= Plastic shear strain,
A, α, α_i and γ	= Regression coefficients (γ was set to 1.0 for Equation 6)
N	= Number of load repetitions,
γ_e	= Elastic shear strain

The results of the nonlinear regression analyses are shown in Table 3. For each of the equations shown above the table includes the best fit values of the regression coefficient for the different mixes, as well as the standard error of the fit, the student t-value for determination of the confidence interval, the probability of observing a value exceeding the t-value and finally the 95% confidence interval for the regression coefficient. Apart from a regression for the different mixes an analysis was also run for the combined data of all mixes. The results for this analysis are shown in the last column of the table.

Table 3: Permanent deformation regression coefficients

	Mix 1			Mix 2			Mix3			HiMA mix			Mix4			Mix 5			Combined data		
Eqn. 7	α_1	α_2	α_3	α_1	α_2	α_3	α_1	α_2	α_3	α_1	α_2	α_3	α_1	α_2	α_3	α_1	α_2	α_3	α_1	α_2	α_3
Values	**28.50**	**-1.093**	**0.494**	**0.461**	**0.334**	**0.254**	**0.058**	**0.648**	**0.337**	**0.114**	**0.149**	**0.490**	**18.28**	**-0.707**	**0.380**	**87.84**	**-1.046**	**0.341**	**471.9**	**-1.388**	**0.271**
Std. Error	6.264	0.044	0.007	0.121	0.064	0.005	0.015	0.062	0.006	0.032	0.066	0.010	2.792	0.031	0.005	7.434	0.019	0.003	26.50	0.012	0.002
t value	4.549	-24.89	67.74	3.807	5.241	49.18	3.937	10.43	55.68	3.576	2.243	47.51	6.548	-22.70	77.08	11.82	-55.82	114.2	17.81	-112.7	118.1
Pr(>|t|)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.025	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2.5%	18.39	-1.177	0.479	0.273	0.211	0.245	0.036	0.537	0.326	0.064	0.018	0.470	13.42	-0.770	0.370	74.93	-1.081	0.335	427.2	-1.410	0.267
97.5%	43.69	-1.006	0.508	0.763	0.462	0.264	0.091	0.765	0.349	0.197	0.285	0.511	24.79	-0.644	0.389	102.9	-1.011	0.347	521.1	-1.366	0.276
Eqn. 6	**A**	α		**A**	α		**A**	α		**A**	α		**A**	α		**A**	α		**A**	α	
Values	**0.120**	**0.638**		**1.779**	**0.252**		**0.760**	**0.337**		**0.205**	**0.490**		**0.607**	**0.457**		**0.871**	**0.422**		**1.480**	**0.309**	
Std. Error	0.006	0.005		0.082	0.005		0.044	0.006		0.020	0.010		0.020	0.004		0.026	0.003		0.034	0.003	
t value	21.39	120.5		21.67	48.99		17.24	54.93		10.21	47.45		29.95	118.2		33.70	126.7		43.60	118.9	
Pr(>|t|)	0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000	
2.5%	0.109	0.628		1.634	0.242		0.679	0.326		0.168	0.470		0.569	0.449		0.824	0.416		1.418	0.304	
97.5%	0.131	0.648		1.935	0.261		0.849	0.349		0.247	0.510		0.647	0.464		0.920	0.428		1.543	0.314	
Eqn. 5	**A**	α_1	γ_1	**A**	α_1	γ_1	**A**	α_1	γ_1	**A**	α_1	γ_1	**A**	α_1	γ_1	**A**	α_1	γ_1	**A**	α_1	γ_1
Values	**-1.498**	**11.10**	**5.758**	**-7.849**	**11.14**	**1.753**	**-0.816**	**5.514**	**4.030**	**-0.615**	**10.210**	**7.505**	**0.018**	**8.023**	**5.918**	**-3.175**	**8.215**	**2.747**	**-4.181**	**8.063**	**2.175**
Std. Error	0.268	0.479	0.493	1.677	1.646	0.095	0.401	0.147	0.454	0.459	2.352	2.121	0.146	0.318	0.415	0.335	0.279	0.087	0.400	0.376	0.057
t value	-5.595	23.19	11.67	-4.681	6.767	18.53	-2.034	37.41	8.874	-1.341	4.341	3.539	0.123	25.20	14.25	-9.473	29.49	31.47	-10.45	21.45	38.36
Pr(>|t|)	0.000	0.000	0.000	0.000	0.000	0.000	0.042	0.000	0.000	0.180	0.000	0.000	0.902	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2.5%	-2.089	10.44	4.908	-12.29	8.368	1.572	-1.738	5.290	3.290	-1.683	8.315	4.866	-0.271	7.557	5.233	-3.906	7.703	2.582	-5.057	7.364	2.066
97.5%	-1.005	12.44	6.918	-5.006	15.53	1.965	-0.162	6.001	5.120	0.091	23.546	15.46	0.271	8.776	6.787	-2.547	8.842	2.929	-3.432	8.893	2.293

4.3 Implementation into SAPDM

Using the regression parameters shown in Table 3, Equations 5, 6 and 7 can now be used to predict permanent deformation (rutting) in HMA, provided that the elastic shear strain in the material is known. The CalME approach uses the elastic shear strain at the outer edge of the dual wheel at a depth of 50 mm as an input for the permanent strain prediction. Based on the analysis of a number of typical South African pavement structures by Denneman et al (2011), the elastic shear strain at the edge of the tyre, but in the centre of the pavement layer will be used for the SAPDM.

To illustrate the use of the damage models developed in this paper a simple simulation of a heavy vehicle simulator (HVS) test is performed. The example also serves to provide an initial assessment of the predictive performance of the laboratory calibrated models developed in this study. The data of HVS test 442A4 was used as reported by Steyn & Fisher (2008). The pavement consisted of a 40 mm continuously graded HMA layer on a granular base. The pavement structure and HVS loading was modelled as described by Denneman et al, 2008. The HMA mix design used for the HVS test was identical to Mix 4 tested as part of the SAPDM project. Therefore the moduli and permanent deformation regression coefficients determined for Mix 4 as shown in Table 3 could be used in the analysis.
Figure 5 shows the average permanent deformation recorded in the HVS test as well as the permanent deformation predicted using the different models. The HVS test was run at 40℃ up to 240 000 repetitions, where the temperature was increased to at 50℃ at 310 000 repetitions the temperature was again increased to 60℃ up to end of test at 370 000 repetitions. The permanent deformation recorded in the test was restricted to deformation in the asphalt layer, as was confirmed by means of test pits dug after the tests were completed. The graph also shows the predictive performance of the original MEPDG equation using the vertical strain in the centre of the layer as input. Note that the original MEPDG equation was calibrated based on field data as well as laboratory data while the models developed for this paper are based on laboratory data only.

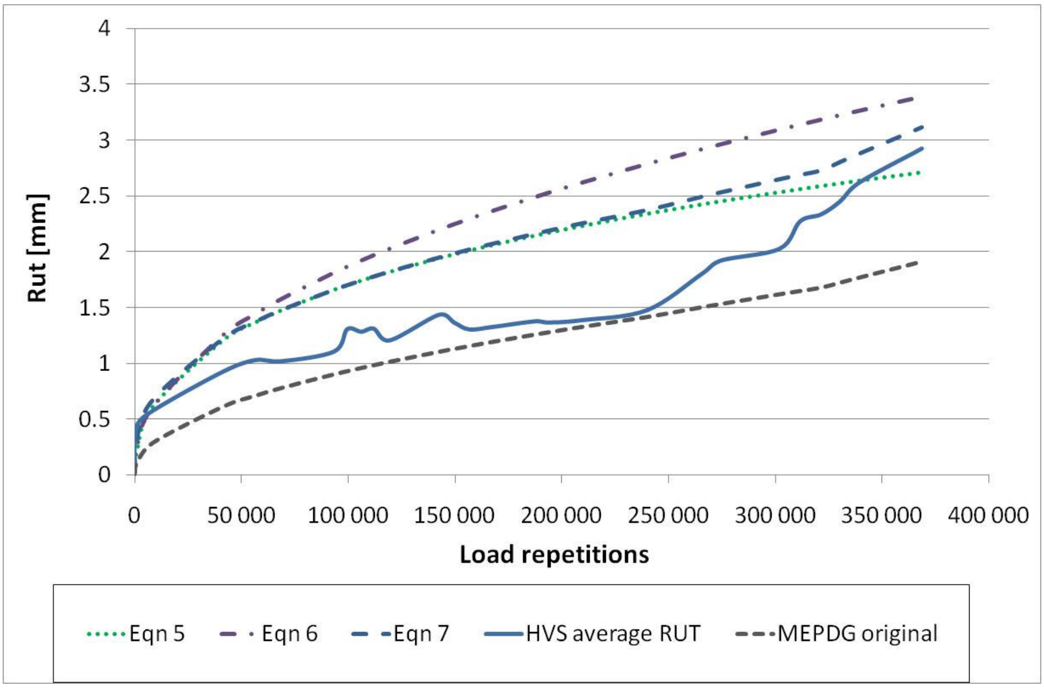

Figure 5 Predictive performance of mix specific models
Figure 6

5 CONCLUSIONS

The objective of this paper is to present permanent deformation and fatigue damage models that were calibrated against a set of laboratory data for six South African asphalt mix designs.

Models were developed based on regression analysis of the results obtained from laboratory experiments. The form of the permanent deformation and fatigue prediction equations is based on the models in the MEPDG and CalME, with certain adjustments to better suit the purposes of the SAPDM project.

The various damage functions allow a suitable fit to the laboratory data for both fatigue and permanent deformation. The next phase of the project will focus on the further calibration of the damage models presented in this paper against local accelerated pavement testing and long term pavement performance data.

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KEY WORDS

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