Evaluation of reflective cracking performance of AC mixes with asphalt rubber binder using HVS tests and recursive mechanistic-empirical analysis

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ABSTRACT: A major research project has been conducted at the University of California Pavement Research Center (UCPRC) to evaluate the performances of several AC overlays that contained asphalt rubber binder. This paper presents the analysis with respect to reflection cracking using a recursive mechanistic-empirical analysis procedure, known as *CalME*. For reflection cracking a simple model for calculating the strain in the overlay over an existing crack is presented. The fatigue properties required by the procedure for different AC mixes were determined from laboratory test data. The analysis procedure was validated by simulating full-scale pavement testing using the Heavy Vehicle Simulator (HVS). The results of the HVS tests were imported to the *CalME* database and the simulations were done in increments of one hour. Most HVS tests show a considerable increase in deflections during the test, due to damage of the pavement layers. This implies that other response parameters, such as stresses and strains, also change considerably during the test. As these response parameters are used in the empirical relationships to predict pavement performance, it is very important that correct values are used at any point in time during the simulation. Care was taken to ensure that the simulations with *CalME* would produce response values (deflections in this case) that were reasonably close to the measured responses. The predicted changes in the moduli of the overlays and the relationship between predicted damage and observed cracking indicate that recursive mechanistic-empirical approach with the reflection cracking model worked satisfactory.

1 INTRODUCTION

The goal of this study was to evaluate the reflection cracking performance of asphalt mixes used in overlays for rehabilitating cracked asphalt concrete pavement in California. The main objective was to compare the performance of three overlays with mixes containing binders using Caltrans' MB specification (binders including recycled tire rubber and polymers blended at the refinery) against two control overlay mixes [dense-graded asphalt concrete (DGAC) and gap-graded rubberized asphalt concrete (RAC-G)]. These control overlays represent typical pavement structures currently used throughout California.

The project was divided into two phases. In the first phase, the uniform test pavement, which consisted of six test sections, was trafficked with the HVS to induce fatigue cracking on the asphalt concrete layer. The original pavement consisted of 77 to 88 mm of DGAC on a design thickness of 410 mm of aggregate base (AB) on a clay subgrade. The AB consisted of 100% recycled building waste material with a high percentage of crushed concrete. Reactive cement was found in the AB. In the second phase, selected overlay mixes were placed to evaluate:

- Reflection cracking (expected failure mode) under HVS trafficking at moderate temperatures, and
- Rutting performance under HVS loading at high temperature.

Only the reflection cracking tests are included in this paper. They were performed by applying HVS trafficking directly over the previously cracked Phase I test sections. A laboratory study, primarily investigating the shear and fatigue properties of the mixes, was undertaken in parallel with the HVS study.

The six reflection cracking test sections, constructed as part of the second phase of the study, were as follows:

1. Half-thickness (45 mm) MB4 gap-graded overlay with minimum 15 percent recycled tire rubber (referred to as “MB15” in this paper)
2. Half-thickness rubberized asphalt concrete gap-graded (RAC-G) overlay
3. Full-thickness (90 mm) DGAC overlay (split into two subsections in the analysis)
4. Half-thickness MB4 gap-graded overlay
5. Full-thickness MB4 gap-graded overlay
6. Half-thickness MAC15TR gap-graded overlay with minimum 15 percent recycled tire rubber.

The test sections were instrumented with Multi Depth Deflectometers (MDDs) and thermocouples. At regular intervals during the HVS tests the resilient deflections were recorded at several depths using the MDDs and at the pavement surface using a Road Surface Deflectometer (RSD, similar to a Benkelman beam). The permanent deformations were also recorded by the MDDs and the pavement profile was measured using a laser profilometer. Any distress at the surface of the pavement was recorded. During HVS testing the temperature was controlled using a climate chamber. Falling Weight Deflectometer (FWD) tests were carried out before and after the HVS tests. Details on the HVS and the instrumentation can be found in Harvey et al., 1998 and on the overall study in Jones et al, 2007.

2 SIMULATION OF HVS TESTS USING *CalME*

The HVS tests were simulated using an incremental-recursive program known as *CalME* (Ullidtz et al., 2007). Data from each HVS test were imported into a *CalME* database. The data comprised information on loads (time of application and load level), temperatures at different levels, RSD results, MDD resilient and permanent deformations and pavement profiles.

The backcalculated layer moduli from the last FWD test before commencement of the HVS loading were used as the initial layer moduli (for asphalt layers at the reference temperature of 20 ºC). Layer moduli were backcalculated using *CalBack*. For asphalt layers the master curve was obtained from frequency sweep tests on beams in the laboratory, with the exception of the original DGAC layer where the master curve was based on FWD backcalculated moduli. For the subgrade the change in stiffness with changing stiffness of the pavement layers and with changing load level was obtained from FWD backcalculated values. These parameters were used with the response model (*LEAP*, Symplectic Engineering Corporation, 2004) to calculate stresses, strains and deflections in the pavement structure. The strain in the overlay over an existing cracked asphalt layer was calculated using the reflection cracking model described below.

To predict the pavement performance, in terms of cracking and permanent deformation, a number of models were used. Parameters for prediction of asphalt damage were obtained from controlled strain fatigue tests on beams. Repeated Simple Shear Tests at Constant Height (RSST-CH) were used to determine the parameters for predicting permanent deformation in the asphalt layers. A crushing model was developed for the self-cementing base layer, consisting of recycled building waste material with a high content of crushed concrete. Cracking at the pavement surface was calculated from the reflection damage to the surface layer, using a model developed based on previous simulations of HVS tests and the WesTrack experiment, with coefficients modified based on the results of the present experiment.

An incremental-recursive process was used to simulate the performance of the test sections. The time increment used was one hour. For the first hour of the simulation the program would read the temperatures from the database and calculate the moduli, for a constant wheel speed of 9.6 km/h, the approximate speed of the HVS wheel. The number of loads during the first hour, as well as the load level and the tire pressure, were also read from the database. The modulus of the subgrade would be adjusted to the stiffness of the pavement layers and to the load level. If the test had wheel wander, five different positions of the wheel would be considered. For the first wheel position the stresses and strains at the center line of the test section were calculated and used to determine the decrease in moduli and the increase in permanent deformation of each of the pavement layers. The output from these calculations were used, recursively, as input to the calculation for the next wheel position. Because of the changes to moduli, response, damage, and permanent deformation the “time hardening” procedure was used (Deacon et al. 2002).

The first step in the simulation is to make sure that the calculated pavement response is reasonably close to the actual pavement response during the test. The calculated pavement response is used to predict the pavement performance (damage and permanent deformation). Therefore, if the calculated response is not reasonably correct it would be futile to try to use it for calibration of the performance models. For the HVS tests used for this paper, response measurements were available in the form of resilient MDD deflections and/or RSD deflections.

Once the resilient deflections are predicted reasonably well during the simulations, it is possible to calibrate the performance models so that the permanent deformation of each layer, the decrease in layer moduli and the observed surface cracking, are reasonably well predicted.

3 REFLECTION CRACKING MODEL

Reflection cracking damage was calculated using the method developed by Wu (2005). 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 2D and 3D finite element calculations, and assumes a dual wheel on a single axle:

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

$$E_{an} = {E_a}/{E_s}, E_{bn} = {E_b}/{E_s}, E_{un} = {E_u}/{E_s}, \sigma_n = {\sigma_o}/{E_s},$$

$$LS_n = {LS}/{a}, H_{an} = {H_a}/{a}, H_{un} = {H_u}/{a}$$

Equation 1 Strain, in μstrain, 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, $\beta 1$ = -0.73722, $\beta 2$ = -0.2645, $\beta 3$ = -1.16472, $a1$ = 0.88432, $b1$ = 0.15272, $b2$ = -0.21632, $b3$ = -0.061, $b4$ = 0.018752.

To predict reflection cracking, the resulting strain was used with the model for the master curve of the damaged asphalt, which has the format:

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

Equation 2 Modulus of damaged asphalt.

where δ, α, β, and γ are constants, tr is reduced time in sec and the damage, ω, is calculated from:

$$\omega = \left(\frac{MN}{MNp} \right)^{\alpha}$$

$$MNp = A \times \left(\frac{\mu\varepsilon}{\mu\varepsilon_{ref}} \right)^{\beta} \times \left(\frac{E}{E_{ref}} \right)^{\beta/2} = A' \times \left(\frac{SE}{SE_{ref}} \right)^{\beta/2}$$

Equation 3 Damage as a function of number of loads, strain, and modulus.

where E is the modulus of damaged material,
E_i is the modulus of intact material,

MN is the number of load repetitions in millions ($N/10^6$),
$\mu\varepsilon$ is the strain at the bottom of the asphalt layer in µstrain,
SE is the strain energy, and
A, A', α, β, $\mu\varepsilon_{ref}$, E_{ref}, and SE_{ref} are constants

The initial (intact) modulus, E_i, corresponds to a damage, ω, of 0 and the minimum modulus, $E_{min}=10^{\delta}$, to a damage of 1.

From previous calibration studies on cracking of new pavements it has been found that the damage at crack initiation may be determined from:

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

Equation 4 Damage at crack initiation.

where h_{AC} is the combined thickness of the asphalt layers.

From a calibration study using WesTrack data it was found that the propagation of cracking could be approximated by:

$$Cr\% = \frac{100\%}{1+\left(\dfrac{\omega}{\omega_o}\right)^{-8}}$$

Equation 5 Cracking in percent as a function of damage.

where *Cr%* is the cracking in percent of wheelpaths,
ω is the calculated damage, and
ω_o, is a constant determined by assuming 5% cracking at crack initiation.

During HVS testing cracking was measured in m/m^2, so the equation was changed to:

$$Cr\ m/m^2 = \frac{A}{1+\left(\dfrac{\omega}{\omega_o}\right)^{\alpha}}$$

Equation 6 Cracking in m/m^2 as a function of damage.

The maximum recorded cracking during the HVS experiments was about 8-9 m/m^2. It is reasonable to assume that about 10 m/m^2 would correspond to 100% cracking.

4 SIMULATION OF PAVEMENT RESPONSE

As mentioned previously, the deflections normally increase considerable during an HVS test, as a result of damage to the bound layers (asphalt and self-cementing AB in this case). This means that the stresses and strains in the pavement layers, which are used in calculation of the pavement performance, also change during the test. To ensure that the pavement response calculated by *CalME* was reasonably correct for the duration of the test, the surface deflections and the deflections at the depths of the MDD modules were calculated by *CalME* and compared to the RSD and MDD measurements.

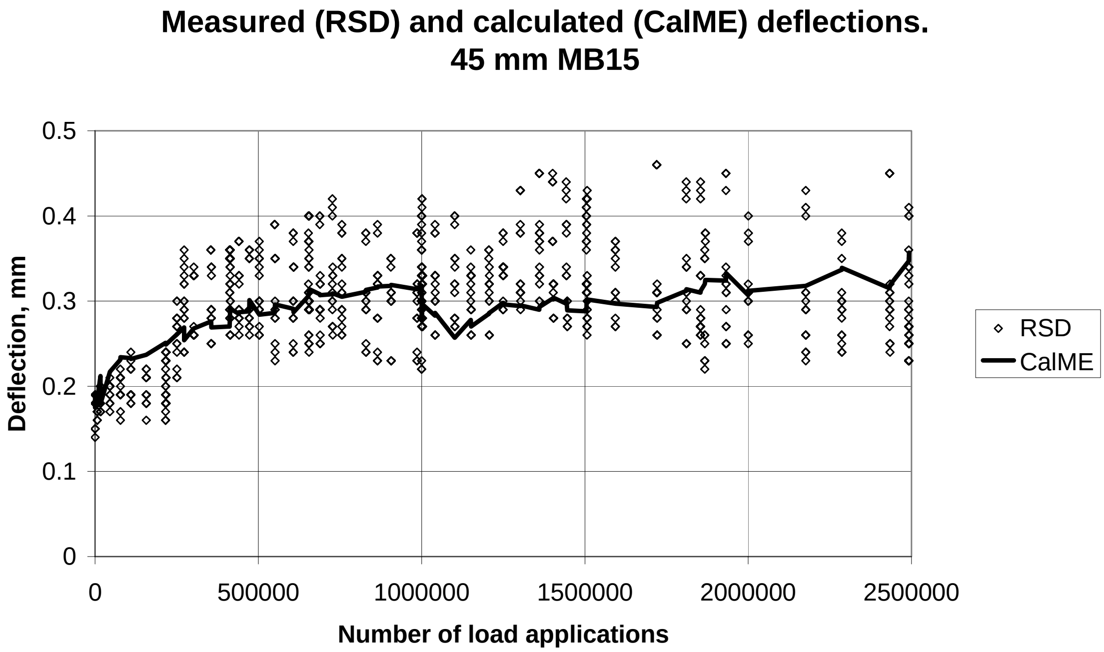

Figure 1 Example of deflections calculated by *CalME* compared to measured surface deflections.

Figure 1 shows a comparison for the test section with a 45 mm MB15 overlay. Even though the test section is only 6 m long the surface deflections vary considerably over the area of the test section. The coefficient of variation on the RSD measurements varies from less than 10% to more than 20%. It may be noticed that the deflection increases by more than 50% within the first one million load applications. The drop in deflection after one million load applications is due to the temperature being reduced from 20 ºC to 15 ºC.

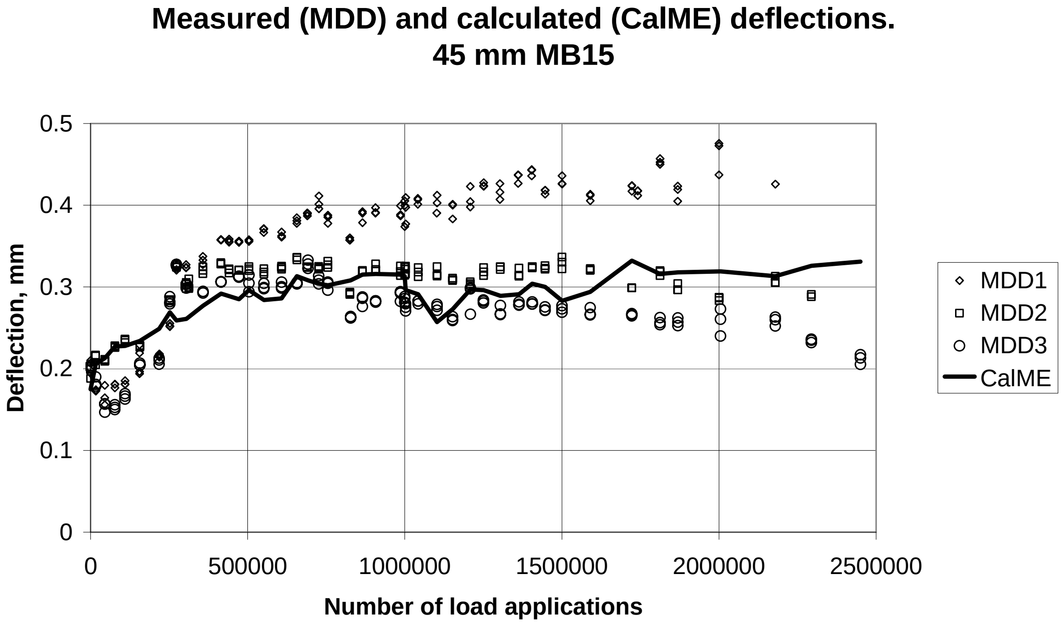

Figure 2 Example of deflections calculated by *CalME* compared to measured deflections at top of base.

The three MDDs shown in Figure 2 measured the deflection at (approximately) the top of the aggregate base. They also indicate a considerable variation within the test section, and show the same trend as the RSD deflections. The deflections calculated by *CalME* are seen to be in good agreement with the measured deflections.

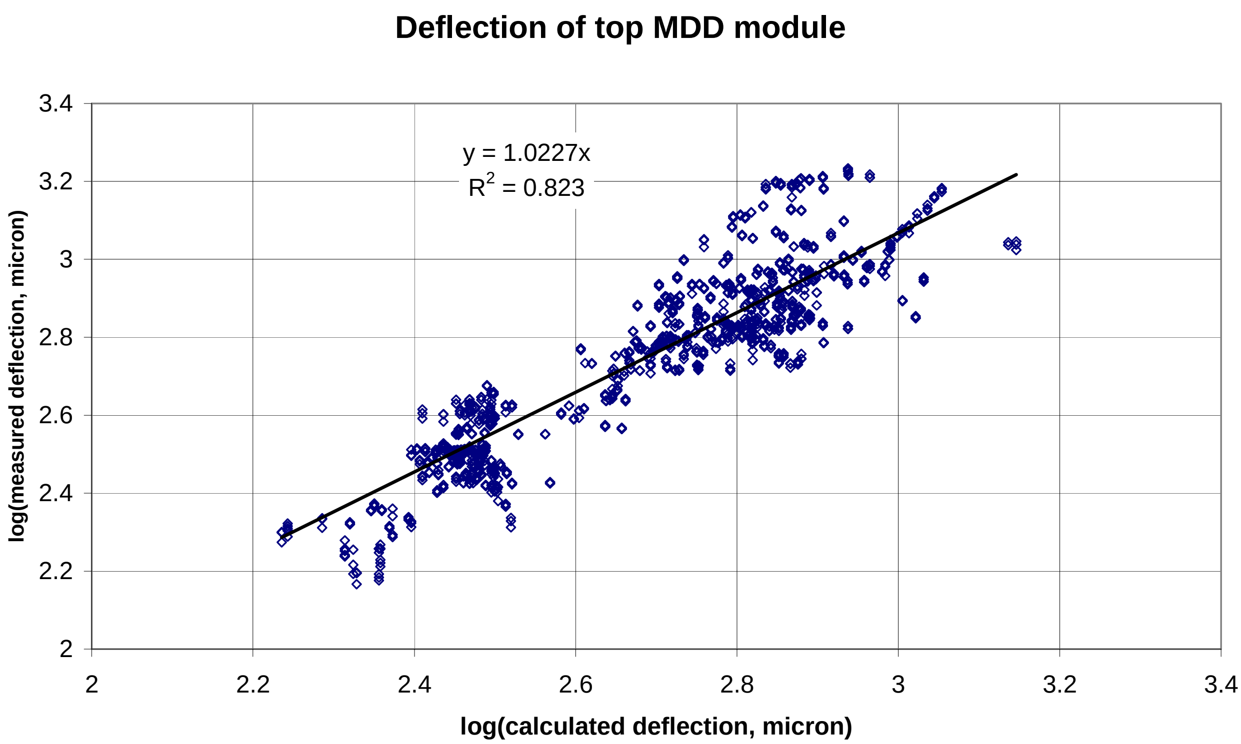

Figure 3 Comparison of deflections measured by all top MDD modules to *CalME* deflections.

Figure 3 compares all of the deflections measured by the MDD modules nearest the pavement surface, for all of the HVS tests, to the deflections calculated by *CalME*.

The section with 90 mm DGAC overlay was split into two sections in the analysis, approximately at the middle of the section, because the behavior of the two sections were distinctly different.

5 DAMAGE AND CRACKING

The terminal moduli predicted by *CalME* should be similar to the moduli backcalculated from FWD tests following the HVS experiment if the damage to the pavement layers has been correctly calculated. For the present tests there was a decrease in the moduli of the asphalt layers and of the self-cementing AB. The modulus of the subgrade also decreased with the decrease in the stiffness of the pavement layers.

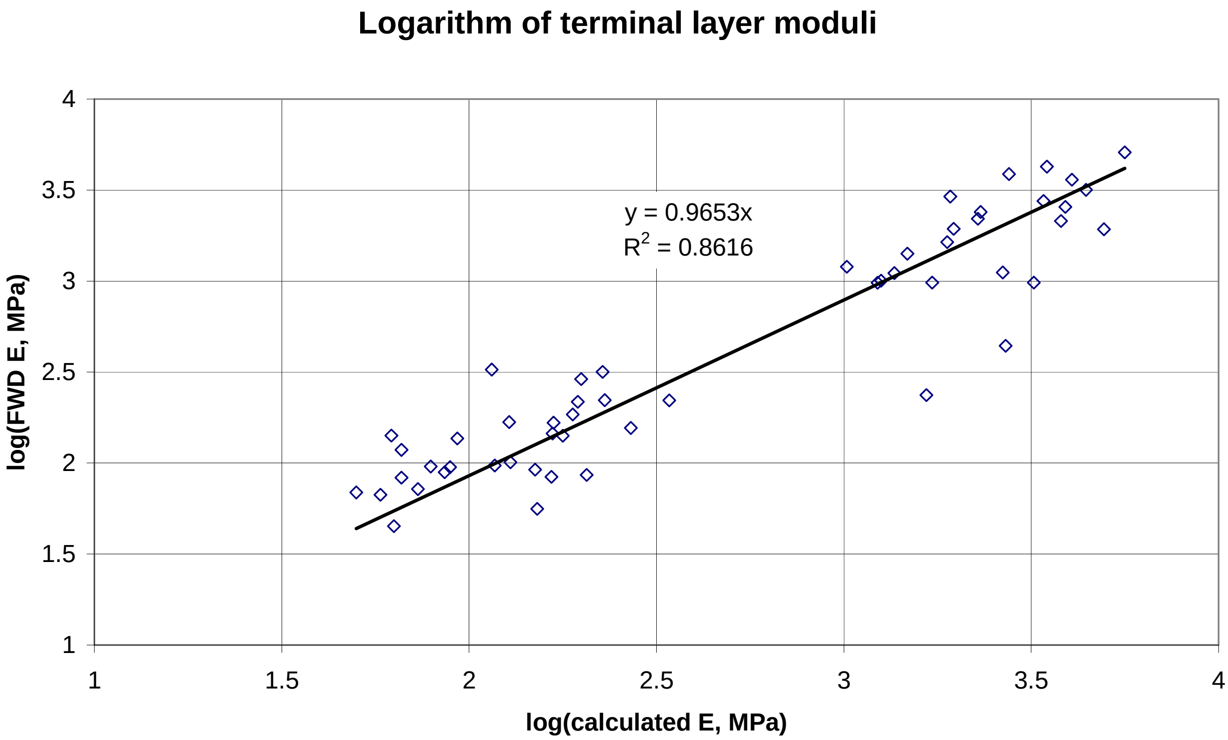

Figure 4 Logarithm of terminal moduli, FWD tests versus simulation with *CalME*.

In Figure 4 the moduli backcalculated from the first FWD test series after the completion of each HVS experiment are compared to the terminal moduli from the simulation with *CalME*. To show the full range of moduli for all of the pavement layers, the logarithm of the moduli, in MPa, are shown.

For prediction of reflection cracking the coefficients of Equation 4 and Equation 6 were modified to the values shown in Equation 7 and Equation 8.

$$\omega_{initiation} = \frac{1}{1+\left(\frac{h_{AC}}{390\,mm}\right)^{-1}}$$

Equation 7 Coefficients for crack initiation model.

$$Cr\ m/m^2 = \frac{10}{1+\left(\frac{\omega}{\omega_o}\right)^{-3.5}}$$

Equation 8 Coefficients for crack propagation model.

Figure 5 shows the observed reflection cracking (Obs), in m/m^2, versus the reflection damage, ω, calculated by *CalME* for the surface layer. Also shown is the cracking as calculated from Equation 7 and Equation 8 (Calc), with the assumptions that crack initiation corresponds to 0.5 m/m^2 of cracking. The calculated cracking is shown for layer thicknesses of 125 and 170 mm, which correspond roughly to the combined AC thickness for the sections with thin and thick overlays, respectively.

The figure illustrates that the visible cracking follows the development of damage.

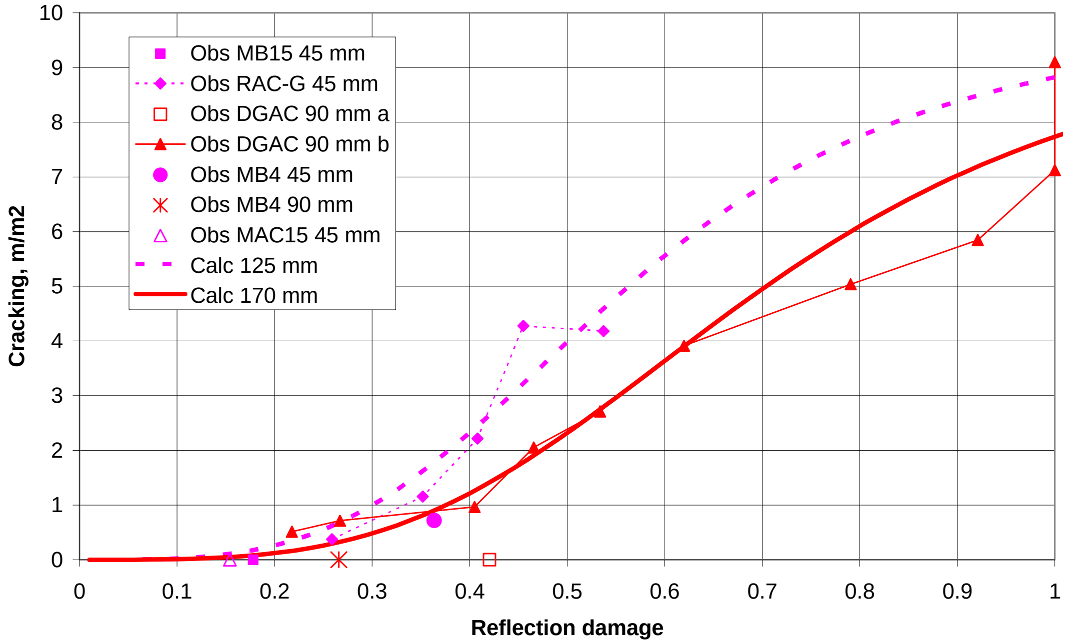

Figure 5 Observed and calculated cracking versus damage from *CalME*.

In Figure 6 the reflection cracking predicted using Equation 7 and Equation 8 is shown as a function of the observed reflection cracking.

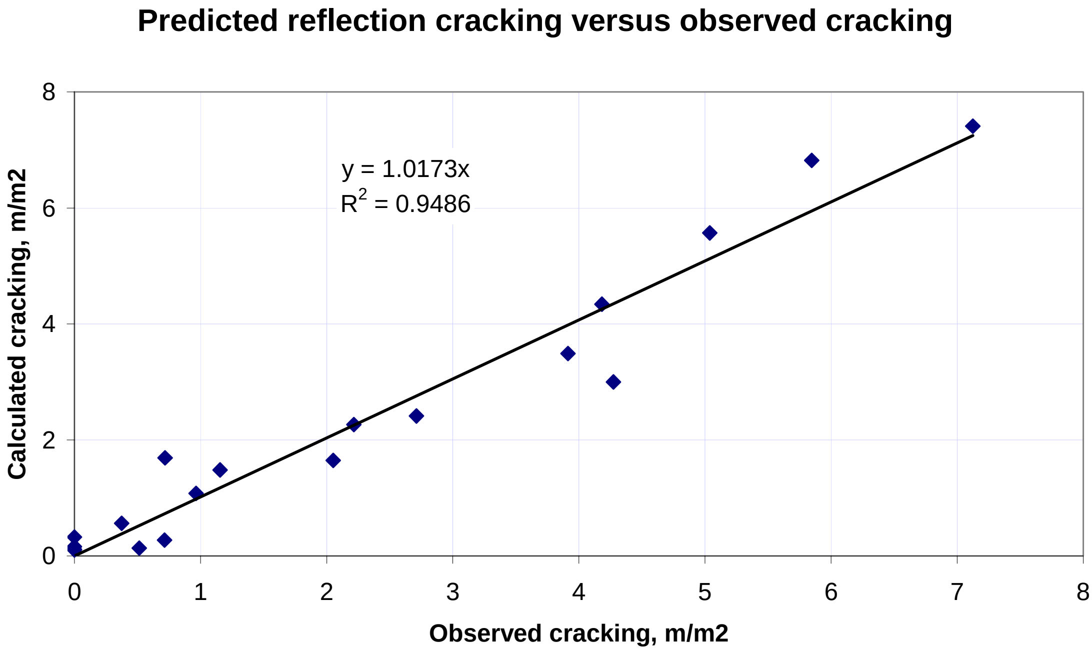

Figure 6 Predicted versus observed reflection cracking.

6 CONCLUSION

The main conclusion of the analysis was that it was possible to simulate the pavement response and performance using *CalME*. The resilient pavement deflections, at the surface and at different depths, were predicted reasonably well for the whole duration of the HVS tests. The empirical relationships derived from laboratory experiments, to determine permanent deformation and damage, also predicted the pavement performance reasonably well for the duration of the experiment, when appropriate shift factors were applied to allow for the difference between the laboratory experiments and the HVS loading. The simple reflection cracking model appeared to work correctly with respect to the development of reflection damage and cracking.

Another important conclusion of the experiment was that a recursive mechanistic-empirical method is needed to interpret the results of HVS (or other full scale) testing. This is exemplified by the test section with 90 mm DGAC overlay. Although the original pavement was constructed to be uniform over all six test sections it was found that it had quite large variations, both spatially and with respect to time. The two subsections of the 90 mm DGAC overlay section showed very different performances. This difference was predicted reasonably well using the recursive mechanistic-empirical approach, but would hardly have been possible to consider in a purely empirical analysis.

Once the models of the recursive mechanistic-empirical method have been calibrated the different overlay materials may be compared through simulations, where the underlying pavement, the loading, and the climatic conditions are exactly the same for each material. The sensitivity to the condition of the underlying pavement, the loading, and the climate can likewise be studied. Such studies are presently being carried out.

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