

## Using four-point bending tests in calibration of the California mechanistic-empirical pavement design system

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**ABSTRACT:** The University of California Pavement Research Center (UCPRC) has been supporting the California Department of Transportation's effort to implement mechanistic-empirical (ME) pavement design by developing and calibrating ME flexible pavement design models. These models have been incorporated into a draft software program called *CalME*. *CalME* introduces an Incremental-Recursive Mechanistic-Empirical (IRME) method in which the materials properties for the pavement are updated in terms of damage as the pavement life simulation progresses. Two of the key components of *CalME* are the stiffness master curve and fatigue damage model for hot mix asphalt materials. This paper presents how *CalME* uses four-point bending (4PB) tests to identify parameters for these models. Calibration of the *CalME* design system is also demonstrated using Heavy Vehicle Simulator (HVS) test data. It is shown that the 4PB tests can provide vital parameters that are necessary for *CalME* in predicting accurate complete performance history of a flexible pavement structure.

### 1 INTRODUCTION

The use of mechanistic-empirical methods has recently mushroomed in the U.S. pavement design community. The best known example of such a method is the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed by AASHTO (ARA Inc., 2004). MEPDG simulates a pavement structure throughout its design life and provides complete performance histories for various distress mechanisms. However, the asphalt fatigue model in MEPDG does not update the stiffness of asphalt during simulation, and uses a linear cumulative damage model ("Miner's law") to assess damage under different loads and temperature conditions (ARA Inc., 2004).

The University of California Pavement Research Center (UCPRC) has completed a large number of Heavy Vehicle Simulator (HVS) (Harvey *et al.*, 1998) tests for the California Department of Transportation (Caltrans) to evaluate different flexible pavement designs and hot mix asphalt (HMA) characteristics. In HVS tests, measured surface deflections often increase markedly, sometimes becoming more than twice as high at the end of the test as they were at the beginning because of damage to the asphalt concrete caused by the repeated wheel loads. However, MEPDG does not consider any decrease in the HMA modulus as a result of fatigue damage (except for rehabilitation designs). In fact, it includes a model for aging that predicts a continuous increase in the stiffness of the HMA layers across the life of the pavement, which results in increased stiffness and smaller predicted deflections as the pavement is subjected to trafficking. While aging can be important, the effect of updating stiffness for aging and not updating it for fatigue damage can result in very unrealistic calculated elastic responses in the pavement during its life. This makes it impossible to use the model to simulate an HVS test and, inversely, to use HVS tests to calibrate the model, or to use response measurements (deflections or strains) from test tracks to calibrate response models, except for pavements with extremely thick asphalt

concrete layers where little fatigue is expected to develop. This difficulty led the UCPRC to develop new mechanistic-empirical design models which are included in a software called *CalME* (Ullidtz *et al.*, 2006) for new flexible pavements and rehabilitation. *CalME* has allowed the UCPRC to evaluate and develop models for various distress mechanisms such as reflective cracking, rutting, HMA aging, etc.

*CalME* includes an incremental-recursive (IR) approach in which the material properties for the pavement are updated in terms of layer stiffness as the simulation of the pavement life progresses. The IR approach requires a model for describing stiffness degradation of each layer caused by fatigue damage under traffic loading. Although *CalME* has an aging model, it is not applied in the calibration study shown here because of the short testing duration.

The objective of this paper is to introduce a fatigue damage model for HMA layers and the identification of model parameters using stiffness reduction curves obtained in four-point bending (4PB) beam fatigue tests (Tayebali *et al.*, 1996). Other related components of the pavement design system are presented. To demonstrate the effectiveness of the approach, calibration of the fatigue model using HVS tests is also presented.

## 2 PAVEMENT DESIGN SUBSYSTEM FOR FATIGUE DAMAGE IN *CALME*

When calculating fatigue damage, *CalME* idealizes flexible pavement structures as multilayer elastic systems. Stiffness for HMA layer depends on loading time and loading temperature, while the stiffness of unbound layers depend on confinement provided by the upper layers and the load amplitude applied by the truck wheel loads.

### 2.1 Master Curves for Asphalt Materials

The model for intact HMA modulus versus reduced time follows the model in MEPDG (ARA Inc., 2004):

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

where  $E_i$  = the intact modulus in MPa;  $tr$  = reduced time in sec;  $\alpha$ ,  $\beta$ ,  $\gamma$ , and  $\delta$  = experimentally determined constants, and  $\log$  = logarithm with base 10. Reduced time is found from (ARA Inc., 2004):

$$tr = lt \times \left( \frac{visc_{ref}}{visc} \right)^{aT} \quad (2)$$

where  $lt$  = the loading time in sec;  $visc_{ref}$  = the binder viscosity at the reference temperature;  $visc$  = the binder viscosity at the present temperature, and  $aT$  = constant. This model accounts for the temperature and loading time dependency of HMA modulus. The binder viscosity is calculated using the following equation:

$$\log_{10} \log_{10}(visc) = A + VTS \cdot \log_{10} T \quad (3)$$

where  $T$  is binder temperature in Rankine while  $A$  and  $VTS$  are model parameters.

### 2.2 HMA Fatigue Damage Model

In *CalME*, it is assumed that fatigue damage causes the HMA modulus to decrease according to the following relationship:

$$\log(E) = \delta + \frac{\alpha \times (1 - \omega)}{1 + \exp(\beta + \gamma \log(tr))} \quad (4)$$

where  $\omega$  = the damage, which can be calculated from the following equations:

$$\omega = \left( \frac{MN}{\phi \times MN_p} \right)^\alpha \quad (4)$$

$$\alpha = \exp \left( \alpha_0 + \alpha_1 \times \frac{t}{1^\circ C} \right) \quad (5)$$

$$MN_p = A \times \left( \frac{\mu\varepsilon}{\mu\varepsilon_{ref}} \right)^\beta \times \left( \frac{E}{E_{ref}} \right)^\gamma \times \left( \frac{E_i}{E_{ref}} \right)^\delta \quad (6)$$

Where  $MN$  = the number of load applications in millions;  $\mu\varepsilon$  = the tensile strain at the bottom of the asphalt layer;  $E$  = the (current) damaged modulus;  $E_i$  = the intact modulus;  $t$  = HMA average temperature;  $\phi$  = t shift factor that will need to be calibrated;  $A$ ,  $\alpha_0$ ,  $\alpha_1$ ,  $\beta$ ,  $\gamma$ ,  $\delta$  = model parameters; and  $\mu\varepsilon_{ref}$  and  $E_{ref}$  = reference strain and modulus, respectively, used for normalizing the ratios. Typically,  $\beta = 2\gamma$  in order to comply with the concept that fatigue damage is driven by strain energy. Note that constants and exponents in Equations 3-6 are not related to those in Equation 1.

### 2.3 Moduli of Unbound Layers

It has been found that the moduli of unbound materials sometimes vary with the stiffness of the layers above them. This may happen as a result of either a change in stiffness of asphalt layers due to change in temperature or increasing damage to the asphalt layers. For granular layers this effect is the opposite of what would be expected based on an understanding of the nonlinearity 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 reasoning is in agreement with observation made by analyzing Long-Term Pavement Performance (LTTP) data (Richter, 2006) in which it was found that the moduli of granular layers, back-calculated from FWD tests, tend to decrease, instead of increase, with increasing bulk stress. An explanation for this behavior could be due to the decrease in the confining effect of the layers above the granular material as explained in (Ullidtz *et al.*, 2006) with analysis using the distinct element method.

To allow for this effect, the stiffness of each unbound layer was modeled as a function of the flexural stiffness of the layers above it.

$$E = E_0 \times \left[ 1 - \left( 1 - \frac{S}{S_{ref}} \right) \times \text{stiffness factor} \right] \quad (7)$$

with:

$$S = \left( \sum_1^{n-1} h_i \times \sqrt[3]{E_i} \right)^3 \quad (8)$$

where  $E_0$  = the modulus at the reference condition;  $S$  = the combined stiffness of layers above layer  $n$ ;  $S_{ref}$  = the normalizing stiffness (value of 3500 N-mm was used here); and  $h_i$  = the thickness of layer  $i$  (mm).

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 non-linear models, with modulus of granular materials increas-

ing with increasing bulk stress, and the modulus of the cohesive subgrade decreasing with increasing deviator stress. However, truck axle load level had to be used instead of stress because of the effects of confinement.

### 3 MODEL CALIBRATION USING HVS DATA

#### 3.1 Overview of the Calibration Procedure

The IR models included in *CalME* were used to predict performance of all twenty-seven flexible pavement HVS tests performed so far as part of the accelerated pavement testing program operated for the California Department of Transportation (Caltrans) by the University of California Pavement Research Center (UCPRC).

During HVS testing, pavement response - in terms of deflections at the surface and/or at multiple depths - were measured using both Road Surface Deflectometer (RSD) for surface deflections and Multi-Depth Deflectometer (MDD) (Harvey *et al.*, 1996) for deflections at multiple depths.

In order to accurately predict the gradual degradation of a pavement, the response model must be able to predict measured deflections with reasonable accuracy even though this does not guarantee accurate prediction of stress and strain. Accordingly, the research team's first concern was to make sure that resilient deflections were predicted reasonably well for the duration of the test and for all load levels.

During an HVS test, measured surface deflections should increase as the number of load applications increases due to stiffness degradation of all the pavement layers and weakening of the bond between different layers. Fatigue damage causes stiffness degradation in the HMA layer, which in turn leads to stiffness reduction in the underlying unbound layer due to the non-linear elastic characteristics of unbound layers and the effect of confinement on granular layers.

Once reasonably good agreement was achieved between the measured and the calculated deflections, models for other distresses such as permanent deformation could be calibrated with confidence.

The purpose of using HVS calibrations was to evaluate the overall trends of the *CalME* damage models against those of the HVS test results. This was accomplished by comparing deflections calculated using moduli determined from initial measurements and *CalME* damage calculations with measured deflections under HVS loading. Note that the only parameter to be calibrated is the shift factor  $\phi$  defined in Equation 4. It was found that a value of 3 may be used for  $\phi$  for HMA layers in all of the HVS tests. In other words, it took three HVS load repetitions at a given strain, modulus, and temperature to produce the same damage as one load repetition in the laboratory bending beam test.

To demonstrate the procedure proposed in this paper, the following section presents calibration results using data from one of the HVS tests..

#### 3.2 Introduction of Example HVS Calibration

One HVS section was used here to illustrate the calibration process. Details of the test section can be found in (Harvey *et al.*, 1999). The section had two layers of HMA, an aggregate base (AB), and an aggregate subbase (ASB). The subgrade was clay, with varying plasticity across the pavements. The design and as-built layer thicknesses are shown in Table 1.

Table 1. Design and As-built Thicknesses for the HVS Section

Layer	Design Thickness (mm)	As-built Thickness (mm)
HMA top lift	61	63
HMA bottom lift	76	84
Aggregate base (AB)	274	274
Aggregate subbase (ASB)	229	215
Subgrade (SG)	-	-

A dual-wheel load of 40 kN was used to traffic the section for the first 150,000 repetitions to prevent excessive initial deformation of the newly constructed asphalt concrete. After that, the load of the dual-wheel was increased to 80 kN for the next 50,000 repetitions. After 200,000 repetitions were applied, the trafficking load was further increased to 100 kN and maintained onward for the remainder of the test to accelerate pavement cracking.

Bias-ply tires on a dual wheel were used for these tests. The wheel load was a dual wheel with a centerline distance of 305 mm and a tire pressure of 690 kPa for all load levels.

The test section was instrumented with two Multi-Depth Deflectometers (MDD), one at Station 4 and the other at Station 12. Both MDDs were anchored at 3000 mm depth. A plan view of the test section along with the location for MDD locations are shown in Figure 1.

Although actual wheel speeds varied, they were approximately 7.6 km/h during HVS testing and 1.8 km/h during deflection measurement on the MDDs.

The test section is enclosed by a temperature control chamber. Air temperatures inside the chamber ranged from 16° C to 25° C during the entire testing period. The target air temperature was 20°C±4°C. The temperature range is expected to promote fatigue cracking and minimize rutting of the HMA layer. The recorded actual pavement temperatures were used in calibration of *CalME* models.

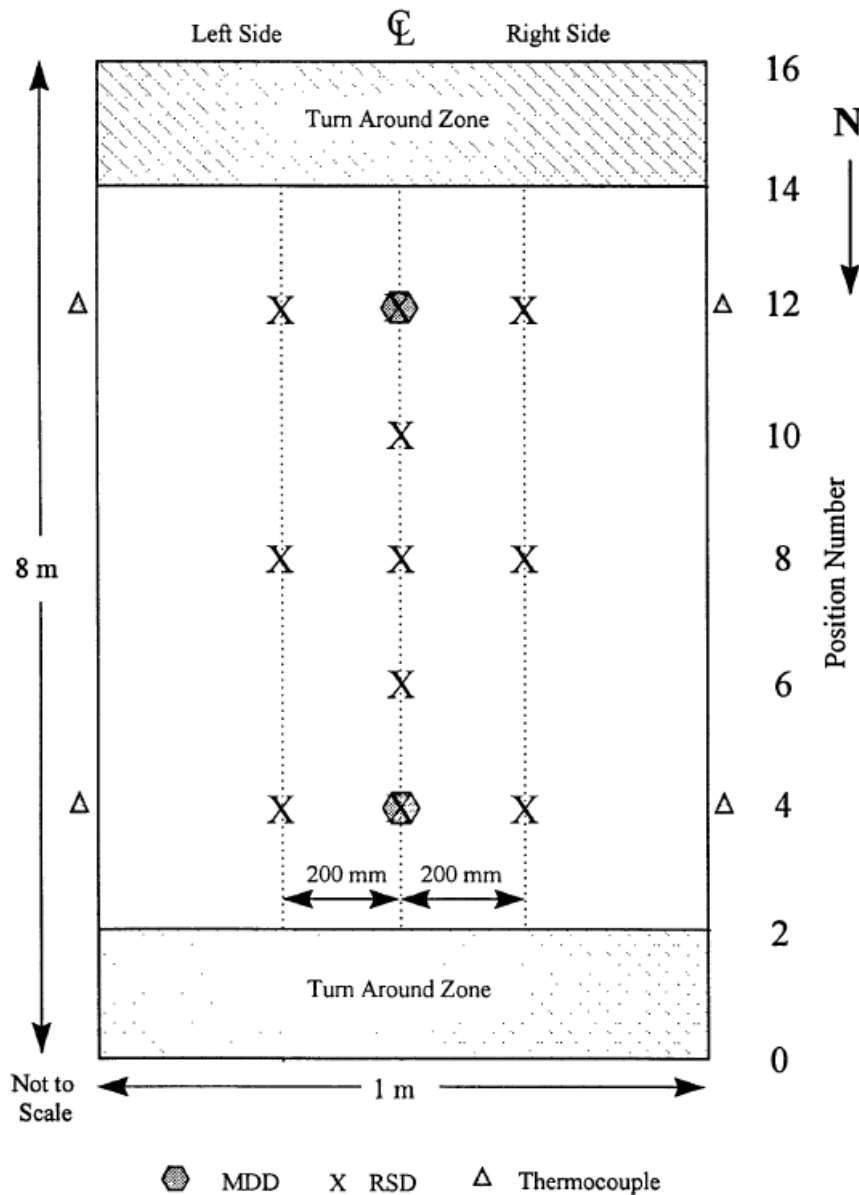


Figure 1. Plan view of the test section and location for instrumentation

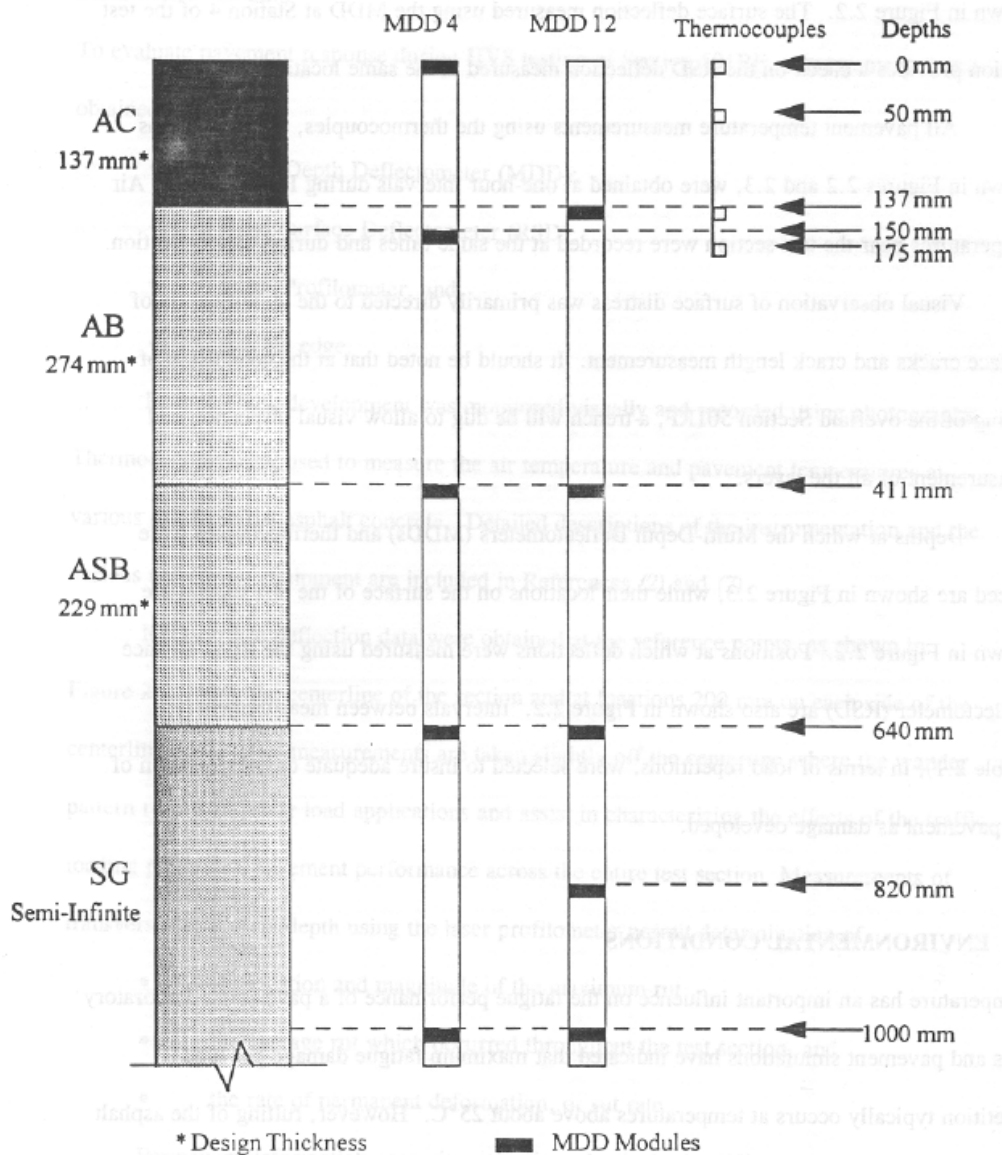


Figure 2. Pavement structure with MDD and thermocouple positioning

### 3.3 Model Parameter Identification Using 4PB Test Data

The applications of 4PB beam test in characterizing the HMA properties can be categorized into the flexural fatigue testing for characterizing the fatigue damage model and the flexural frequency sweep testing for characterizing the stiffness master curve.

#### 3.3.1 Determination of HMA stiffness master curve

4PB beam flexural frequency sweep testing characterizes the effects of loading frequency and temperature on the initial flexural complex modulus of an HMA mix. This allows *CalME* to take the vehicle speed and temperature variation into account when conducting an incremental/recursive mechanistic-empirical prediction of fatigue performance. The loading sequence of flexural frequency sweep test for each of three temperatures, 10, 20, and 30C, is normally conducted, from quick to slow, at 15, 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz. The flexural complex modulus ( $E^*$ ) master curve is then constructed based on the MEPDG stiffness model by fitting the laboratory results.

Table 2 and Figure 3 summarize the stiffness model parameters and the corresponding fitting results for both top and bottom lifts of HMA layer, respectively. As can be seen in Figure 3, the introduced stiffness model produces an appropriate fitting result. Figure 3 shows that in general the bottom lift has higher stiffness than that of the top lift for all the loading times used.

Table 2. Stiffness model parameters of HMA material for use in CalME

Layer Material	$\delta$	$\alpha$	$\beta$	$\gamma$	$E_{ref}$ (MPa)	aT	VTS	A
Top Lift	2.301	1.855	-0.3984	0.938905	9028	1.354133	-3.5047	9.6307
Bottom Lift	2.301	1.944	-0.4012	0.976841	11168	1.282477	-3.5047	9.6307

Note:

The reference moduli ( $E_{ref}$ ) are defined at a loading time of 0.015 sec. and a reference temperature of 20°C.

### 3.3.2 Determination of HMA fatigue damage model parameters

Four-point bending flexural fatigue testing is used to measure the fatigue damage or stiffness deterioration characteristics of specific mixes over a range of traffic (in terms of testing stress or strain levels in the laboratory) and environmental conditions (testing temperatures) so that the fatigue considerations can be incorporated into the process of designing asphalt concrete pavements. One of the advantages of using 4PB beam fatigue test is that the middle one-third part of the beam is theoretically subjected to pure bending without any shear deformation. Notice that the fatigue resistance of an HMA is its ability to withstand repeated bending without fracture. Hence, this makes the four-point bending beam test a good candidate to characterize the fatigue properties of asphalt mix.

The power fatigue damage model introduced here (Equations 3-6) is used to predict laboratory fatigue test results at various temperatures and test strain levels. Model parameters are determined by matching calculated and measured stiffness reduction curves through a nonlinear optimization process. During this process the fatigue shift factor  $\phi$  is set to 1.0.

Table 3 and Figure 4 summarize the fatigue damage model parameters and fitting results for both top and bottom lifts of the HMA layer using the laboratory 4PB beam fatigue test results. As seen in Figure 4, the calculated stiffness was fit reasonably well for stiffness ratio higher than 0.5 ~ 0.6; however, the calculated stiffness tends to produce a more conservative prediction which leads to less fatigue damage when the stiffness ratio is lower than 0.5~0.6. The deviation between calculated and measured stiffness grows as the stiffness ratio continues to decrease. This is expected since data with residual stiffness ratio below 0.3 is ignored. The reason for disregarding the low stiffness ratios is that once the beam starts to develop macro-cracks it will behave quite differently from a pavement layer that develops macro-cracking. The beam risks rapid failure, whereas the pavement will, probably, fail slower due to different boundary conditions..

A comparison between calculated and measured stiffness reduction curves are shown in Figure 5. As shown in the figure, the damage model captures the stiffness degradation process very well.

Table 3. Fatigue damage model parameters of AC material for use in CalME

Layer Material	$\alpha_0$	$\alpha_1$	A	$\mu\epsilon_{ref}$	$\beta$	$E_{ref}$ (MPa)	$\gamma$
Top Lift	0.09845	0	14.47997	200	-3.75102	3000	-1.87551
Bottom Lift	-0.0138	0	13.69337	200	-4.10023	3000	-2.05012

Note:

Please refer to Equations 4, 5, and 6 for parameter definitions.

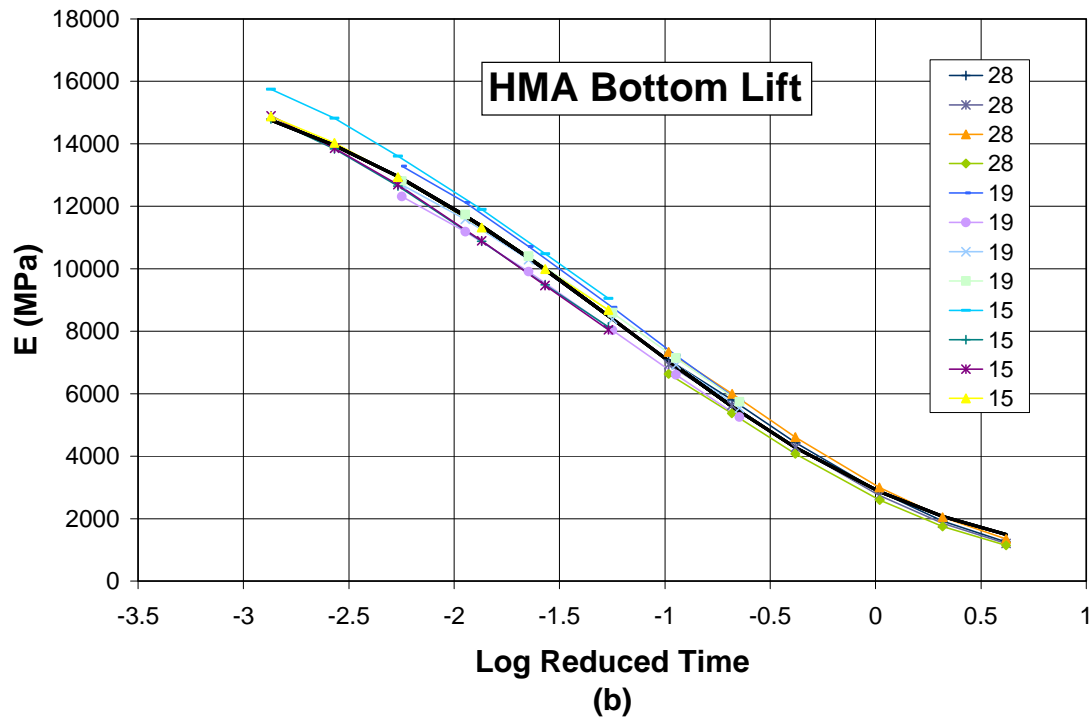
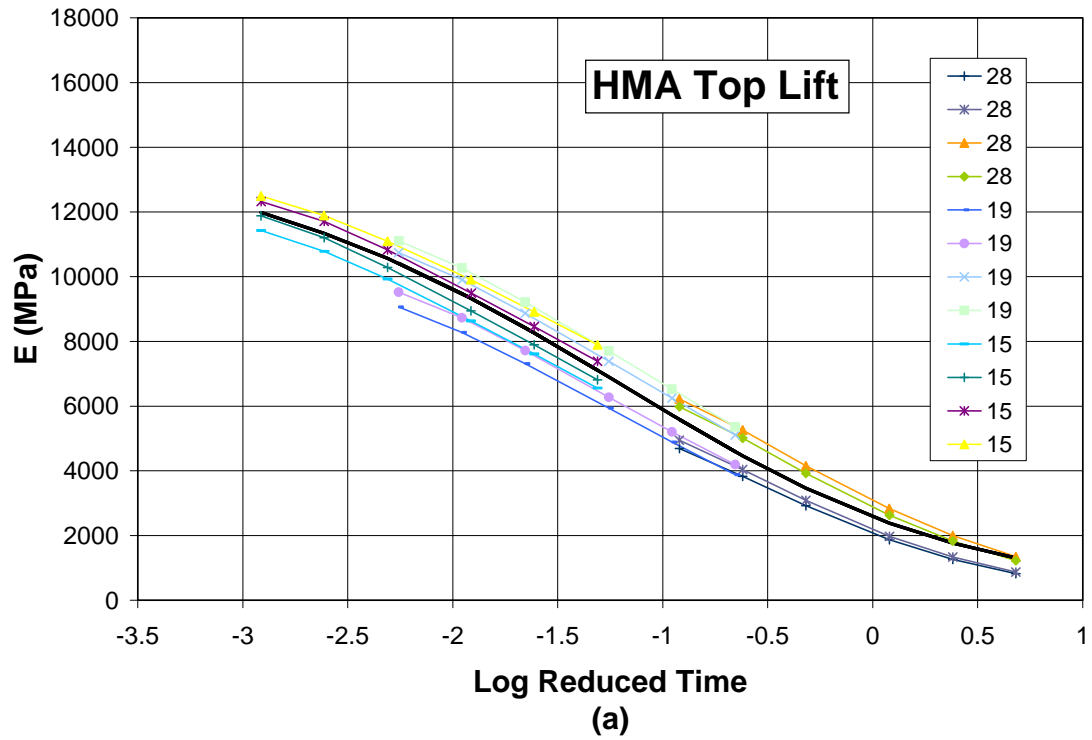


Figure 3. Summary of flexural frequency sweep test results and the fitted curves for (a) top lift and (b) bottom lift of HMA layer. Legend indicates testing temperature in degrees Celsius. The thick solid lines without marker indicate the fitted results.

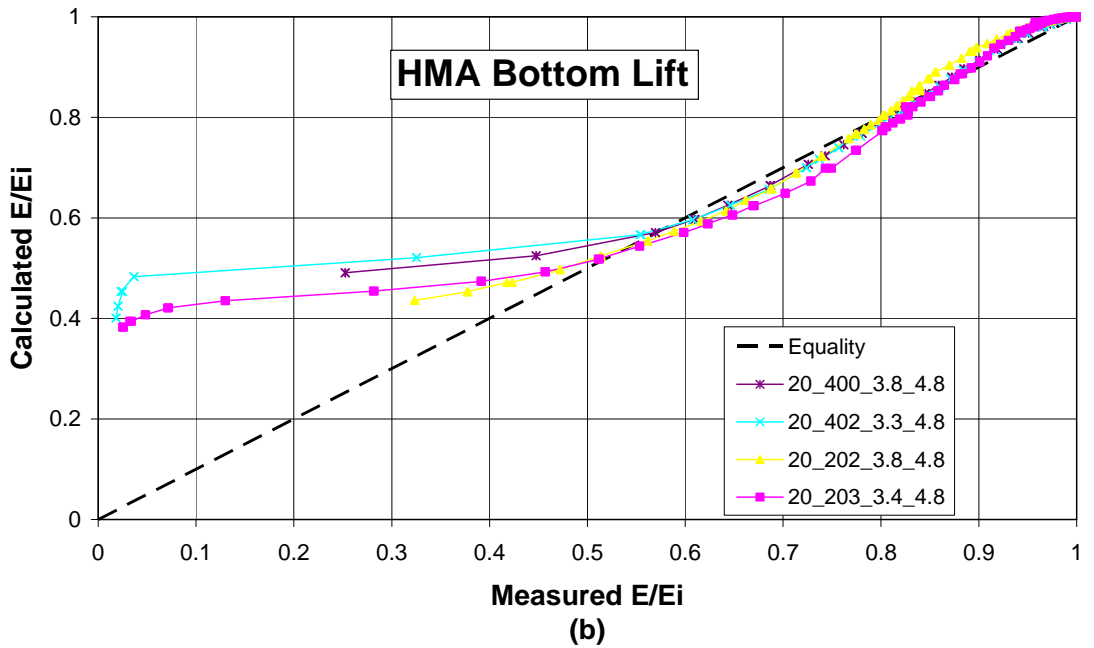
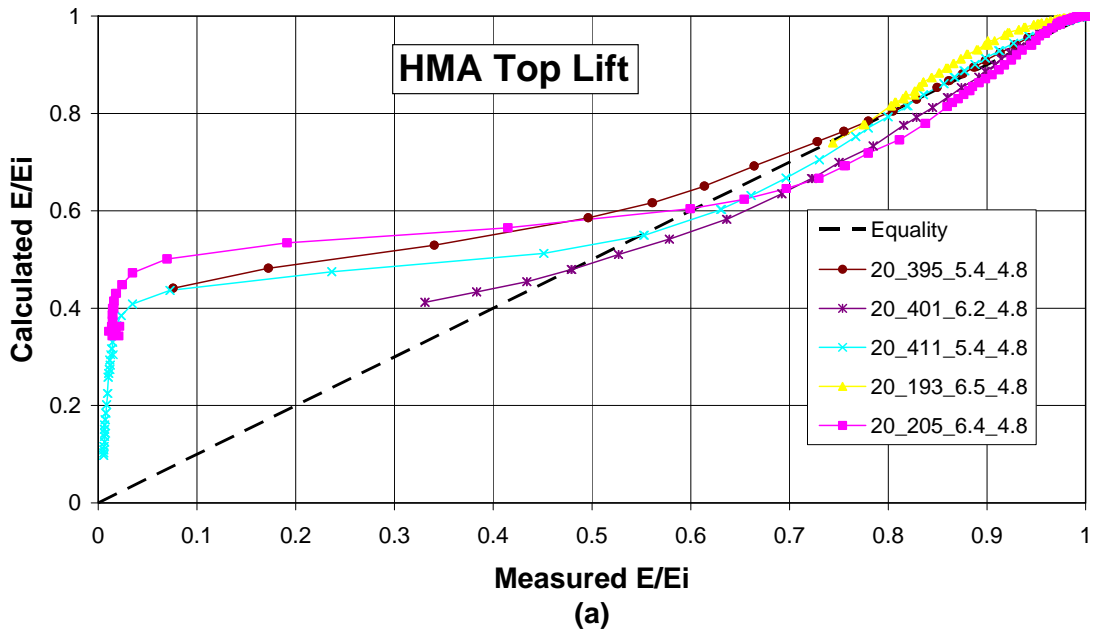


Figure 4. Comparison of calculated and measured normalized stiffness from flexural fatigue test results. Legends indicate testing temperature (20°C), strain levels in microstrain (around 200 or 400), air void content, and binder content by weight of aggregate (Note: E is the current stiffness, Ei is the initial stiffness)

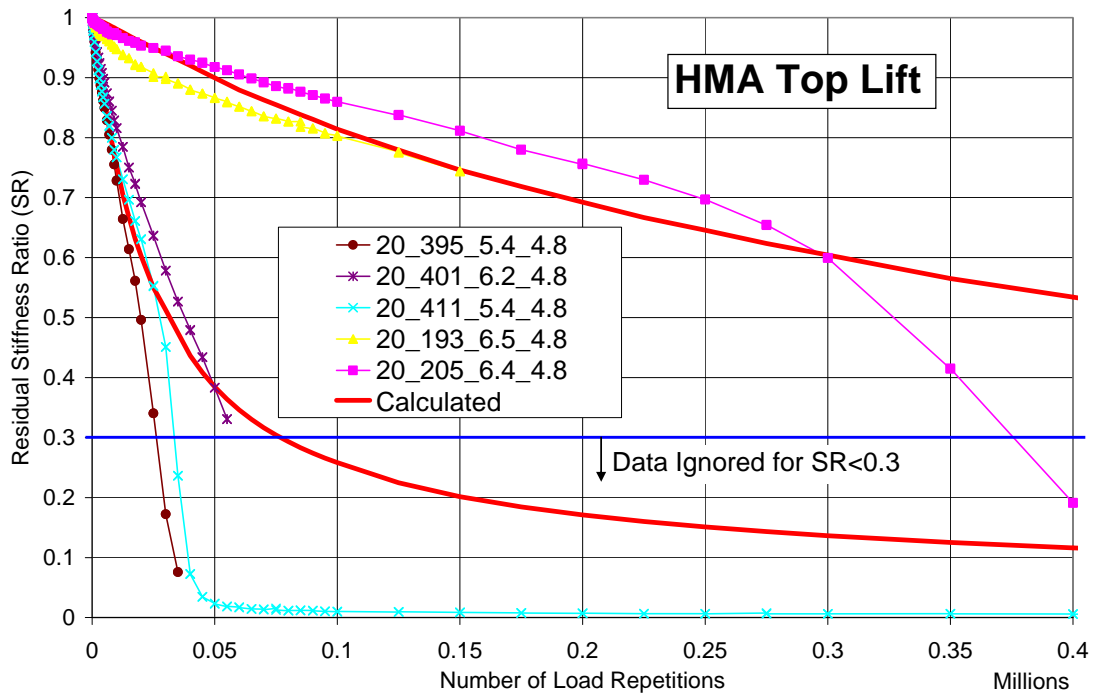


Figure 5. Comparison between calculated and measured stiffness reduction curves for HMA top lift, the two calculated curves are for strain levels of 205 and 411 microstrain.

### 3.4 CalME Simulation and Results

The pavement response was calculated for each hour of the HVS test, and it was assumed that each HVS loading sequence was evenly distributed over time (note that with an almost constant temperature this has little significance). The load was placed at five transverse positions across the 100 mm width of the loaded area to account for the wander pattern used for the HVS trafficking. Damage to the asphalt for each wheel position was accumulated using the “time hardening” method, i.e., by first calculating the number of load applications required to cause the present damage, given the present pavement response and conditions, then by calculating the damage that would be caused by that number of load applications plus the number of loads applied at the wheel position during the hour under consideration.

Damage to the asphalt was based on the maximum principal tensile strain at the bottom of the lowest asphalt layer, when the layers were bonded. After debonding, the principal strain at the bottom of the top layer was used to calculate the damage of the top layer. Apparent damage to the unbound layers was calculated using the damage to the asphalt layers, the degree of bonding between asphalt layers, and the apparent damage to any unbound layer above the layer under consideration. Temperature variation in the asphalt layers also affects the apparent damage of the unbound layers.

It was assumed that the wheels distributed the load over two circular areas; this assumption was reasonably correct for the low load level (40 kN) but not for the high load level (100 kN), where the actual load distribution was closer to two rectangles with one side twice the length of the other. Poisson’s ratio was assumed to be 0.35 for all layers.

The following figures show the deflection of all the LVDT modules in each MDD. They are first shown for a wheel load of 40 kN, then for a wheel load of 100 kN. Slip was assumed to have been almost fully developed after 2 MESAL (million ESAL, calculated using a 4.2 exponent), which corresponds to the time shortly after the load was increased to 100 kN. The slip values used was 1.0, with 0 representing full slip and 100000 for no slip. Slip value decreased from 100000 to 1.0 following a log linear curve.

In the figures below, measured deflections are indicated by an M, followed by a number revealing MDD module depth, and calculated deflections are indicated by the letter C, again followed by a number denoting depth.

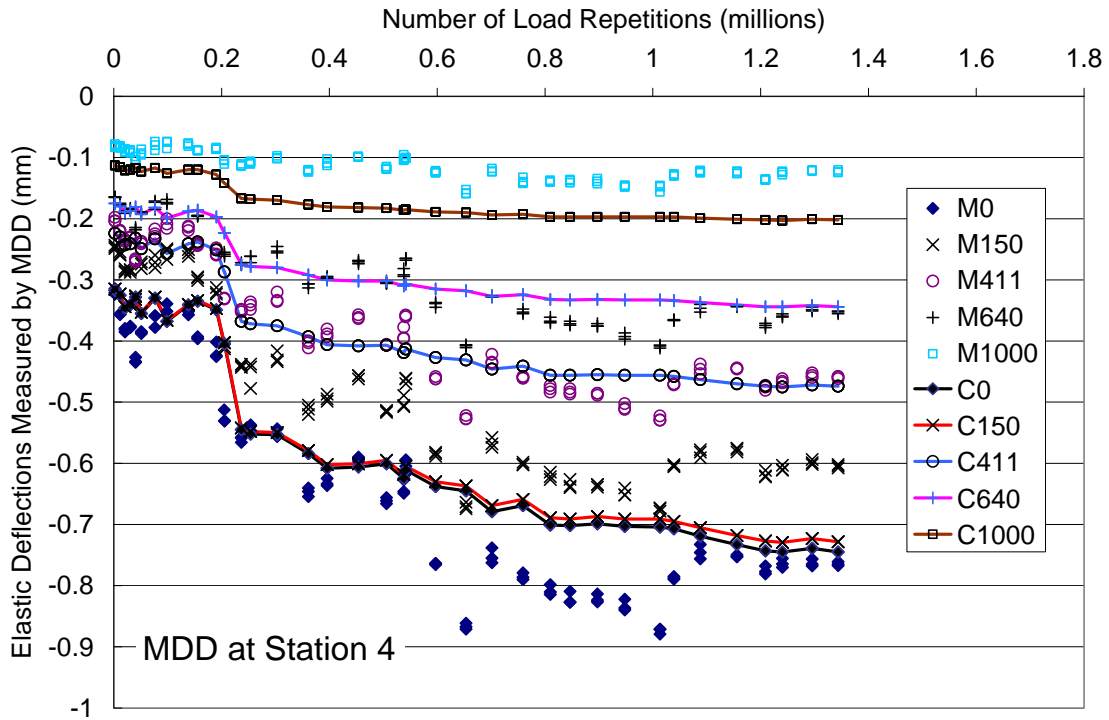


Figure 6. Comparison of measured and calculated elastic deflections for MDD at station 4 under 40kN load, legends indicate vertical deflections (measured vs. calculated) and LVDT module depth

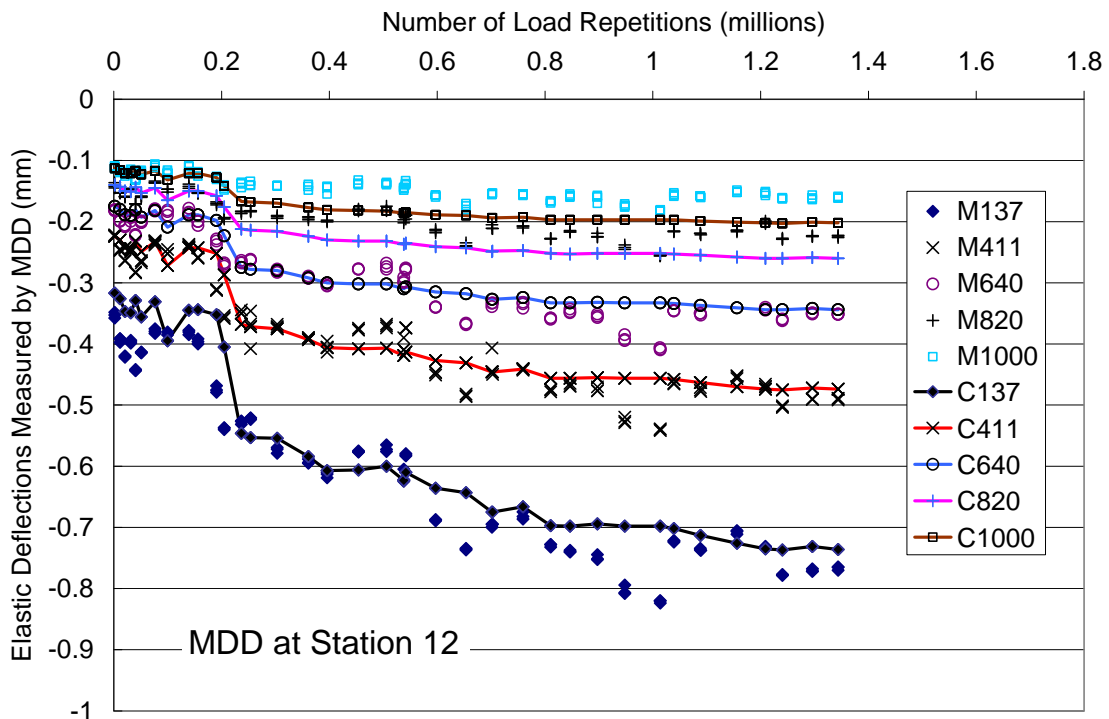


Figure 7. Comparison of measured and calculated elastic deflections for MDD at station 12 under 40kN load, legends indicate vertical deflections (measured vs. calculated) and LVDT module depth

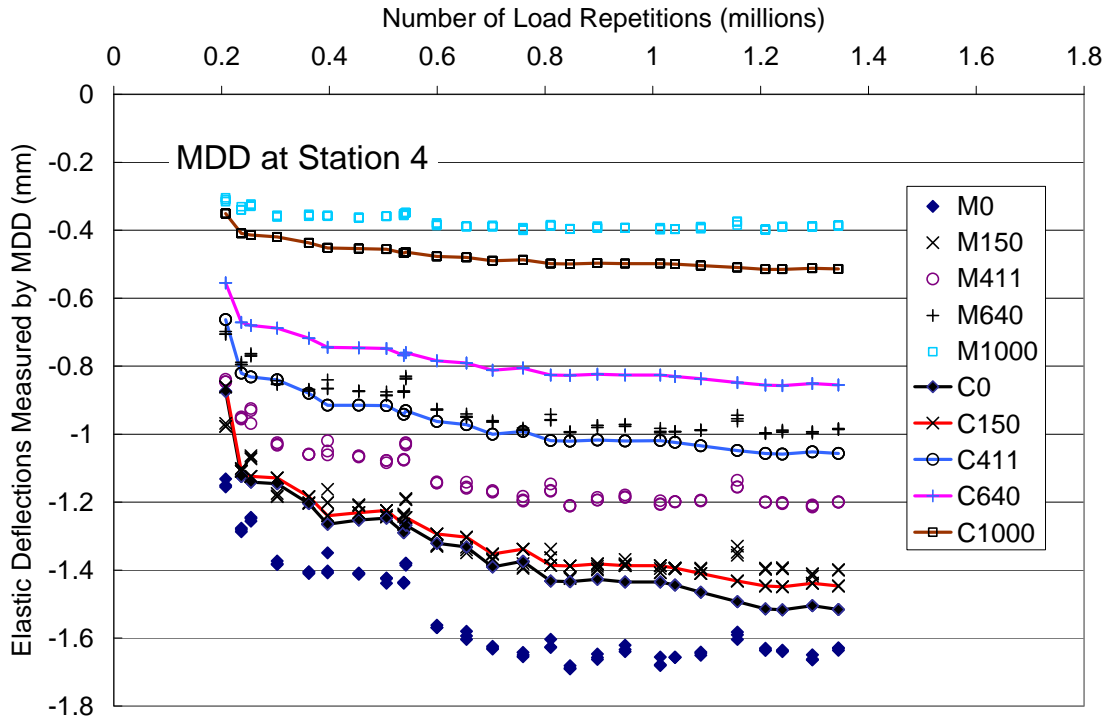


Figure 8. Comparison of measured and calculated elastic deflections for MDD at station 4 under 100kN load, legends indicate vertical deflections (measured vs. calculated) and LVDT module depth

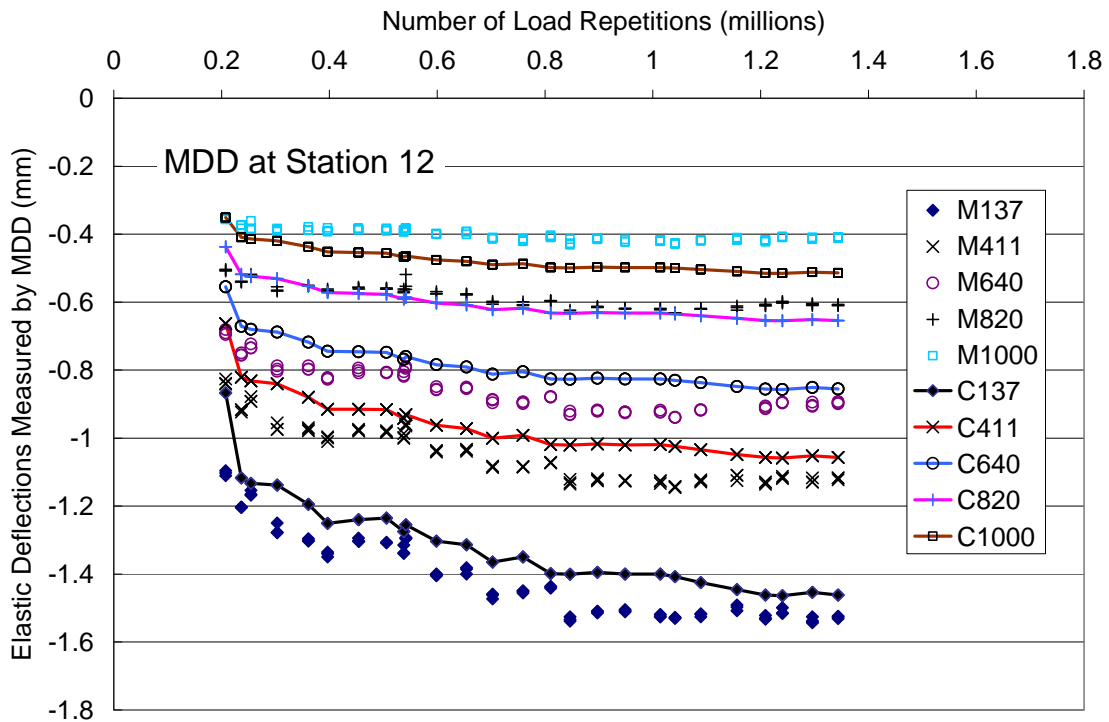


Figure 9. Comparison of measured and calculated elastic deflections for MDD at station 12 under 100kN load, legends indicate vertical deflections (measured vs. calculated) and LVDT module depth

Given the scatter in the measured data, the agreement between measured and calculated deflections is very good. The calculated stiffness degradation history during the HVS test is shown in Figure 10, for HVS load of 40 kN and a wheel speed of 1.8 km/h. Layer damage and

surface crack density histories are shown in Figure 11. According to the figure, the curve that represents surface crack length history has a different shape from the curve for damage history. The relationship between HMA layer damage and measured surface crack length is one of the areas that are still under research by the authors of this paper.

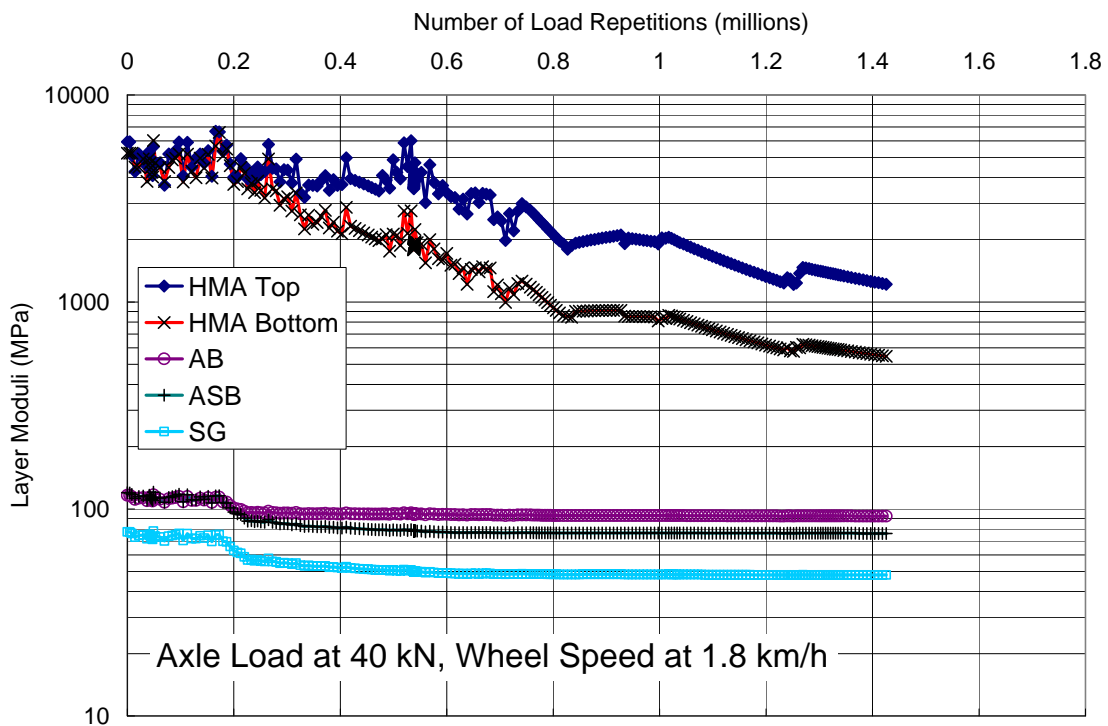


Figure 10. Layer moduli degradation history for axle load of 40 kN and wheel speed of 1.8 km/h

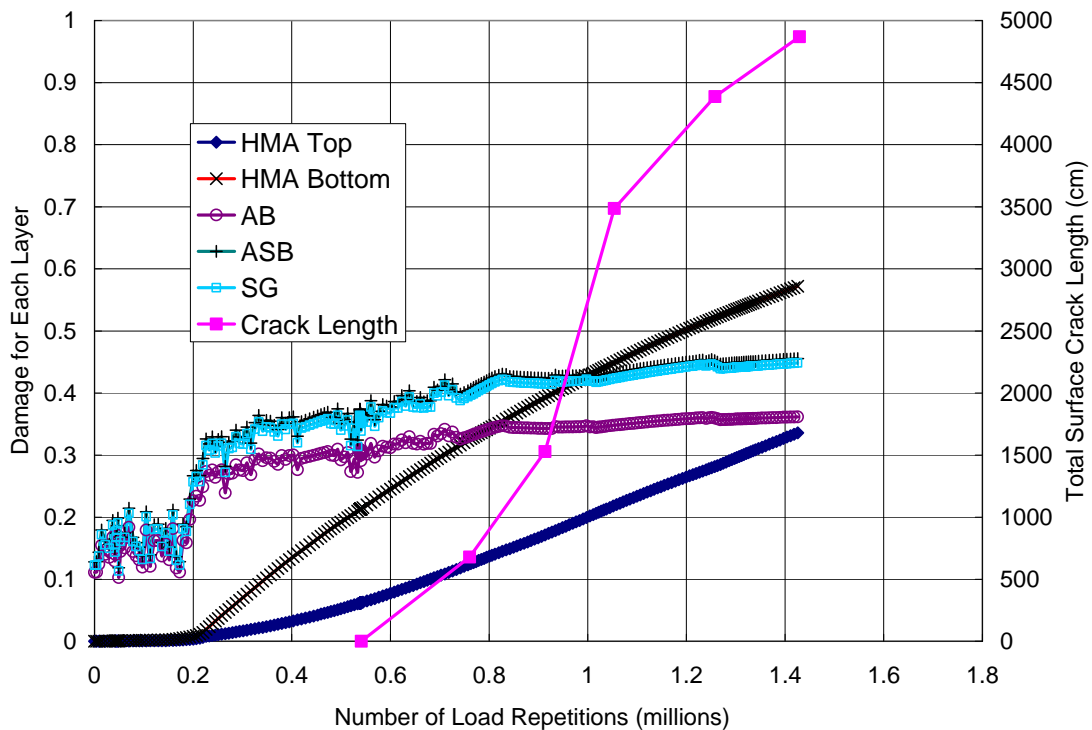


Figure 11. Layer damage and observed surface cracking length history

## 4 CONCLUSIONS

This paper presents the subsystem of *CalME* for predicting fatigue damage and an example of its calibration using surface deflection data collected from Heavy Vehicle Simulator (HVS) tests. Stiffness data from 4PB flexural frequency sweep data are used to identify master curve model parameters, while stiffness reduction curves from 4PB flexural fatigue tests are used to identify fatigue damage model parameters. This allows *CalME* to implement an incremental-recursive mechanistic empirical (IRME) design method in which stiffness of all layers (including HMA) are updated as the pavement life progresses.

With an IRME approach, surface deflections calculated by *CalME* increases as truck traffic accumulates. Calibration using data collected from HVS tests indicates that the surface deflections predicted by *CalME* matched the ones measured by multi-depth deflectometers (MDD) very well. The same approach can be used for calibration using test track data with strain gauges and where stiffnesses are back-calculated from deflection data as the loading progresses. It is believed that *CalME* provides a practical tool for simulating complete performance history of flexible pavements.

To make the fatigue subsystem complete, one needs to establish a proper relationship between fatigue damage and surface cracking density. This can be done with either a deterministic or a probabilistic approach.

## 5 ACKNOWLEDGEMENTS

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