Process of Developing a Mechanistic-Empirical Asphalt Pavement Design System for California

P. Ullidtz
Dynatest International, Copenhagen, Denmark

J. Harvey
University of California Pavement Research Center, Davis, California, USA

I. Basheer
California Department of Transportation, Sacramento, California, USA

R. Wu
University of California Pavement Research Center, Davis, California, USA

J. Lea
University of California Pavement Research Center, Davis, California, USA

ABSTRACT: Mechanistic-Empirical (ME) asphalt pavement design systems have two components: a mechanistic model for calculating the critical primary responses (stresses, strains, and displacements) in the pavement, and empirical models which relate the calculated responses to the pavement distress or performance, in terms of cracking, rutting, and surface roughness. When developing an ME design system it is of utmost importance to pay considerable attention to both of these components. This paper describes the process of developing an effective flexible pavement design system, based on the authors' experience from development of the flexible pavement design system known as *CalME,* for the California Department of Transportation (Caltrans). This paper also outlines the process used in developing both the primary response model and distress prediction models (fatigue cracking, rutting, and roughness) based on the incremental-recursive damage approach of the California ME asphalt pavement design system. The paper also presents the benefits of considering the mechanistic and empirical components separately, and using the two types of accelerated pavement testing facilities (Heavy Vehicle Simulator and test tracks) in the mechanistic model validation and distress prediction calibration efforts.

KEY WORDS: Mechanistic-empirical, incremental-recursive, full scale testing, calibration, validation.

1 INTRODUCTION

As the name implies, a Mechanistic-Empirical (ME) asphalt pavement design system has two components: a mechanistic (or analytical) model for calculating the critical stresses and strains in the pavement, and empirical models, relating the response thus calculated, to the pavement performance, in terms of cracking, rutting, and surface roughness. When developing an ME design system it is of utmost importance to pay attention to both of these components. It is also important to realize that a pavement constantly undergo changes, as a consequence of changes in temperature and moisture, age hardening and damage. This means that the pavement response constantly changes throughout the life of the pavement. To track these changes in pavement response, an incremental-recursive method may be used, where the increase in damage is calculated for a short increment in time, and the new pavement condition is used, recursively, as input to the next time increment.

This paper describes the process of developing such a design system, based on the experience gained to date in connection with development of the design system known as *CalME*, for the California Department of Transportation (Caltrans). The development relied heavily on the two Heavy Vehicle Simulators (HVSs) owned by Caltrans, and used for verification of the mechanistic primary response model as well as for the calibration of the empirical performance models. The California HVS tests were complemented by track tests, such as WesTrack, NCAT (National Center for Asphalt Technology), CEDEX (Centro de Estudios y Experimentación de Obras Públicas) and MnRoad and some HVS testing by the Swedish National Road and Transport Research Institute.

A guideline for the development process has been to start from the simplest possible models and only accept more complex models if the added complications resulted in a significant improvement to the system.

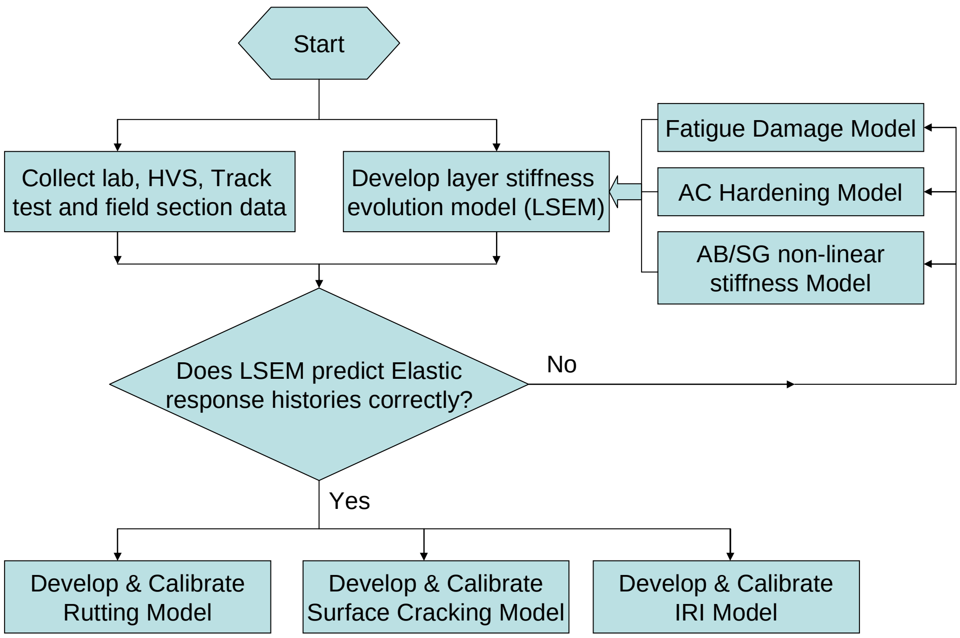

Figure 1 Flow chart for model development.

The first step in the development process is to ensure that the mechanistic model is capable of predicting the pavement primary responses at any point in time during the life of the pavement. This requires that the moduli of the pavement layers, and their changes as functions of time, loading, temperature, stress conditions etc. are known. The first step, therefore, involves the mechanistic model as well as the layer stiffness evolution models (LSEM in Figure 1).

Accelerated pavement tests (APT) may be used for this purpose if the test sections are instrumented so that the pavement response can be monitored throughout the duration of the experiment. If the response cannot be correctly predicted, the models used to provide the elastic parameters (i.e. the inputs to the mechanistic model) or the mechanistic model itself, must be adjusted, until agreement is obtained. There is no purpose in trying to adjust the empirical models to an incorrectly predicted response. The paper will present examples where it has been necessary to depart from commonly accepted models, in order to enable prediction of the actual pavement response.

It is evident that this first step also implies the modification/verification of asphalt fatigue damage models, as the damage, which reduces the stiffness, is part of the LSEM and influences the response. Once the correct response has been achieved, the empirical models for permanent deformation (rutting) and for relating visual cracking to fatigue damage, may be adjusted/calibrated.

It is crucial that the models for permanent deformation relate to the actual mechanism causing this deformation. It was found that for asphalt layers most of the permanent deformation (except for a relatively small amount of traffic compaction) is related to critical shear stress and shear strain in the upper part of the asphalt layer.

Pavement roughness is a function of variability of structure, materials and loads. With no variability the pavement may crack and rut, but it will be evenly and uniformly, and will not result in a rough ride quality. The model for pavement roughness is still under development by the authors and has not yet been verified, but the basic elements of the proposed model will be presented in this paper.

In the subsequent sections of the paper, a brief description of the various components needed to build the California ME flexible pavement design system is presented.

2 MECHANISTIC MODEL

The mechanistic model (called analytical model in Europe) is a theoretical, mathematical model for calculating the pavement primary response (stress, strain, displacement) under load. All existing models used for pavement design are based on solid mechanics (continuum mechanics) but pavement materials are not solid. They consist of solid grains, liquids and air, and how they deform under a load can be quite different from the deformation of solid materials. Even if the Distinct Element Method, DEM (Cundall, 1978) could be developed to a point where it could be used for pavement design, it would still require many simplifications with respect to reality.

Because reality differs from the assumptions on which the theoretical model is based it is necessary to verify the theoretical model and, if needed, to modify it to better reflect reality. One of the complications that cannot be avoided, if pavement layer moduli are to be backcalculated from deflections measured with a Falling Weight Deflectometer (FWD), is the non-linearity of the subgrade. Ignoring the non-linearity and treating all layers as linear elastic will often produce the “inverted layer” problem, where a crushed aggregate base apparently has a much lower modulus than a soft clay subgrade. One of the products from the development of the ME pavement design system for Caltrans was a backcalculation program called CalBack (Lu et al., 2009), which includes an Odemark-Boussinesq option with a non-linear subgrade, with decreasing modulus for increasing deviator stress.

It is well known that the moduli of granular materials are also non-linear. Triaxial tests show the modulus to be increasing with increasing bulk stress. The same phenomenon is observed from deflection measurements with Multi Depth Deflectometers (MDDs) during HVS testing. As the load was increased the stiffnesses of the granular layers also increased. This non-linearity should be considered either in the mechanistic model or in the LSEM, but there are other effects, not easily modeled using continuum mechanics. The non-linearity of granular materials should result in increasing moduli, when the modulus of an upper asphalt layer is decreasing, either as a result of increasing temperature or increasing damage, because the bulk stress in the granular layers would be increasing. The opposite phenomenon has, however, been repeatedly, although not always, observed both from MDD measurements during HVS testing and from FWD testing, indicating that the moduli of unbound layers tend to increase with increasing confinement from the layers above.

Figure 2 shows an example of backcalculated moduli of unbound layers (AB: aggregate base, SG: subgrade) as a function of the stiffness of the upper layers, *S*, calculated as indicated in Equation (1).

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

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

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 parameters of Equation (1) may be determined from the best fitting lines, $y = a \times x + b$, of Figure 2, through $Eo = a + b$ and $Stiffness\ factor = a/(a + b)$.

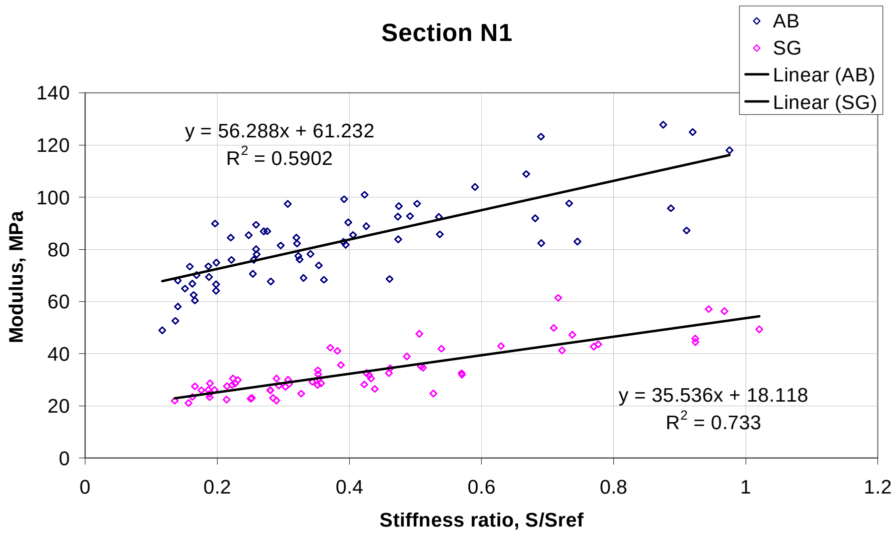

Figure 2 Moduli of unbound layers as a function of the stiffness of upper layers.

During HVS testing a rather large increase in deflection is normally observed. Figure 3 shows an example of deflections measured with an MDD during an HVS test. Deflections were measured under a 40 kN load at the surface (M0), at the top of the base (M137), and at the top of the subgrade (M640). During the first 0.6 million loads the surface deflection is seen to more than double, from about 0.3 mm to about 0.8 mm. This increase is caused by micro-cracking in the asphalt layer. The first visible, hairline crack was observed after 0.6 million load applications. If only the initial pavement condition is used with the mechanistic model, the predicted pavement response will be very incorrect for most of the experiment. It is essential that the changes to the pavement materials due to damage are taken into consideration, as explained in the following. The resulting deflections calculated with *CalME* are shown by dotted lines in Figure 3 for the three depths (C0, C137 and C640), and are seen to be reasonably close to the measured deflections.

Although the above mentioned non-linearities and effects of confinement and of damage, complicate the mechanistic model and/or the LSEM, they cannot be ignored. They must be taken into consideration for the model to produce reasonably correct response values for the duration of the experiment, or for the life of the pavement. Another complication may be due to debonding between asphalt layers, which may have a pronounced effect on the pavement response. For rehabilitation of old pavements, cracking in the existing asphalt layer or joints in a Portland Cement Concrete (PCC) layer, will produce strains in the new overlay that are quite different from the strains calculated using layered elastic theory. The California ME pavement design system (*CalME*) has models to consider debonding and to simulate the strains at joints and cracks that cause reflection cracking.

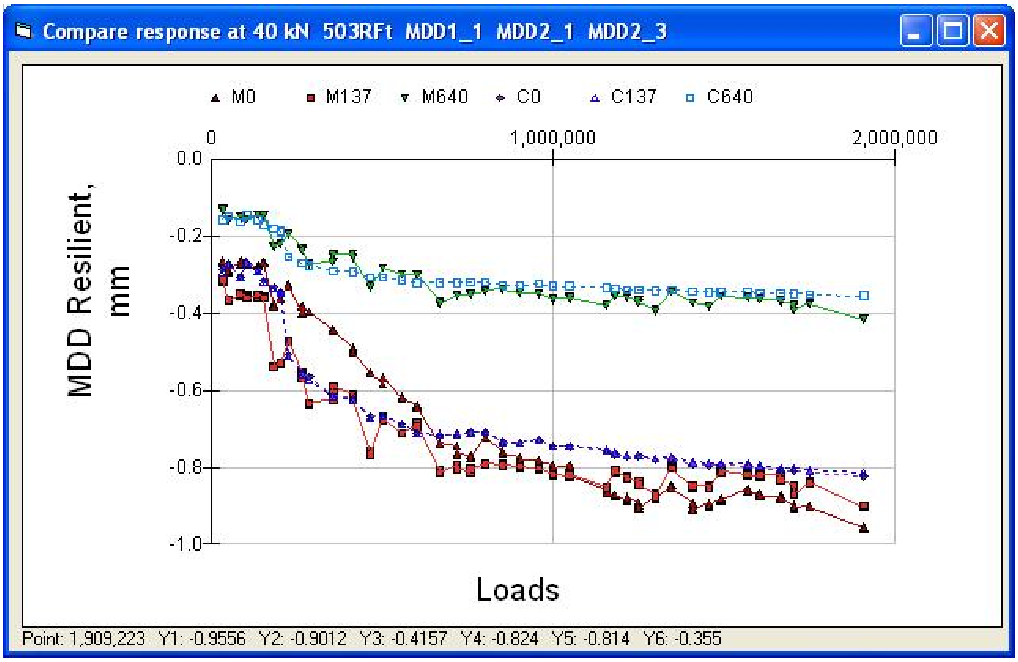

Figure 3 Measured and calculated deflections during HVS tests.

3 EMPIRICAL RELATIONSHIPS

For each time increment in the simulation, the moduli of the pavement layers are first determined using the LSEMs. The moduli may depend on the temperatures at different depths, the vehicle speed, seasonal variations with moisture or freeze/thaw, load level, confinement, ageing and any previous damage to the materials. The mechanistic model is then used to calculate the pavement response at critical locations, for each wheel load at each lateral location, and these response values are finally used with empirical relationships to determine the increase in damage or permanent deformation of the layers.

3.1 Fatigue Damage

As illustrated by Figure 3 the damage caused by fatigue of bitumen or cement bound materials must be considered in the simulation of both response and performance. For asphalt concrete, it has been found that a sigmoidal relationship may be used to describe the master curve:

$$\log(E) = \delta + \frac{\alpha \times (1-\omega)}{1+\exp(\beta + \gamma \log(tr))} \tag{2}$$

where E is the modulus,

tr is reduced time,
α, β, γ, and δ are constants determined from frequency sweep or FWD tests, and
ω is the damage calculated from:

$$\omega = \left(\frac{MN/SF}{MN_p} \right)^{\alpha}, \quad MN_p = A \times \left(\frac{\mu\varepsilon}{\mu\varepsilon_{ref}} \right)^{\beta} \times \left(\frac{E}{E_{ref}} \right)^{\gamma} \times \left(\frac{E_i}{E_{ref}} \right)^{\delta} \tag{3}$$

where MN is the number of load applications in millions,
SF is the shift factor from laboratory to in situ loading,
$\mu\varepsilon$ is the tensile strain at the bottom of the asphalt layer,
E_i is the intact modulus (ω = 0),
$\mu\varepsilon_{ref}$ and E_{ref} are reference constants, and
A, α, β, γ, and δ are constants (not related to the constants of Equation (2)).

The constants α, β, γ, and δ in Equation (2) may be determined from frequency sweep tests in the laboratory or from FWD tests at different temperatures, and the constants A, α, β, γ, and δ of Equation (3) may be determined from laboratory flexural beam fatigue tests through a least squares method. (An example is shown in a companion paper on simulation of a CEDEX test section). With no damage (ω = 0), Equation (2) describes the master curve for intact asphalt and for a damage ω = 1, the asphalt reaches its lowest modulus value ($E_{min} = 10^{\delta}$). The shift factor, SF in Equation (3), is determined from the HVS or test track experiment.

This damage model has been found to be capable of predicting asphalt damage during HVS tests with constant temperatures very well. For experiments with large seasonal temperature variations, or for in-situ road pavements, there appears to be a need for consideration of additional effects. As with all other ME design systems, the above model tends to predict most damage at high temperatures and least damage at low temperatures, which is in conflict with some field observations. The HVS experiments must, therefore, be supplemented by full scale track testing.

For materials stabilized with cement and/or foamed asphalt or asphalt emulsion and for other recycled materials containing binders, models for the decrease in modulus, based on the compressive stress or strain at the top of the layer, or on the tensile stress or strain at the bottom of the layer, may be used.

Ageing is important for both bitumen and cement bound materials in test tracks and the field. CalME includes ageing (i.e. stiffening of the elastic parameters) models for both types of materials. Some consideration of ageing effects on damage models can be included but have not been calibrated at this time.

3.2 Relationship between Damage and Visual Cracking

As mentioned above, a large amount of damage may take place, in the form of micro-cracking, before visible macro-cracks can be observed. For a number of full scale tests, it has been found that the damage at crack initiation could be determined as a function of the thickness of the asphalt layers, and that crack propagation could be determined using a sigmoidal function of the damage as follows:

$$\omega_i = \frac{A}{1 + \left(\frac{h_{AC}}{h_{ref}} \right)^{\alpha}}; \quad Cr = \frac{Cr_{\max}}{1 + \left(\frac{\omega}{\omega_o} \right)^{\beta}} \tag{4}$$

where ω_i is the damage at crack initiation,
h_{AC} is the combined thickness of the asphalt layers,
h_{ref} is a reference thickness,
Cr is the cracking (as % of wheel path, m/m^2 crack length to area ratio, or some other measure)
Cr_{max} is the maximum amount of cracking, and
A, α, β and ω_o are constants determined using data obtained from full scale tests.

3.3 Permanent Deformation

Permanent deformation of asphalt layers may be due both to post-construction compaction of the material under traffic and to shear deformation. Traffic compaction is normally limited to a few percent of the thickness of the asphalt layer whereas shear deformation is more serious and may continue to develop. The shear deformation has been found to be related to the shear stress and the shear strain at a depth of about 50 mm under the edge of the tire (Monismith et al., 2000). The resistance to shear deformation has been quantified using a Repeated Simple Shear Test at Constant Height (RSST-CH) in the laboratory, although other repeated load permanent deformation tests could be used, and the models recalibrated. For many materials, the permanent or inelastic shear strain, γ_i, may be described by a gamma function of the number of load repetitions:

$$\gamma_i = \exp\left(A + \alpha \times \left[1 - \exp\left(-\ln(N)/\gamma\right) \times \left(1 + \ln(N)/\gamma\right)\right]\right) \times \exp\left(\beta \times \tau/\tau_{ref}\right) \times \gamma_e \qquad (5)$$

where γ_i is the inelastic shear strain,
N is the number of load applications,
τ is the shear stress,
τ_{ref} is a reference shear stress,
γ_e is the elastic shear strain, and
A, α, β and γ are constants determined from the RSST-CH test.

The RSST-CH tests should be done on samples having an air voids content corresponding to the traffic compacted condition of the material. The permanent deformation, dp, due to shearing may then be related to the inelastic shear strain through:

$$dp\ (mm) = K \times \sum h_i \times \gamma_i^i \qquad (6)$$

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

For unbound materials the permanent deformation, dp, has been found to be related to the vertical strain at the top of the layer:

$$dp\ (mm) = A \times MN^{\alpha} \times \left(\frac{\mu\varepsilon}{\mu\varepsilon_{ref}}\right)^{\beta} \times \left(\frac{E}{E_{ref}}\right)^{\gamma} \qquad (7)$$

where MN is the number of load applications in millions,
$\mu\varepsilon$ is the vertical, compressive strain at the top of the layer,

$\mu\varepsilon_{ref}$ is a reference strain,
E is the modulus of the material,
E_{ref} is a reference modulus, and
A, α, β and γ are constants.

To date, no laboratory method has been used for determining the constants of Equation (7), and full scale testing has been used for characterization. It has been found that the same parameters could be used for many different materials, except for materials where the moisture content increased significantly during the testing.

3.4 Surface Roughness

Pavement roughness is due to variability of the pavement structure, of the pavement materials or the loads. With no variability in layer thicknesses, pavement materials or loads, the pavement may crack and rut, but it will never become rough.

At the AASHO Road Test (1962), roughness was determined as the Slope Variance (SV) over a distance of one foot. This may be calculated as:

$$\begin{aligned} SV &= \frac{1}{N-2} \times \sum \left[\frac{(rd_{i+1} - rd_i)}{0.3} \right]^2 \\ &= \frac{1}{0.3^2 \times (N-2)} \sum \left[(rd_{i+1} - \mu)^2 + (rd_i - \mu)^2 - 2 \times (rd_{i+1} - \mu) \times (rd_i - \mu) \right] \\ &\cong \frac{2 \times \sigma^2}{0.09} \times (1 - \rho) \end{aligned} \qquad (8)$$

where rd_i is the rut depth in mm at point i,
μ is the average rut depth in mm,
σ is the standard deviation of the rut depth in mm, and
ρ is the autocorrelation coefficient for a distance of 300 mm.

A Monte Carlo simulation technique has been incorporated in *CalME* to allow the user to determine the standard deviation of the rut depth as a function of the variability of various parameters that affect performance (e.g. traffic, climate, construction and material variability, etc.). However, the roughness model has not yet been calibrated, and it would require a method for determining the autocorrelation coefficient. It is believed, however, that roughness must necessarily be related to variability.

The Monte Carlo simulation may also be used to calculate the propagation of cracking or rutting, which may provide a within project reliability.

4 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 of the equations may change from increment to increment. The first step in the process is, therefore, to calculate the "effective" number of load applications that would have been required, with the present parameters, 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 process is illustrated in Figure 4 where three different damage curves are shown, corresponding to different values of strain, modulus, temperature or other conditions. For the first 4000 load repetitions, applied at what corresponds to the "middle" condition, the increase in damage is simply calculated using the "middle" relationship. If the next 4000 loads are applied at conditions corresponding to curve 2, the number of load applications that would have been required, at this condition, to achieve the present damage, must first be determined. At condition 2, 9000 load applications would have been needed to produce the existing damage of 0.095, rather than the 4000 loads at condition 1. For condition 2 the increase in damage is calculated for the loads from 9000 to 13000.

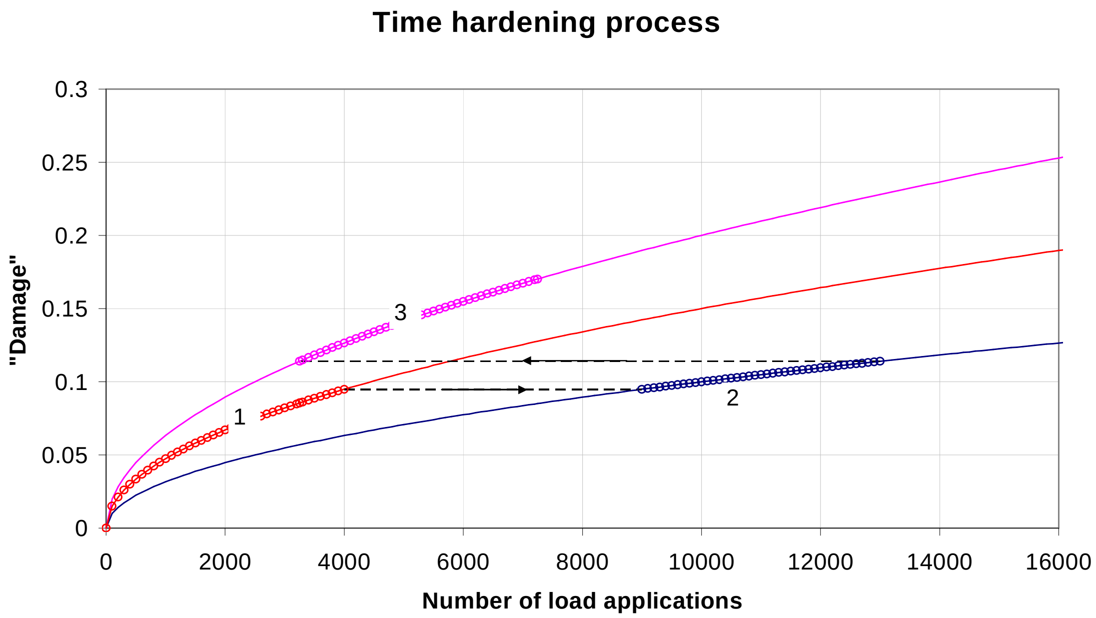

Figure 4 Illustration of time hardening process.

5 CALIBRATION AND VERIFICATION USING FULL SCALE TESTING

Full scale tests are essential for verification of mechanistic component of the ME design system and calibration of the empirical component of the system. In order to ensure successful verification and calibration efforts, a set of ideal conditions must be maintained and efficient instrumentation must be implemented in full scale tests. The following briefly discuss a few issues pertaining to full scale testing that can play a major role in the development of an effective ME pavement design system.

In order to verify that the pavement response calculated with the mechanistic model is reasonably close to the actual response in the pavement, it is necessary to instrument the pavement with gauges and to measure the pavement response during the experiment. This is not a trivial matter, as the presence of a gauge tend to change the pavement response, and the aim is to determine the response that would have been there without the gauge. Gauges also need to be sufficiently sturdy to survive being installed in pavement layers and the subsequent environment and loading.

Multi-depth deflectometers (MDDs) have been found to be very useful for HVS testing. Not only do they provide the deflections at several depths under the rolling wheel load, from which the elastic compression of the individual layers may be determined, but they also provide the permanent deformations, again with the possibility of calculating the contribution from each individual layer. Strain gauges or stress cells embedded in the pavement structure during construction are also very useful. Once cracking develops in a layer, however, it may have a pronounced influence on the measured values, depending on the location of the gauge with respect to the cracks.

Characterization of the elastic parameters of the pavement materials, before any loading is applied, is essential. The experimental sections should be tested with an FWD at a range of temperatures and with a density of testing providing at least twenty test points for each section, in order to determine both the master curve of the asphalt, the effects of load level and of confinement on unbound materials and the distribution of the parameters. A section of pavement that will not be subjected to loading should also be tested, and followed during the experiment in order to evaluate the effects of ageing. It is very important that the effects of temperature, ageing and damage can be separated. If possible, FWD testing should also be carried out frequently during the experiment (impossible under an HVS) or other means of determining the moduli, such as wave propagation or Light Weight Deflectometers (LWDs), should be used to monitor the changes in elastic parameters.

During the experiment, the temperature should be measured frequently, at least hourly, at several depths, and the changes in moisture content of the unbound materials should be recorded. At regular intervals, the longitudinal and transverse pavement profile should be measured and any visible distress at the pavement surface must be recorded. At the completion of the experiment, a forensic study should be carried out to identify the origin of any damage or deformation.

6. SIMULATION OF EXPERIMENT AND EXTENSION TO IN SITU CONDITIONS

CalME has a facility for importing the results of the experiment, in terms of hourly loads, temperatures, response measurements, backcalculated moduli, permanent deformations and cracking into the database. The experiment may then be simulated on the computer. This considerably facilitates modifications and calibrations. The first step in this “virtual” experiment is to make sure that the pavement response is correctly predicted. Once this is achieved, the empirical models may be calibrated.

This facility is also useful when analyzing test track or APT comparison studies, for example when “identical” sections have been built to compare different materials. Comparison sections are never identical due to construction, time of testing, subgrade and other variability. Once the models are calibrated to match the “as-tested” results, simulations can be run under identical conditions (except for the experiment variables) to produce “virtual” results for comparison under same conditions.

HVS tests and track tests are situated somewhere between experiments in the laboratory on small samples and the reality of real highway pavements, but they are still simplifications of reality. An HVS or other APT test is a large scale laboratory test, with detailed control of pavement structure, loads and climate, and is recommended as the first step of the calibration process. It must, however, be complemented by full scale track testing, with longer duration, more realistic loads and real climatic conditions, as a second step. Using in situ pavement sections directly for calibration of the models is almost impossible. Even for experimental pavement sections the level of data detail and precision is seldom sufficient for a direct calibration. But as a third step in the calibration process they are indispensable, although this final step in the “calibration” must rely, to a large extent, on engineering judgment.

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