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Development of the *CalME* Standard Materials Library

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Partnered Pavement Research Center (PPRC) Contract Strategic Plan Element 3.18.1:
Update *CalME* Standard Materials Library

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16. ABSTRACT

The main purpose of the project is to improve the ability of Caltrans pavement designers to use mechanistic-empirical (ME) pavement design procedures that were developed and calibrated for California conditions as part of Partnered Pavement Research Center Strategic Plan Element (PPRC SPE) 4.1 and refined in SPE 3.4. Specifically this project is part of a long-term series of tasks to collect regional materials data for use by Caltrans in ME flexible pavement designs and rehabilitations. This technical memorandum documents the current state of the Standard Materials Library (SML) for CalME, the ME analysis and design computer program for flexible pavements developed by the UCPRC for Caltrans. This technical memorandum summarizes the role of the SML in the ME design process, the classifications of different pavement materials, the relevant models applicable to each material group, and the processes for identifying various model parameters. The technical memorandum also describes how construction variabilities are accounted for in the SML. A full list of materials included in the SML at the completion of this task in 2014 is presented, including the research conducted up to that time on the materials and how their model parameters were determined.

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8. RELATED DOCUMENTS
Zhou, J, J. M. Signore, J. T. Harvey. 2013. Superpave Implementation Phase I: Determining Optimum Binder Content (UCPRC-TM-2012-03)

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PROJECT GOAL AND OBJECTIVES

The goal of this project, SPE 3.18.1, "Update *CalME* Standard Materials Library," was to expand the database of standard materials in the *CalME* Mechanistic-Empirical design system by means of field sampling, to perform laboratory and field testing to characterize selected materials, and to develop performance model coefficients. To achieve this goal, this project included the following tasks:

- Identify materials to be added to the updated Standard Materials Library
- Characterize selected asphalt-bound materials through laboratory testing
- Characterize selected non-asphalt-bound materials through field testing
- Review repeated load triaxial (RLT) testing (modified AASHTO TP 79) as a substitute for the repeated simple shear test at constant height (RSST-CH, AASHTO T 320) for characterizing the rutting performance of asphalt-bound materials
- Develop performance model coefficients for selected materials
- Update the Standard Materials Library in *CalME*

This technical memorandum presents results from each of the above tasks except the review of the RLT. In addition, the materials presented in this technical memorandum also include the materials characterized in SPEs 4.1 and 3.4. The findings from the RLT review are presented in a separate report: Superpave Implementation Phase II: Comparison of Performance-Related Test Results (UCPRC-RR-2015-01).

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LIST OF ABBREVIATIONS

AB	Aggregate base
AC	Asphalt concrete
AS	Aggregate subbase
APT	Accelerated pavement testing
ATIRC	Advanced Transportation Infrastructure Research Center
ATPB	Asphalt-treated permeable base
CalME	California Mechanistic-Empirical
CFIPR	Cold foam in-place recycling
CIR	Cold In-place Recycling
COV	Coefficient of variance
CTB	Cement-treated Base
CTE	Coefficient of thermal expansion
CTPB	Cement-treated permeable base
DMI	Distance measurement instrument
EICM	*Enhanced Integrated Climatic Model* software
FDR	Full-depth reclamation
FHWA	Federal Highway Administration
FMLC	Field-mixed, laboratory-compacted
FSF	Fatigue shift factor
FWD	Falling weight deflectometer
GPR	Ground penetrating radar
HDM	Highway Design Manual
HMA	Hot mix asphalt
HVS	Heavy Vehicle Simulator
ICM	Integrated Climatic Model
IRI	International Roughness Index
LCB	Lean concrete base
LMLC	Laboratory-mixed, laboratory-compacted
LTS	Lime-treated subbase
MDTP	Mixture Design and Testing Protocol
ME	Mechanistic-Empirical
MEPDG	Mechanistic-Empirical Pavement Design Guide
NCHRP	National Cooperative Highway Research Program
PCC	Portland cement concrete
PG	Performance grade
PPRC	Partnered Pavement Research Center
RAP	Reclaimed asphalt pavement
RHCA	Recycled hardened concrete aggregate
RHMA	Rubberized hot mix asphalt
RLT	Repeated load triaxial
SDF	Standard deviation factor
SML	Standard Materials Library
STOA	Short-term oven aging
UC	University of California
UCPRC	University of California Pavement Research Center
USCS	Unified Soil Classification System

1 INTRODUCTION

1.1 Purpose and Background

The California Department of Transportation (Caltrans) is transitioning from using an empirical method of flexible pavement design and rehabilitation to a mechanistic-empirical (ME) method. One of the major benefits of the ME method is the capability it provides to account for conditions that vary by region, such as climate, traffic, and pavement materials. Making this ME method work most effectively requires an extensive collection of regional climate, traffic, and materials data from the different parts of California so that representative categories for each condition can be determined. The project described in this technical memorandum is part of a continuing effort by the University of California Pavement Research Center (UCPRC) to collect regional materials data for Caltrans to use in its ME flexible pavement designs and rehabilitations.

In order to use a specific material in ME design, the material must first be characterized by testing in either the laboratory or the field. For routine designs it is not financially feasible to perform a complete project-specific set of material tests to characterize the materials before the project is designed. That is why a *somewhat* comprehensive library of pre-characterized materials representative of California's various regions is essential for maximizing the utility of ME design. In addition, for large projects it is recommended that project-specific materials be characterized in order to validate the performance prediction for the chosen design. In these cases an available library with characterizations of typical materials specific to the project region can help designers evaluate preliminary designs even before any project-specific materials become available.

A comprehensive materials library can also help establish reasonable performance criteria when developing performance-based construction specifications for asphalt-bound materials. Since ME design takes into account material performance, construction specifications can then include requirements for material performance. A materials library can help establish reasonable and achievable performance criteria based on true materials testing data.

The materials library being developed by the UCPRC for Caltrans ME design is referred to as the *Standard Materials Library* (SML). The main purpose of this current project is to continue expansion of the SML that started in Partnered Pavement Research Contract (PPRC) Strategic Plan Element (SPE) 4.1, "Development of the First Version of a Mechanistic-Empirical Pavement Design," and was extended and refined from 2004 to 2008 in SPE 3.4, "ME Design Implementation," which ran from 2008 to 2011. The current project has run for three years, from 2011 to 2014 under SPE 3.18.1, "Update *CalME* Standard Materials Library." This project is closely related to SPE 3.18.3, "Superpave Implementation," in terms of the asphalt-bound material selection. This technical memorandum presents an overview of the Standard Materials Library resulting from SPEs 4.1, 3.4, and 3.18.1.

1.2 Caltrans Implementation of ME Design

The UCPRC has developed a computer program called *CalME* (California Mechanistic-Empirical) to enable Caltrans to implement the ME design method. *CalME* 2.0, the latest version of the program, was released to Caltrans district design offices in September 2014. The Standard Materials Library described in this technical memorandum is the one included in the *CalME* 2.0 installation package, in an MS *Access* database file named *CalME.mdb*.

1.3 Goals, Objectives, and Deliverables

The goal of this project, SPE 3.18.1, "Update *CalME* Standard Materials Library," was to expand the database of standard materials in the *CalME* Mechanistic-Empirical design system by means of field sampling, to perform laboratory and field testing to characterize selected materials, and to develop performance model coefficients. To achieve this goal, this project included the following tasks:

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This technical memorandum presents results from each of the above tasks except the review of the RLT. In addition, the materials presented in this technical memorandum also include the materials characterized in SPEs 4.1 and 3.4. The findings from the RLT review are presented in a separate report: Superpave Implementation Phase II: Comparison of Performance-Related Test Results (UCPRC-RR-2015-01).

2 GENERAL APPROACH

The Standard Materials Library should include most if not all of the materials typically used in flexible pavements. This chapter first presents a brief description of the ME design process and the role of material characterization. Next, it describes how the different materials are classified, what material models are needed, and how the model parameters are identified in *CalME* 2.0.

2.1 ME Design Process and Material Characterization

2.1.1 ME Design Process and CalME 2.0

ME design is an iterative process in which trial pavement designs are adjusted repeatedly either manually or automatically based on predicted performance until an optimal design is reached. A key component of any ME design system is a module that predicts the performance of a given pavement design. This module and the pavement distresses included in it can vary from one ME design system to another, depending on the specific project. In *CalME* 2.0, the module's predicted distresses include fatigue cracking, reflective cracking, rutting, and smoothness. It is expected that in future versions of *CalME* additional pavement distresses will be added.

CalME uses an incremental-recursive performance prediction process. Figure 2.1 shows a flowchart of this process and it illustrates both the "incremental" and the "recursive" parts of the module. Specifically, "incremental" refers to the part of the process where pavement performance is predicted for each time increment and "recursive" refers to the part where the pavement condition is updated using the distress states (or levels) predicted for the preceding time increment before the incremental pavement distresses are predicted for the next time increment.

CalME uses Monte Carlo simulation for evaluating the statistical reliability of a given pavement design. Essentially, *CalME* generates a set of random pavement structures that together provide a representative sample of the as-built structures for a given pavement design. This accounts for the construction variability. In addition, a designer can elect to include the uncertainties associated with predicting future climate conditions. *CalME* then uses the process shown in Figure 2.1 to predict the performance of each individual pavement structure with the corresponding climate condition and uses the performance statistics to determine the reliability of the given design.

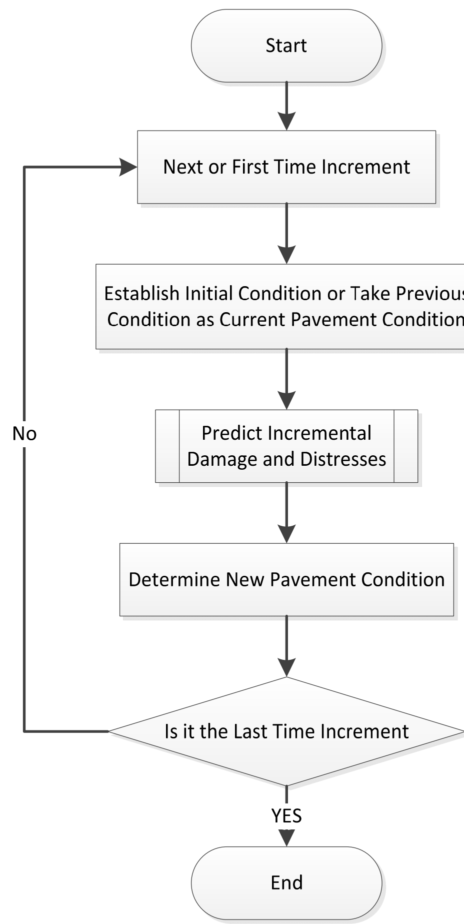

Figure 2.1: Flowchart of the incremental-recursive performance prediction process used in *CalME.*

2.1.2 Roles of Material Characterization in CalME

As shown in Figure 2.1, a key part of the incremental-recursive performance prediction process is the subprocess that predicts incremental damage and distresses. This subprocess is referred to as the *incremental damage prediction process,* which applies the environmental and traffic loading for the given time increment and predicts the incremental damage (loss of stiffness or permanent deformation) and resultant change in distresses in the pavement. This subprocess involves interaction between material characterization and the other components of the ME design, as illustrated in Figure 2.2.

As shown in Figure 2.2, there are three levels of damages or distresses predicted by *CalME:* primary, secondary, and tertiary. The primary distresses are damages such as fatigue damage, reflective cracking damage, and permanent deformation in each layer, which do not depend on other distresses. The secondary distresses are the ones that depend on primary distresses, while tertiary distresses are the ones that depend on primary and/or secondary distresses. For example, surface cracking is a result of fatigue damage and reflective cracking damage and therefore it is a secondary distress. Similarly, surface rutting is a function of layer permanent deformations and therefore it is also a secondary distress. In *CalME*, IRI is a function of surface rut variability and surface cracking, and therefore is a tertiary distress.

Figure 2.2 indicates that material characterization is not involved in the predictions of the secondary and tertiary distresses in *CalME* 2.0. Instead, these distresses can be determined based on a damage value alone, no matter what materials are used in the pavement. The role of material characterization is to provide models for predicting pavement conditions (temperature, moisture contents, etc.), critical primary responses (stress, strain, and/or deformation at critical locations in the pavement that are related to distress development), and the resulting primary distresses.

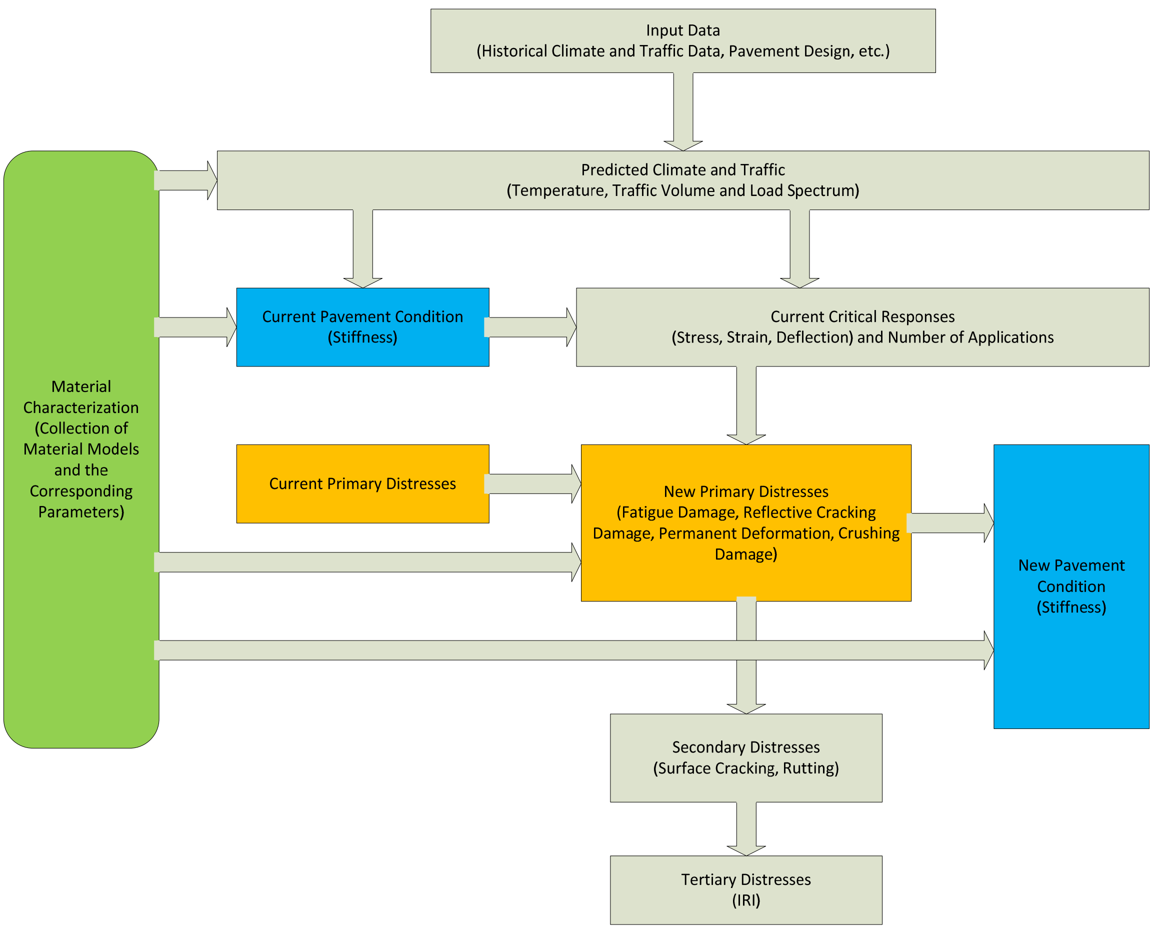

Figure 2.2: Interaction between material characterization and other components of the incremental distress prediction process for *CalME* 2.0.

In *CalME* 2.0, pavement structures are simplified as multilayer elastic systems when calculating critical responses for predicting fatigue damage and permanent deformation. Accordingly, pavement responses only depend on layer stiffnesses since the Poisson's ratio of each material in the pavement structure is assumed to remain constant throughout the analysis life. In order to calculate the strain that drives reflective cracking damage in the new asphalt layer (e.g., overlay), joints and cracks in the underlying layer are introduced into the multilayer elastic system. The joint or crack characteristics such as spacing and opening width, however, are structural properties and not material properties.

Even without asphalt fatigue damage, which reduces stiffnesses, many important pavement materials do not have constant stiffnesses. For example, hot mix asphalt (HMA) stiffness depends on loading duration and HMA temperature. Similarly, subgrade stiffness typically demonstrates nonlinearity with respect to stress level, seasonal moisture content variation, and the freeze/thaw cycle. Fatigue damage and reflective cracking damage from traffic loading then add an additional element of change to the layer stiffness. Asphalt-bound material characterization describes how the stiffness of a material changes with loading duration and asphalt temperature as well as fatigue and reflection cracking damage.

Material properties also affect the prediction of environmental conditions for the pavement. Specifically, temperature profile in a pavement is affected by the thermal diffusivities of its layers. *CalME* 2.0 does not account for effects of any climate conditions other than temperature profile on the asphalt-bound material.

In summary, there are three groups of functions that material characterizations in *CalME* can potentially provide:

1) Environmental models: models that affect pavement response to environmental conditions, e.g., a heat transfer model that is used to determine pavement temperature
2) Stiffness models: models for layer stiffness given all of the potential relevant factors such as loading duration, material temperature, loading stress, time of the year, age, fatigue damage, etc.
3) Physical evolution models: models for changing the physical conditions of a material. These are the models needed for updating primary distresses/damage given all potential critical primary responses (stress, strain, deflection), the corresponding number of traffic load applications, and the current damage. Examples of physical evolution models include an asphalt mix fatigue damage model and a cement-treated material curing model. Note that physical evolution can include both damage and stiffening (such as aging and curing).

In essence, material characterization involves selecting the appropriate set of material models and identifying the corresponding model parameters through laboratory and/or field testing. Different types of materials require different materials characterization parameters for each of the above three functional groups of models. Accordingly, types of materials in the *CalME* Standard Materials Library can be classified into functional groups, each with its own type of material models and therefore their own materials characterization needs. The material models selected for *CalME* 2.0 for each of the functional groups are presented below along with the material classification.

2.2 Classification of Standard Materials for *CalME* 2.0

There are different ways to classify different pavement materials. For *CalME*, a decision was made to classify materials based on their mechanistic (stiffnesses, strain and stress-based damage models) and empirical (distresses) behaviors. Specifically, materials are classified by the sets of models needed to describe how they will perform in the ME design process. Figure 2.3 shows the unified modeling language (UML) diagram for hierarchical classification of materials used in *CalME* 2.0.

Figure 2.3 illustrates that the natural hierarchy of materials in the Standard Materials Library arises out of the relations between different material classifications. For example, Figure 2.3 shows that *asphaltic material* has an "is a" relationship with *pavement material*. In other words, *asphalt material* is a specialized type of *pavement material*. This implies that all models selected for *pavement material* (heat transfer and linear elasticity) are applicable to *asphaltic material* as well. In addition, asphaltic material has its own set of material models, including asphaltic stiffness master curve, asphaltic stiffness aging, etc. In *CalME, asphaltic material* has two stiffness models: the linear elasticity model inherited from the generic *pavement material*, and the asphaltic stiffness master curve model that is specific to the *asphaltic material*. The asphaltic stiffness master curve accounts for the effects of loading time and temperature, and provides the Young's modulus needed for the linear elasticity model.

Note that not all distress models are material-dependent and therefore distress models are not always used for material classification.

Further discussion of Figure 2.3 is presented in Section 2.2.1 through Section 2.2.6, which describe each material and the associated material models.

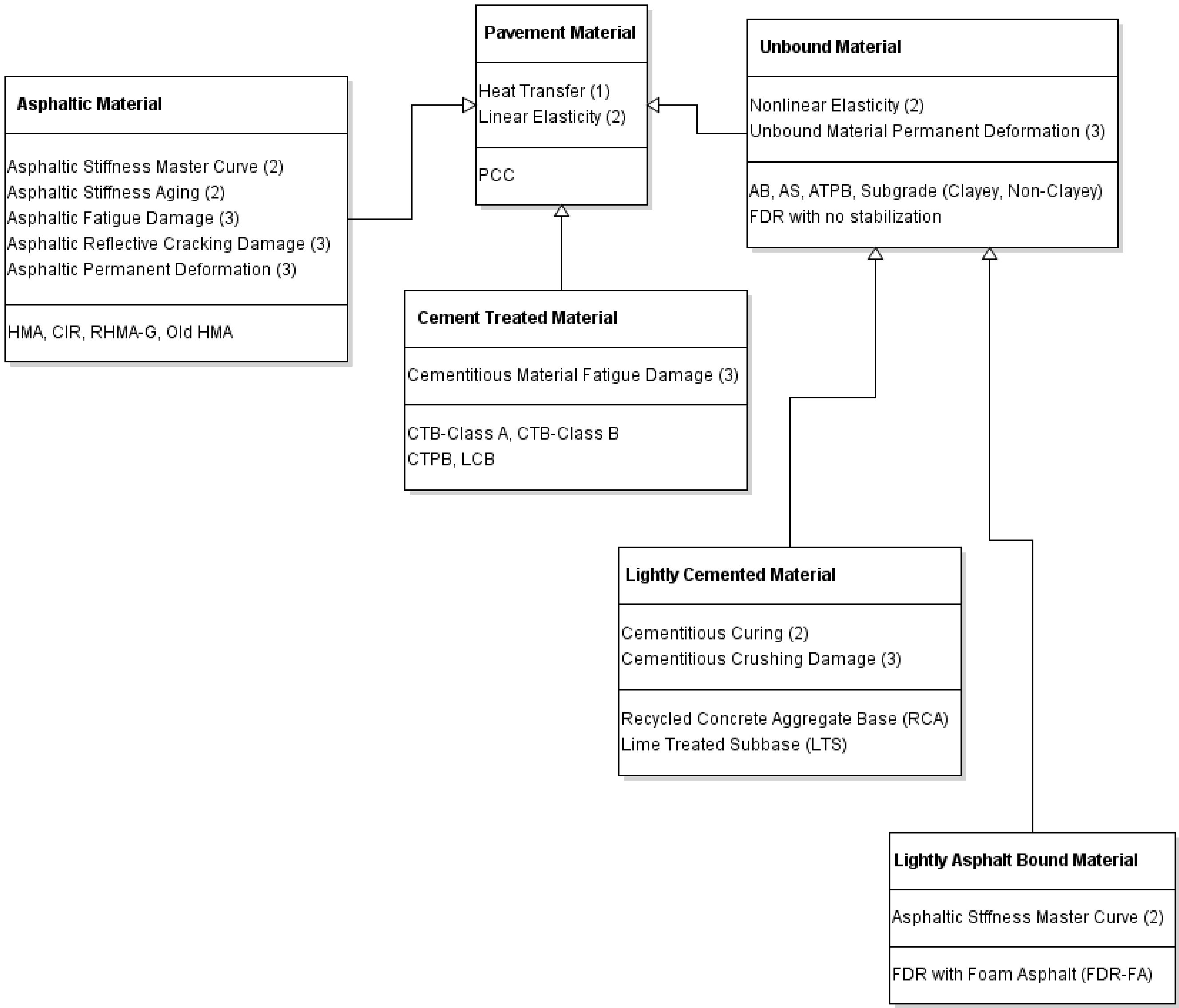

Figure 2.3: Hierarchy of materials in the Standard Materials Library.
(Note: **each box contains the name of the material group followed by a list of models required for the material group, with first letter of the functional group of each model in parenthesis, and a list of example materials using the abbreviations of that appear in the Caltrans *Highway Design Manual.* The arrows connecting different boxes indicate the "is a" relationship.)**

2.2.1 Pavement Material

Every material used in *CalME* 2.0 is a specialized type of *pavement material*, which is defined by two properties: heat transfer and linear elasticity. An example of this material is portland cement concrete (PCC).

CalME 2.0 includes three groups of pavement materials: *asphaltic material*, *cement-treated material*, and *unbound material*. The specific sets of models required for each specialized material are shown in Figure 2.3.

Generally speaking, asphaltic materials are bound by asphalt binder, cement-treated materials are bound by cement, and unbound materials are either not bound or only lightly bound by either asphalt binder or cement.

2.2.1.1 Heat Transfer

A one-dimensional, coupled heat and moisture flow model called the *Integrated Climatic Model* (ICM) was developed in the late 1980s by Lytton et al. to simulate temporal variations in the temperature, moisture, and freeze/thaw conditions internal to the pavement and their impact on key pavement material properties *(1)*. This program is recognized as the most comprehensive model addressing the effects of climate on pavements.

The *Enhanced Integrated Climatic Model* (EICM)*(2)* is an improved version of ICM that was developed for the Federal Highway Administration (FHWA) and adopted as the climatic model in the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) software developed under National Cooperative Highway Research Program (NCHRP) Project 1-37A *(3)*. EICM is intended to help predict or simulate the changes in behavior and characteristics of pavement and unbound materials in conjunction with varying environmental conditions over years of service.

EICM was found to be too slow and complex to be run within *CalME*. Instead, *CalME* uses a simplified thermal model to predict a pavement's temperature profile during its service life. The model is based on surface temperatures generated by EICM and a constant deep soil temperature. Specifically, *CalME* divides California into nine climate zones, each of which is represented by a "super weather station" that has thirty years *(4)* of historical weather data ranging from 1961 to 1990 that can be used as inputs to EICM to calculate pavement surface temperatures over that same thirty-year period. *CalME* assumes that pavement temperature at a depth of four meters remains constant and sets this value as the annual average surface temperature. *CalME* then solves for pavement temperature profile by using a 1-D Finite Element formulation with a finite difference time step *(5)*.

CalME further assumes that pavement temperatures are cyclic and that the 30-year period is longer than the temperature cycle. Accordingly, *CalME* uses the 30 years of historical temperature data to represent future pavement temperatures and repeats itself every thirty years. *CalME* 2.0 allows the user to randomize the point in the temperature history that aligns with the construction date. This is a simplification and it is believed that the error introduced is minimal.

Solving for pavement temperature profile with known top (surface) and bottom (i.e., 4 meter depth) temperature history is essentially a heat transfer problem, which is governed by the following partial differential equation (in 1-D) called Fourier's Law of conduction:

$$\frac{\partial T}{\partial t} = \alpha \frac{\partial^2 T}{\partial z^2} \quad \textbf{(1)}$$

where: T is temperature that varies with time t and depth z
α is the thermal diffusivity

CalME starts with an initial uniform temperature profile using the average annual surface temperature as the fixed value. It solves Equation (1) hour by hour. The only model parameter required here is α, i.e., the thermal diffusivity of the material in each layer.

2.2.1.2 Linear Elasticity

In *CalME* 2.0, pavement structures are simplified as multilayer elastic systems. All materials are assumed to be linear elastic when calculating the critical responses of the pavement. This is true even for rate-dependent materials such as *asphaltic materials* and stress-dependent materials such as *unbound materials,* as shown in Figure 2.3. This is possible because all of the models in *CalME* 2.0 that affects layer stiffness (i.e., models that belong to Functional Group 2 defined in Section 2.1.2) are non-iterative. In other words, if layer stiffness is affected by certain factor, that factor cannot in turn be affected by the same layer stiffness.

To characterize a linear elastic material in *CalME* 2.0, the user needs to provide the layer stiffness and Poisson's ratio. Stiffness refers to the apparent Young's modulus of a material under a given loading condition, such as loading rate, temperature, age, confinement, or stress state. For in-service pavements, the layer moduli are determined from backcalculation using FWD basins.

2.2.2 Asphaltic Material

Asphaltic material refers to asphalt-bound materials such as hot mix asphalt (HMA) and cold in-place recycled (CIR) materials. To be classified as an asphaltic material, the bond provided by asphalt binder must be strong enough to allow production of laboratory specimens such as beams and cores for characterization testing. Asphaltic materials are defined by the following models:

- *Asphaltic stiffness master curve* describes how the material's stiffness changes with loading duration and temperature.
- *Asphaltic stiffness aging* describes how the material's stiffness changes with age.
- *Asphaltic fatigue damage* describes how fatigue damage is accumulated in the material and how the fatigue damage affects its stiffness.
- *Asphaltic reflective cracking damage* describes how reflective cracking damage is accumulated in the material when applicable.
- *Asphaltic permanent deformation* describes how permanent deformation is accumulated in the material.

Each of these models is described in more detail in below.

2.2.2.1 Asphaltic Stiffness Master Curve

Asphalt-bound materal stiffness is modeled in *CalME* as a function of temperature and loading time, commonly referred to as the stiffness master curve. The stiffness master curve in *CalME* is a slightly simplified version of the model used in the NCHRP 1-37A *Mechanistic-Empirical Pavement Design Guide* (MEPDG) model *(3). CalME* uses the same equation as MEPDG for the relation between mix stiffness and the reduced time:

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

where: E is the stiffness in MPa,
tr is reduced time in sec,
α, β, γ, and δ are constants, and
logarithms are to base 10.

Reduced time is a function of both actual loading time (i.e., loading duration) and temperature:

$$tr = \frac{lt}{a(T)} \tag{3}$$

where: lt is the loading duration (in sec), and
a is the temperature shift factor that depends only on temperature T.

The loading time depends on the vehicle speed and layer thickness assuming that stress dissipates through depth at a 45° angle and a tire contact area diameter of 200 mm:

$$lt = \frac{200mm + 2\times z_{1/3}}{V} \tag{4}$$

where $z_{1/3}$ is the depth of the layer at the upper 1/3 division point. The temperature shift factor $a(T)$ is in turn a function of temperature that uses binder viscosity as the intermediate variable:

$$a = \left(\frac{\eta}{\eta_{ref}}\right)^{aT} \tag{5}$$

where: η is the binder viscosity at the loading temperature,
η_{ref} is the binder viscosity at the reference temperature, and
aT is a constant.

The binder viscosity can be calculated from temperature using the following equation:

$$\log\log\eta = A + VTS \cdot \log T \tag{6}$$

where: η is the binder viscosity in cPoise,
T is binder temperature in Kelvin, and
A and VTS are regression constants.

For MEPDG Level 1 and 2 inputs, *A* and *VTS* are determined through regression analysis based on binder viscosity data that is measured either directly or indirectly. For Level 3 inputs, default values for *A* and *VTS* are used based on the binder grade.

In *CalME*, stiffness master curve model parameters are only determined by fitting laboratory frequency sweep testing data for the mix. It was found that any pair of *A* and *VTS* values give an equally good match between measured and predicted stiffness. Accordingly, *A* and *VTS* are fixed at 9.6307 and -0.5047 respectively, using the default values for binders with penetration grade 40–50. Note that the difference in value for *A* is due to the fact that *CalME* uses Kelvin temperature while MEPDG uses Rankine temperature for Equation (6).

The above model for the stiffness master curve has the following model parameters: $\delta, \alpha, \beta, \gamma, \text{and } aT$. The reference temperature is arbitrary and it is fixed at 20°C in *CalME*. Delta (δ) correponds to the minimum stiffness while $\delta + \alpha$ correspond to the maximum stiffness. In *CalME* 2.0, it is further assumed that the minimum stiffness of asphaltic material is 200 MPa, corresponding to the typical stiffness of the compacted aggregate base. This implies $\delta = 2.3010$, so the only parameters to be identified for the asphaltic stiffness master curve are $\alpha, \beta, \gamma, \text{and } aT$.

2.2.2.2 Asphaltic Stiffness Aging

In *CalME* 2.0, the effect of aging is represented by an increase in binder viscosity. Specifically, it is assumed that aging will cause the model parameter *A* in Equation (6) to increase, and the amount of increase is a function of aging time and pavement temperature:

$$\Delta A = b_{Aging} \times \frac{\log(t_{month}+1)}{1-a_{Aging}\times\log\left(\frac{T}{10°\text{C}}\right)} \qquad \textbf{(7)}$$

where: ΔA is the increase in model parameter A in Equation (6),
t_{month} is the pavement age in months,
T is the pavement temperature in °C measured at 1/3 depth of each layer, and
$a_{Aging} = 0.7$ and $b_{Aging} = 0.007$ are model parameters.

Equation (7) is derived from the work by Houston et al. *(6)* with some additional assumptions. The model parameter a_{Aging} depends on the *A-VTS* relationship of the binder, but the variations are very small and an average value of 0.7 may be used for most binders as indicated above. The model parameter b_{Aging} can be adjusted to account for different types of material. The default values shown above were determined based on preliminary field data collected in California.

Just increasing the viscosity will make the master curve shift to the right, which implies that there will be no hardening effect at high or low temperatures. To allow the hardening effect at both high and low temperatures, an aging factor has been introduced and it is defined as the ratio of the modulus of hardened material to the modulus of the original material. The aging factor is determined by evaluating the effect of a viscosity increase due to aging for the temperature corresponding to a modulus of the original material of $10^{\delta + \alpha/2}$ under 10 Hz loading frequency. The aging factor is then used to increase the modulus at all temperatures. In essence, applying a uniform aging factor for all temperatures is equivalent to increasing δ. An example of how the aging factor is determined along with the unaged and aged stiffness master curves are shown in Figure 2.4. Note that the aging factors are different for different materials even if ΔA is the same because the aging factor depends on the parameters of the stiffness master curve model.

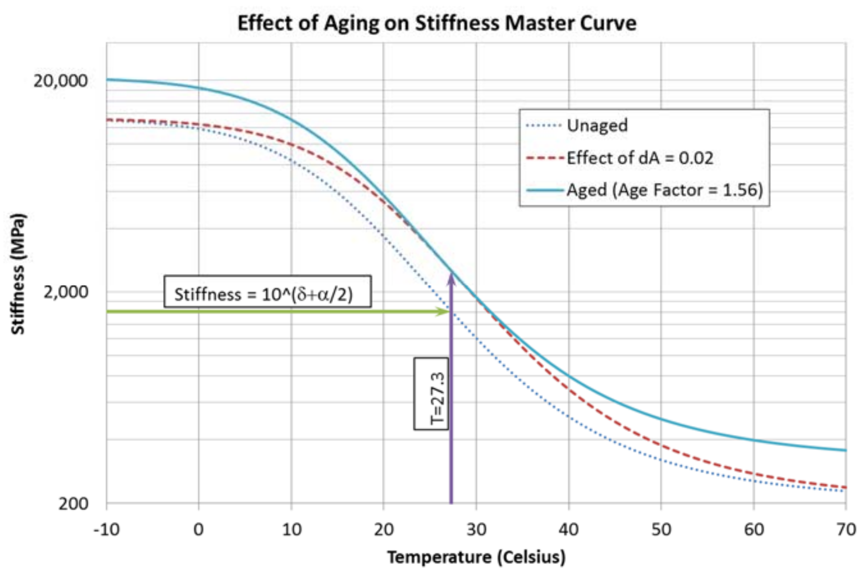

Figure 2.4: Demonstration of the effect of increase in A (i.e., dA) and the corresponding unaged and aged stiffness master curves, $\boldsymbol{\beta = 0.7482, \delta = 2.3010, \gamma = 0.8718}$ and $\boldsymbol{\alpha = 1.8195}$, loading frequency = 10Hz.

As an example to demonstrate how they change with time and climate, aging factors have been calculated for the HMA layer in flexible pavements in the North Coast and Desert climate zones respectively over a thirty-year period. The structure has 150 mm HMA over 300 mm AB-Class 2 followed by subgrade with CL soil. The name of the HMA material is “HMA Type A (Mix 01) RAP00 PG 64-28 Blasted Granite.” (See Table A.1 for a complete list of materials in the current standard materials library.) The changes of aging factors over time are shown in Figure 2.5, which indicates that the HMA layer stiffness will increase by approximately 50 to 70 percent over 30 years due to aging depending on the climate zone the pavement is in.

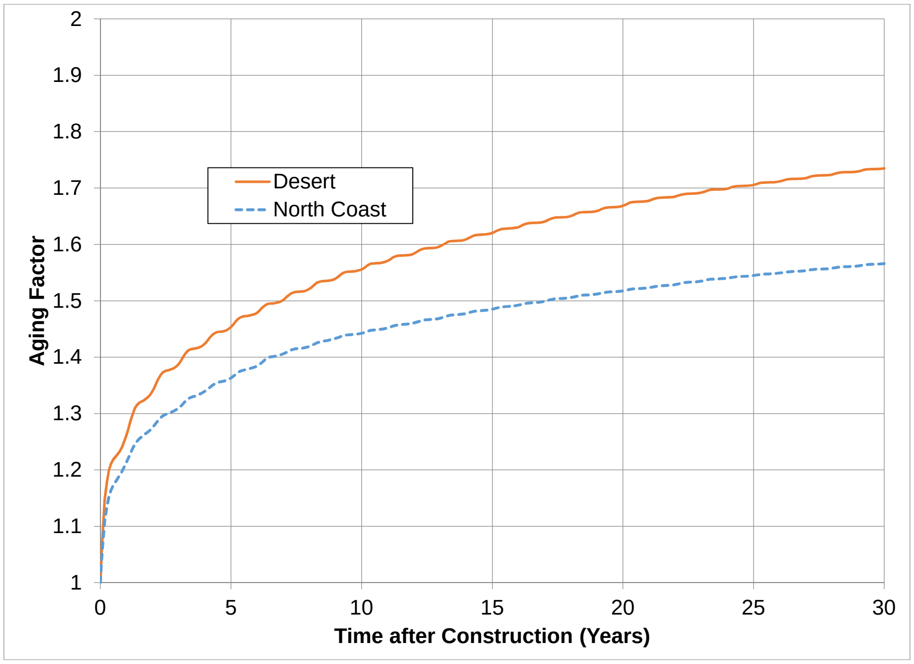

Figure 2.5: Aging factor calculated for a typical flexible pavement in the North Coast and Desert climate zones respectively.

2.2.2.3 Asphaltic Fatigue Damage

Fatigue damage in asphaltic materials is caused by the repeated application of tensile strains due to both traffic loading and daily temperature cycles. In *CalME* 2.0, only traffic-related fatigue damage is considered. Fatigue damage affects the stiffness master curve of asphaltic materials. Specifically, the equation between mix stiffness and reduced time (i.e., Equation (2)) for asphaltic material with fatigue damage becomes:

$$\mathrm{Log(E)} = \delta + \frac{\alpha \times (1-\omega)}{1+\exp(\beta+\gamma \log(tr))} \quad \textbf{(8)}$$

where ω is the fatigue damage, which is in turn calculated from the following equation:

$$\omega = \left(\frac{MN}{FSF \times MN_p}\right)^{\alpha_f} \quad \textbf{(9)}$$

where: *MN* is the number of load applications in millions,
MN_p is the allowable number of load repetitions in millions,
FSF is the fatigue shift factor, and
α_f is a material dependent model parameter.

The fatigue shift factor *FSF* is the empirical part of the fatigue equation. It converts laboratory fatigue performance to the field fatigue performance. *FSF* is set to 1.0 when fitting laboratory test results. MN_p is calculated in turn using the following equation:

$$MN_p = A \times \left(\frac{\varepsilon}{\varepsilon_{ref}}\right)^{\beta} \times \left(\frac{E}{E_{ref}}\right)^{\beta/2} \quad \textbf{(10)}$$

where: ε = bending strain at the bottom of the asphalt layer in με, negative for tensile,
ε_{ref} = -200 microstrain is the reference bending tensile strain,
E_{ref} = 3,000 MPa is the reference stiffness, and
A and β are material constants.

The above set of fatigue equations require the following material-dependent parameters: α_f, *A,* and β. In addition, *FSF* also needs to be determined as part of the calibration process to account for the difference in laboratory and field fatigue performance.

2.2.2.3.1 Effect of Damage on Stiffness

By definition, fatigue damage affects the stiffness of asphaltic materials. According to Equations (2) and (8), the damaged stiffness $E_{damaged}$ and undamaged stiffness $E_{undamaged}$ have the following relationship:

$$\log\left(E_{damaged}\right) - \log\left(E_{undamaged}\right) = -\frac{\alpha \cdot \omega}{1+\exp[\beta+\gamma \cdot \log(tr)]} \quad \textbf{(11)}$$

This can be further simplified as:

$$\log\left(E_{damaged}\right) - \log\left(E_{undamaged}\right) = -[\log(E_{undamaged}) - \delta] \cdot \omega \quad \textbf{(12)}$$

The ratio between damaged and undamage stiffness (*SR*) can then be calculated as:

$$SR = \frac{E_{damaged}}{E_{undamaged}} = \left(\frac{10^{\delta}}{E_{undamaged}}\right)^{\omega} \quad \textbf{(13)}$$

There are no additional model parameters needed to account for the effect of damage on asphaltic material stiffness.

2.2.2.3.2 Influence of Rest Periods

Laboratory fatigue testing has shown the beneficial influence of rest periods on the fatigue life of asphalt materials. One of many examples was provided by Francken and Clauwaert *(7).* The effect of rest period on fatigue life can be described using a shift factor, SF_{RP}, that can be approximated by

$$SF_{RP} = 1 + \left(\frac{RP}{RP_{ref}}\right)^{\varphi} \quad \textbf{(14)}$$

where: *RP* is the rest period,
RP_{ref} is a reference rest period, and
φ is a constant.

The values of RP_{ref} and φ are material-dependent constants which must be determined experimentally. For the three mixes presented by Francken (dense-graded HMA, stone-filled sand sheet, and base-course–type HMA) the value of φ ranged from 0.35 to 0.68. The experiments also indicate that the shift factor reaches a maximum level for long rest periods, from 5 to 30 times the loading time, for the three mixes tested.

NCHRP Web-only Document 134 *(8)* suggests that time temperature superposition may be used for rest periods (as it is for loading time). Assuming this is correct, the effects of rest periods could be accounted for by changing Equation (10) to:

$$MN_p = A \times \left(\frac{\varepsilon}{\varepsilon_{ref}}\right)^{\beta} \times \left(\frac{E}{E_{ref}}\right)^{\beta/2} \times \left[1 + \left(\frac{RP}{RP_{ref}} \times \left(\frac{\eta(T_{ref})}{\eta(T)}\right)^{aT}\right)^{\varphi}\right] \qquad \textbf{(15)}$$

where: $\eta(T)$ is the viscosity at temperature T, and

aT is the parameter defined in Equation (5).

Using the time temperature superposition causes the beneficial effects of rest periods to decrease with time, as the binder hardens. This appears to be reasonable and could account for part of the deterioration of fatigue characteristics with aging.

In addition, Equation (15) further implies that the effect of rest period increases with temperature, as the binder viscosity decreases. This appears to be reasonable and could account for the typical increase in fatigue life when temperature increases.

Presently *CalME* 2.0 by default includes the effects of rest periods for all asphaltic materials with $RP_{ref} =$ 10 seconds and $\varphi = 0.4$. The rest period is calculated as the time interval between two axle applications, assuming all the axle applications are uniformly distributed in time.

2.2.2.3.3 Methods of Adding Static and Dynamic Stress

Another potential reason for the difference between the beam fatigue tests and in situ pavements is that daily and seasonal temperature changes cause changes to the asphalt materials that affect the fatigue properties. Cooling of an asphalt beam will cause the beam to contract, but in the pavement layer the material is restrained from contracting. In a linear elastic material this constraint would cause a semi-static tensile stress in the material.

For fatigue of metals several methods are used to add the effects of static and dynamic tensile stresses. The dynamic stress is normally sinusoidal, with an amplitude of σ_a, on which a static stress of σ_m is superimposed.

Goodman's method of adding dynamic and static stresses states that failure is reached when:

$$\frac{\sigma_a}{S_N} + \frac{\sigma_m}{S_u} = 1 \tag{16}$$

where: σ_a is the amplitude of the dynamic stress,
σ_m is the static stress,
S_N is the fatigue stress for N load applications, and
S_u is the static strength.

If the fatigue equation for purely dynamic loading is used, the effect of the static stress may be considered by multiplying the amplitude of the dynamic stress by a factor f:

$$f = \frac{S_u}{S_u - \sigma_m} \tag{17}$$

For asphalt it appears that strain is more important than stress. For metals stress and strain are practically proportional, but for a viscoelastic material such as asphalt concrete that is not the case. The coefficient of thermal expansion (CTE) for asphalt is typically 40 to 50 microstrain/°C. Cooling an asphalt beam by 10˚C would cause it to contract by about 500 microstrain. To bring it back to the original length that strain must be imposed on the material. This will create a stress that relaxes over time, but the strain will remain.

When an asphalt pavement cools down it will contract in the vertical direction but not in the horizontal direction (at least not longitudinally). This will not create a measurable strain in the material, but on the level of the grain size it will. A strain will develop in the binder film when the grain contracts. The condition of the material will be the same as in a beam that is cooled and then strained to gain its original length. It makes sense, therefore, to consider a strain caused by temperature changes, although it would not be possible to measure such a strain in the material.

One possible way of including this strain would be to use the method given above, but with the static (temperature-induced) strain added to the dynamic (load-induced) strain. This would require a calculation of the static strain, and to do this the temperature at which the static strain is zero must be known. It is uncertain whether this temperature is constant during the year since it could be changing as a result of permanent deformation in the material, so it would be interesting to measure the contraction or expansion on cores or slabs cut from asphalt layers at different times and temperatures. The Goodman method would also require a maximum permissible static strain (or minimum temperature), which could possibly be related to the low temperature grade for PG grade materials.

An option for including temperature strains using the Goodman method has been added to *CalME* 2.0. By default, this option is not activated. When it is activated, a user must provide a temperature at which temperature strain becomes zero ($T_{\varepsilon=0}$). By default, $T_{\varepsilon=0} = 20°\text{C}$. The temperature corresponding to maximum allowable temperature strain ($T_{\min}$) is set to -20°C. The inverse of dynamic strain multiplier can be calculated as:

$$\frac{1}{f} = \frac{S_u - \sigma_m}{S_u} = 1 - \frac{\sigma_m}{S_u} = 1 - \frac{(T - T_{\varepsilon=0}) \times CTE}{(T_{\min} - T_{\varepsilon=0}) \times CTE} = 1 - \frac{T - T_{\varepsilon=0}}{T_{min} - T_{\varepsilon=0}} \quad \textbf{(18)}$$

where T is the current HMA layer temperature.

2.2.2.4 Asphaltic Reflective Cracking Damage

In *CalME* 2.0, reflective cracking damage for asphaltic materials is calculated using the same equations as fatigue damage. The only difference is how the strains are calculated. Instead of using multilayer elastic theory, regression equations are developed *(9)* to relate pavement conditions (such as existing crack spacing, layer thicknesses, and stiffnesses) and applied traffic load to a critical strain that drives reflective cracking damage.

2.2.2.5 Asphaltic Permanent Deformation

The shear-based approach developed by Deacon et al. *(10)* for predicting permanent deformation (rutting) of the asphalt layer has been used in *CalME* 2.0. Rutting in the asphalt is assumed to be controlled by shear deformation. The rutting estimates used computed values of shear stress, τ, and elastic shear strain, γ^e, at a depth of 50 mm (2 in.) beneath the edge of the tire. This approach also assumes that rutting occurs solely in the top 100 mm (4 in.) of the HMA layer.

Rutting in the HMA layer due to the shear deformation is determined from the following:

$$rd_{AC} = K \times \gamma^i \times h \quad \textbf{(19)}$$

where: rd_{AC} is the vertical rut depth in the asphalt concrete, in millimeters,
γ^j is the permanent (inelastic) shear strain at 50 mm depth,
K is a value relating permanent shear strain to rut depth (mm), and
h is the thickness of the HMA layer in millimeters, with a maximum value of 100 mm.

The permanent strain may be calculated from a gamma function:

$$\gamma^i = A \cdot \exp\left(\alpha \times \left[1 - \exp\left(-\ln(N)/\gamma\right) \times \left(1 + \ln(N)/\gamma\right)\right]\right) \times \gamma^e \quad \textbf{(20)}$$

where: γ^e = corresponding elastic shear strain (m/m),
N = equivalent number of load repetitions, which is the number of load repetitions at the stress and strain level of the next time increment to reach the permanent shear strain calculated at the end of current time increment, and
A, α, and γ are material-dependent model parameters.

The model parameter K in Equation (19) is 2.0 for all HMA mixes and CIR material based on calibrations using Heavy Vehicle Simulator (HVS) test data. Rubberized HMA with gap-graded aggregates (RHMA-G) however has a K value of 0.5. The effects of other binder modifications on the value of K have been found to be insignificant.

2.2.3 *Cement-Treated Materials*

Cement-treated materials include cement-treated base (CTB, both Caltrans Class A and Class B), cement-treated permeable base (CTPB), and lean concrete base (LCB). These materials are subject to fatigue damage, as described below.

2.2.3.1 Cementitious Material Fatigue Damage

A damage function similar to the one used for fatigue damage of asphaltic materials is used for these materials:

$$\omega = A \times MN^{\alpha} \times \left(\frac{\varepsilon}{\varepsilon_{ref}}\right)^{\beta} \times \left(\frac{E}{E_{ref}}\right)^{\gamma} \qquad \textbf{(21)}$$

where: ω = damage for the layer,
MN = the number of load repetitions in millions,
ε = horizontal tensile strain at the bottom of the layer,
E = the modulus of the cement-treated layer after adjustment for damage,
ε_{ref} = 45 $\mu\varepsilon$ is the reference strain,
E_{ref} = 10,000 MPa is the reference stiffness, and
A, α, β and γ are model parameters.

The modulus of the layer E is reduced by multiplying the intact modulus by $(1-\omega)$:

$$E = E_i \times (1-\omega) \qquad \textbf{(22)}$$

where E_i is the intact stiffness.

Damage model parameters are $A = 1.0$ and $\alpha = 0.25$, while β and γ depend on the initial stiffness E_i:

$$\beta = 0.25 + 0.9 \times \frac{E_i}{E_{ref}} \qquad \textbf{(23)}$$

$$\gamma = 0.05 + 0.9 \times \frac{E_i}{E_{ref}} \qquad \textbf{(24)}$$

2.2.4 Unbound Materials

For *unbound materials,* layer stiffnesses are determined through backcalculation using surface deflection data collected using the falling weight deflectometer (FWD). To identify various non-linearities, FWD testing may be needed multiple times with different loads and at different times of the day when pavement temperatures are different. This will be explained in more detail in Section 2.2.4.1 and Section 2.2.4.2.

The effects of seasonal moisture variation and freeze/thaw on unbound layer stiffnesses have been deactivated in *CalME* 2.0. Research is being conducted by the UCPRC to develop more comprehensive models for predicting seasonal variation of soil conditions such as moisture content and freeze/thaw. The findings from this research will be implemented in a future version of *CalME* once they become available.

Examples of unbound material used in *CalME* include the following:

- Aggregate base (AB): AB-Class 2
- Aggregate subbase (ASB): ASB-Class 1, ASB-Class 2, and ASB-Class 3
- Asphalt-treated permeable base (ATPB)
- Subgrade: all Unified Classification System (UCS) categories of soil, from heavy clay to well-graded gravel, or more generically clayey or non-clayey soil
- Aggregate base from full-depth recycled material without stabilization

As shown in Figure 2.3, there are two subgroups of materials in the *unbound material* group: *lightly cemented materials* and *lightly asphalt-bound materials*. These materials behave like unbound material but have some additional material behaviors that make them different from unbound material. Lightly cemented materials are lightly bound by either cement or lime, while lightly asphalt-bound materials are lightly bound by asphalt binder.

2.2.4.1 Nonlinear Elasticity

During the calibration of *CalME* models using data from flexible HVS test sections and the twenty-six original WesTrack sections, it was found that the stiffness of unbound materials could vary with the stiffness of the layers above them, beginning with the asphalt surface layer. This occurred both when the variation in unbound stiffness was due to temperature variation and fatigue damage to the asphalt, or to changes in stiffness from other causes in other layers above a given unbound layer. The change in stiffness was the opposite of what would be expected for granular layers due to the nonlinearity of the material. The following relationship is used to describe this stiffness variation as a function of confinement in the unbound layers from the layers above them:

$$E_n = E_{n,ref} \times \left[1 - \left(1 - \frac{S_n^3}{S_{ref}^3}\right) \times SF\right] \qquad \textbf{(25)}$$

where: E_n is the stiffness of unbound layer n, counting from the surface,
S_n = bending stiffness for layer n (defined in Equation (26)),
$E_{n,ref}$ = the stiffness of layer n when bending stiffness $S = S_{ref}$,
S_{ref} = 3,500 $(N \cdot mm)^{\frac{1}{3}}$ is the normalizing bending stiffness and,
SF = stiffness factor is a model parameter.

The bending stiffness S for layer n is calculated as:

$$\mathrm{S} = \sum_{i=1}^{n-1} \mathrm{h_i} \times \sqrt[3]{\mathrm{E_i}} \qquad \textbf{(26)}$$

where: h_i is the thickness of layer i, counting from the surface, and
E_i is the stiffness of layer i.

If full slip has developed between two or more layers, their combined bending stiffness is found from:

$$\mathrm{S} = \left(\sum_{i=1}^{n-1} \mathrm{h_i^3} \times \mathrm{E_i}\right)^{\frac{1}{3}} \qquad \textbf{(27)}$$

If partial slip has developed between layers, a linear interpolation is done between full and no slip.

The unbound layers for some HVS tests also showed the well-known nonlinearity of granular materials, with the modulus of granular layers increasing with increasing bulk stress and the modulus of cohesive materials decreasing with increasing deviator stress. In *CalME,* this nonlinearity is treated as a function of the wheel load rather than as a function of the stress condition to avoid the interdependence between stiffness and stress:

$$\mathrm{E_P} = \mathrm{E_{40kN}} \times \left(\frac{\mathrm{P}}{\mathrm{40kN}}\right)^{\alpha} \qquad \textbf{(28)}$$

where: E_P is the stiffness of an unbound layer at wheel load P,
$E_{40\,kN}$ is the layer stiffness at a wheel load of 40 kN, and
α is a constant.

The model parameter α in Equation (28) may be positive or negative depending on whether stiffness increases or decreases with the wheel load.

Combining Equations (25) and (28) leads to the following complete model for unbound layer stiffness:

$$\mathrm{E_n} = \mathrm{E_{n,ref}} \times \left[1 - \left(1 - \frac{\mathrm{S_n^3}}{\mathrm{S_{ref}^3}}\right) \cdot \mathrm{SF}\right] \times \left(\frac{\mathrm{P}}{\mathrm{40kN}}\right)^{\alpha} \qquad \textbf{(29)}$$

The remaining undetermined material dependent model parameters include stiffness factor *SF* and loading effect parameter α.

2.2.4.2 Unbound Material Permanent Deformation

Permanent deformation of unbound layers such as lightly cemented or unbound materials is based on the vertical resilient strain at the top of the layer $\mu\varepsilon$ and stiffness of the layer E:

$$d_p = A \times MN^{\alpha} \times \left(\frac{\mu\varepsilon}{\mu\varepsilon_{ref}}\right)^{\beta} \times \left(\frac{E}{E_{ref}}\right)^{\gamma} \qquad \textbf{(30)}$$

where: d_p = the permanent deformation in the layer,
MN = the number of load applications in millions,
$\mu\varepsilon_{ref}$= the normalizing strain,
E_{ref} = the normalizing stiffness, and
A, α, β and γ are material specific model parameters

Permanent deformations in all of the unbound or lightly bound layers can be calculated using Equation (30). The model parameters however are material dependent.

2.2.5 Lightly Cemented Material

Lightly cemented materials include recycled hardened concrete aggregate (RHCA) base and lime-treated subbase (LTS). Both of them are subject to the stiffness increase caused by continuous curing and damage caused by crushing.

2.2.5.1 Cementitious Curing

Cementitious curing results in increasing layer stiffness. For lightly cemented materials, stiffness is a function of time:

$$E(Age) = E(Initial\ Age) \times \frac{A_{age} \times \ln(Age)+1}{A_{age} \times \ln(Initial\ Age)+1} \qquad \textbf{(31)}$$

where: $E(Age)$ = stiffness for given age, in days
$E(Initial\ Age)$ = stiffness at the initial age in days, and
A_{age} is a model parameter.

A default value of -0.6 is used for A_{age}. An example of a calculated stiffness factor, defined as the ratio between $E(Age)$ and $E(Initial\ Age)$, for an initial age of 90 days is shown in Figure 2.6. As shown in the figure, stiffness for lightly cemented materials will increase to about 2.6 times its initial value twenty years after construction.

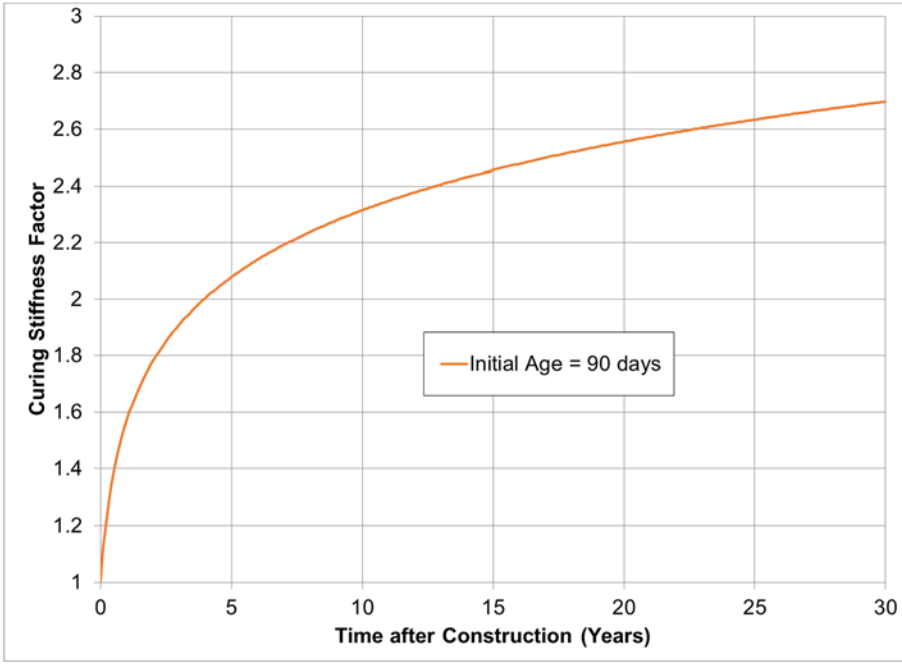

Figure 2.6: Example of calculated curing stiffness factor for a twenty-year period with A_{age} = -0.6.

2.2.5.2 Cementitious Crushing Damage

For lightly cemented materials, a function similar to the one used for the damage of cement-treated materials is used:

$$\omega = A \times MN^{\alpha} \times \left(\frac{\sigma_z}{\sigma_{ref}}\right)^{\beta} \times \left(\frac{E}{E_{ref}}\right)^{\gamma} \quad \textbf{(32)}$$

where: ω = damage for the layer,
MN = the number of load repetitions in millions,
σ_z = vertical stress at the top of the layer,
E = the modulus of the cement-treated layer after adjustment for damage,
σ_{ref} = the reference strain,
E_{ref} = the reference stiffness, and
A, α, β and γ are model parameters.

The modulus of the layer is reduced by multiplying the intact stiffness by $(1-\omega)$:

$$E = E_i \times (1-\omega) \quad \textbf{(33)}$$

where E_i is the intact stiffness.

The damage model parameters are $A = 1.0$ and $\alpha = 0.25$, while γ, σ_{ref}, and E_{ref} depend on the initial stiffness E_i:

$$\beta = 0.25 + 0.8 \times \frac{E_i}{10{,}000\ MPa} \quad \textbf{(34)}$$

$$\gamma = 0.05 + 0.8 \times \frac{E_i}{10{,}000\ MPa} \qquad \textbf{(35)}$$

$$\sigma_{ref} = 0.02 \times \left(\frac{E_i}{10{,}000\ MPa}\right)^{-0.9} \qquad \textbf{(36)}$$

$$E_{ref} = E_i \qquad \textbf{(37)}$$

2.2.6 Lightly Asphalt-Bound Material

The difference between *lightly asphalt-bound materials* and *unbound materials* is that the stiffness for the former is rate and temperature dependent. A stiffness master curve is required to fully describe lightly asphalt-bound materials. There is only one lightly asphalt-bound material in *CalME* 2.0: full-depth recycled material with foamed asphalt as the stabilizing agent (FDR-FA).

The same set of equations for the asphaltic materials stiffness master curve is used for lightly asphalt-bound materials. The difference is that the model parameters can only be derived from field test results because lightly asphalt-bound materials are not viable for making beam specimens used for AASHTO T 321 frequency sweep test.

2.3 Determination of Model Parameters

In this section, the material models used in *CalME* 2.0 are grouped based on their function as defined in Section 2.1.2. Brief descriptions of how the model parameters are identified are also presented.

2.3.1 Environmental Models

Environmental models are those that estimate environmental conditions such as pavement temperature and moisture condition. *CalME* 2.0 has only one environmental model: heat transfer.

2.3.1.1 <u>Heat Transfer</u>

The only model parameter required for heat transfer is the thermal diffusivity of the material in each layer. Currently *CalME* uses typical values of thermal diffusivity for various types of materials based on the data and model found in Reference *(11).* Each material has to be assigned one of the thermal codes shown in Table 2.1. Laboratory testing may be needed to classify an unbound material based on the USCS (Unified Soil Classification System) before the correct thermal code can be assigned.

Table 2.1: Typical Values for Thermal Diffusivities in *CalME* 2.0

Material Type	Thermal Code	Thermal Diffusivity (m^2/s)
Portland Cement Concrete	CC	1,696
Cement Treated Base	CT	1,696
Lean Concrete Base	LC	1,696
Asphalt Concrete	AC	2,000
Rubberized Asphalt Concrete	RA	2,000
Hot Recycled Asphalt Concrete	HR	2,000
Cold Recycled Asphalt Concrete	CR	2,000
Asphalt-Treated Permeable Base	AT	2,000
Bed Rock	BR	3,333
Gravel – Well graded	GW	3,490
Gravel – Poorly graded	GP	4,540
Silty Gravel	GM	3,215
Clayey Gravel	GC	3,086
Sand – Well graded	SW	3,706
Sand – poorly graded	SP	2,952
Silty Sand	SM	1,963
Clayey Sand	SC	2,647
Silt – Low plasticity	ML	1,598
Clay – Low plasticity	CL	1,360
Organic Clay – Low plasticity	OL	1,166
Silt – Low plasticity	MH	1,472
Clay – Low plasticity	CH	1,292
Organic Clay – Low plasticity	OH	937
Peat	PT	688

2.3.2 Stiffness Models

Stiffness models are used to estimate the pavement layer stiffness. *CalME* 2.0 includes the basic linear elasticity model as well as different models that describe the change of stiffness with various factors such as temperature, loading rate, age, confinement, load level, and damage.

2.3.2.1 Linear Elasticity

Model parameters for linear elasticity in *CalME* 2.0 include stiffness and Poisson's ratio. Poisson's ratio is assumed to be 0.20 for cementitious materials such as PCC and cement-treated materials and 0.35 for everything else. Layer stiffness, on the other hand, needs to be determined either from laboratory or field testing depending on the material type.

Due to the fact that it can be affected by various factors, the stiffness model parameter actually refers to the stiffness of a layer under the corresponding reference conditions:

- Intact condition when a material is subjected to damage,
- 20°C when a material is temperature dependent,
- 0.015 second loading time for rate-dependent materials,
- 40 kN for load-dependent materials, and
- 3,500 $N \cdot mm$ for confinement-dependent materials.

The actual stiffness of a pavement layer is then adjusted based on the actual loading conditions and damage.

Stiffness parameters for pavement materials are generally determined either based on in situ stiffness backcalculated from falling weight deflectometer test data or based on typical values reported in the literature. For asphaltic materials and lightly asphalt-bound material, the time and temperature dependency for stiffness is described by stiffness master curves.

2.3.2.2 Asphaltic Stiffness Master Curve

Both asphaltic materials and lightly asphalt-bound materials have asphaltic stiffness master curves whose model parameters can be identified using test data that include stiffnesses measured at different loading rates and temperatures. For *CalME*, the stiffness master curve parameters for asphaltic materials were determined using data from frequency sweep testing on four-point bending beam specimens following test method AASHTO T 321. Since lightly asphalt-bound materials beams cannot be produced, an alternative method was used to develop the master curve parameters.

The typical experiment design for characterizing asphaltic stiffness master curves for *CalME* is shown in Table 2.2. As the table shows, six specimens will be needed for the test and a total of 66 data points will be obtained with each specimen tested under 11 loading frequencies. These data are used to fit the equations shown in Section 2.2.2.1 and to determine the unknown model parameters $\alpha, \beta, \gamma,$ and aT for each asphaltic material. An example of a stiffness master curve fitted using frequency sweep stiffness data is shown in Figure 2.7. This is for material named “HMA Type A (Mix 08) RAP00 PG 64-28PM Crushed Alluvial.” (See Table A.1 for a complete list of materials in the current standard materials library.) As shown in the figure, the model matches the measured stiffness very well except for when loading frequency is less than 0.02 Hz. This deviation is expected because asphalt materials have a minimum assumed stiffness of 200 MPa (see Section 2.2.2.1). Furthermore, the measured stiffnesses at low loading frequencies are not reliable due to the low load signals and lack of confinement in the specimens at the highest testing temperatures.

Table 2.2: Typical Experiment Design for Characterizing Stiffness Master Curves

Factorial	Number of Levels	Values	Unit
Temperature	3	10, 20, 30	°C
Strain Amplitude	1	100 or 200*	microstrain
Frequency Combination	1	0.01, 0.02, 0.05, 0.1, 0.2, 0.5, 1.0, 2.0, 5.0, 10.0, 15.0	Hz
		Number of Replicates	2
		Total Number of Specimens	6

*: Each specimen is typically tested under 100 microstrain but 200 microstrain can be used when necessary to make the applied load measurable.

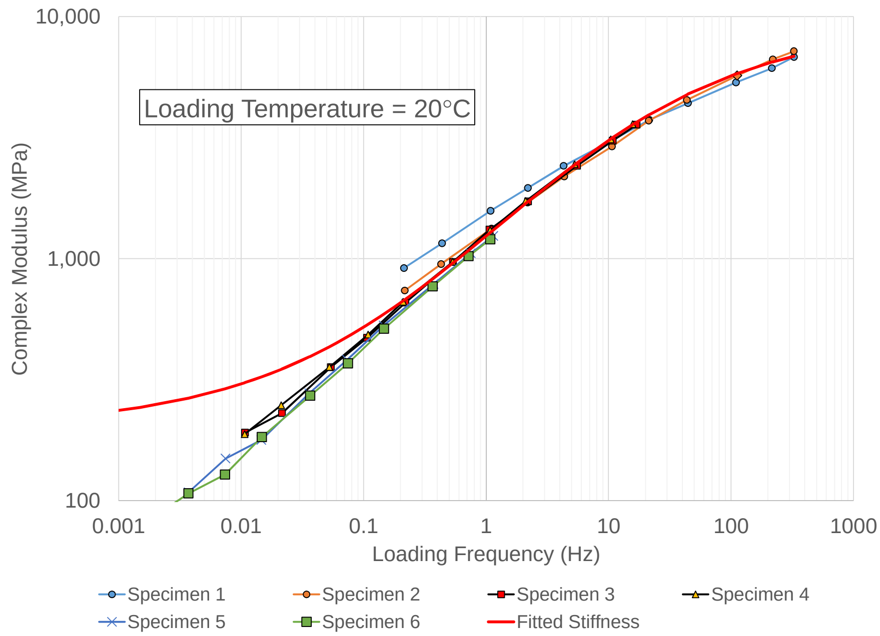

Figure 2.7: Example of a stiffness master curve fitted using frequency sweep stiffness data.

For lightly asphalt-bound materials, a somewhat ad hoc approach is adopted for identifying their model stiffness master curve parameters. Specifically, the model parameters are determined based on the following data if they are available:

- Backcalculated stiffness using FWD testing data
- Sensitivity to temperature change based on either backcalculated in situ stiffness or laboratory test data
- Minimum stiffness based on either backcalculated in situ stiffness or laboratory test data

2.3.2.3 Unbound Material Nonlinear Elasticity

The remaining undetermined model parameters for unbound material nonlinear elasticity include the stiffness factor *SF* and loading effect parameter α. As examples, the range of values for WesTrack materials *(12)* are listed in Table 2.3.

Table 2.3: Unbound Material Nonlinear Elasticity Parameters for WesTrack Materials

Description	SF	α
Aggregate base (AB)	0 to 0.65	0.6
Subgrade (SG)	0	-0.2

To identify the stiffness factor SF and loading effect parameter α, one needs to provide stiffnesses for an unbound material under different confinements and wheel loads. This was achieved by conducting FWD testing at the same location under different surface temperatures and drop heights.

2.3.2.4 Cementitious Curing for Lightly Cemented Material

As shown in Section 2.2.5.1, a default value of -0.6 is used for A_{age} to describe the effect of curing on the stiffness of lightly cemented materials. No additional testing was conducted for materials in *CalME*.

2.3.3 Physical Evolution Models

As mentioned in Section 2.1.2, physical evolution models are needed to update material conditions such as damage and aging, given all the potential critical responses such as stress, strain, the corresponding number of applications, and the current status of damage. Physical evolutions considered in *CalME* 2.0 include damage (fatigue and crushing damage), permanent deformation, and aging.

2.3.3.1 Asphaltic Fatigue Damage

For *CalME* 2.0, the model parameters for asphaltic fatigue damage defined in Equations (9) and (10) are determined by fitting the stiffness reduction curves from bending beam fatigue tests (AASHTO T 321). The typical experiment design for characterizing the asphaltic fatigue damage model for *CalME* is shown in Table 2.4. Theoretically, the asphaltic fatigue damage model parameters can be identified using data from other fatigue tests as long as stiffness reduction curves and strain histories are recorded. Note that only one temperature and one loading frequency are used in the fatigue testing. Only one temperature is used because the fatigue asphaltic damage model included cannot account for the effect of temperature very well, and 20°C was chosen because it is believed to be near the critical temperature at which an asphalt concrete pavement is most susceptible to fatigue damage. The choice of 10 Hz as the loading frequency represents a loading time of 0.016 seconds, which is approximately the time required for the strain pulse caused by a standard axle load to pass a point at the asphalt concrete layer bottom at a speed of 100 km/h (~60 mph).

Table 2.4: Typical Experiment Design for Characterizing Asphaltic Fatigue Damage

Factorial	Number of Levels	Values	Unit
Temperature	1	20	°C
Strain Amplitude	2	200, 400	microstrain
Loading Frequency	1	10.0	Hz
		Number of Replicates	3
		Total Number of Specimens	6

Figure 2.8 shows an example of the comparison between the calculated and measured residual stiffness ratio after fitting the fatigue test data. A comparison between the calculated and measured stiffness reduction curves for Specimen 5 in Figure 2.8 is shown in Figure 2.9. These are for the material named "HMA Type A (Mix 11) RAP00 PG 64-16 Crushed Alluvial Hveem." (See Table A.1 for a complete list of materials in the current standard materials library.)

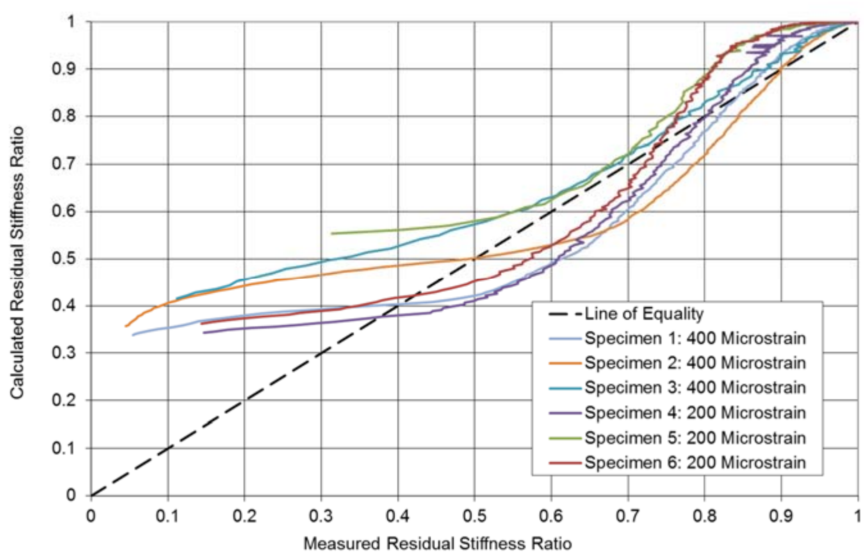

Figure 2.8: Example comparison between calculated and measured residual stiffness ratio after fitting the fatigue test data.

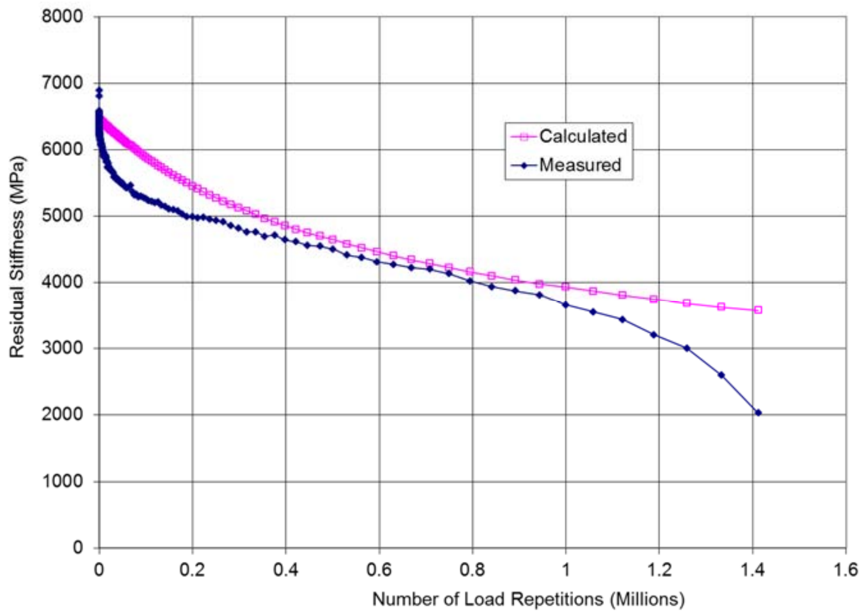

Figure 2.9: Comparison of calculated and measured stiffness reduction curves for Specimen 5 shown in Figure 2.8 (A=29.1, $\alpha_f = 1.03$, and $\beta = -5.99$).

2.3.3.2 Asphaltic Permanent Deformation

For *CalME*, the rutting model parameters defined in Equation (20) for asphaltic permanent deformation are determined by fitting the permanent shear strain accumulation curves obtained from repeated simple shear test at constant height (RSCH) following AASHTO T 320. Theoretically, the model parameters can also be identified using data from other permanent deformation tests as long as the elastic and permanent strain histories are recorded.

The typical experiment design for characterizing the asphaltic fatigue damage for *CalME* is shown in Table 2.5. As shown in the table, a total of eighteen specimens are needed for each material. Each specimen is 6 in. (150 mm) in diameter and 2 in. (50 mm) in height.

Table 2.5: Typical Experiment Design for Characterizing Asphaltic Permanent Deformation

Factorial	Number of Levels	Values	Unit
Temperature	2	45, 55	°C
Shear Stress Amplitude	3	70, 100, 130	kPa
Loading Frequency	1	0.1 second loading, 0.6 second rest	N/A
		Number of Replicates	3
		Total Number of Specimens	18

Figure 2.10 shows an example of the comparison between the calculated and measured permanent shear strain after fitting the test data. The comparison between the calculated and measured permanent shear strain accumulation curves for Specimen 16 in Figure 2.10 is shown in Figure 2.11. These are for the material named "HMA Type A (Mix 12) RAP00 PG 64-16 Crushed Alluvial Superpave." (See Table A.1 for a complete list of materials in the current standard materials library.)

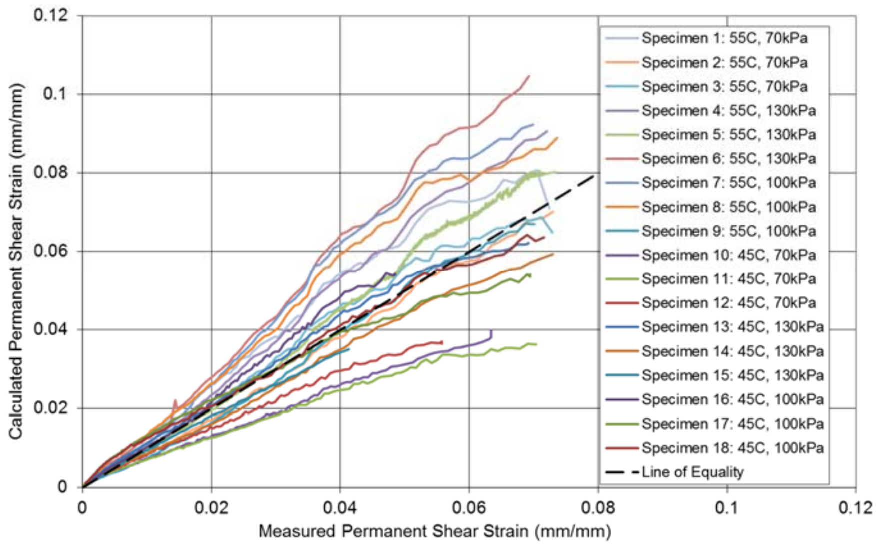

Figure 2.10: Example of a comparison between the calculated and measured permanent shear strains after fitting the permanent shear strain accumulation data.

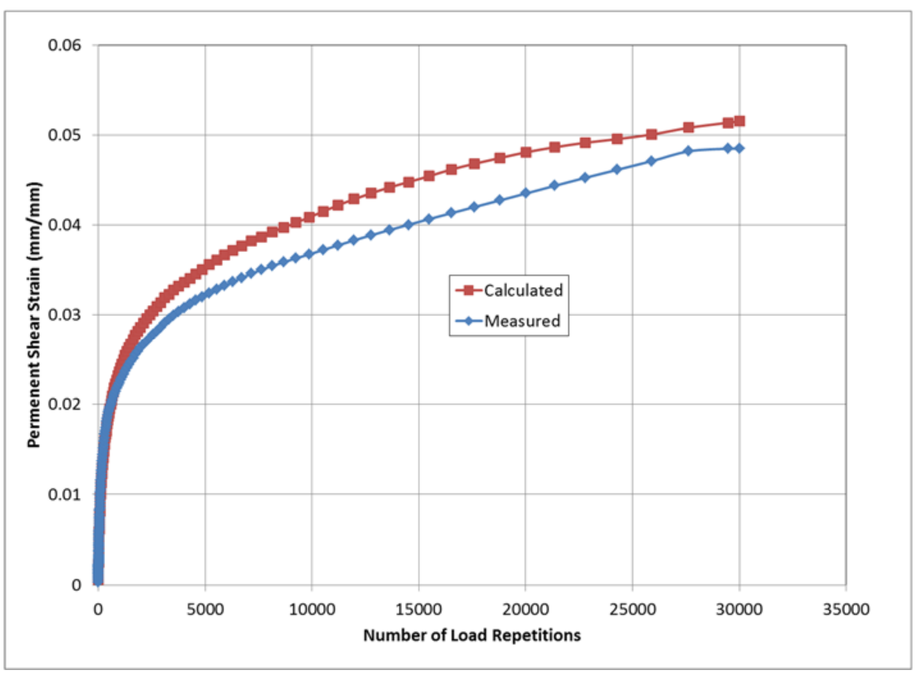

Figure 2.11: Comparison of calculated and measured permanent shear strain accumulation curves for Specimen 16 shown in Figure 2.10 (A=0.67, $\alpha = 3.60$, and $\gamma = 2.25$).

2.3.3.3 Asphaltic Stiffness Aging

As discussed in Section 2.2.2.2, default aging model parameters are used for all asphaltic materials. No additional testing was conducted to characterize individual asphaltic material in terms of aging.

2.3.3.4 Cementitious Material Fatigue Damage

The model parameters for cementitious material fatigue damage defined in Equation (21) either have default values or are related to intact stiffness. No additional testing was conducted to determine their values in *CalME*.

2.3.3.5 Cementitious Crushing Damage

Similar to cementitious material fatigue damage, the cementitious material crushing damage model parameters defined in Equation (32) either have default values or are related to intact stiffness. No additional testing was conducted to determine their values in *CalME*.

2.4 Considerations for Construction Variability

CalME can be run in deterministic or Monte Carlo (probabilistic) mode. In deterministic mode, *CalME* provides the mean prediction for the performance of a given pavement design with the assumption that all material inputs represent mean values.

In Monte Carlo mode, *CalME* generates design inputs based on the corresponding statistical distributions under the assumption that these inputs are random variables. To make the Monte Carlo simulation manageable, only a select set of inputs are treated as random variables, as listed below, to account for construction variabilities:

- Structural variability:
    - Layer thicknesses: normal distribution
- Material variability:
    - Layer stiffnesses: lognormal distribution
    - Asphaltic fatigue damage model: lognormal distribution for parameter *A*
    - Asphaltic permanent deformation model: lognormal distribution for parameter

The input for structural variability (i.e., layer thickness) is the coefficient of variance. All the inputs related to material variability follow lognormal distribution. Suppose X is a random variable following lognormal distribution, its logarithm $\log(X)$ follows a normal distribution with mean μ and standard deviation σ:

$$\log(X) \sim N(\mu, \sigma) \quad \textbf{(38)}$$

In *CalME* 2.0, a lognormal distribution is specified by providing an apparent average defined as 10^{μ} and a standard deviation factor (SDF) defined as 10^{σ}. Note that the coefficient of variance (COV) for random variable X is:

$$COV(X) = \sqrt{e^{[\ln(SDF)^2]} - 1} \quad \textbf{(39)}$$

For example, the *COV* corresponding to a SDF of 1.3 is 0.27, which is approximately equal to 0.3.

Note that *CalME* allows two other types of variabilities, namely, traffic and climate but using them is not recommended as they can slow down the computations and may have no significance considering the high uncertainity associated with these two particular variables.

2.5 Classification of Materials for Caltrans Empirical Design

Although its main focus is the ME performance prediction module, *CalME* also includes a module that implements the Caltrans empirical procedures defined in the Highway Design Manual *(13)* for the design and rehabilitation of flexible pavements to provide a starting point for the ME design process.

The Caltrans empirical design procedure requires classifying each material into one of the following categories:

- RHMA-G: rubberized hot mix asphalt with gap-graded aggregates
- RHMA-O: rubberized hot mix asphalt with open-graded aggregates
- HMAB: asphalt-treated base
- HMA: hot mix asphalt
- ATPB: asphalt-treated permeable base
- CTPB: cement-treated permeable base
- CTB-Class A: cement-treated base Class A
- CTB-Class B: cement-treated base Class B
- OGFC: open-graded friction course
- LTS: lime-treated subbase
- LCB: lean concrete base
- AB: aggregate base
- AS: aggregate subbase
- CIR: cold in-place recycled material

This empirical classification for each material is listed in Appendix Table A.2.

2.6 Summary

Mechanistic-empirical (ME) design procedures need to provide pavement performance predictions regarding different distresses that are considered critical. Each critical distress requires a computational model to describe how the distress develops in each pavement layer under various loading conditions.

The current version of *CalME,* version 2.0, has been developed by the UCPRC to enable Caltrans to design flexible pavements in California. The critical distresses in *CalME* 2.0 include fatigue cracking, reflective cracking, surface rutting, and ride quality deterioration in terms of smoothness measured using the International Roughness Index (IRI). Future enhancement of *CalME* will consider other important distresses, such as thermal cracking, top-down cracking, etc.

Each of the computational models for the distresses included in *CalME* has a set of model parameters that need to be determined. In order to use a material as part of a pavement design in *CalME* 2.0, one first needs to characterize the material by providing parameters for the computational models that predict fatigue damage, reflective cracking damage, and permanent deformation under different traffic and environmental loadings.

A Standard Materials Library (SML) has been introduced into *CalME* to provide a list of predefined materials for use in pavement design. The SML is essentially a collection of materials that have been characterized through previous studies. Specifically, model parameters and the associated uncertainties when applicable have been determined for these materials. Each material in the library has been classified in one of three groups—asphaltic material, cement-treated material, and unbound material—based on the models needed for that material.

The *CalME* SML continues to grow. In terms of material characterization, most of the current effort has focused on asphaltic materials, which are defined as materials bounded by asphalt binder and that are typically used in surface layers. These materials must be strong enough to allow production of viable laboratory specimens because a series of lab tests will be conducted on them to determine the fatigue and permanent deformation resistance of each material.

On the other hand, most of the models for nonasphaltic materials use default model parameters and require no additional laboratory testing for them to be characterized. The only exception is the stiffness of a pavement layer. Typically, layer stiffnesses are estimated with falling weight deflectometer (FWD) tests and the resulting data are used to backcalculate layer stiffness and to provide an estimate of the variability of the stiffnesses for Monte Carlo simulation.

3 LIST OF MATERIALS AND PARAMETERS

3.1 Introduction

California is a large and diverse state with such a large number of materials that can potentially be used in pavements that it is neither practical nor necessary to include every single possibility in the Standard Materials Library (SML). Instead, it is intended that the library include materials that are representative of the range of typical materials in each major materials category, and that new materials be added incrementally. *CalME* 2.0 allows minor adjustments on each material in the SML when necessary so that general local conditions can be accounted for.

For a properly designed flexible pavement, the surface layer is the most critical design component influencing its overall performance. On California highways, hot mix asphalt (HMA) with dense-graded aggregates and HMA with rubberized asphalt binder and gap-graded aggregates (RHMA-G) are the most commonly used structural surface layer materials for flexible pavements. Therefore, a decision was made to focus on developing model parameters for different regional HMA and RHMA-G mixes. But for other materials, such as lime-treated subbase (LTS), asphalt-treated permeable base (ATPB), etc., only one representative of each material type has been included.

Some of the materials included in *CalME* 2.0 SML have "HDM 2012" as part of the material name. Although these names suggest they are related to the 2012 version of Highway Design Manual (HDM 2012) *(13),* these materials are not defined in HDM 2012 with corresponding Mechanistic-Empirical design inputs. Instead, in 2013 Caltrans proposed adding a section called "Resilient Modulus" to Chapter 610, "Pavement Engineering Considerations," of the 2012 version of HDM. In the proposed addition, recommended values for stiffness and Poisson's ratio were given for various base, subbase, and subgrade materials. These values were adopted for those materials in *CalME* 2.0 whose names include "HDM 2012." The phrase "HDM 2012" will be removed in future versions of *CalME*.

Section 3.2 through Section 3.7 below present a complete, category-by-category list of the materials included in the SML released with *CalME* 2.0. For each material category, an introduction of the material is first given and this is followed by any relevant data collected for characterizing the material and a description of the materials in the SML that belong to the category.

3.2 Pavement Materials

3.2.1 Portland Cement Concrete

Although *CalME* is a tool for designing pavements with flexible surfaces, a portland cement concrete (PCC) layer has been included in the SML to represent the situation where an old cracked concrete layer is overlaid with an asphalt concrete surface (e.g., using the crack-and-seat-and-overlay with HMA strategy).

The PCC layer is not expected to have either damage or permanent deformation because a PCC layer cannot be used as the surface layer in a flexible pavement. The distresses in the PCC layer are not likely to deteriorate further after being overlaid with a relatively thick structural asphalt concrete overlay. Therefore the only model parameters needed are those for describing heat transfer and linear elasticity. The default model parameters for PCC can be found in Sections 2.3.1.1 and 2.3.2.1.

Only one PCC material is included in the SML and it is named as "PCC." *PCC Stiffness* and its variability included in the SML are based on a layer stiffness value backcalculated in an accelerated pavement testing study conducted on crack and seated PCC slabs overlaid with 150 mm of hot mix asphalt. The variability is relatively low compared to other materials and should be replaced with actual values that can for example be determined using in-situ stiffness backcalculation results.

Table 3.1: List of Material Dependent Parameters for PCC Materials

Material Name	Stiffness (MPa/ksi)	SDF for Stiffness
PCC	35,000/5,076	1.1

3.3 Asphaltic Materials

3.3.1 HMA and RHMA-G

There are a total of twenty-three mixes included in *CalME* 2.0. They are numbered consecutively from Mix #01 to Mix #25 (Mix #18 and Mix #20 were assigned to a material that was later removed) in the order they were added to the library. These materials represent fresh HMA and RHMA-G in newly placed surface layers.

3.3.1.1 <u>Material Selection</u>

The Standard Materials Library released in *CalME* 2.0 includes mixes produced in three rounds of testing for HMA and RHMA-G. Round One and Round Two focused on providing a wide range of performance for typical HMA and RHMA-G mixes used in California, while Round Three aimed at providing the performance of typical production mixes used by contractors throughout the state. Some of the mixes were prepared in the laboratory and some were field mixed during normal production with a contractor's asphalt mixing plant, details of which are provided later. Details of the mixes used in the three rounds of testing are shown below:

- Round One: to provide a wide range of mix performance for typical dense-graded HMA with conventional binders by using two very different aggregate rock types.
    - Binder PG grades: four binder grades to cover all typical binders used for dense-graded HMA (see Table 632.1 of the Highway Design Manual *[13]*)
        - PG 64-10
        - PG 64-16
        - PG 64-28
        - PG 70-10
    - Aggregate types: two aggregate types to cover heavily and lightly crushed aggregates respectively
        - Granite from a hard rock source
        - Partially crushed alluvial gravel
    - Total number of mixes: six (two of the aggregate/binder combinations were deemed not practical)
    - Mix designations: Mix #01 to Mix #06
- Round Two: to provide a wide range of mix performance for typical dense-graded HMA with modified binders, and a special dense-graded HMA called "rich-bottom" mix. This was achieved again by using two very different aggregate rock types.
    - Binder types:
        - PG 64-28 PM for regular HMA
        - PG 64-10 for rich-bottom HMA
    - Aggregate types: two aggregate types to cover good and poor aggregates respectively. These were sourced from the same quarry as the ones used in Round One.
        - Granite from a hard rock source
        - Partially crushed alluvial gravel
    - Total number of mixes: four
    - Mix designations: Mix #07 to Mix #10
- Round Three: to provide performance of a selected subset of mixes recommended by various Caltrans districts as being widely used and important. Caltrans district material engineers identified a pool of 15 mixes that were designed by contractors using the Hveem method. Each was modified by the UCPRC to meet the Caltrans Superpave specification (Caltrans Standard Special Provision *[14]* Section 39 Version 12-29-11), yielding 30 possible mixes. This was the final determination made:
    - Six Superpave mixes were selected to represent different California regions
    - Three Hveem mixes that correspond to three of the six selected Superpave mixes
    - Total number of mixes: nine
    - Mix designations: Mix #11 to Mix #21 (except Mix #18 and Mix #20)

In addition, the following six mixes are also included in the *CalME* 2.0 Standard Materials Library:

- Two HMA mixes designed for the long-life projects on I-5
  - Mix #22: an HMA mix with PG 64-10 binder, 1.2 percent lime, and 25 percent reclaimed asphalt pavement (RAP) designed for the long-life project on I-5 near Red Bluff in Tehama County
  - Mix #23: an HMA mix with PG 64-28 PM binder, 1.2 percent lime, and 15 percent RAP designed for the long-life project on I-5 near Weed in Siskyou County
- Two mixes used in the accelerated pavement testing track built for the SHRP II R21 project *(15)* at the UCPRC facility in Yolo County:
  - Mix #24: an RHMA-G mix with PG 64-16 base binder, no RAP with crushed alluvial aggregates
  - Mix #25: an HMA mix with PG 64-28 PM binder, no RAP with crushed alluvial aggregates
- Two additional HMA mixes designed for the long-life projects on I-5:
  - Mix #26: an HMA Rich Bottom mix with PG 64-10 binder, 1.2 percent lime and no RAP, designed for the pilot long-life project on I-5 near Red Bluff, this material has to be placed more than 100 mm below the surface because no permanent deformation is expected in this layer.
  - Mix #27: an HMA mix with PG 64-28 PM binder, 1.2 percent lime, and 15 percent RAP designed for the pilot long-life project on I-5 near Weed, this mix is different from Mix #23 in terms of the binder content and aggregate source.

A list of the HMA and RHMA-G materials included in *CalME* 2.0 is shown in Table 3.2. More details about each of the materials can be found in Appendix A.

3.3.1.2 Material Acquisition and Specimen Preparation

The mixes listed in Table 3.2 were prepared in two different ways: laboratory mixed and laboratory compacted (*LMLC)* and field mixed and laboratory compacted (*FMLC).* Laboratory compaction means collecting loose field mix samples and then compacting the samples in the laboratory after reheating.

LMLC specimens were prepared in the laboratory using aggregates and binders provided by suppliers throughout the state. The UCPRC acquired aggregates at the HMA plants and asphalt binder at the refinery through a combination of long-haul trucking and staff pickup. Once received, aggregates were dried, bulk-sieved to individual sizes, and batched according to the mix design formula. Asphalt binder was split into one-gallon samples and stored at a temperature of 20°C. Standard procedures were then followed to mix, compact, and cut specimens for testing. Short-term oven-aging (STOA) was applied to simulate the aging of mix that occurs during mix production, transport, and placement. Details of the specimen preparation procedure may be found in the final report for SPE 3.18.3, *Superpave Implementation Phase II: Comparison of Performance-Related Test Results (16).*

Table 3.2: HMA and RHMA-G Materials included in *CalME* 2.0

Testing Round	Binder Grade	Mix Design Specification	Aggregate Type/Source	District	Mix #	Preparation Method[*]
Round One	PG 64-28	Hveem	3/4" Blasted Granite	N/A	01	LMLC
		Hveem	3/4" Crushed River Gravel		N/A	N/A
	PG 64-16	Hveem	3/4" Blasted Granite	N/A	02	LMLC
		Hveem	3/4" Crushed River Gravel		03	LMLC
	PG 64-10	Hveem	3/4" Blasted Granite	N/A	04	LMLC
		Hveem	3/4" Crushed River Gravel		05	LMLC
	PG 70-10(16)	Hveem	3/4" Blasted Granite	N/A	06	LMLC
		Hveem	3/4" Crushed River Gravel		N/A	N/A
Round Two	PG 64-28PM[y]	Hveem	3/4" Blasted Granite	N/A	07	LMLC
		Hveem	3/4" Crushed River Gravel		08	LMLC
	PG 64-10 Rich Bottom	Hveem	3/4" Blasted Granite	N/A	09	LMLC
		Hveem	3/4" Crushed River Gravel		10	LMLC
Round Three	PG 64-16	Hveem	3/4" Crushed River Gravel	3	11	LMLC
		Superpave			12	LMLC
	PG 64-16	Hveem	3/4" Blasted Basalt	4	13	LMLC
		Superpave			14	LMLC
	PG 64-16 Rubberized	Superpave	1/2" Blasted Basalt	4	15	LMLC
		Hveem			16	LMLC
	PG 64-10 Rubberized	Superpave	3/4" Blasted Granite	8	17	LMLC
		Hveem			~~18~~[x]	~~LMLC~~
	PG 64-28PM[y]	Superpave	1" Blasted Granite	8	19	LMLC
		Hveem			~~20~~[x]	~~LMLC~~
	PG 70-10	Superpave	3/4" Blasted Granite	6	21	LMLC
I-5 near Red Bluff or Weed	PG 64-10	Long Life	3/4" Crushed River Gravel 25% RAP, 1.2% lime	2	22	LMLC
	PG 64-28PM[y]	Long Life	3/4" Crushed River Gravel 15% RAP, 1.2% lime	2	23	LMLC
UCPRC Test Track	PG 64-16 Rubberized	Hveem	1/2" Crush River Gravel	3	24	FMLC
	PG 64-28PM[y]	Hveem	3/4" Crushed River Gravel	3	25	FMLC
I-5 near Red Bluff or Weed	PG 64-10 Rich Bottom	Long Life	3/4" Crushed River Gravel 0% RAP, 1.2% lime	2	26	LMLC
	PG 64-28PM[y]	Long Life	3/4" Crushed River Gravel 15% RAP, 1.2% lime	2	27	LMLC

[*]: LMLC = Laboratory-mixed and laboratory-compacted; FMLC = field-mixed and laboratory-compacted
[x]: Mix #18 and Mix #20 were assigned but never tested, and therefore they have not been included in the standard materials library.
[y]: PM = polymer modified

As mentioned above, FMLC specimens were prepared from loose mix sampled from haul trucks in the field. The loose mix was reheated until the compaction temperature was reached uniformly, approximately two hours, and then it was compacted. No STOA was applied because the field mixes have already been aged during production at the plant.

3.3.1.3 Laboratory Testing

Laboratory testing conducted for HMA and RHMA, along with the experiment designs, were described in the preceding sections:

- 2.3.2.2: Asphaltic Stiffness Master Curve
- 2.3.3.1: Asphaltic Fatigue Damage
- 2.3.3.2: Asphaltic Permanent Deformation

In addition, repeated load triaxial (RLT) tests (AASHTO TP 79) were conducted as part of PPRC SPE 3.18.3 to see if a correlation could be found between RSCH and RLT parameters. The findings from that testing can be found in the SPE 3.18.3 final report *(16).*

3.3.2 Cold In-Place Recycled (CIR) Material

The same set of laboratory tests were required to characterize CIR materials as HMA and RHMA-G. The laboratory specimens needed included beams for AASHTO T 321 and cores for AASHTO T 320. These specimens were produced from slabs taken from the field.

Only one CIR material is included in *CalME* 2.0 under the name *Cold In-Place Recycled Asphalt.* The specimens were sampled from State Route 16 (SR-16) in Colusa County near the SR-20 intersection on April 30, 2010. Figure 3.1 shows the saw cut pattern and the slabs taken from the field.

(a) Saw cut pattern

(b) Slab samples, porous part is the CIR layer

Figure 3.1: Taking slabs from the field to produce beam and core specimens in the laboratory

The CIR layer was constructed under Contract #03-2M4604 between July and August 2007 from PM R3.4 to PM 7.3. The existing pavement has 100 to 150 mm of HMA over 0 to 450 mm of aggregate base. The CIR layer was 50 mm thick and had asphalt emulsion as the binding agent. The CIR layer was coated with a light application of asphaltic emulsion and sand and opened to traffic for seven days before a 30 mm overlay of HMA was placed.

The maximum specific gravity of the CIR layer was determined in the lab to be 2.377 following AASHTO T 209. A total of 18 beams and 26 cores were produced from the field slabs. The air-void contents of the beams and cores ranged from 13 percent to 22 percent with an average of 16 percent for beams and 15 percent for cores based on bulk specific gravities measured following AASHTO T 331 with CoreLok.

The model parameters for the *Cold In-Place Recycled Asphalt* material were determined by running the same set of laboratory tests listed in Section 3.3.1.3.

3.3.3 Old Hot Mix Asphalt

When using *CalME* to simulate the performance of a rehabilitation design, model parameters are needed for the existing (and hence old) HMA layer. To accommodate this situation, *CalME* 2.0 includes a material named *Old HMA* to represent generic old HMA layers. The model parameters for Old HMA were developed based on laboratory test results from old HMA specimens sampled from various locations. It is expected that this material will be adjusted in actual designs based on FWD testing so at a minimum its stiffness will be characterized. Additionally, fatigue testing can be conducted on samples taken from old existing pavement to characterize cracking performance of the in-situ material.

3.3.3.1 General Strategy

Given the potential broad variability in old HMA layer properties such as age, mix design, and condition, great uncertainty is introduced in predicted pavement performance by using a single generic material to represent all old HMA layers. It is therefore desirable to do laboratory characterization of the old HMA layers for each project if they are considered critical for the pavement performance. However, since it is not always possible to characterize these old layer materials in the laboratory, large variability has been built into the model parameters for the generic *Old HMA* material.

As shown in Section 3.3.1.3, one needs to provide model parameters for the asphaltic stiffness master curve, asphaltic fatigue damage model, and asphaltic permanent deformation model since old HMA is treated as an asphaltic material.

3.3.3.1.1 *Stiffness Master Curve*

FWD testing on the existing pavement should always be conducted as part of the Mechanistic-Empirical design process to characterize the asphaltic stiffness master curve. Layer stiffness backcalculation can be performed using the FWD data to yield the in situ stiffness and its variability in the old HMA layer. These values can be entered in *CalME* 2.0 as modifications to the *Old HMA* material without changing the shape of the master curve of the material.

If beam specimens can be produced for the existing HMA layer, frequency sweep testing using the four-point bending beam (AASHTO T 321) should always be conducted as part of the laboratory testing of old HMA. This will yield the full stiffness master curve for old HMA layer. However, one should still use the in situ stiffness of the existing HMA layer developed from the stiffness backcalculation so that the weakening effects of fatigue and cracking can be accounted for. The shape of the stiffness master curve however should come from the laboratory test results.

3.3.3.1.2 *Asphaltic Fatigue Damage Model*

In *CalME*, fatigue damage is accumulated for all HMA layers no matter how deep they are. However, old HMA layers are typically in deeper parts of the pavement and are likely to experience very low tensile strain and therefore very little fatigue damage. Some literature has suggested that 70 microstrain *(17)* is the endurance limit below which fatigue damage can practically be ignored for the HMA layer. Therefore, it is recommended that laboratory testing for fatigue resistance is only needed for the old HMA layer if the horizontal tensile strain at the bottom of the old HMA layer will exceed 70 microstrain under the typical axle load. This strain could be numerically evaluated using multilayer elastic analysis with the backcalculated stiffness of the old HMA and other layers in the pavement section and a reasonably assumed stiffness and thickness for the new HMA overlay.

3.3.3.1.3 *Rutting Resistance*

In *CalME* it is assumed that rutting occurs in only the top 100 mm (4 in.) of HMA layers. It is therefore only necessary to conduct laboratory testing for rutting resistance when the old HMA layer will be in the top 100 mm.

3.3.3.2 Generic Old HMA Material

The generic *Old HMA* material included in *CalME* 2.0 is based on laboratory testing results UCPRC conducted on several old HMA materials sampled from various highways in California over the years. A list of the available laboratory test data is shown in Table 3.3. In the table, “Full” indicates a full factorial for the experiment design shown in Table 2.2 for stiffness, in Table 2.4 for fatigue, and in Table 2.5 for rutting. “None” indicates that no data is available.

Table 3.3: List of Old HMA Laboratory Test Data

Old HMA #	County	Route	Year Placed	Year Sampled	Age (Years)	Stiffness	Fatigue	Rutting
1	Kings	SR-198	1963	2008	55	Full	Full	55°C only
2	Kings	SR-198	1999	2008	9	Full	Full	None
3	Lake	SR-53	1956	2007	51	Full	Full	70 kPa and 130 kPa only
4	Contra Costa	I-580	1989	2007	18	Full	Full	None
5	Humboldt	US 101 PM 127.5	1992	2007	15	Full	Full	None
6	Humboldt	US 101 PM127.5	2002	2007	5	Full	Full	None
7	Humboldt	US 101 PM129.9	1992	2007	15	Full	Full	None
8	Humboldt	US 101 PM129.9	2002	2007	5	Full	Full	Full
9	San Joaquin	I-580	1996	2005	9	None	None	Full
10	Shasta	I-5	1987	2005	18	None	None	Full

3.3.3.2.1 Stiffness Master Curve

Since the minimum stiffness in *CalME* is fixed and the maximum stiffness will be indirectly determined by the backcalculated in-situ stiffness, the shape of a stiffness master curve is only controlled by β and γ. So, parameters δ, β and γ will come from the generic material, while α will come from the backcalculated stiffness.

The stiffness master curves of various old HMA materials at 10 Hz loading frequency are shown in Figure 3.2, which indicates that the shapes of most of the stiffness master curves are actually quite similar. Mix #05 in Table 3.3 (15 year-old HMA on State Route 101 PM 127.5 in Humboldt County) has been selected as the generic material for providing the shape of stiffness master curve.

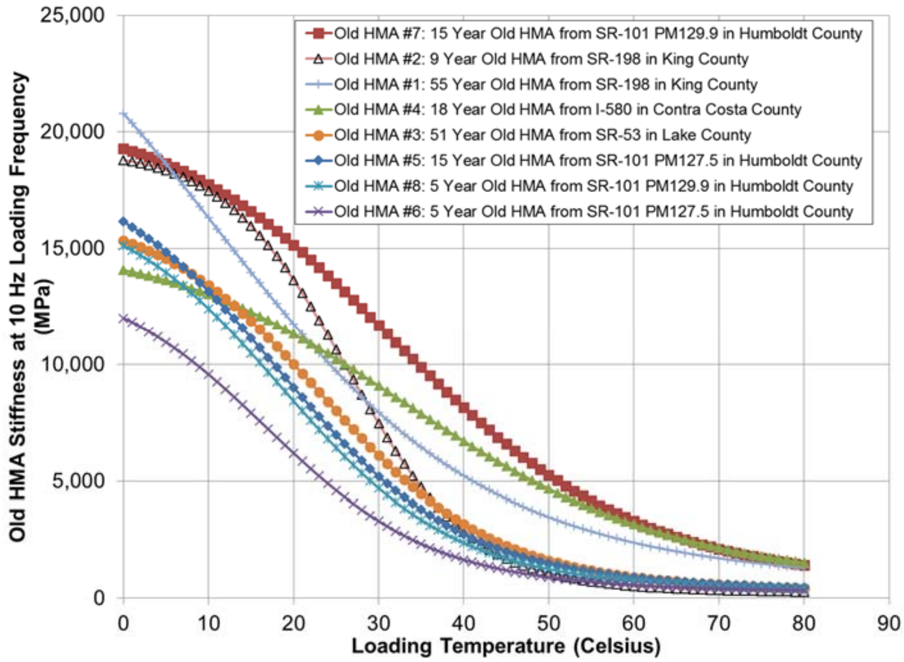

Figure 3.2: Stiffness master curves of different old HMA under 10 Hz loading frequency (no stiffness data for Mix #9 and Mix #10).

3.3.3.2.2 Asphaltic Fatigue Damage Model

Figure 3.3 shows the stiffness reduction curves of various old HMA materials tested at 20°C, 10 Hz, and 200 microstrain. There are large variabilities in the curves. The *fatigue life*, defined as the number of load repetitions to a 0.5 residual stiffness ratio, ranges from 60,000 to 5,000,000. A Q-Q plot of the natural log of fatigue life for the old HMA included in Figure 3.3 is shown in Figure 3.4. Figure 3.4 indicates that the fatigue life can be approximated by the lognormal distribution reasonably well although many other distributions can work equally well given the small sample size. Assuming lognormal distribution, the average fatigue life is 486,000 and the standard deviation factor (SDF) is 4.8, which means one standard deviation of fatigue life corresponds to 4.8 times longer or shorter than the average value.

The SDF for old HMA is significantly higher than the default SDF of 1.15 for HMA fatigue life, indicating much a larger uncertainty associated with using a generic old HMA material to represent all old HMA.

According to Figure 3.3, the average fatigue resistance can be represented by Mix #02 in Table 3.3, (nine-year-old HMA from SR-198 in Kings County). However, the SDF for model parameter A, which controls fatigue life, is 4.8.

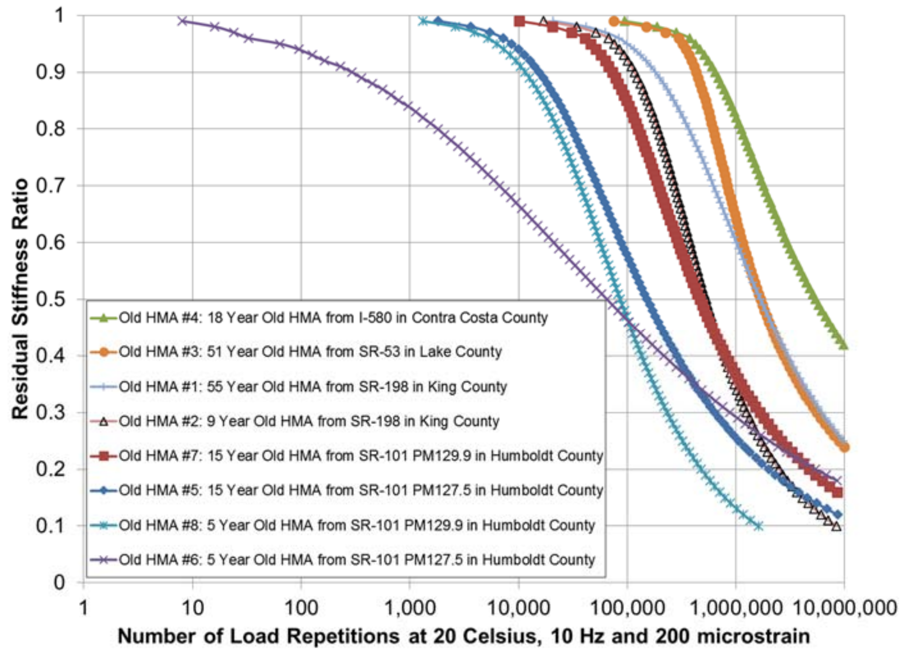

Figure 3.3: Stiffness reduction curves for various old HMA materials tested under 20°C, 10 Hz, and 200 microstrain.

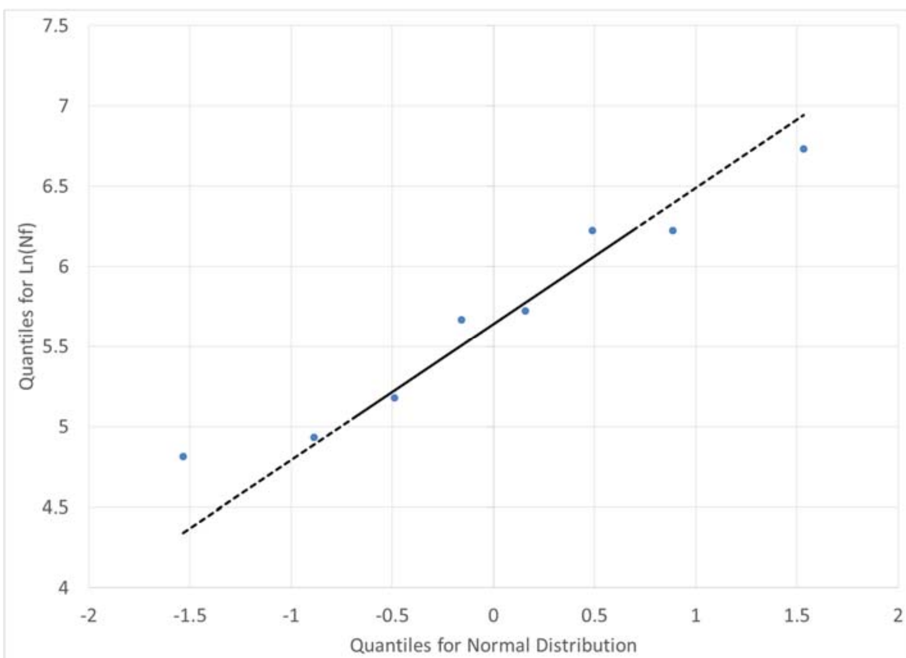

Figure 3.4: Q-Q plot of natural log of fatigue life for the old HMA shown in Figure 3.3.

3.3.3.2.3 Rutting Resistance

In *CalME*, variability in rutting resistance is accounted for by assuming $\exp(A)$ follows a lognormal distribution (see Equation (20) for the definition). The rutting equations indicate that exp(A) controls the amount of inelastic shear strain when everything else is the same.

Figure 3.5 shows the accumulation of inelastic shear strain for different old HMA materials under repeated shear loading at 100 kPa. Large variability is shown in the figure. The amount of inelastic shear strains after 10,000 repetitions are 0.00162, 0.00359, 0.00475, and 0.0101 for the four mixes. Assuming lognormal distribution, the average inelastic shear strain is 0.0041 with a standard deviation factor of 2.1. This means the average rutting resistance can be roughly represented by Mix #10 (18-year-old HMA from I-5 PM 29.2 in Shasta County) (see Table 3.3) with an SDF of 2.5. Note that the default SDF for rutting resistance of new HMA mixes is 1.2.

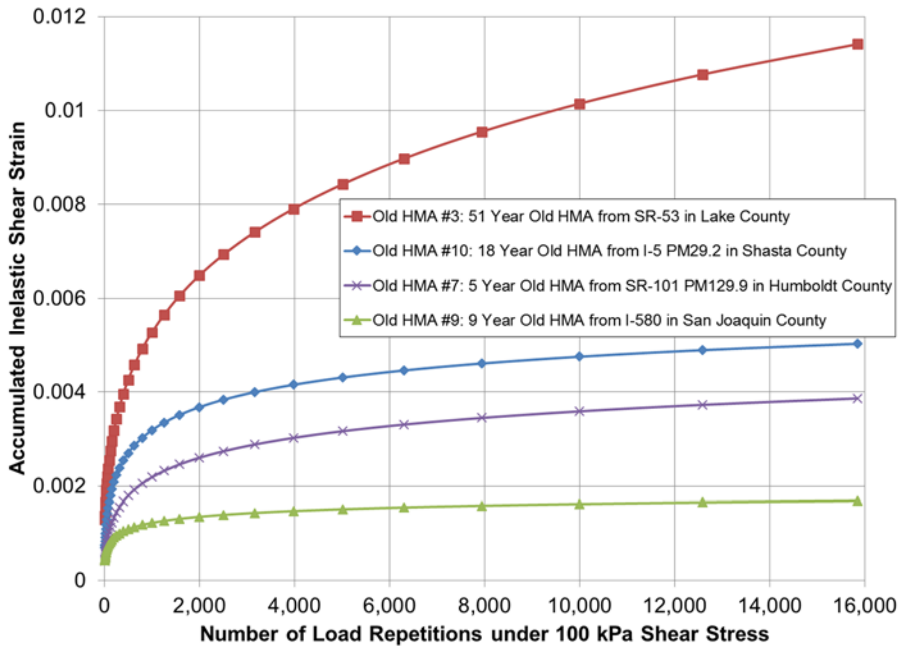

Figure 3.5: Accumulation of inelastic shear strain for different old HMA materials under repeated shear loading at 100 kPa.

3.3.3.2.4 Overall Model Parameters

As noted in the discussion above, in order to represent the average performance of old HMA in *CalME*, in terms of the master curve, fatigue resistance, and rutting resistance, the material *Old HMA* was formulated by combining the behaviors of three different old HMA mixes (Mix #05, Mix#02, and Mix#10). In addition, laboratory test results on available old HMA samples yielded uncertainties in fatigue and rutting resistance that were much higher than the default values for old HMA. Table 3.4 lists the sources and values of different properties for the generic old HMA material.

Table 3.4: Sources and Values of Different Properties for the Generic Old HMA Material

Property	Subproperty	Source	Value
Stiffness master curve	In situ stiffness	Backcalculation	
	Shape of the master curve	Mix 5 in Table 3.3	
	SDF of in situ stiffness	Backcalculation	
Fatigue resistance	Model parameters	Mix 2 in Table 3.3	
	SDF of parameter A		4.8
Rutting resistance	Model parameters	Mix 10 in Table 3.3	
	SDF of parameter $\exp(A)$		2.1

3.3.3.3 <u>Summary</u>

CalME model parameters for old HMA layers can be obtained either by conducting laboratory tests for the specific material or by using the *Old HMA* material included in *CalME* 2.0 when no test data is available.

Laboratory testing for old HMA layer stiffness is often preferred over estimates, but fatigue testing is recommended when an old HMA layer will have strains at the bottom that are higher than 70 microstrain under a typical axle load. Rutting tests are only necessary when the old HMA layer will be in the top 100 mm.

When model parameters are derived from project-specific laboratory testing data, the default variability in fatigue and rutting resistance can be used. Specifically, the standard deviation factors for fatigue and rutting resistance are 1.15 and 1.2 respectively.

When no laboratory test data is available, an old HMA material can be represented by a generic old HMA material named *Old HMA* in *CalME* 2.0 that represents the average performance of several old HMA mixes tested by the UCPRC in the past. The ages of these old HMA mixes range from five to fifty-five years and they were sampled from various sites throughout California. Using the generic *Old HMA* material property requires the inclusion of a much higher variability in fatigue and rutting resistance. In particular, the SDF for fatigue life is 4.8 while the SDF for accumulated rut is 2.1.

3.3.4 Model Parameters for Asphaltic Materials in CalME 2.0

Model parameters for all asphaltic materials included in *CalME* 2.0 are listed in Appendix B.

3.4 Cement-Treated Materials

3.4.1 Cement-Treated Base

Caltrans uses two cement-treated base (CTB) materials: CTB-Class A and CTB-Class B (Table 663.1B in the Highway Design Manual *[13]* and Section 27 in Standard Specifications *[18]*). The UCPRC has conducted falling weight deflectometer (FWD) testing on several California highway locations that include CTB layers. The locations and FWD-backcalculated CTB layer stiffnesses are listed in Table 3.5, which indicates large variations in the in situ stiffness for CTB-Class A layer. It is believed that this large variation reflects the range of parent materials, cement contents, and fatigue damage at different locations.

Table 3.5: List of Locations and Backcalculated CTB Layer Stiffnesses

No.	Location	Lane Number	Year Placed	Date Tested	Material	Average	SDF	Reference
1	I-80 near Auburn	N/A	N/A	7/11/2005	Class A	2,550	1.4	*(19)*
2	I-580 near Richmond (PM 2.6 to 1.7)	3/3	1989	8/7/2007 and 8/9/2007	Class A	7,023 to 10,427	1.3	N/A
3	SR-198 near Lemoore (PM 9.2 ~17.9)	2/2	1963	2/6/2008 ~2/7/2008	Class B	2,499 to 4,833	1.5	*(20)*
4	SR-101 in Santa Rosa (PM 22.40 to 22.30)	Outside shoulder	2008~ 2010	5/19/2010	Class B	6,619	1.4	N/A

3.4.1.1 CTB Materials in *CalME*

The generic CTB materials in *CalME* 2.0 are named *CTB-Class A, HDM 2012* and *CTB-Class B, HDM 2012.* The names indicate that their stiffnesses are based on recommendations that Caltrans proposed for use in the HDM 2012, as mentioned in Section 3.1. The stiffnesses and the corresponding standard deviation factor (SDF) values for CTB materials are listed in Table 3.6.

Table 3.6: List of CTB Materials in *CalME* 2.0

Material Name	Stiffness (MPa/ksi)	SDF for Stiffness
CTB-Class A, HDM 2012	9,653/1,400	1.20
CTB-Class B, HDM 2012	7,584/1,100	1.35

Compared to the backcalculation results shown in Table 3.5, these values are high in stiffness and low in SDF because they represent the initial condition of these materials.

3.4.2 Cement-Treated Permeable Base

No field testing data are available for characterizing cement-treated permeable base (CTPB). The generic CTPB material in *CalME* 2.0 is named as *CTPB, HDM 2012*. The name indicates that its stiffness is based on recommendations in the proposed Caltrans change to HDM 2012 mentioned in Section 3.1. The stiffness and the corresponding standard deviation factor (SDF) values for CTPB materials are listed in Table 3.7. Note that these parameters for CTPB are exactly the same as those listed in Table 3.6 for CTB-Class B.

Table 3.7: CTPB Material in *CalME* 2.0

Material Name	Stiffness (MPa/ksi)	SDF for Stiffness
CTPB, HDM 2012	7,584/1,100	1.35

3.4.3 Lean Concrete Base

To estimate the in situ stiffness for lean concrete base (LCB), FWD testing was conducted on US 101 southbound Lane #1 over a 1,141 m section between PM 27.309 and PM 26.600 near Santa Rosa. The FWD testing interval was about 16 m (50 ft). The as-built structure included 335 mm (1.10 ft) of asphalt-bound material over 147 mm (0.48 ft) of LCB, followed by 200 mm (0.66 ft) of aggregate subbase over subgrade based on coring and dynamic cone pentation data. Coring also indicates disintegrated LCB layers in two of five locations. The LCB layer was constructed between 2008 and 2010 as part of Contract No. 04-0A10U4.

The backcalculated LCB layer stiffnesses are listed in Table 3.8. According to the data, the short section can be clearly divided into two subsections, each with a distinct LCB layer stiffness. The difference in LCB layer stiffness cannot be attributed to traffic because there is no on- or off-ramp to separate these two subsections. With no additional field data, the conservative design inputs used for the LCB layer are shown in Table 3.9.

Table 3.8: Backcalculated LCB Layer Stiffnesses from US 101 near Santa Rosa

Sub-section	Starting PM	Ending PM	Number of Points	Stiffness (MPa/ksi)	SDF
1	26.770	26.600	18	25,507/3,699	1.52
2	27.309	26.780	42	7,193/1,043	1.79
Overall	27.309	26.600	70	10,175/1,475	2.19

Note: 1 ksi = 1,000 psi

Table 3.9: Conservative Parameters for LCB Material in *CalME* 2.0

Material Name	Stiffness (MPa/ksi)	SDF for Stiffness
Lean Concrete Base	6,000/870	1.35

3.5 Unbound Materials

3.5.1 Aggregate Base

Caltrans has specifications for two types of aggregate base (AB): Class 2 and Class 3 (Section 26, *[18]*) but typically only Class 2 is used. Accordingly, only *AB-Class 2* is included in *CalME* 2.0.

The UCPRC has conducted FWD testing and backcalculated AB-Class 2 stiffnesses from various locations including highway, county roads, and test tracks. Backcalculated AB-Class 2 stiffnesses range from 100 to 400 MPa (~15 to 60 ksi) with SDF values range from 1.20 to 1.50.

3.5.2 Aggregate Subbase

Caltrans has specifications (Section 25, *[18]*) for three types of aggregate subbase (AS): Class 1, Class 2, and Class 3. AS-Class 4 and AS-Class 5 are listed in the specification as "reserved sections" and have not been defined. Accordingly only *AS-Class 1, AS-Class 2,* and *AS-Class 3* are included in *CalME* 2.0.

The UCPRC has conducted FWD testing on a test track *(21),* constructed for accelerated pavement testing using the Heavy Vehicle Simulator, in which the subbase layer can be classified as AS-Class 1. The backcalculated subbase/base combined layer stiffness has an average between 105 and 285 MPa (~15 and 40 ksi) and a coefficient of variance between 0.25 and 0.51 (Table 4.8, *[21]*), which correspond to SDFs of 1.28 and 1.62 respectively.

No backcalculated layer stiffnesses are available for AS-Class 2 and AS-Class 3.

3.5.3 Subgrade

Caltrans uses the Unified Soil Classification System (ASTM D2487) to classify soils. According to the Caltrans HDM (Section 614.2, *[13]*), highly organic soils must be removed before placing additional layers for a pavement.

The UCPRC backcalculated in situ stiffnesses from FWD testing on various types of subgrade soils. However, a comprehensive database for typical stiffness based on soil classification is still under development at the time of writing of this technical memorandum. Suggested values for the various soil types are available in *CalME 2.0* and also in the Caltrans *Highway Design Manual* (HDM), Chapter 630. Note that these values will be needed when designing a new pavement structure, whereas when designing overlays of existing pavements the subgrade stiffness will be backcalculated from FWD testing and automatically uploaded into *CalME 2.0*.

3.5.4 Asphalt-Treated Permeable Base

Caltrans has only one generic classification for asphalt-treated permeable base (ATPB) (Section 29, *[18]*). It is produced the same way as hot mix asphalt except with lower binder content that has a default value of 2.5 percent by weight of aggregate. It has historically been used almost exclusively as a 75 mm (0.25 ft) thick drainage layer directly below the dense-graded asphalt concrete layers.

The UCPRC conducted FWD testing on eastbound Interstate 580 in Richmond (PM 2.6 to PM 1.7) in 2007. The pavement was constructed in 1989 under Contract No. 04-108744 and contains a 90 mm ATPB layer. Coring indicates that the ATPB has disintegrated into loose rocks at PM 2.6 but is relatively intact at PM 1.7. The backcalculated stiffness for the ATPB layer is about 600 MPa (~85 ksi) on average with coefficients of variance around 0.30.

In a study conducted by the UCPRC for investigating the performance of drained and undrained flexible pavements *(22),* ATPB was constructed as the drainage layer in test sections for accelerated pavement testing using the HVS. It was found that the ATPB layer stiffness decreased from around 1,500 MPa before HVS trafficking to between 200 and 400 MPa after HVS trafficking. This significant reduction in the stiffness of the ATPB was due to stripping resulting from loading and the intrusion of fines from the aggregate base, regardless of the presence of water. Caltrans and UCPRC researchers have noted that ATPB has about a 50 percent chance of stripping in the field within 10 years of construction.

It was decided to use ATPB in its stripped condition in *CalME* 2.0. Specifically, ATPB is treated as an unbound material with stiffness similar to a Class 2 aggregate base.

3.5.5 Full-Depth Reclamation with No Stabilization

The process of full-depth reclamation with no stabilization (FDR-NS) is also referred to as *pulverization,* which is a roadway rehabilitation strategy that involves in-place recycling of the entire existing flexible pavement layer and some of the existing underlying granular base material *(23).* It has primarily been used in District 2. The pulverization process transforms an existing distressed flexible pavement into base for a new flexible pavement structure. The Caltrans 2010 Standard Specification allows up to 100 percent reclaimed aggregate from "reclaimed processed asphalt concrete, PCC, LCB, or CTB" (Section 25 and 26, *[18]*) for use in both aggregate base and subbase.

The UCPRC has conducted extensive research on FDR-NS (pulverized) material *(24).* Comprehensive laboratory and field tests of the pulverized materials were conducted and the results were compared with those of typical aggregate materials. It was concluded that the pulverized material is stiffer than typical aggregate base material and the permanent deformation resistance of the pulverized material was worse than that of the typical aggregate base material in California at low stress levels but better at higher stress levels. *CalME* simulations (Table 27, *[24]*) suggested that the difference in accumulated permanent deformation in the aggregate base layer after 20 years of trafficking were minimal between the pavements using pulverized material and the ones using typical aggregate material.

It was decided to consider FDR-NS material as typical aggregate base, i.e., AB-Class 2.

3.5.6 Unbound Materials in CalME *2.0*

All of the unbound materials included in *CalME* 2.0 are listed in Table 3.10. As mentioned earlier, the "HDM 2012" suffix in material names indicates that the corresponding stiffness came from the change to the HDM proposed in 2012.

Table 3.10: List of Unbound Materials in *CalME* 2.0

Type	Classification	Name in *CalME*	Stiffness (MPa/ksi)	SDF for Stiffness
Aggregate Base	AB-Class 2	AB-Class 2, HDM 2012	310/45.0	1.20
Aggregate Subbase	AS-Class 1	AS-Class 1, HDM 2012	241/34.9	1.20
	AS-Class 2	AS-Class 2, HDM 2012	206/29.9	1.20
	AS-Class 3	AS-Class 3, HDM 2012	172/24.9	1.20
Subgrade	GW	GW, HDM 2012	262/38.0	1.20
	GM	GM, HDM 2012	206/29.9	1.20
	GP	GP, HDM 2012	199/28.9	1.20
	SW	SW, HDM 2012	144/20.9	1.20
	SM	SM, HDM 2012	144/20.9	1.20
	GC	GC, HDM 2012	137/19.9	1.20
	SP	SP, HDM 2012	117/17.0	1.20
	SC	SC, HDM 2012	96/13.9	1.20
	ML	ML, HDM 2012	75/10.9	1.20
	CL	CL, HDM 2012	62/9.0	1.20
	MH	MH, HDM 2012	41/5.9	1.20
	CH	CH, HDM 2012	27/3.9	1.20
ATPB	ATPB	Asphalt-Treated Permeable Base	300/43.5	1.20
FDR-NS	AB-Class 2	Full-Depth Reclamation with no Stabilization	300/43.5	1.20

Note: 1 ksi = 1,000 psi

3.6 Lightly Cemented Materials

3.6.1 Recycled Hardened Concrete Aggregate

The Caltrans 2010 Standard Specification allows up to 100 percent reclaimed aggregate from "reclaimed processed asphalt concrete, PCC, LCB, or CTB" (Section 25 and 26 *[18]*) for use in both aggregate base and subbase in California. Recycled hardened concrete aggregates (RHCA) are considered to have the same quality as natural aggregates. Although there is no limitation on using RHCA, there is no benefit to using it either.

In *CalME* 2.0, recycled hardened concrete aggregate is considered a lightly cemented material, based on observations of recementation in HVS test tracks and highways with base layers made of various percentages of RHCA *(25)*. The available data suggest that recementation will lead to stiffness increases for the RHCA layer. The extent of recementation was found to depend on the percentage of RHCA in the material. After reviewing the existing data, the stiffnesses for aggregate base layers with different percentages of RHCA are shown Table 3.11.

Table 3.11: Aggregate Base Stiffness with Different Percentages of RHCA *(25)*

Percentage of Crushed Recycled Hardened Concrete Aggregate in Aggregate Base (%)	Suggested Stiffness Modulus MPa (ksi)
100	2,500 (350)
50 to 100	1,000 (150)
<50	250 (40)

Note: 1 ksi = 1,000 psi

As discussed in Section 2.2.5, recementation in lightly cemented material is accounted for by the cementitious curing model, while the loss of recemented bond caused by trafficking is described by the cementitious crushing damage model.

3.6.1.1 Recycled Hardened Concrete Aggregate Material in *CalME* 2.0

Given the limited amount of data available, it was decided to adopt a conservative value for the stiffness of an RHCA layer when it is used as aggregate base. There is only one RHCA material in *CalME* 2.0 called *Recycled Hardened Concrete Aggregate,* which represents aggregate base layers with at least 50 percent RHCA. According to Table 3.11, the recommended stiffness for the Recycled Hardened Concrete Aggregate material in *CalME* is 1,000 MPa (~145 ksi). It is further assumed that this stiffness gain due to recementation will take one year based on observations from HVS test sections (Figure 4.5, *[26]*).

In *CalME*, the default initial age of a new flexible pavement is ninety days, corresponding to the delay between placement and opening to traffic. The ninety-day stiffness for RHCA needs to be 669 MPa in order to have a one-year stiffness of 1,000 MPa using the default model parameters for cementitious curing (Section 2.2.5.1).

Accordingly, the initial stiffness for RHCA material was set to 650 MPa in *CalME* 2.0, and a default SDF value of 1.20 for stiffness is used.

3.6.2 Lime-Treated Subbase

Lime-treated subbase (LTS) is covered as "Lime Stabilized Soil" in Section 24 of Caltrans 2010 Standard Specification *(18)*. In the Highway Design Manual *(13),* LTS is covered in Chapter 660, titled "Base and Subbase," among other places.

3.6.2.1 <u>Field Testing Conducted by the UCPRC</u>

The UCPRC conducted FWD testing on County Road 27 (CR-27) in Yolo County east of Interstate 505 between station 244 m (8+00 ft) and 1,798 m (59+50 ft). The pavement consists of 300 mm (12 in.) of lime-treated subbase (LTS), 445 mm (17.5 in.) of AB-Class 2, and 75 mm (3 in.) of HMA. The LTS layer was the result of treating a clay-silt subgrade with 5 percent lime (4 percent by dry weight). CR-27 is a two-lane road (one lane in each direction). It was reconstructed in the year 2000 and was extensively cracked in the wheelpaths at the time of FWD testing.

FWD tests were done at 25 m intervals in each direction between the two wheelpaths, and staggered at 12.5 m in opposite directions to allow better coverage. Tests were conducted on May 14, 2010, and May, 18, 2010, for cold and warm sessions respectively. The pavement was ten years old at the time of testing.

A ground-penetrating radar (GPR) scan was conducted on June 2, 2010. Layer thickness data was taken at every 0.25 m interval. In order to find the actual layer thickness for each FWD drop, the distance measurement instrument (DMI) data recorded for each drop was corrected by matching the beginning and ending locations between FWD and GPR.

At each FWD drop location, layer thickness was taken as the average value from GPR scans for points within a 1.0 m distance. A four-layer system consisting of HMA, aggregate base, aggregate subbase, and subgrade layer was used for backcalculation. All layers were assumed to be linear elastic and all four layers were used in the backcalculation for each test section.

The backcalculated LTS layer stiffness showed significant variation, with most values falling between 200 MPa (10th percentile) and 2,000 MPa (90th percentile). Assuming a lognormal distribution, the average LTS layer stiffness on CR-27 was 628 MPa with an SDF of 2.44.

3.6.2.2 Recommended Design Parameters for LTS from Literature

There is an extensive literature available that covers the construction and performance of pavement structures incorporating lime-stabilized materials. A comprehensive review was given by Mallela et al. *(27)* on the consideration of lime-stabilized layers in Mechanistic-Empirical pavement design.

Mallela et al. (*27*) recommended that the resilient modulus (M_r) of lime-stabilized soils can be estimated from the unconfined compressive strength (q_u) of the cured soil-lime samples obtained in accordance with Mixture Design and Testing Protocol (MDTP, *[28]*). The 28-day q_u is measured and determined in accordance with ASTM D5102 as part of the MDTP. The resilient modulus can subsequently be estimated from q_u as follows (Thompson *[29]*):

$$M_r = 0.124 \times q_u + 9.98 \qquad \textbf{(40)}$$

where: M_r = resilient modulus in ksi, and
q_u = unconfined compressive strength in psi

This equation was also adopted in the change to the Caltrans Highway Design Manual proposed in 2012. When strength data is not available, it is believed *(27)* that resilient moduli of 200 MPa (30 ksi) to 400 MPa (60 ksi) can be readily achieved for reactive soils (with 25 percent passing the No. 200 sieve and a PI of at least 10). However, the exact extent of the increase is a function of the soil mineralogy and lime content.

Lime-treated soils may show continuous stiffness increases due to pozzolanic reactions between soil and lime. The extent to which the reaction proceeds is influenced primarily by natural soil properties.

3.6.2.3 LTS Material in *CalME* 2.0

One LTS material is included in *CalME* 2.0, named *LTS, HDM 2012*. As mentioned before, the "HDM 2012" suffix indicates that LTS stiffness came from the change to the HDM proposed in 2012. The initial stiffness for the *LTS, HDM 2012* material is 600 MPa, which corresponds to an unconfined compressive strength of 700 psi. The corresponding SDF for the LTS stiffness is 1.20. The stiffness is achievable based on the above discussions but it represents a lime-treated reactive soil because it is considered a lightly cemented material in *CalME* so its stiffness will continue to increase. The in situ stiffness will increase to 900 MPa after one year of in situ curing.

3.7 Lightly Asphalt-Bound Materials

3.7.1 Full-Depth Reclamation with Foamed Asphalt

Full-Depth Reclamation with Foamed Asphalt (FDR-FA) was previously referred to as cold foam in-place recycling (CFIPR). It is an in-place flexible pavement rehabilitation strategy that transforms existing asphalt concrete (AC) into a stabilized base for a new pavement surface layer using foamed asphalt as the stabilizing agent *(30)*.

3.7.1.1 Mechanical Characteristics of FDR-FA Material

UCPRC has conducted extensive research on FDR-FA material and its mechanical characteristics in flexible pavements *(31,32)*. It was found that FDR-FA layer stiffness is significantly affected by in situ moisture content and layer temperature. The effect of long-term curing on FDR-FA stiffness was believed to be minimal even when up to 2 percent of cement was used. This research showed that cementitious active fillers should be considered in all FDR-FA projects to provide early strength for early trafficking of the rehabilitated road.

A conceptual illustration of the moisture sensitivity of FDR-FA stiffness *(31),* which was developed based on both field and laboratory testing data, is shown in Figure 3.6. The field data indicates that stiffness reduction of the FDR-FA layer under field moisture conditions in wet seasons could be as much as 45 percent compared to that in the dry seasons. Based on backcalculation results (pg. 57, *[31]*), the stiffness for the full-depth reclamation with foamed asphalt (FDR-FA) layer with 1 to 2 percent cement ranged from 600 to 1,100 MPa for wet seasons and 900 to 2,000 MPa for dry seasons.

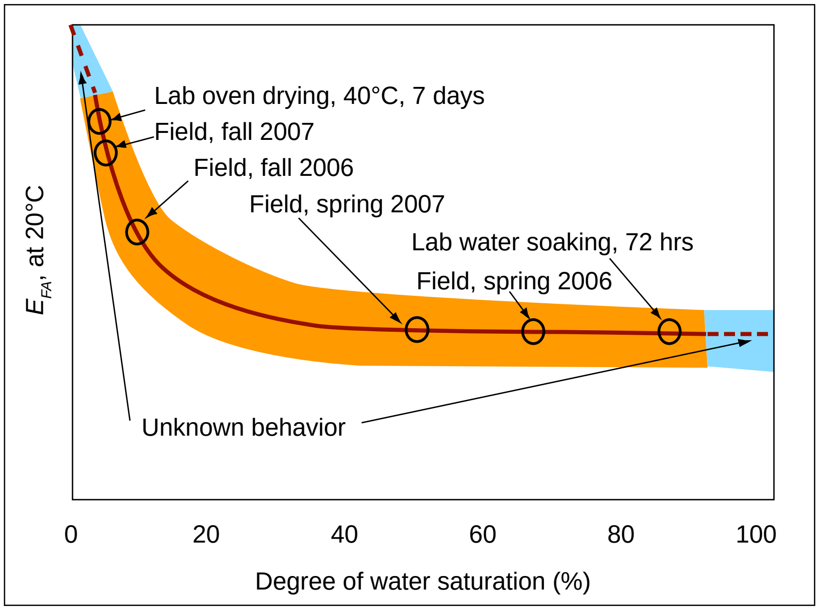

Figure 3.6: Conceptual illustration of moisture sensitivity of FDR-FA stiffness *(31).*

Field data indicate that FDR-FA layer stiffness will increase by 3.75 percent for every temperature drop of 1 degree Celsius (pg. 63, *[31]*) based on backcalculation results. In other words, FDR-FA stiffness will double every time its temperature decreases by 19°C. Note that typical HMA mix stiffness will double for every 8 to 10°C drop in temperature.

A limited series of triaxial tests were performed to compare the permanent deformation resistance of different mixes under different curing and soaking conditions. Most of these specimens had already been subjected to triaxial resilient modulus tests before the permanent deformation tests were carried out. It was assumed that the resilient modulus tests were essentially nondestructive, considering that the stress levels applied in the permanent deformation tests are much higher than those applied during the resilient modulus tests.

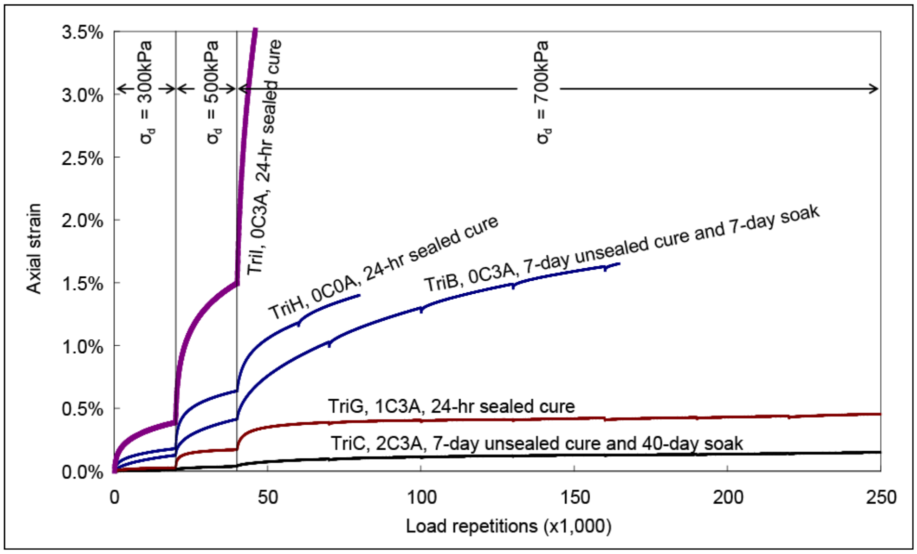

Figure 3.7: Triaxial permanent deformation test results (Section 9.10, *[31]*): labels indicate cement and asphalt binder contents and curing conditions, 0C0A indicates no cement and no foamed asphalt.

The axial strain development of the five specimens is shown in Figure 3.7. The mix design, and the curing and soaking condition for each specimen prior to testing are also shown. It can be seen that FDR-FA material has different resistance to permanent deformation depending on the mix design, and the curing and soaking condition. In particular, compared to the untreated material (TriH), FDR-FA material will have:

- Significantly worse performance when not properly cured (TriI)
- Slightly better performance when no cement is used but properly cured, even when soaked after being properly cured (TriB)
- Significantly better performance when cement is used (TriG and TriC), even when soaked (TriC)

3.7.1.2 FDR-FA Material in *CalME* 2.0

In *CalME* 2.0, FDR-FA material is treated as a lightly asphalt-bound material. Its stiffness depends on temperature and loading frequency, as described by the asphaltic stiffness master curve.

The moisture dependency of FDR-FA layer stiffness cannot be accounted for in *CalME* 2.0 because the feature has been temporarily deactivated. The UCPRC is conducting research (under SPE 3.31, "Improved ME Design") to identify a model for estimating moisture contents in a pavement structure so that in future versions of *CalME* layer stiffness can be related to in situ moisture content. In the meantime, to be conservative, a decision was made to apply the wet season stiffness of the FDR-FA layer to the entire year. Specifically a reference stiffness of 800 MPa for a loading frequency of 10 Hz and temperature of 20°C have been used, corresponding approximately to the average backcalculated wet season stiffness for an FDR-FA layer with 1 to 2 percent cement. As shown in Figure 3.8, a side effect of this decision is that the maximum stiffness of FDR-FA material is much lower than typical HMA materials. An SDF value of 1.40 has been selected for FDR-FA layer stiffness. This is higher than the typical value of 1.20 used for other materials to account for the uncertainties in the percentage of reclaimed asphalt pavement (RAP) and aggregate base.

Although an FDR-FA layer may be subject to fatigue damage under trafficking, long-term deflection testing and backcalculation has shown little damage over periods of up to 15 years after construction. It is expected that an asphalt overlay will be replaced within 15 years. Deflection testing and backcalculation on accelerated pavement testing tracks at the UCPRC has shown that FDR-FA layer stiffness decreases from an initial value of 5,000 MPa to about 1,500 MPa after being subjected to accelerated trafficking *(33).* The use of wet stiffness for the FDR-FA layer has more than accounted for the effect of fatigue damage. It was therefore decided to ignore the fatigue damage in the FDR-FA layer.

As discussed in the section above, when it is properly cured, FDR-FA material has better resistance to permanent deformation than the corresponding untreated material. Nevertheless, a decision was made to assume that an FDR-FA layer would have the same permanent deformation model parameter as an AB-Class 2 material. As part of SPE 4.59, "Developing Guidelines for Project Selection and Mechanistic-Empirical Design of Full-Depth Reclaimed Pavements in California," the UCPRC is conducting research to calibrate a permanent deformation model for the FDR-FA layer.

There is only one FDR-FA material in *CalME* 2.0 and it is named *Full-Depth Reclamation with Foamed Asphalt.* It has the same model parameters as *AB-Class 2, HDM 2012* material except for the stiffness master curve, whose parameters are listed in Table 3.12.

The master curve model parameters for FDR-FA material are listed in Table 3.12. A comparison of the stiffness master curves for FDR-FA in *CalME* 2.0 and a typical HMA mix is shown in Figure 3.8. As the figure shows, the FDR-FA material has slightly higher minimum stiffness at high temperature (283 MPa compared to 200 MPa) than HMA Mix #02 but a significantly lower maximum stiffness at low temperature (1,161 MPa compared to 15,685 MPa) due to the decision to use wet-season stiffness. The average temperature sensitivity in the 15°C to 25°C range for the FDR-FA material is a 4 percent increase in stiffness per every 1°C drop in temperature, which is consistent with the discussion in the previous section.

Table 3.12: Master Curve Model Parameters for FDR-FA Material in *CalME* 2.0

δ	α	β	γ	aT	A	VTS	E_{ref} (MPa)*
2.4523	0.6124	0.9041	1.058	1.30	10.0406	-3.68	800

*: Reference stiffness E_{ref} is calculated using other model parameters for a loading time of 0.015 second

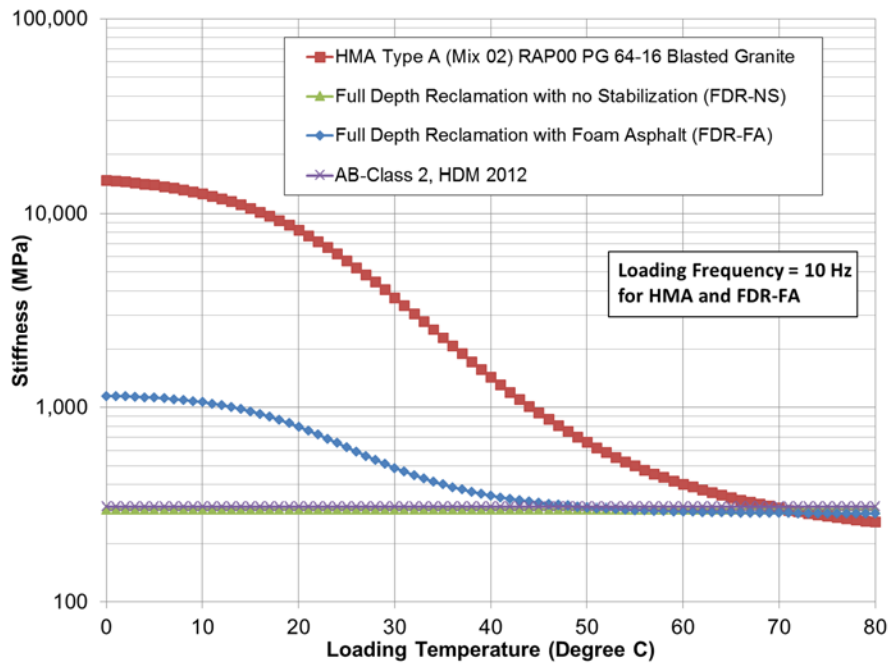

Figure 3.8: Comparison of stiffness master curves for FDR-FA in *CalME* 2.0 and an example HMA mix along with FDR-NS and AB-Class 2.

The FDR-FA material in *CalME* 2.0 has the same resistance to permanent deformation as AB-Class 2 material. This is conservative when FDR-FA layer is properly cured even if no cement is used in the FDR-FA layer. It is not conservative if the FDR-FA layer is not properly cured, i.e., when no cement is used and the compaction (or initial) moisture in the FDR-FA layer is not allowed to evaporate.

In summary, the FDR-FA material in *CalME* 2.0 represents an FDR-FA layer with 1 to 2 percent cement. It is conservative both because wet season stiffness is used throughout the year and because no improved resistance to permanent deformation is included when it is compared to aggregate base. It is expected this material will be updated with less conservative values after incorporating recent findings from the SPE 4.59 project.

3.8 Summary

A complete list of the materials included in the Standard Materials Library (SML) for *CalME* 2.0 is shown in Appendix A. All the coefficients discussed in this chapter have been loaded into the *CalME* database.

Note that new materials may be added to the list by changing the database itself, but not through the user interface in *CalME* 2.0. Also note that the following model parameters can be changed in the program by the user, if desired:

- Reference stiffness
- Parameters related to construction variability:
    - SDF for the reference stiffness
    - SDF for the *A* parameter for asphaltic fatigue resistance
    - SDF for the *A* parameter for asphaltic permanent deformation resistance

4 SUMMARY

This project is part of the continual effort undertaken by the University of California Pavement Research Center (UCPRC) to collect regional materials data for use by Caltrans in Mechanistic-Empirical (ME) flexible pavement designs and rehabilitations. The ultimate goal of the project is to establish a comprehensive Standard Materials Library (SML) that provides typical behaviors of various materials used in California pavements along with the corresponding uncertainties. In essence, the SML is a collection of various material models, the corresponding parameters for the models, and the corresponding uncertainties for selected parameters.

This technical memorandum summarizes the role of the SML in the ME design process, the classifications of different pavement materials, the relevant models applicable to each material group, and the processes for identifying various model parameters. The technical memorandum also describes how construction variabilities are accounted for in the SML. A full list of materials included in the current SML is presented, including the research conducted so far on the material and how its model parameters were determined.

It should be noted that the SML is still under development, and therefore the database will continue to expand as more materials are periodically added. Model parameters will be refined and updated. New models will be added. Existing models may be revised and the corresponding model parameters will be re-determined. New materials will be added to the SML. However, the SML will remain valid even with these continual changes because the laboratory and field testing data used to identify the model parameters will remain unchanged.

Additional work programmed for the UCPRC to expand the SML during the period 2014 to 2017 includes the following:

- Under Strategic Program Element (SPE) 4.36, "Recycling of Rubberized HMA in RAP and FDR Projects," the UCPRC will conduct research on full-depth reclamation/recycling at the Advanced Transportation Infrastructure Research Center (ATIRC) facility at UC Davis. The study will include laboratory testing and accelerated pavement testing (APT). Phase II of the study will be performed under SPE 4.59, "Developing Guidelines for Project Selection and Mechanistic-Empirical Design of Full-Depth Reclaimed Pavements in California." *CalME* model parameters for various full-depth reclamation materials will be developed as part of the study through additional laboratory testing and accelerated pavement testing (APT), as well as Mechanistic-Empirical modeling.
- SPE 3.30, "Standard Materials Library and Guidance," includes work for characterizing new HMA and RHMA-G mixes designed with the Superpave method and adding them to the library through plant sampling and laboratory testing. Additional in situ stiffness data for lean concrete base (LCB), cement-treated base (CTB), cold in-place recycled (CIR) material, and various full-depth reclamation (FDR) materials will be added to the library through falling weight deflectometer testing and backcalculation.

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APPENDIX A: LIST OF ALL MATERIALS

Table A.1: Complete List of Materials Included in *CalME* 2.0 Standard Materials Library

Material No.	Material Name	Classification
1	PCC	Pavement Material
2	Old HMA	Asphaltic Material
3	HMA Type A (Mix 01) RAP00 PG 64-28 Blasted Granite	
4	HMA Type A (Mix 02) RAP00 PG 64-16 Blasted Granite	
5	HMA Type A (Mix 03) RAP00 PG 64-16 Crushed Alluvial	
6	HMA Type A (Mix 04) RAP00 PG 64-10 Blasted Granite	
7	HMA Type A (Mix 05) RAP00 PG 64-10 Crushed Alluvial	
8	HMA Type A (Mix 06) RAP00 PG 70-16 Blasted Granite	
9	HMA Type A (Mix 07) RAP00 PG 64-28PM Blasted Granite	
10	HMA Type A (Mix 08) RAP00 PG 64-28PM Crushed Alluvial	
11	HMA Rich Bottom (Mix 09) RAP00 PG 64-10 Blasted Granite	
12	HMA Rich Bottom (Mix 10) RAP00 PG 64-10 Crushed Alluvial	
13	HMA Type A (Mix 11) RAP00 PG 64-16 Crushed Alluvial Hveem	
14	HMA Type A (Mix 12) RAP00 PG 64-16 Crushed Alluvial Superpave	
15	HMA Type A (Mix 13) RAP00 PG 64-16 Blasted Basalt Hveem	
16	HMA Type A (Mix 14) RAP00 PG 64-16 Blasted Basalt Superpave	
17	RHMA-G (Mix 15) RAP00 PG 64-16 base Blasted Basalt Superpave	
18	RHMA-G (Mix 16) RAP00 PG 64-16 base Blasted Basalt Hveem	
19	RHMA-G (Mix 17) RAP00 PG 64-16 base Blasted Granite Superpave	
20	HMA Type C (Mix 19) RAP00 PG 64-28PM Blasted Granite Superpave	
21	HMA Type A (Mix 21) RAP00 PG 70-10 Blasted Granite Superpave	
22	HMA Type A (Mix 22) RAP25 PG 64-10 Crushed Alluvial with Lime	
23	HMA Type A (Mix 23) RAP15 PG 64-28PM Crushed Alluvial with Lime	
24	RHMA-G (Mix 24) RAP00 PG 64-16 base Crushed Alluvial (SHRP II R21)	
25	HMA Type A (Mix 25) RAP00 PG 64-28PM Crushed Alluvial (SHRP II R21)	
26	HMA Rich Bottom (Mix 26) RAP00 PG 64-10 Crushed Alluvial with Lime	
27	HMA Type A (Mix 27) RAP15 PG 64-28PM Crushed Alluvial with Lime	
28	Cold In-Place Recycled Asphalt	
29	CTB-Class A, HDM 2012	Cement Treated Material
30	CTB-Class B, HDM 2012	
31	CTPB, HDM 2012	
32	Lean Concrete Base	
33	AB-Class 2, HDM 2012	Unbound Material
34	AS-Class 1, HDM 2012	
35	AS-Class 2, HDM 2012	

Material No.	Material Name	Classification
36	AS-Class 3, HDM 2012	
37	ML, HDM 2012	
38	GM, HDM 2012	
39	SC, HDM 2012	
40	GW, HDM 2012	
41	GP, HDM 2012	
42	GC, HDM 2012	
43	SP, HDM 2012	
44	SW, HDM 2012	
45	CH, HDM 2012	
46	MH, HDM 2012	
47	SM, HDM 2012	
48	CL, HDM 2012	
49	Asphalt-Treated Permeable Base	
50	Full-Depth Reclamation with no Stabilization	
51	Recycled Hardened Concrete Aggregate	Lightly Cemented Material
52	LTS, HDM 2012	
53	Full-Depth Reclamation with Foamed Asphalt	Lightly Asphalt-Bound Material

Table A.2: Material Codes Used in Caltrans Empirical Design Method

Material No.	Code	Material No.	Code	Material No.	Code
1	PCC	19	RHMA-G	37	SG
2	HMA	20	HMA	38	SG
3	HMA	21	HMA	39	SG
4	HMA	22	HMA	40	SG
5	HMA	23	HMA	41	SG
6	HMA	24	RHMA-G	42	SG
7	HMA	25	HMA	43	SG
8	HMA	26	HMA	44	SG
9	HMA	27	HMA	45	SG
10	HMA	28	CIR	46	SG
11	HMA	29	CTB-Class A	47	SG
12	HMA	30	CTB-Class B	48	SG
13	HMA	31	CTPB	49	ATPB
14	HMA	32	LCB	50	AB
15	HMA	33	AB	51	AB
16	HMA	34	AS	52	LTS
17	RHMA-G	35	AS	53	AB
18	RHMA-G	36	AS		

Table A.3: Description of All Materials

Material No.	Material Description
1	PCC, UCPRC, HVS Study Goal 6, Modulus = 5,075 ksi
2	Asphalt-bound materials in the existing pavement for rehabilitation projects, old HMA
3	HMA Type A mix with PG 64-28 binder, 3/4" Crushed Granite from Watsonville, no RAP, binder content=5%, air-void content=6%, Hveem mix design
4	HMA Type A mix with PG 64-16 binder, 3/4 Crushed Granite from Watsonville, no RAP, binder content=5%, air-void content=6%, Hveem mix design
5	HMA Type A mix with PG 64-16 binder, 3/4" Crushed River Gravel from Sacramento, no RAP, binder content=5%, air-void content=6%, Hveem mix design
6	HMA Type A mix with PG 64-10 binder, 3/4" Crushed Granite from Watsonville, no RAP, binder content=5%, air-void content=6%, Hveem mix design
7	HMA Type A mix with PG 64-10 binder, 3/4" Crushed River Gravel from Sacramento, no RAP, binder content=5%, air-void content=6%, Hveem mix design
8	HMA Type A mix with PG 70-16 binder, 3/4" Crushed Granite from Watsonville, no RAP, binder content=5%, air-void content=6%, Hveem mix design
9	HMA Type A mix with PG 64-28PM binder, 3/4" Crushed Granite from Watsonville, no RAP, binder content=5%, air-void content=6%, Hveem mix design
10	HMA Type A mix with PG 64-28PM binder, 3/4" Crushed River Gravel from Sacramento, no RAP, binder content=5%, air-void content=6%, Hveem mix design
11	HMA Rich Bottom mix with PG 64-10 binder, 3/4" Crushed Granite from Watsonville, no RAP, binder content=5.5%, air-void content=3%, Hveem mix design
12	HMA Rich Bottom mix with PG 64-10 binder, 3/4" Crushed River Gravel from Sacramento, no RAP, binder content=5.5%, air-void content=3%, Hveem mix design
13	HMA Type A mix with PG 64-16 binder, 3/4" Alluvial from Sacramento, no RAP, binder content=5%, air-void content=6%, Hveem mix design
14	HMA Type A mix with PG 64-16 binder, 3/4" Alluvial from Sacramento, no RAP, binder content=5.5%, air-void content=6%, Superpave mix design
15	HMA Type A mix with PG 64-16 binder, 3/4" Basalt from Vallejo, no RAP, binder content=5.2%, air-void content=6%, Hveem mix design
16	HMA Type A mix with PG 64-16 binder, 3/4" Basalt from Vallejo, no RAP, binder content=6.3%, air-void content=6%, Superpave mix design
17	RHMA-G mix with PG 64-16 base binder, 1/2" Basalt from Vallejo, no RAP, binder content=8.3%, air-void content=6%, Superpave mix design
18	RHMA-G mix with PG 64-16 base binder, 1/2" Basalt from Vallejo, no RAP, binder content=8%, air-void content=6%, Hveem mix design
19	RHMA-G mix with PG 64-16 base binder, 3/4" Crushed Granite from Corona, no RAP, binder content=8.8%, air-void content=6%, Superpave mix design
20	HMA Type C mix with PG 64-28PM binder, 1" Crushed Granite from Corona, no RAP, binder content=6.4%, air-void content=6%, Superpave mix design
21	HMA Type A mix with PG 70-10 binder, 3/4" Crushed Granite from Edmonston, no RAP, binder content=5.9%, air-void content=6%, Superpave mix design
22	HMA Type A mix with PG 64-10 binder and 1.2% lime, 3/4" Crushed River Gravel from Redding, RAP25, binder content=5.38%, air-void content=6%, Long Life mix design
23	HMA Type A mix with PG 64-28PM binder and 1.2% lime, 3/4" Crushed River Gravel from Redding, RAP15, binder content=5.2%, air-void content=6%, Long Life mix design
24	RHMA-G mix with PG 64-16 base binder, 1/2" Crush River Gravel from Crushed Alluvial, no RAP, binder content=7.5%, air-void content=9%, Hveem mix design
25	HMA Type A mix with PG 64-28PM binder, 3/4" Crushed River Gravel from Crushed Alluvial, no RAP, binder content=5%, air-void content=4%, Hveem mix design
26	HMA Rich Bottom mix with PG 64-10 binder and 1.2% lime, must be at least 4" deep, 3/4" Crushed Alluvial from Redding, no RAP, binder content=5.5%, air-void content=3%, Long Life mix design
27	HMA Type A mix with PG 64-28PM binder and 1.2% lime, 3/4" Crushed Alluvial from Montague, RAP15, binder content=5.73%, air-void content=6%, Long Life mix design

Material No.	Material Description
28	Cold In-Place Recycled Asphalt, SR16 Near Colusa, Modulus = 290 ksi
29	CTB-Class A, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 1400ksi
30	CTB-Class B, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 1100 ksi, R-value = 80, Gravel Factor = 1.2
31	Cement Treated Permeable Base, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 1100 ksi
32	Lean Concrete Base, Sonoma County 101 near Santa Rosa, Modulus = 870 ksi
33	AB-Class 2, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 45 ksi, R-value = 78, Gravel Factor = 1.1
34	AS-Class 1, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 35 ksi, R-value = 65, Gravel Factor = 1
35	AS-Class 2, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 30 ksi, R-value = 55, Gravel Factor = 1
36	AS-Class 3, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 25 ksi, R-value = 45, Gravel Factor = 1
37	Inorganic silts, very fine sands, rock four, silty or clayey fine sands, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 11 ksi, R-value = 20, Gravel Factor = 0
38	Silty gravels, gravel-sand-silt mixtures, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 30 ksi, R-value = 54, Gravel Factor = 0
39	Clayey sands, sand-clay mixtures, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 14 ksi, R-value = 23, Gravel Factor = 0
40	Well-graded gravels and gravel-sand mixtures, little or no fines, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 38 ksi, R-value = 68, Gravel Factor = 0
41	Poorly graded gravels and gravel-sand mixtures, little or no fines, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 29 ksi, R-value = 52, Gravel Factor = 0
42	Clayey gravels, gravel-sand-clay mixtures, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 20 ksi, R-value = 34, Gravel Factor = 0
43	Poorly graded sands and gravelly sands, little or no fines, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 17 ksi, R-value = 31, Gravel Factor = 0
44	Well-graded sands and gravelly sands, little or no fines, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 21 ksi, R-value = 37, Gravel Factor = 0
45	Inorganic clays of high plasticity, fat clays, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 4 ksi, R-value = 6, Gravel Factor = 0
46	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 6 ksi, R-value = 9, Gravel Factor = 0
47	Silty sands, sand-silt mixtures, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 21 ksi, R-value = 37, Gravel Factor = 0
48	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 9 ksi, R-value = 16, Gravel Factor = 0
49	Asphalt-Treated Permeable Base, Various Laboratory and Field Data, Assumed Stripped, Modulus = 44 ksi
50	Full-Depth Reclamation with no Stabilization, Various State Routes, Modulus = 44 ksi, R-value = 78, Gravel Factor = 1.1
51	Recycled Hardened Concrete Aggregate, Goal 9 - MB Road, Modulus = 94 ksi, R-value = 78, Gravel Factor = 1.1
52	Lime-Treated Subbase/Subgrade, stiffness and Poisson's ratio based on HDM Version 2012, Modulus = 87 ksi
53	Full-Depth Reclamation with Foamed Asphalt, Various State Routes, Modulus = 116 ksi

APPENDIX B: MODEL PARAMETERS FOR ASPHALTIC MATERIALS

Table B.1: Stiffness Master Curve Model Parameters for All Asphaltic Materials and Lightly Asphalt-Bound Materials

Material Number*	δ	β	γ	aT	A	VTS
2	2.30103	-0.56735	0.61764	1.100549	9.630746	-3.5047
3	2.30103	0.612633	0.949345	1.089008	9.630746	-3.5047
4	2.30103	-0.31122	0.799129	1.218652	9.630746	-3.5047
5	2.30103	-0.0733	0.579741	1.246853	9.630746	-3.5047
6	2.30103	-0.31485	0.702574	1.10128	9.630746	-3.5047
7	2.30103	-0.13586	0.809248	1.365802	9.630746	-3.5047
8	2.30103	-0.13953	0.48993	1.400474	9.630746	-3.5047
9	2.30103	0.578436	0.921341	1.192067	9.630746	-3.5047
10	2.30103	0.90358	0.998651	1.050757	9.630746	-3.5047
11	2.30103	-0.21959	0.763492	1.198944	9.630746	-3.5047
12	2.30103	-0.19288	0.692977	1.266528	9.630746	-3.5047
13	2.30103	-0.16266	0.813244	1.239024	9.630746	-3.5047
14	2.30103	0.081589	0.775585	1.208629	9.630746	-3.5047
15	2.30103	0.111756	0.798999	1.299547	9.630746	-3.5047
16	2.30103	0.221841	0.829792	1.197749	9.630746	-3.5047
17	2.30103	0.156404	0.757764	1.189432	9.630746	-3.5047
18	2.30103	0.127077	0.863051	1.100173	9.630746	-3.5047
19	2.30103	0.286394	0.798686	1.201783	9.630746	-3.5047
20	2.30103	1.368361	0.662101	1.134975	9.630746	-3.5047
21	2.30103	-0.66656	0.676331	1.278408	9.630746	-3.5047
22	2.30103	0	0.627688	1.211117	9.630746	-3.5047
23	2.30103	0.293632	0.720078	1.105295	9.630746	-3.5047
24	2.30103	0.647748	1.011773	1.031791	9.630746	-3.5047
25	2.30103	1.430486	1.098128	1.025107	9.630746	-3.5047
26	2.30103	0.374174	0.830243	0.937828	9.630746	-3.5047
27	2.30103	0.386426	0.788663	1.107079	9.630746	-3.5047
28	2.30103	0.542631	0.292639	1.73183	9.630746	-3.5047
53	2.4523	0.90411	1.058	1.3	10.0406	-3.68

*: Refer to Table A.1 for the material corresponding to a given material number.

Table B.2: Fatigue Damage Model Parameters for All Asphaltic Materials

Material Number*	A	SDF for A	α_f	β	γ
2	12.33823	4.8	1.891	-4.16181	-2.0809
3	423.5491	1.15	0.403	-5.84163	-2.92082
4	676.9679	1.15	1.369	-6.3575	-3.17875
5	170.1362	1.15	0.784	-7.67492	-3.83746
6	689.2024	1.15	0.76	-6.36741	-3.18371
7	15.66926	1.15	1.003	-4.31425	-2.15713
8	533485.1	1.15	0.668	-11.4372	-5.71862
9	2006.004	1.15	0.361	-6.05154	-3.02577
10	4887.232	1.15	0.234	-7.68991	-3.84496
11	100.2491	1.15	0.884	-4.69363	-2.34682
12	59.46049	1.15	0.914	-4.17104	-2.08552
13	29.11709	1.15	1.031	-5.99175	-2.99587
14	651.2938	1.15	0.746	-7.74111	-3.87056
15	13.90033	1.15	0.976	-5.02097	-2.51049
16	30.81378	1.15	0.785	-5.36726	-2.68363
17	1302.871	1.15	0.397	-4.72962	-2.36481
18	1511.816	1.15	0.378	-4.65549	-2.32774
19	3652.606	1.15	0.289	-6.26942	-3.13471
20	314374.3	1.15	0.164	-4.67655	-2.33828
21	109.6611	1.15	1.11	-6.38398	-3.19199
22	160.0059	1.15	0.712	-6.02665	-3.01332
23	1.02E+08	1.15	0.143	-9.32615	-4.66307
24	17.28333	1.15	0.747	-5.49391	-2.74695
25	2519.093	1.15	0.218	-5.9927	-2.99635
26	2491.481	1.15	0.477	-6.49081	-3.24541
27	3.47E+08	1.15	0.136	-8.09194	-4.04597
28	30.34296	1.15	0.363	-8.84917	-4.42459

*: Refer to Table A.1 for the material corresponding to a given material number.

Table B.3: Permanent Deformation Model Parameters for All Asphaltic Materials

Material Number*	A	SDF for A	α	γ	K
2	1.648721	2.1	2.996327	2.776249	2
3	6.264486	1.2	2.252411	3.013509	2
4	3.272621	1.2	2.600627	1.893956	2
5	0.864726	1.2	4.026561	2.012488	2
6	3.830625	1.2	2.613355	2.545258	2
7	2.233708	1.2	3.223059	1.806295	2
8	1.006314	1.2	3.999895	1.998261	2
9	6.264486	1.2	2.252411	3.013509	2
10	3.448216	1.2	2.549526	3.107731	2
11	0.962998	1.2	3.929296	1.654542	2
12	1.257548	1.2	4.058931	1.971074	2
13	2.236832	1.2	3.400171	2.988937	2
14	1.956544	1.2	3.598819	2.245117	2
15	1.0	1.2	4.270282	2.278821	2
16	3.440548	1.2	3.14551	2.667491	2
17	1.001001	1.2	4.313473	2.047842	0.5
18	1.0	1.2	3.864841	2.162672	0.5
19	1.675726	1.2	3.995917	2.676816	0.5
20	1.208925	1.2	3.447602	2.004351	2
21	1.441329	1.2	4.48827	2.972329	2
22	1.0	1.2	3.578284	2.456087	2
23	2.045513	1.2	2.599597	2.372825	2
24	1.426294	1.2	3.946311	1.717846	0.5
25	6.870353	1.2	2.64575	4.233602	2
26[X]	1.0	1.2	0	0	2
27	2.06789	1.2	3.108326	2.865329	2
28	1.006523	1.2	5.985675	3.971506	2

*: Refer to Table A.1 for the material corresponding to a given material number.
[X]: This is a Rich Bottom HMA mix assumed to have no permanent deformation, must be placed at least 100 mm deep