Calibration of Mechanistic-Empirical Models for Cracking and Rutting of New Pavements Using Heavy Vehicle Simulator Tests

Per Ullidtz[1], John Harvey[2], Khalid Ghuzlan[3], Bor-Wen Tsai[4], Bruce Steven[5] and Carl Monismith[6]

Abstract

The Heavy Vehicle Simulator (HVS) is ideally suited for initial calibration of Mechanistic-Empirical models for pavement design. The HVS may be seen as a large scale laboratory equipment, with detailed control of materials, loads and environment, and with the possibility of carrying the tests through to failure. In-situ pavements, used for long term observation of pavement performance, are normally designed with a high level of reliability, resulting in very few failures within the normal service life. HVS testing may be used to close the gap between the common, small scale laboratory tests and the long term observation of in situ pavement performance.

The two HVSs owned by the California Department of Transportation (Caltrans) have been used for initial calibration of the Mechanistic-Empirical models of a computer program known as CalME. CalME has an incremental-recursive procedure, making it possible to follow the gradual deterioration of the pavement during the HVS loading test. 13 new flexible pavements, with different materials and layer thicknesses, have been tested at moderate temperatures (≈ 20 ºC) and 16 sections at high temperatures (40-50 ºC). Elastic moduli were determined from Falling Weight Deflectometer (FWD) tests and from frequency sweep tests on beams in the laboratory. Fatigue parameters were determined from constant strain beam tests, and permanent deformation parameters from Repeated Simple Shear Tests at Constant Height (RSST-CH). The models derived from laboratory tests were directly used in CalME, with calibration factors to match the HVS tests.

The sections tested at moderate temperature were all instrumented with Multi Depth Deflectometers (MDDs), which record both resilient and permanent deformations at several depths in the pavement structure. Surfaces deflections were also measured with a Road Surface Deflectometer (RSD, similar to a Benkelman beam) and the surface profiles were recorded by a laser profilometer. The resilient deflections changed markedly during the tests, on average the surface deflection increased by a factor of 2.4 from the beginning to end of the test. It is essential that this change in response is modeled correctly for the full duration of the test, otherwise any attempts at calibrating the empirical models would be futile. The change in response is due to the damage to the materials caused by the loads, so the validation of the response model and the calibration of the fatigue damage models for the materials are mutually dependent. Once the pavement response has been modeled correctly for the complete duration of the test, the empirical models for permanent deformation can be calibrated.

The MDDs also record the permanent deformation of the individual layers in the pavement during the test. These measurements as well as the pavement surface profiles are used for calibrating the empirical models for permanent deformation at moderate temperature. The high temperature tests had relatively few load applications and were only used for calibrating the permanent deformation models of different asphalt materials.

Keywords: HVS, Mechanistic-Empirical, Cracking, Rutting, CalME, Recursive
[1]Dynatest International, Naverland 32, Glostrup, DK 2600, DENMARK, Email: pullidtz@dynatest.com, [2] University of California, Davis, California, USA, Email: jtharvey@ucdavis.edu, [3] California Department of Transportation, California, USA, Email: Khalid_Ghuslan@dot.ca.gov, [4] University of California, Berkeley, California, USA, Email: bwtsai@berkeley.edu, [5] University of California, Davis, California, USA, Email: bdsteven@ucdavis.edu, [6] University of California, Berkeley, California, USA, Email: clm@newton.berkeley.edu

Introduction

Modern methods of pavement design aim at predicting the gradual functional and structural deterioration of the pavement layers, over the whole life time of the pavement. This is achieved using an Incremental-Recursive method based on Mechanistic-Empirical principles (IRME). For each increment of time the materials parameters are determined as a function of climate, aging, loading conditions and previous damage and the critical response (stresses and strains) is calculated using a mechanistic model. The calculated response is then used with empirical relationships to predict the damage caused during the increment, and the output from the current increment is used, recursively, as input to the next time increment.
Calibrating an IRME procedure is a great challenge. Long term pavement performance studies must eventually be used in the calibration process, but there is usually a wide gap between the limited knowledge of materials characteristics, normally from laboratory testing, and the in situ performance with uncertainties on pavement structure, traffic loading and climatic conditions. The Heavy Vehicle Simulator (HVS) is an excellent tool to help pave this gap. The short test section of the HVS can be carefully constructed, with detailed knowledge of the pavement materials. The test section can be instrumented and surveyed, so that the pavement response and performance can be measured frequently during testing. The climate can be controlled or closely monitored. Each load application is known exactly with respect to magnitude, speed and position and, very importantly, the test can be carried on to failure. Real roads are normally designed with a high level of reliability. If a long term pavement performance test section was designed with a reliability of 95% there is only a 5% “chance” that it will fail before the end of the design life.

HVS tests

Since 1995 the California Department of Transportation (Caltrans) has owned and operated two HVS equipments in cooperation with the University of California Pavement Research Center (UCPRC). The HVS tests described in this paper were all done on flexible pavements and are from the period 1995 to 2004.
During HVS testing, pavement response - in terms of deflections at the surface and/or at multiple depths - was measured at regular intervals (Harvey et al., 1996). A Road Surface Deflectometer (RSD) was used to measure deflections at the surface between the tires of a dual wheel, similarly to the Benkelman Beam. Multi-Depth Deflectometers (MDDs) were installed in a number of the test sections to measure both the resilient deflections and the permanent deformations at multiple depths. The pavement profile was measured using a laser profilometer, and any distress at the surface of the pavement was recorded.

The HVS test series were grouped by “Goals”. The temperature was controlled for all of the tests. The Goals modeled, and their controlled test temperatures and conditions were:

- *Goal 1*, a comparison of new pavement structures with and without Asphalt Treated Permeable Base (ATPB) layer, tested under dry conditions, moderate temperatures (20 °C)
- *Goal 3 Cracking*, a comparison of reflection cracking performance of Asphalt Rubber Hot Mix – Gap Graded (ARHM-GG) and Dense Graded Asphalt Concrete (DGAC) overlays placed on the cracked Goal 1 sections, dry conditions, 20 °C
- *Goal 3 Rutting*, a comparison of rutting performance of ARHM-GG and DGAC overlays of previously untrafficked areas of Goal 1 pavements, dry conditions, 40 °C and 50 °C at 50 mm depth, four different tire/wheel types (bias-ply duals, radial duals, wide-base single and aircraft)
- *Goal 5*, a comparison of new pavement structures with and without ATPB layer under wet conditions (water introduced into base layers), moderate temperatures, 20 °C
- *Goal 9*, initial cracking of asphalt pavement with six replicate sections in preparation for later overlay, new pavement, ambient rainfall, 20 °C.

Details on the testing and analysis can be found in Ullidtz et al. (2007)

CalME, an Incremental-Recursive Mechanistic-Empirical model (IRME)

CalME is a pavement design program, for new pavement design as well as for rehabilitation design. CalME has three levels of design:

1. Caltrans current empirical methods, the “R-value” method for flexible structures and the “Deflection Reduction” method for rehabilitation design,
2. a “Classical” Mechanistic-Empirical design, largely based on the Asphalt Institute method, using ESALs and a weighted mean annual environmental condition, and
3. an IRME model in which the materials properties are updated in terms of damage for each time increment, using the “time hardening” approach, and used (recursively) as input to the next time increment. This approach predicts the pavement conditions at any point in time during the pavement life.

The IRME mode may also be used to simulate HVS tests or sections from test tracks. For this mode the climatic conditions and the loading during the test is imported into the CalME database. Temperatures measured at different depths and the number of applications of different loads and their load levels, are imported for each hour of the test. This data is used by CalME to determine the layer parameters and for calculating the response, for each one hour increment of the simulation. For the simulations described here the response model LEAP was used (Symplectic Engineering

Corporation, 2004). LEAP allows partial slip between the layers, which was observed at a number of the test sections.
The measured pavement response (resilient deflections at different depths in this case) and the permanent deformations are also imported, so that the results of the simulation can be quickly compared to actual test data. If backcalculated layer moduli from FWD testing are available, this may also be imported into the database for comparison to simulated values.
Some of the sub-models used in CalME are briefly described in the following.

Master curve for asphalt materials

The master curve was determined from frequency sweep tests supplemented by FWD testing. The format used for the master curve is the same as used in the MEPDG (NCHRP, 2004). For intact asphalt the format is:

$$\log(E_i) = \delta + \frac{\alpha}{1 + \exp(\beta + \gamma \log(tr))}$$

Equation 1: Asphalt modulus versus reduced time.

where E_i is the modulus in MPa,
tr is reduced time in seconds and
α, β, γ, and δ are constants determined from frequency sweep tests.
Log is the logarithm to base 10.

Reduced time is found from:

$$tr = lt \times \left(\frac{visc_{ref}}{visc} \right)^{aTg}$$

Equation 2: Reduced time as a function of loading time and viscosity

where *lt* is the loading time (in sec),
viscref is the binder viscosity at the reference temperature,
visc is the binder viscosity at the present temperature, and
aTg is a constant.

Damage to asphalt materials

During HVS testing the resilient deflections normally show a considerable increase. An example is shown in Figure 1. The initial deflection, under a 40 kN load, is a little more than 0.2 mm, whereas the deflection (under the same wheel load) is at about 0.9 mm towards the end of the test. This means that the pavement response is changing dramatically during the test. It is essential that the damage causing this change in response is captured in the simulation, otherwise there would be no purpose in trying to calibrate the empirical sub-models for predicting pavement performance.
For damaged asphalt concrete the modulus was determined from:

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

Equation 3: Modulus of damaged asphalt concrete (variables same as in Equation 1).

where the damage, ω, was calculated from:

$$\omega = A \times MN^{\alpha} \times \left(\frac{\mu\varepsilon}{\mu\varepsilon_{ref}}\right)^{\beta} \times \left(\frac{E}{E_{ref}}\right)^{\gamma} \times \exp(\delta \times t)$$

Equation 4: Damage as a function of loads, strain, modulus and temperature.

where MN is the number of load applications in millions,
$\mu\varepsilon$ is the tensile strain at the bottom of the asphalt layer,
E is the modulus,
t is the temperature in ºC,
$\mu\varepsilon_{ref}$ is a reference constant with 200 µstrain value,
E_{ref} is a reference constant with 3000 MPa value, and
A, α, β, γ, and δ are constants (not related to the constants of Equation 1).

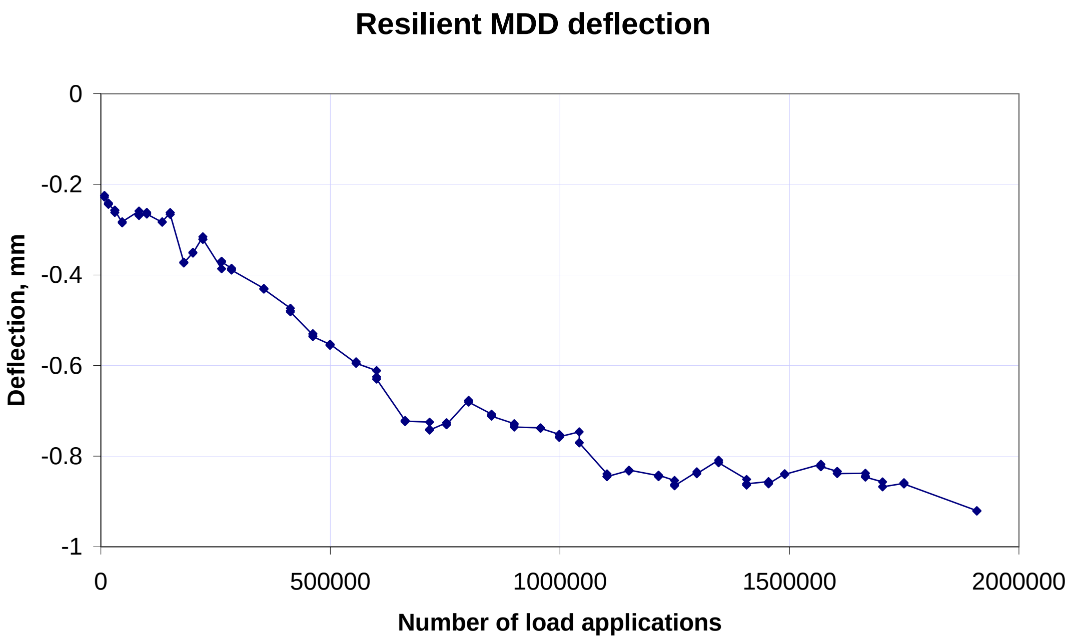

Figure 1 Example of increase in resilient deflection during HVS test.

The constant γ in Equation 4 was assumed equal to $\beta/2$, making damage a function of the strain energy. The parameters of Equation 4 were determined from four point beam, controlled strain, fatigue testing, by minimizing the Root Mean Square (RMS) of the difference between the measured modulus and the modulus calculated from Equation 3. The minimization was done in Excel using Solver.

Permanent deformation of asphalt materials

Permanent deformation of the asphalt may be caused by post compaction of the material or by shearing. The post compaction is normally small and is assumed to be

proportional to the reduction in air voids. In CalME it may be imposed during the initial loading phase. The shear deformation is more important and is determined using a shear-based approach, developed by Deacon et al. (2002). The phenomenon is roughly illustrated by Figure 2, where the triangular area slides downwards pushing material outwards and upwards.
The permanent, or inelastic, shear strain, γ^i, will depend on the shear stress, τ, the elastic shear strain, γ^e, and the number of load repetitions. The relationship is determined from Repeated Simple Shear Tests at Constant Height (RSST-CH) in the laboratory. The best fitting relationship for the materials used was found to be a gamma function:

$$\gamma^i = \exp\left(A + \alpha \times \left[1 - \exp\left(-\ln(N)/\gamma\right) \times \left(1 + \ln(N)/\gamma\right)\right]\right) \times \exp\left(\beta \times \tau/\tau_{ref}\right) \times \gamma^e$$

Equation 5: Gamma function for permanent shear strain.

where γ^e is the elastic shear strain,
τ is the shear stress,
N is the number of load repetitions,
τ_{ref} is a reference shear stress (0.1 MPa), and
A, α, β, and γ are constants determined from the RSST-CH.

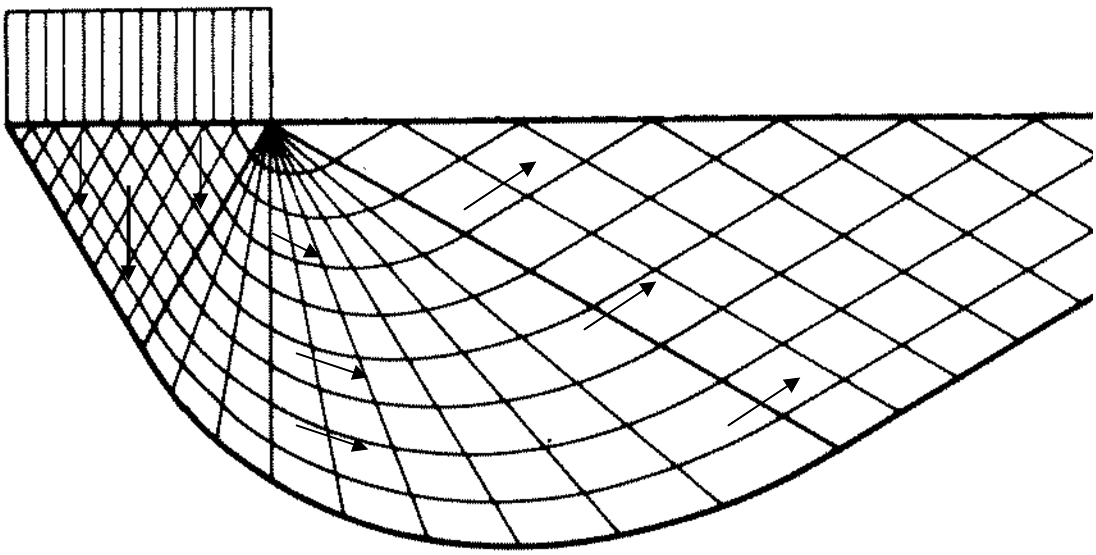

Figure 2 Illustration of shear deformation.

The permanent deformation of the asphalt is calculated from:

$$dp = K \times \sum h_i \times \gamma_i^i$$

Equation 6: Calculation of permanent deformation.

where K is a calibration factor determined from HVS testing,
h_i is the thickness of layer i, and
γ^i_i is the permanent (inelastic) shear strain in layer i.
The summation is done for the top 100 mm of the asphalt.

Moduli of unbound layers

The moduli of the unbound layers were found to be stress dependent following the well known relationship:

$$E = k_1 \times \left(\frac{stress}{p} \right)^{k_2}$$

Equation 7: Non-linear moduli of unbound materials.

where *E* is the modulus,
stress is the bulk stress for granular materials and the deviator stress for cohesive materials,
k_1, k_2 (positive for bulk stress and negative for deviator stress) and *p* are constants (p = 0.1 MPa).

More controversially, it was also found that the modulus of the unbound materials varied with the stiffness of the layers above the material. For granular layers this effect is the opposite of what would be expected based on Equation 7. A decrease in the stiffness of the layers above a granular layer would be expected to cause an increase in the bulk stress in the granular material and, therefore, an increase in the modulus, whereas the opposite effect is observed. The effect is in good agreement with the observation made by Richter (2006) that the moduli of granular layers, backcalculated from FWD tests on LTPP Seasonal Monitoring sections, tend to decrease, instead of increase, with increasing bulk stress.
To allow for this effect, the stiffness of each unbound layer was modeled as a function of the bending stiffness of the layers above it:

$$E = Eo \times \left(1 - \left(1 - S / S_{ref}\right) \times Stiffness\ \ factor\right), \ with$$

$$S = \left(\sum_{1}^{n-1} h_i \times \sqrt[3]{E_i} \right)^3$$

Equation 8: Modulus of each unbound layer as a function of the bending stiffness of the layers above it.

where *Eo* is the modulus (of layer n) at the reference stiffness,
S is the combined bending stiffness of the layers above layer *n*,
S_{ref} is the reference stiffness (a value of 3500^3 N·mm was used here),
h_i is the thickness of layer *i* in mm, and
E_i is the modulus of layer *i* in MPa.

The *Stiffness factor* was determined from regression analyses of moduli backcalculated from FWD tests. Stiffness factor represents “fraction of the decrease in the stiffness of the layers above the one under consideration”.

Permanent deformation of unbound layers

The model for permanent deformation of the unbound layers, *dp*, is given in Equation 9, where *MN* is the number of load applications in millions, $\mu\varepsilon$ is the vertical compressive strain at the top of the layer and *E* is the modulus. The reference constants are $\mu\varepsilon_{ref}$ = 1000 $\mu strain$ and E_{ref} = 40 MPa. The relationship was derived from tests in the Danish Road Testing Machine during the International Pavement Subgrade Performance Study (2005):

$$dp\ mm = A \times MN^{\alpha} \times \left(\frac{\mu\varepsilon}{\mu\varepsilon_{ref}}\right)^{\beta} \times \left(\frac{E}{E_{ref}}\right)^{\gamma}$$

Equation 9: Permanent deformation of unbound layers.

The model agreed well with the measured permanent deformations in the unbound materials, but it should be noted that the permanent deformations were all quite small.

Example of Simulation

The example presented here is from Goal 1 (HVS test numbered 503RF). The parameters used for the models given above are shown in Table 1.

Table 1. Parameters used in simulation of test 503RF

	Modulus						
	α	*β*	*γ*	*δ*	*aT*	*A*	*VTS*
Top AC	1.8738	-0.3987	0.9436	2.301	1.3529	9.6307	-3.5047
Bottom AC	1.9428	-0.4007	0.9807	2.301	1.2824	9.6307	-3.5047
	Unbound						
	Eo	*Stiffness factor*		*Power on load*			
AB + AS	269 MPa	0.43		0.6			
Subgrade	112 MPa	0.21		-0.3			
	Fatigue						
	A	*α*	*β*	*γ*	*δ*	Shift fact	
Top AC	0.00154	0.8695	4.1968	2.0984	0.1619	3	
Bottom AC	0.00125	0.8399	3.9718	1.9859	0.1913	3	
	AC rutting						
	A	*α*	*β*	*γ*	*K*		
Top+Bottom	-1.316	5.218	1.03	2.86	0.08		
	Unbound rut						
	A	*α*	*β*	*γ*			
AB + AS	0.8	0.333	1.333	0.333			
Subgrade	1.1	0.333	1.333	0.333			

The pavement had two layers of conventional dense graded asphalt (AC, top layer 74 mm, bottom layer 88 mm), an aggregate base (AB of 274 mm), and an aggregate subbase (AS of 305 mm) on a clay subgrade. Most of the material parameter values were derived from laboratory tests, the remaining from FWD tests or from calibration using a similar test section (numbered 501RF). A reference temperature of 20 ºC and a reference loading time of 0.015 sec (corresponding roughly to 10 Hz) were used for the AC modulus.
The load was a dual wheel with radial tires at a pressure of 0.69 MPa and a loading speed of approximately 7.6 km/h. The loads were laterally distributed over a width of 1000 mm. The first load level was 40 kN, it was then increased to 80 kN and finally to 100 kN (for most of the load applications).
Some of the resilient deflections measured with the MDDs under a 40 kN wheel load are shown in Figure 3. The legend M is for measured deflections, shown with a fully drawn line, and C is for calculated deflections, shown with a dotted line. Deflections were measured and calculated at the top of the AC (legend 0, for depth 0 mm), close to the top of the AB (legend 137, for depth 137 mm from AC surface) and close to the top of the subgrade (legend 640, for depth 640 mm). The large increase in resilient deflections during the test may be noticed. The first visible cracking was recorded at approximately 650,000 load applications, when almost all of the increase in deflection had already taken place.

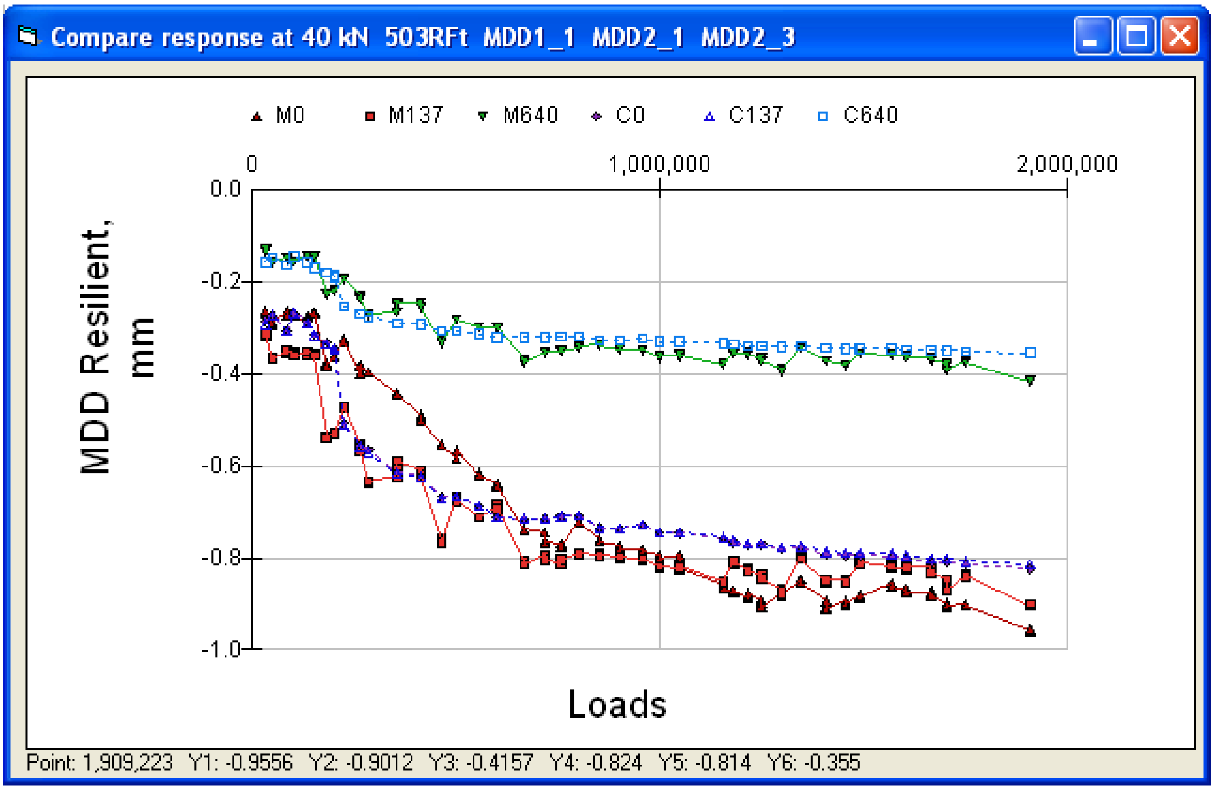

Figure 3. Resilient deflections section 503RF, 40 kN.

The first step in the calibration process is to ensure that the response calculated by the mechanistic model is reasonably correct, for the duration of the test. Once the response model results in a satisfactory prediction of the measured resilient deflections, then the empirical relationships for permanent deformation may be calibrated.

The permanent deformation of the asphalt layers, measured and calculated, are shown in Figure 4, and the total permanent deformation at the pavement surface is shown in Figure 5, as measured by MDD 4 at the surface, as the average of the measured surface profile, measured by laser profilometer, and as calculated by *CalME*.

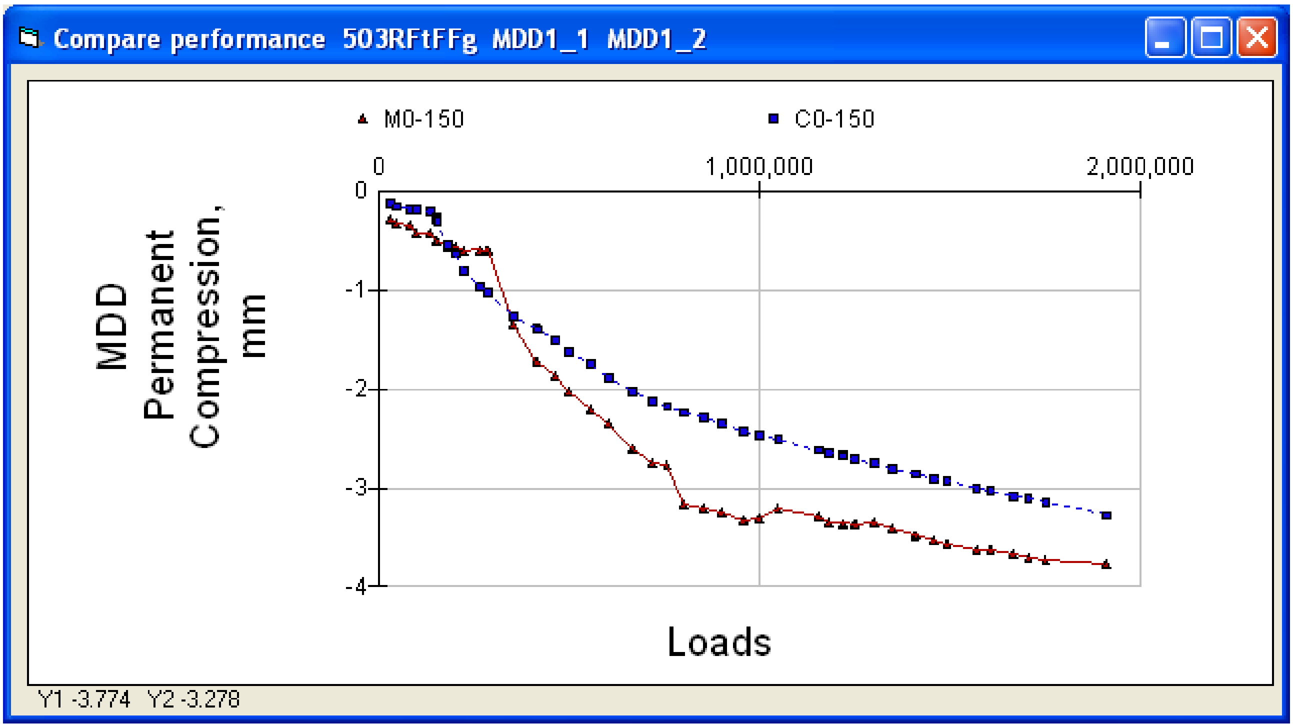

Figure 4. Permanent deformation of the asphalt layers, measured and calculated.

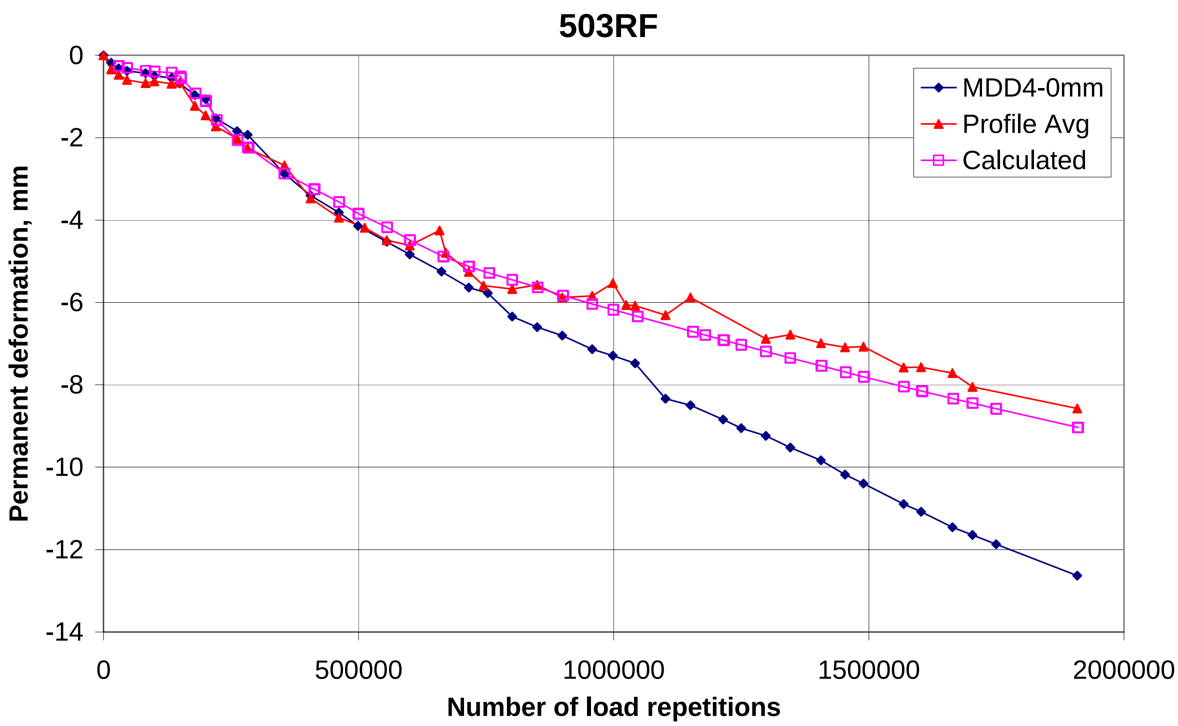

Figure 5. Permanent deformation at the surface of the pavement.

Summary of Results for All Calibration Sections

Figure 6 compares the measured and calculated ratios of final to initial deflection under a 40 kN wheel load for all of the HVS cracking tests. The deflections were measured by MDDs (at or close to the surface) and with the RSD. In general the response model did capture the increase in surface deflections quite well. The standard error of estimate is 0.61 mm.
The measured and calculated final permanent deformations of the asphalt layers are shown in Figure 7 for all of the test sections where it was recorded. The standard error of estimate for the permanent deformation of the asphalt was 1.76 mm. For the granular layers and for the subgrade the agreement between measured and calculated permanent deformation was equally good, but the final permanent deformations in those layers were much smaller, usually less than 3 mm.

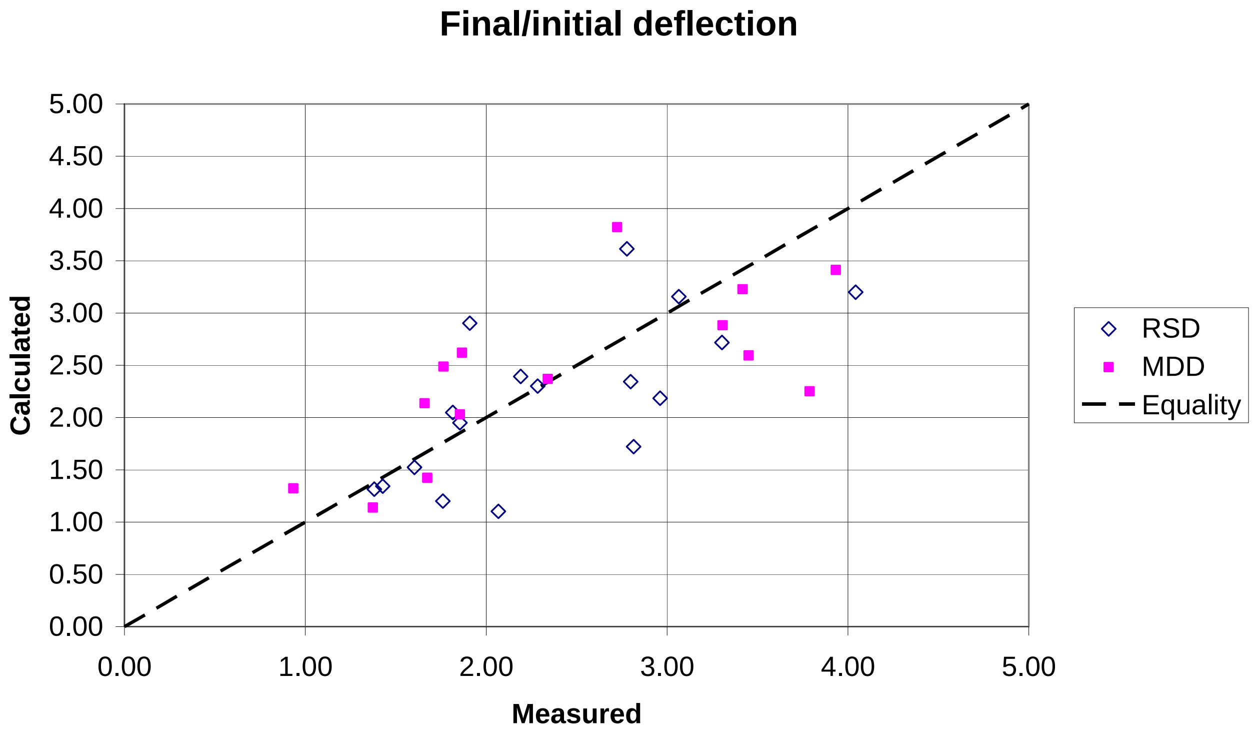

Figure 6: Ratio of final to initial surface deflection for HVS cracking tests.

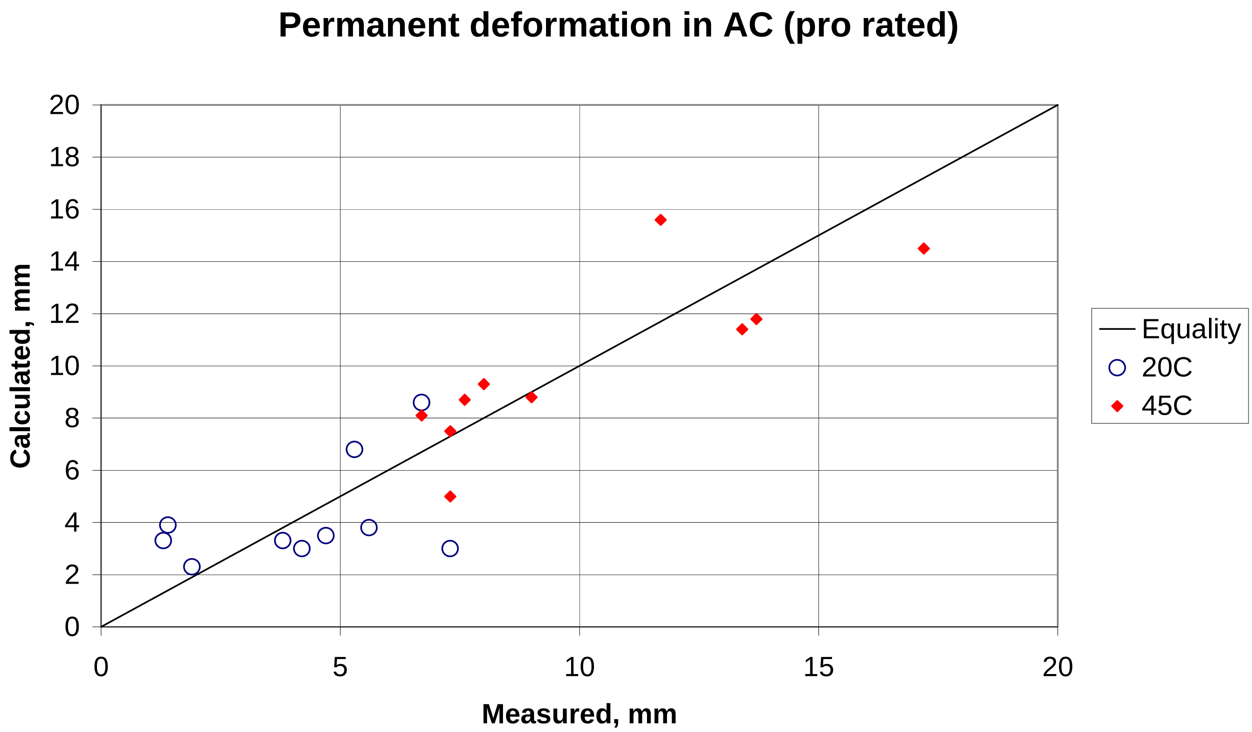

Figure 7: Final permanent deformation of AC layers.

The total permanent deformation at the pavement surface is shown in Figure 8. The standard error of estimate is 2.18 mm. Some of the outliers (two of the Goal 5 tests) were caused by insertion of water directly into the pavement (per the test plan).

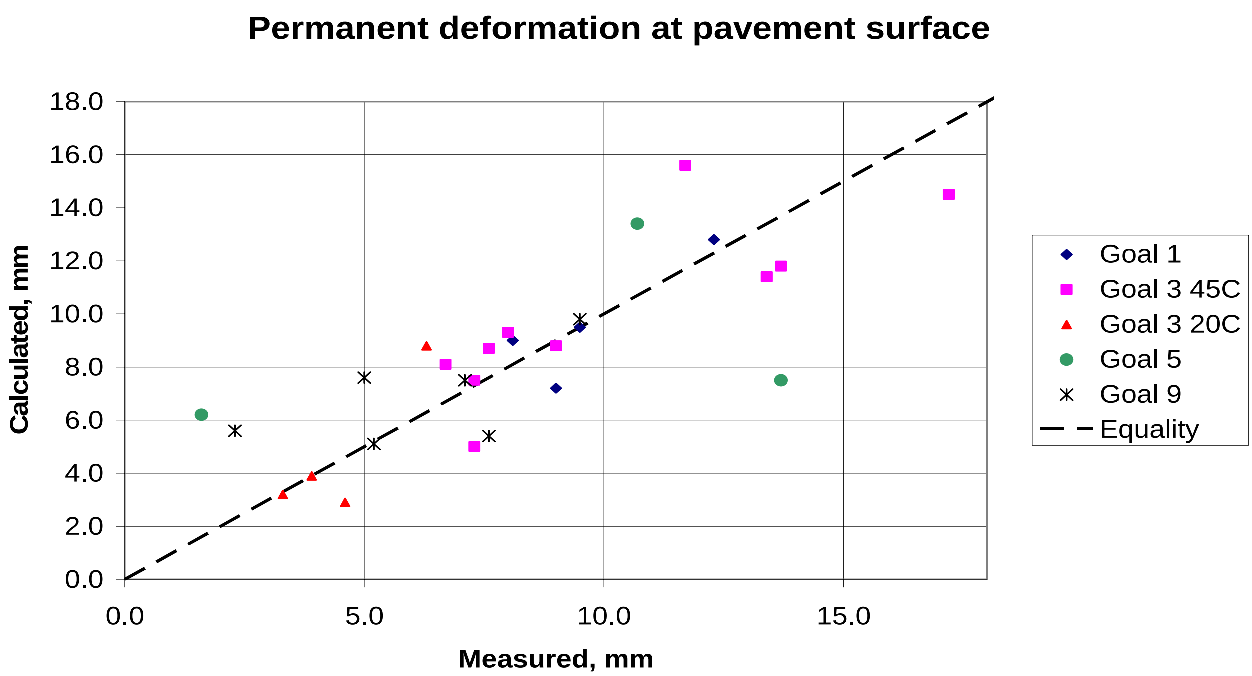

Figure 8. Final permanent deformation at the pavement surface.

The number of loads to crack initiation was calculated or estimated for 17 HVS sections where data was available. The damage calculated by CalME for the top AC layer, at this number of loads, is shown in Figure 9, as a function of the total AC thickness. The ATPB (Asphalt Treated Permeable Base) was included as an AC layer, where present. The signatures that are not filled in Figure 9 indicate drained sections with an ATPB layer.

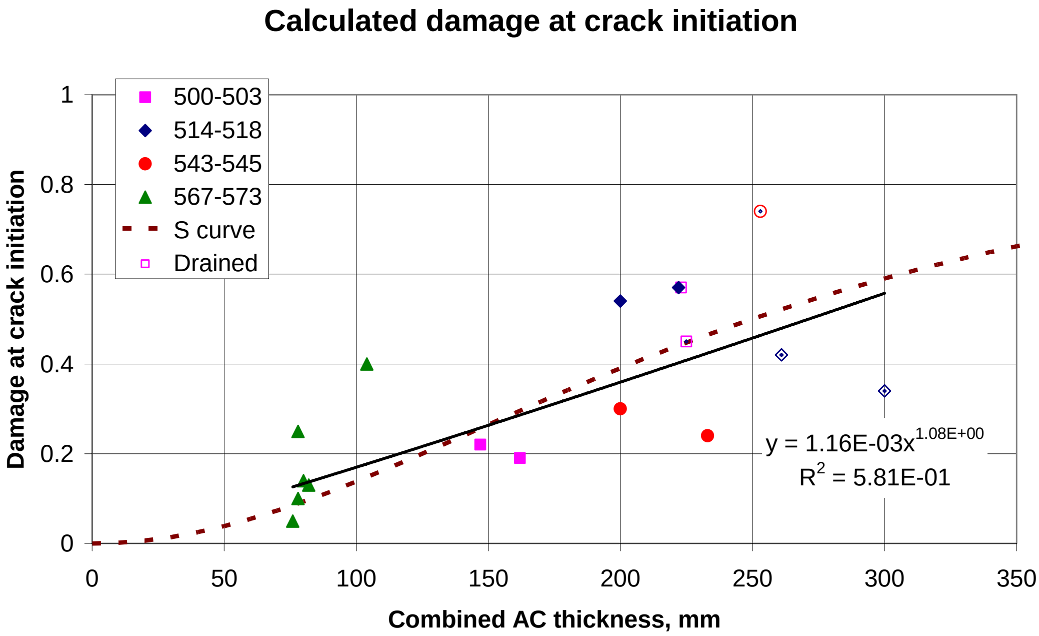

Figure 9. Calculated damage at crack initiation.

The regression equation in Figure 9 is the best fitting linear relationship but for practical purposes (for example to avoid damage larger than 1) the S-shaped (sigmoidal) curve may be preferable. This has the equation:

$$\omega_{initiation} = \frac{1}{1+\left(\dfrac{h_{AC}}{250\,mm}\right)^{-2}}$$

Equation 10. S-shaped curve for damage at crack initiation as a function of AC thickness

where $\omega_{initiation}$ is the damage corresponding to crack initiation, and
h_{AC} is the combined thickness of the asphalt layers.

Conclusions

The combination of models for:

1) modulus of asphalt materials as a function of reduced time,
2) moduli of unbound layers as a function of the stiffness of the layers above and as a function of the load level,
3) decrease of asphalt modulus caused by fatigue, and
4) the development of slip between some asphalt layers,

resulted in a relatively good prediction of the resilient deflections of the pavements, at all load levels and for the whole duration of the tests.

The resilient deflection increased considerably during almost all of the HVS tests. Most of this increase took place before the first visible crack occurred. The increase in visual cracking did not correlate very well with any further development in deflection, or with the calculated decrease in asphalt modulus.
Permanent deformation of the individual layers in the pavement structures was predicted reasonably well and so was the overall permanent deformation at the pavement surface, including predictions for asphalt-rubber overlays. For the granular layers, and particularly for the subgrade, the permanent deformations were very small, making calibration of the models uncertain.
Before the models can be applied to the design of new pavements and rehabilitation overlays a number of issues need to be addressed such as the influence of aging, seasonal variations, wheel speeds and rest periods, and variability of materials, structure, loads and climate, but the calibration using the HVS data reported in this paper is believed to provide a solid foundation for the ongoing calibration effort.

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