Calibration of Mechanistic-Empirical Models for Flexible Pavements Using the California Heavy Vehicle Simulators

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ABSTRACT

Calibration of Mechanistic-Empirical models for pavement design is a very complex process. The Heavy Vehicle Simulator (HVS) is ideal for the first step in this calibration process. The short test section can be carefully constructed with well characterized materials and instrumented to measure the pavement response. The climatic conditions may be controlled or monitored closely, all load applications are known exactly and, maybe most important of all, the pavement may be tested until it fails. This overcomes the problems of real pavements, which have uncertainties regarding materials, loads and climatic conditions and which are normally designed with a high reliability leading to very few failures.

The Mechanistic-Empirical models of an incremental-recursive computer program, known as *CalME*, have been initially calibrated using data from 27 flexible pavement test sections tested with the two HVSs owned by the California Department of Transportation (Caltrans). Most sections were instrumented with Multi Depth Deflectometers (MDDs) to compare the measured pavement deflections (at several depths) to the deflections predicted by the mechanistic model. Resilient deflections were compared for the complete time history of each test, and each test was carried to "failure" in terms of rutting (12.5 mm) and/or cracking (2 m/m^2). This involved the calibration of models for changes in layer moduli, including the effects of asphalt fatigue. The comparison of measured and predicted response is essential to ensure that the pavement response is predicted reasonably well by the mechanistic model. Once this was achieved, models for permanent deformation of the individual pavement layers were calibrated against the measured permanent deformation of the layers, again using the complete time history of each test.

INTRODUCTION

Both parts of a Mechanistic-Empirical method need to be validated and/or calibrated before the method can be used for pavement design and evaluation. Although the mechanistic part for calculating pavement response may be mathematically correct, all mechanistic pavement models are based on simplified assumptions (such as continuum mechanics and linear elasticity) and need to be validated and, possibly, calibrated. The empirical part relates the pavement performance (in terms of cracking, permanent deformation and roughness) to the pavement response (in terms of stresses and strains). This part must be calibrated to the loading, environment and other factors to which the materials are exposed in the pavement.

Collection of reliable response and performance data from in situ pavement sections can be very difficult, due to uncertainties and variations in materials, climate, loads etc and high cost. Accelerated pavement tests, for example using the Heavy Vehicle Simulator (HVS) (1), may provide a very valuable link between small scale laboratory tests, like triaxial tests or bending beam fatigue tests, and in situ pavement sections. With the relatively short test section under the HVS it is possible to obtained detailed and accurate information on the pavement materials. The climatic conditions can be monitored closely or may even be controlled, as was the case for the temperature of the pavements described in this paper, and for every applied load the position, speed, load level, etc. are known.

The HVS pavement section may also be instrumented in order to measure the actual pavement response. Multi Depth Deflectometers (MDD) (2) were installed for most of the test sections described in this paper. The MDDs measured resilient deflections under the moving wheel load at several depths in the pavement. The permanent (or plastic) deformations of individual layers were also measured. Both resilient and permanent deformation were measured at intervals across the duration of the test providing calibration data for the evolution of responses, not only the beginning and end states. This is particularly important for calibrating incremental-recursive models, such as those discussed in this paper, wherein materials properties are updated and responses are calculated using the updated properties at intervals during the performance simulations. Resilient deflections were also measured at the surface of the pavements with a Road Surface Deflectometer (RSD, similar to a Benkelman beam (2)) during most of the tests described in this paper.

Maybe the most important advantage of HVS testing is that it is possible to test the pavement until it fails. In situ pavements are designed with a high level of reliability, and very few sections will fail during their normal service life. If, for example, pavement sections are designed with a reliability of 95% only 5% of the sections will fail within the design period.

This paper describes how 27 flexible pavement sections, loaded to failure with the HVS, were used to calibrate the damage models in the incremental-recursive procedure in *CalME*. *CalME* is a (draft) Mechanistic-Empirical design program for new flexible pavements and rehabilitation in California.

For rehabilitation design, deflections and backcalculated layer moduli from Falling Weight Deflectometer (FWD) testing, determined using a companion program called *CalBack,* may be imported to the *CalME* database. *CalME* has three levels of design, for new pavements as well as for rehabilitation:

1. Caltrans current empirical methods, the “R-value” method for flexible structures and the “Deflection Reduction” method for overlay 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 “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. This approach predicts the pavement conditions at any point in time during the pavement life and was used for the simulations included in this paper.

The resilient deflection was found to increase by a factor of 2.4 on average during HVS testing,. This means that the response, in terms of stresses and strains, also changes dramatically during the test. It is crucial for mechanistic-empirical calibration that the response is predicted correctly throughout the test. Calculation of the response, using the mechanistic part of the method, will depend on:

- the moduli of all of the pavement layers, and changes to these moduli caused by fatigue damage,
- slip between asphalt layers,
- non-linear elastic characteristics of the unbound layers and the effect of confinement on unbound layers.

The permanent deformation models may be calibrated with some confidence once a reasonably good agreement is achieved between the measured and the calculated resilient deflections.

The paper first describes the HVS tests and the models used in CalME. This is followed by a detailed example of a simulation of an HVS test using CalME. Finally a summary of all of the calibrations is given followed by conclusions and recommendations.

The full report of the calibrations described in this paper may be downloaded at (3).

HEAVY VEHICLE SIMULATOR TESTS

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.

MODELS EVALUATED IN *CalME*

Modulus of Asphalt Layers (Master Curve)

The model for intact asphalt concrete modulus versus reduced time was the NCHRP 1-37A model (4, MEPDG):

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

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:

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

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

where lt is the loading time in seconds,
$visc$ is the viscosity,
$visc_{ref}$ is the viscosity at a reference temperature, and
aT is a constant.

The viscosity, in cPoise, is found from:

$$\log(\log(visc\, cPoise)) = A + VTS * \log(t_K)$$

Equation 3: Viscosity as a function of temperature.

where t_K is the temperature in ºKelvin and
A and VTS are constants.

The master curves for the different asphalt materials were determined in the laboratory from frequency sweep tests on beams. The term δ in Equation 1 was determined based on back-calculation from FWD tests rather than flexural frequency sweep results because the laboratory stiffnesses at high temperatures and long loading times were unreasonably low.

Moduli of Unbound Layers

During these and other experiments (5) it has been found that the moduli of unbound materials sometimes vary with the stiffness of the layers above them. This may happen either as a result of change in stiffness of asphalt layers due to change in temperature or caused by increasing damage to the asphalt layers. For granular layers this effect is the opposite of what would be expected based on understanding of the non-linearity of granular material under triaxial testing in the laboratory. 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 in (6) 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. An explanation for this could be the confining effect of the layers above the granular material. This is illustrated in (3) through a calculation with the Distinct Element Method.

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 4: Modulus of each unbound layer as a function of the bending stiffness of the layer above it.

where Eo is the modulus (of layer n) at the reference stiffness,
S is the combined 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.

The stiffness of the unbound layers was simultaneously modeled as a function of the load level using the well known models for non-linearity, with the modulus of granular materials increasing with increasing bulk stress and the modulus of the cohesive subgrade decreasing with increasing deviator stress. However, load level had to be used instead of stress because of the effects of confinement.

The initial moduli of the unbound materials were determined partly from FWD tests and partly from the initial resilient deflections of the MDDs and RSD.

Response Model

The response model used was the layered elastic program *LEAP* (7). Partial bonding between asphalt layers, which occurred during some of the tests, was simulated where appropriate for fatigue analysis. Full bonding was assumed for calculating the shear stress and shear strain at a depth of 50 mm for determination of the permanent shear strain in the asphalt layer for the rutting analysis. This only provides an approximate simulation of the effects of lack of bonding. A more correct simulation requires a 3D Finite Element program.

All materials were assumed to have a Poisson's ratio of 0.35.

Reflection cracking damage was calculated using the method developed by Wu (8). In this method the tensile strain at the bottom of the overlay is estimated using a regression equation. The calculated tensile strain at the bottom of the overlay is used with the fatigue equation described in the next section to calculate damage in the asphalt layers.

The regression equation for tensile strain at the bottom of the overlay is based on many finite element calculations, and assumes a dual wheel on a single axle. The strain, $\mu\varepsilon$ in microstrain, is calculated from:

$$\mu\varepsilon = \alpha \times E_{an}^{\beta 1} \times E_{bn}^{\beta 2} \times (a1 + b1 \times \ln(LS_n)) \times \exp(b2 \times H_{an}) \times (1 + b3 \times H_{un}) \times (1 + b4 \times E_{un}) \times \sigma_n$$

$$E_{an} = {E_a}/{E_s}, E_{bn} = {E_b}/{E_s}, E_{un} = {E_u}/{E_s}, LS_n = {LS}/{a}, H_{an} = {H_a}/{a}, H_{un} = {H_u}/{a}, \sigma_n = {\sigma_o}/{E_s}$$

Equation 5 Strain, in microstrain, over existing crack

where E_a is the modulus of the overlay,
H_a is the thickness of the overlay,
E_u is the modulus of the underlayer,
H_u is the thickness of the underlayer,
E_b is the modulus of the base/sub-base,
E_s is the modulus of the subgrade,
LS is the crack spacing,
σ_o is the tire pressure, and
a is the radius of the loaded area for one wheel.

The following constants were used:

α = 342650, *β1* = -0.73722, *β2* = -0.2645, *a1* = 0.88432, *b1* = 0.15272, *b2* = -0.21632, *b3* = -0.061, and *b4* = 0.018752

Damage to Asphalt Layers

The model for damaged asphalt concrete modulus was:

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

Equation 6: Modulus of damaged asphalt concrete (variables same as in Equation 1).

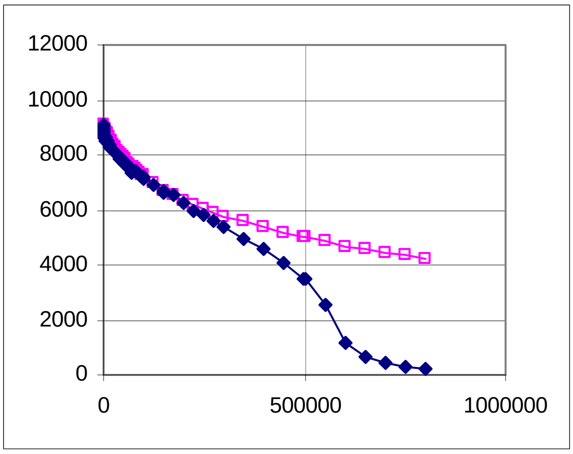
where the damage, ω, was calculated from:

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

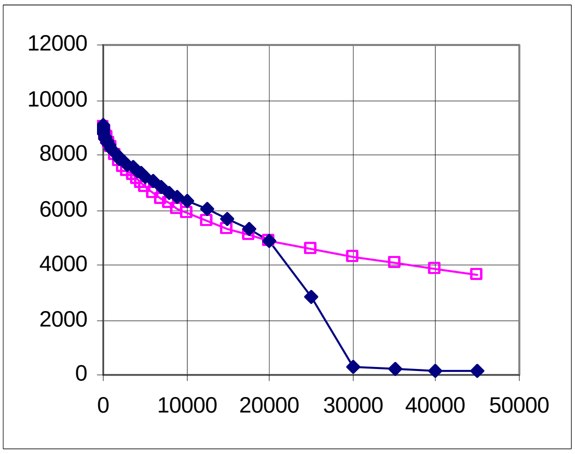
Equation 7: Damage as a function of loads, strain, modulus and temperature.

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,
200 $\mu strain$ and 3000 MPa are reference constants, and
A, α, β, γ, and δ are constants (not related to the constants of Equation 6).

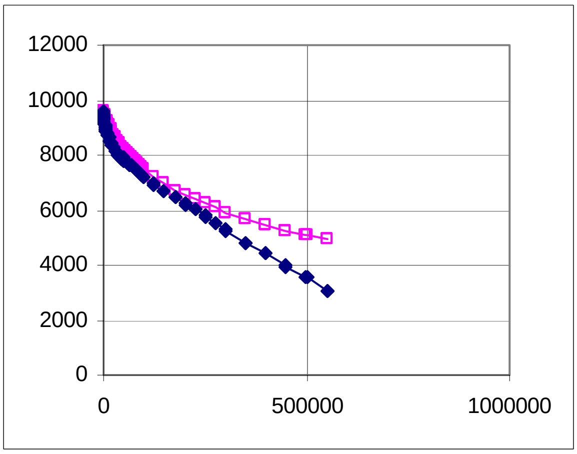
The constant γ in Equation 7 was assumed equal to $\beta/2$, making damage a function of the strain energy. The parameters of Equation 7 were determined from four point beam, controlled strain, fatigue testing, by minimizing the Root Mean Square (RMS) of the difference between the measured modulus and the modulus calculated from Equation 6, ignoring moduli below 30% of the intact value as the laboratory tests at this amount of damage are unlikely to be representative of the in situ conditions of an asphalt layer. The minimization was done in Excel using Solver. An example from the bottom AC layer of Goal1 is shown in FIGURE 1. The ordinate is the modulus in MPa and the abscissa is the number of load applications. The measured moduli are shown as filled diamonds (blue) and the values determined from Equation 6 are shown as open squares (magenta). The average strain during the fatigue test is given below each graph. The example only shows the fatigue tests done at 20 ºC, but the calculated moduli are based on all of the fatigue tests done at temperatures between 5 ºC and 30 ºC.



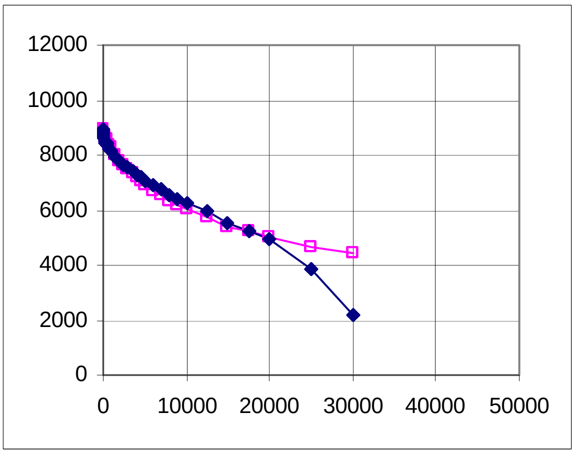
Strain 203 μm/m



Strain 402 μm/m



Strain 202 μm/m



Strain 400 μm/m

FIGURE 1 Example of laboratory fatigue tests from bottom layer of Goal1.

In using Equation 7 for HVS test simulation the number of load applications was divided by a shift factor (= 3, i.e. the damage caused by a laboratory fatigue load is three times that caused by the HVS load, at the same strain, modulus and temperature). This shift factor was used for original, intact asphalt, for all but one test. For reflection cracking tests, where the original asphalt layers had been previously loaded to cracking, but where the moduli had recovered to some extent, a shift factor of 0.6 was used for re-cracking of the materials.

Permanent Deformation of Asphalt and Unbound Layers

A shear-based approach, developed by Deacon et al. (9), for predicting rutting of the asphalt layer was used. Rutting in the asphalt is assumed to be controlled by shear deformation. The permanent, or inelastic, shear strain, γ^i, is determined as a function of the shear stress, τ, the elastic shear strain, γ^e, and the number of load repetitions, 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 8 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.

A constant value of 1.03 for β, determined from previous work (9), was used for all materials. The rest of the parameters were determined by minimizing the RMS of the difference between the measured permanent shear strain and the strain calculated from Equation 8.

The rut depth is calculated for the upper 100 mm of the asphalt layers. The shear stress is calculated at a depth of 50 mm beneath the edge of the tire. For each of the layers within 100 mm of the surface the elastic shear strain, γ^e, is calculated from:

$$\gamma^e = \frac{\tau}{2 \times G_i} = \frac{\tau}{E_i/(1+\nu_i)}$$

Equation 9 Calculation of elastic shear strain in top layers.

where E_i is the modulus of layer i, and
ν_i is Poisson's ratio for layer i.

The permanent shear strain of each layer is calculated from Equation 8, and the permanent deformation is determined from:

$$dp_i = K \times h_i \times \gamma^i$$

Equation 10 Relationship between permanent deformation and permanent shear strain of layer *i*.

where h_i is the thickness of layer *i* (above a depth of 100 mm), and
K is a calibration constant.

The total permanent deformation in the asphalt layers is the sum of the permanent deformations of the layers within the top 100 mm of the pavement. A value of $K = 0.08$ was used for all simulations and all materials, with a few exceptions.

The model for permanent deformation of the unbound layers, dp, is given in Equation 11, 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 1000 $\mu strain$ and 40 MPa. The relation ship was derived from tests in the Danish Road Testing Machine during the International Pavement Subgrade Performance Study (10):

$$dp\ mm = A \times MN^{\alpha} \times \left(\frac{\mu\varepsilon}{1000\ \mu strain} \right)^{\beta} \times \left(\frac{E}{40\ MPa} \right)^{\gamma}$$

Equation 11: Permanent deformation of unbound layers.

Time Hardening Procedure

The models described above are used in an incremental-recursive process. This means that the parameters on the right side of the equal-sign may change from increment to increment of load applications. Therefore, the first step in the process is to calculate the "effective" number of load applications that would have been required, with the present parameter values, to produce the condition at the beginning of the increment. This sometimes requires an iterative procedure. In the second step, the new condition at the end of the increment is calculated for the "effective" number of load applications plus the number of applications during the increment. This must be repeated for each load and load position during the increment.

The method may be illustrated by an example using Equation 11. If, for example, the permanent deformation of the subgrade was 2 mm at the start of an increment, the vertical strain calculated for the first wheel load at the first position was 800 µstrain and the modulus 60 MPa. Then the effective number of load applications at the start of the increment may be found from:

$$MN_{eff} = \left[\frac{2}{1.1 \times \left(800/1000 \right)^{1.333} \times \left(60/40 \right)^{0.333}} \right]^{1/0.333}$$

If the number of repetitions, in millions, of this load, at this position, was *dMN* during the increment, then the permanent deformation after these load applications would be:

$$dp, mm = 1.1 \times \left(MN_{eff} + dMN \right)^{0.333} \times \left(800/1000 \right)^{1.333} \times \left(60/40 \right)^{0.333}$$

The process must be repeated recursively, using the output from each calculation as input to the next, for all loads at each position, before proceeding to the next time increment.

EXAMPLE OF SIMULATION

For each of the HVS experiments the results, comprising temperatures at several depths, loads, and measured resilient and permanent deformations, were imported to the *CalME* database. Each test section was then simulated, hour by hour, for the whole duration of the test, using the recorded temperatures and load applications for each hour. Five wheel positions were considered for tests with wheel wander. The example presented here is from Goal1 (HVS test numbered 503RF). The parameters used for the models given above are shown in TABLE 1.

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 aggregated 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 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).

TABLE 1: Material parameters for HVS test 503RF, Goal 1.

Modulus	*A*	*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*	*B*	*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			

Some of the resilient deflections measured under a 40 kN wheel load are shown in FIGURE 2. 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), at the top of the AB (legend 137) and at the top of the subgrade (legend 640). 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.

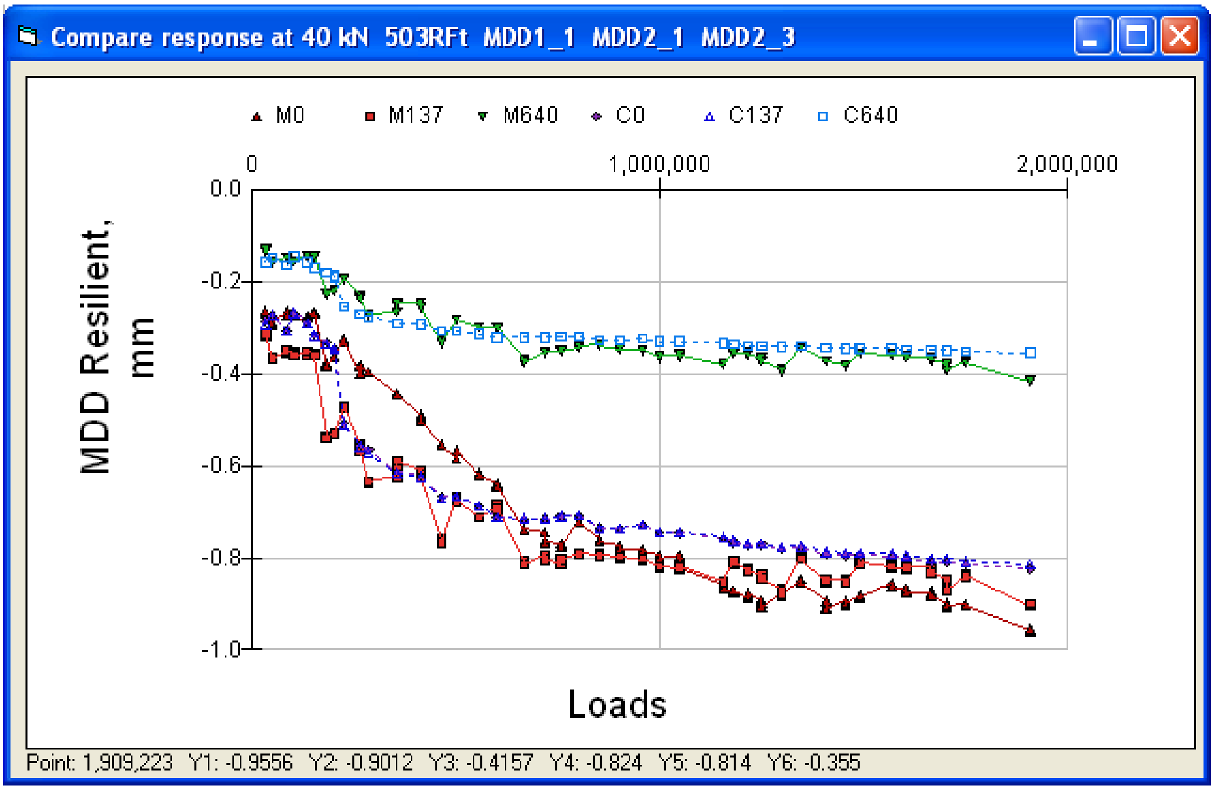


FIGURE 2: Resilient deflections section 503RF, 40 kN.

The effects of asphalt fatigue on the moduli of the asphalt layers are included in the validation of the response model. The response validation, therefore, includes a calibration of the empirical damage function, Equation 7, in terms of the shift factor (3 for these tests).

Once the response model results in a satisfactory prediction of the measured resilient deflections, then the empirical relationships for permanent deformation may be calibrated. For permanent deformation of the AC layers a constant value is determined between the permanent deformation and the permanent shear strain, as calculated by Equation 8, and for the unbound layers the constant *A* of Equation 11 is determined.

Measured and calculated permanent deformation of the AC layers is shown in FIGURE 3. Permanent deformation was calculated for the top 100 mm of the AC only, as explained above.

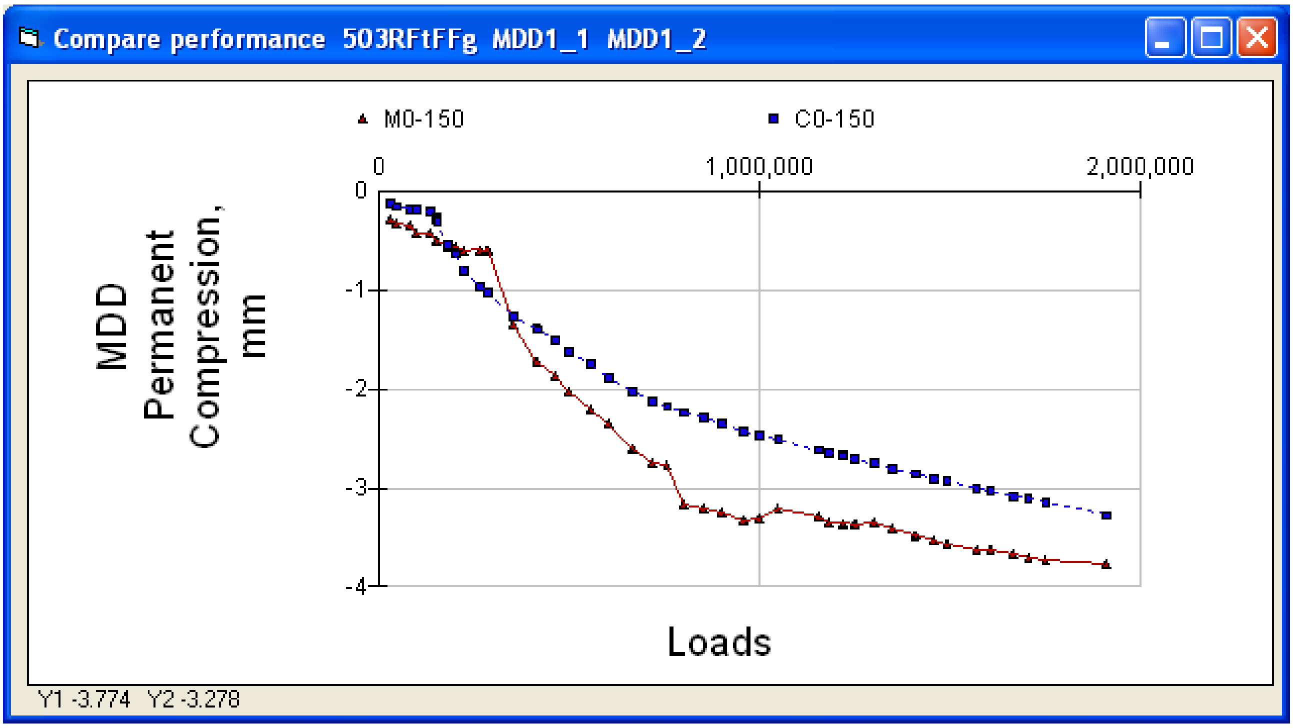


FIGURE 3: Permanent deformation of AC layers 503RF.

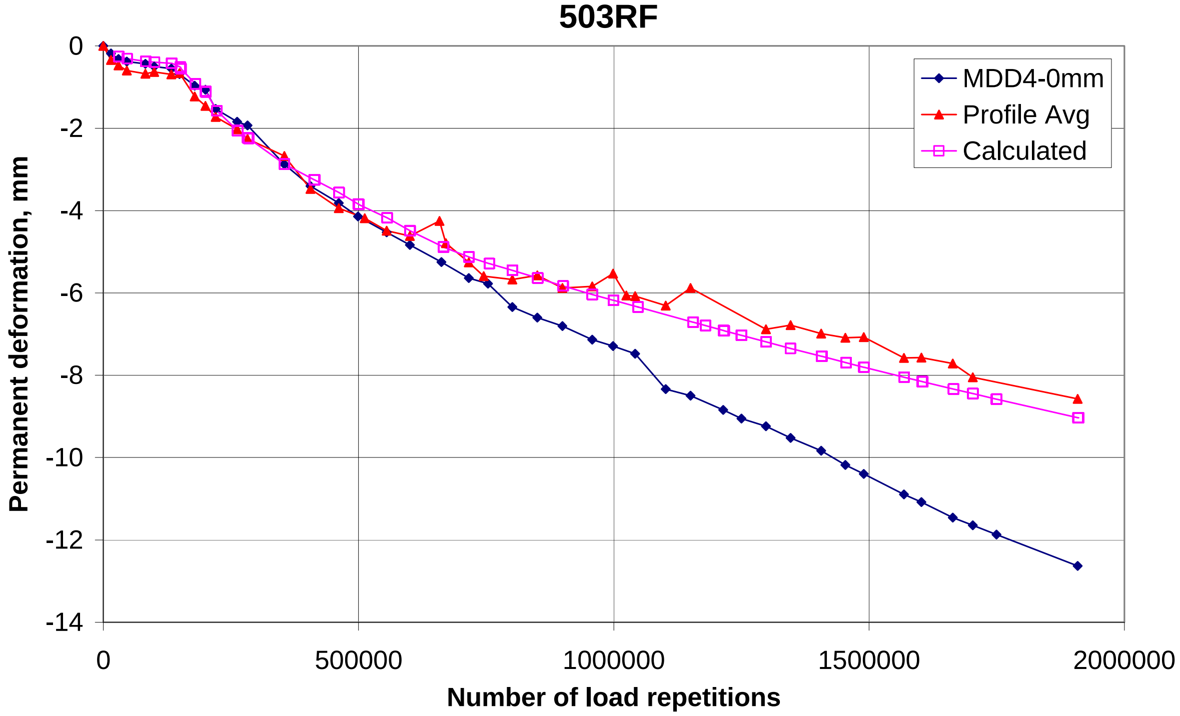


FIGURE 4: Permanent deformation at the pavement surface, 503RF.

The total permanent deformation at the pavement surface is shown in FIGURE 4, 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*.

SUMMARY OF RESULTS FOR ALL CALIBRATION SECTIONS

FIGURE 5 compares the measured and calculated ratios of final to initial deflection under a 40 kN wheel load for all of the HVS cracking tests included in this calibration project. 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.

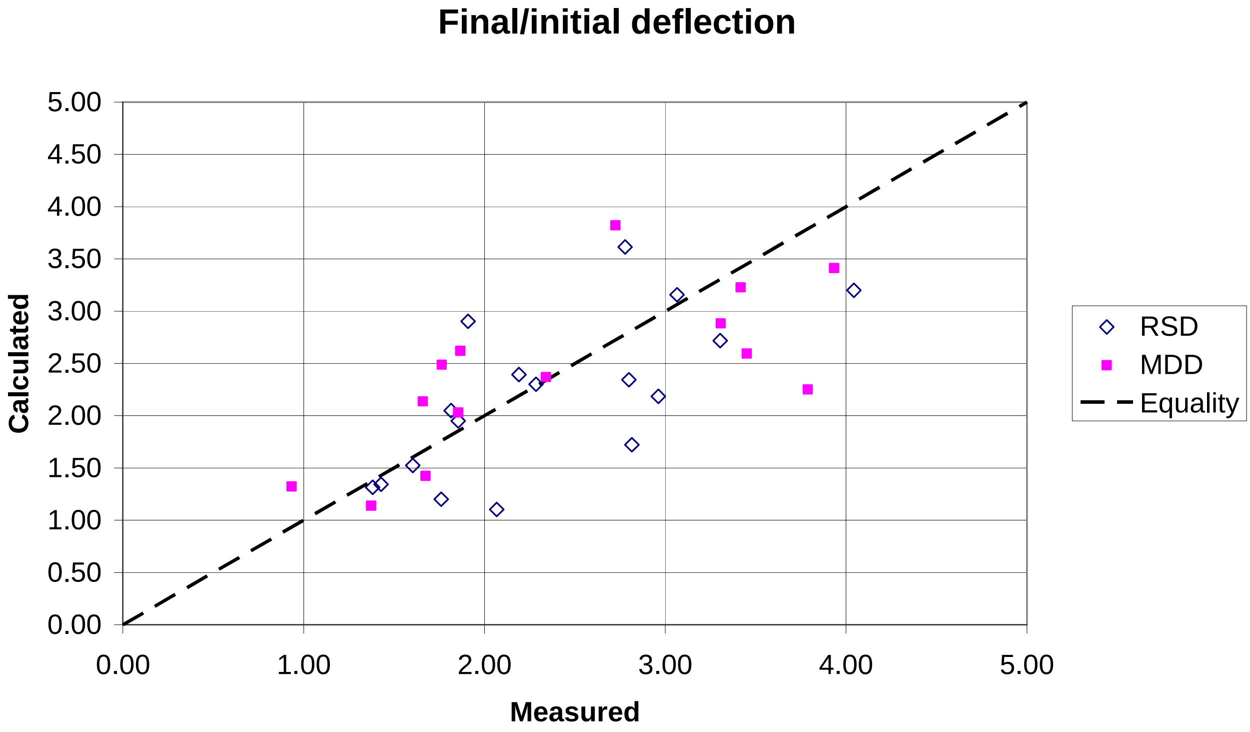


FIGURE 5: Ratio of final to initial surface deflection for HVS cracking tests.

The measured and calculated final permanent deformations of the AC layers are shown in FIGURE 6 for all of the test sections where it was recorded. The standard error of estimate for the permanent deformation of the AC 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.

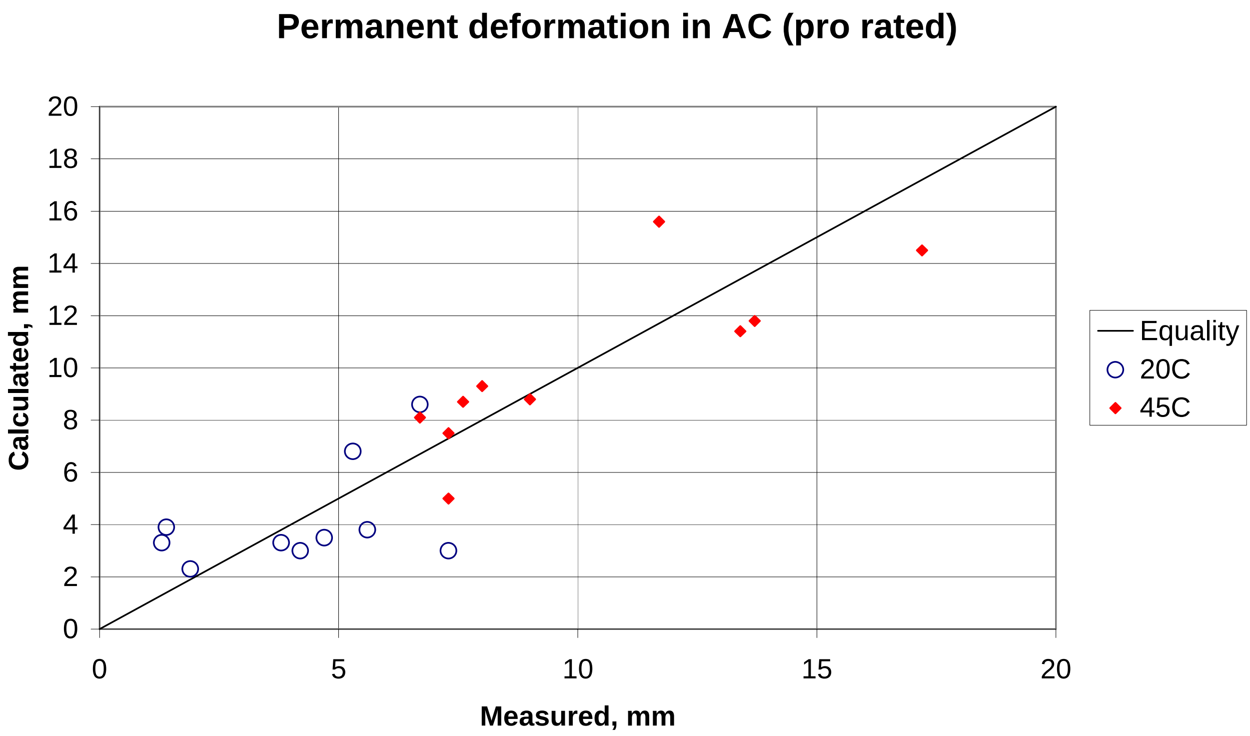


FIGURE 6: Final permanent deformation of AC layers.

The total permanent deformation at the pavement surface is shown in FIGURE 7. 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).

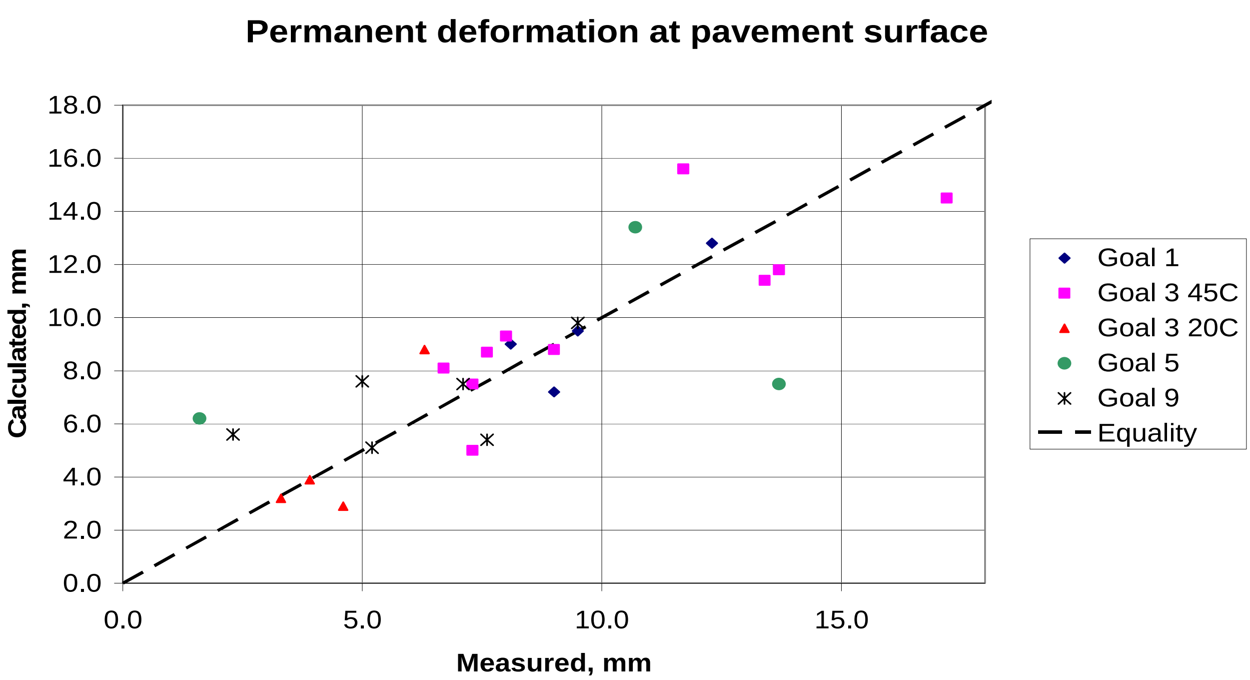


FIGURE 7: Final permanent deformation at the pavement surface.

No simple relationship could be established between the increase in deflection or the decrease in asphalt modulus and the development of observed cracking at the surface (FIGURE 8).

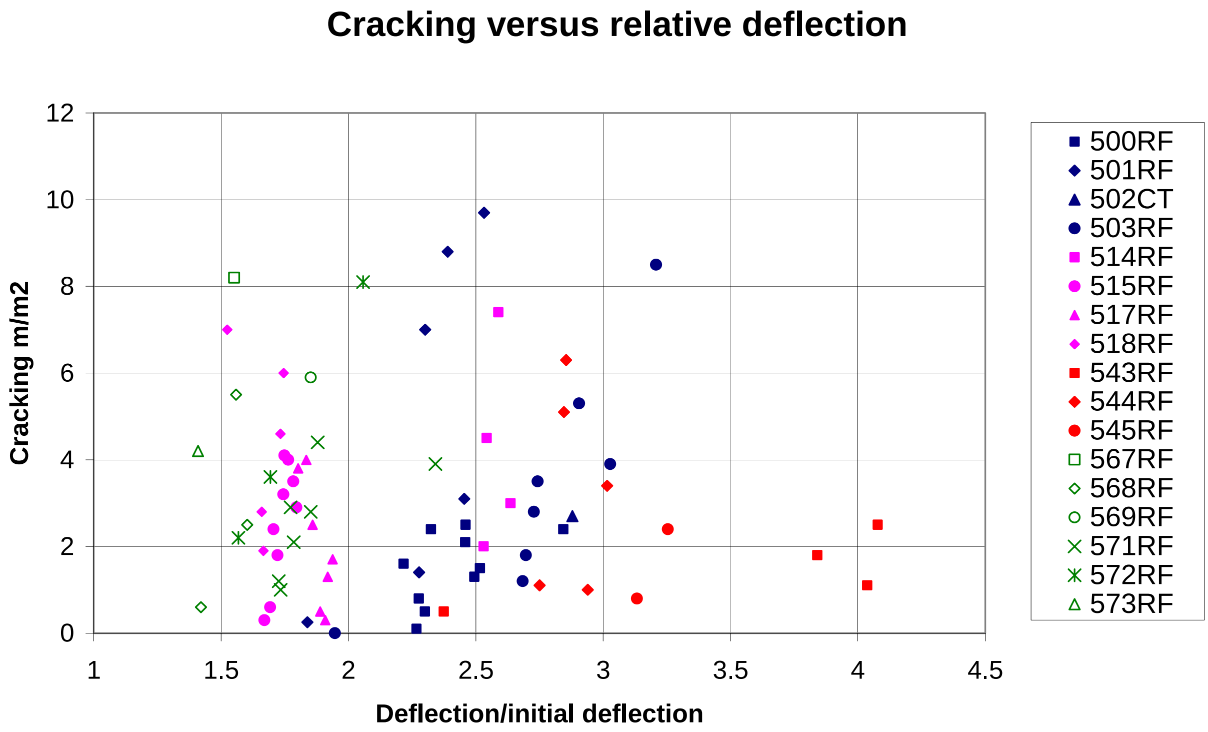


FIGURE 8: Visual cracking versus increase in deflection (tests grouped by Goal for each color).

CONCLUSIONS AND RECOMMENDATIONS

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,
4) strain in overlay caused by existing cracks, and
5) the development of slip between some asphalt layers,

resulted in a relatively good prediction of the resilient deflections at all levels of the pavements 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:

1. The HVS tests are accelerated and the effect of aging is slight. Aging may affect both the stiffness and the fatigue characteristics of the materials. An aging model will be included for field section and test track calibrations.
2. The effects of seasonal variations on the unbound materials need to be established from seasonal monitoring sites.
3. 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.
4. The effects of variability of materials, structure, loads and climate, and of the uncertainty on the models must be established.

The next stage of this research after this validation of the mechanistic models is collection of similar data from available field test sections to develop more realistic empirical shift factors. Presently (July 2007) reports on calibrations of *CalME* models using the WesTrack experiment's 26 original sections and additional HVS tests are being prepared.

The calibration using the HVS data reported in this paper is believed to provide a very solid foundation for the ongoing calibration effort.

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