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Evaluation and modeling of repeated load test data of asphalt concrete for mechanistic-empirical flexible pavement design

Stephen Price
Rowan University

EVALUATION AND MODELING OF REPEATED LOAD TEST DATA OF ASPHALT CONCRETE FOR MECHANISTIC-EMPIRICAL FLEXIBLE PAVEMENT DESIGN

Prepared by:

Stephen Price

Approved by:

Yusuf A. Mehta
Assistant Professor, Rowan University
Thesis Advisor

Douglas Cleary
Associate Professor, Rowan University
Thesis Committee Member

William Riddell
Assistant Professor, Rowan University
Thesis Committee Member

Ralph Dusseau
Civil & Environmental Engineering Chair, Rowan University
Thesis Committee Member

Leslie Myers, Ph.D. P.E.
Federal Highway Administration
Office of Pavement Technology
Thesis Committee Member

EVALUATION AND MODELING OF REPEATED LOAD TEST DATA OF ASPHALT CONCRETE FOR MECHANISTIC-EMPIRICAL FLEXIBLE PAVEMENT DESIGN

by
Stephen Price

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ABSTRACT

Stephen Price
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2006
Advisor: Yusuf A. Mehta, Ph.D., P.E.
Master of Science in Civil Engineering

The Mechanistic-Empirical Pavement Design Guide (MEPDG) uses a power model that incorporates Dynamic Modulus (DM) as a factor in predicting rutting performance of asphalt concrete. The rutting model is empirical and the rutting profiles obtained from the model are largely based on the adjustment of calibration constants not the DM. Various DM values would be capable of producing similar rutting profiles based on the current model. This study evaluated using accumulated strain data obtained from the Repeated Load Test (RLT) as an alternative to using DM. In addition, a model to represent the accumulated strain data obtained from the RLT was developed. The accuracy and behavior of the current model and the model developed in this study in representing a mix from Louisiana were compared. Alternatives for incorporating the RLT data into the MEPDG are also presented.

In addition, a conceptual framework for materials characterization of routinely used HMA mixtures was developed. The conceptual framework will aid state highway agencies in developing a mixture catalog that will help streamline the mix design process, it will also provide guidance in obtaining flexible pavement design inputs for the new MEPDG.

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NOTATION

ABBREVIATIONS

AASHTO ....American Association of State Highway and Transportation Officials

ARA ....Applied Research Associates

ASTM ....American Society for Testing and Materials

DM ....Dynamic Modulus

DOT ....Department of Transportation

ESAL.....................................Estimated Single Axle Load

FHWA...................................Federal Highway Administration

FN ....Flow Number

HMA ....Hot-mix Asphalt

HVS.......................................Heavy Vehicle Simulator

IDT ....Indirect Tensile Test

JMF ....Job Mix Formula

LTPP ....Long-Term Pavement Performance Program

LVDT....................................Linear Variable Differential Transformers

MEPDG.................................Mechanistic-Empirical Pavement Design Guide

NCHRP ....National Cooperative Highway Research Program

RFP ....Request for Proposal

RLT ....Repeated Load Permanent Deformation Test

SHA.......................................State Highway Administration

SPS ....Specific Pavement Studies

VTRC....................................Virginia Transportation Research Center

WIM......................................Weigh in motion

SYMBOLS

D_1....................................slope of linear portion.

D_2....................................y intercept of the exponential.

$|E^*|$....................................dynamic complex modulus, kPa.

$|G^*|$....................................shear complex modulus, kPa.

$|J^*|$....................................dynamic complex creep compliance, kPa^{-1}.

J''....................................loss compliance, kPa^{-1}.

J''_{mix}....................................loss compliance of the specific mix, kPa^{-1}.

N....................................number of cycles.

N....................................number of cycles in primary region

T....................................temperature (°F)

b....................................y intercept of linear portion.

k_1....................................J'' calibration exponent.

k_{r1}, k_{r2}, k_{r3}..............................global calibration factors, current model.

m_1....................................b, mix calibration factor, Alternative 2.

m_2....................................D_1, mix calibration factor, Alternative 2.

m_3....................................D_2, mix calibration factor, Alternative 2.

m_4....................................λ, mix calibration factor, Alternative 2.

m_{r1}, m_{r2}, m_{r3}..............................mix calibration factors, Alternative 1.

r_{r1}, r_{r2}, r_{r3}..............................regional calibration factors, Alternative 1.

r_1, r_2, r_3, r_4 .... regional calibration factors, Alternative 2.

x, y, z .... denotes direction of vector in Cartesian coordinates.

$\beta_{r1}, \beta_{r2}, \beta_{r3}$ .... local calibration factors, current model.

φ .... phase angle, radians.

ε_0 .... maximum amplitude of strain

ε_p .... accumulated permanent strain.

ε_{ps} .... accumulated permanent strain (linear function).

$\varepsilon_{rz} = (\sigma_z - \nu\sigma_x - \nu\sigma_y)/|E^*|$ .. resilient strain, (NCHRP, 2004b)

λ .... rate at which permanent strain accumulates in primary region.

ν .... Poisson's ratio.

σ .... stress

σ_0 .... maximum amplitude of stress

ω .... angular frequency.

CHAPTER 1

Introduction

1.1. Problem Statement

The current Mechanistic-Empirical Pavement Design Guide (MEPDG) uses empirical models that incorporate Dynamic Modulus (DM) to predict rutting performance of asphalt concrete. The Dynamic Modulus is obtained by conducting test procedure ASTM D3497-79, or it is predicted via the Witczak or Hirsch Models using mix aggregate and binder data. The test is conducted within strict strain limits, which keep the specimen within the linear viscoelastic region. However, rutting occurs in the nonlinear viscoelastic region and hot-mix asphalt (HMA) mixtures with similar DM values can have significantly different rutting performance (Von Quintus, 2005).

The repeated load permanent deformation test (RLT) is also under consideration by the National Cooperative Highway Research Program (NCHRP) for predicting the rutting potential of HMA mixtures. The RLT consists of loading an HMA specimen with an 87 psi uniaxial load for 0.1 seconds, followed by a 0.9 second recovery period. The accumulated strain curve output by repeat load testing consists of a primary, secondary and tertiary region. The RLT is conducted until the specimen displays an increasing rate of strain accumulation, which indicates it has reached the tertiary region (which is in the nonlinear viscoelastic region) or 10,000 cycles (Witczak et al., 2002, Myers et al. 2005)). The starting point of the tertiary region is generally defined as the Flow Number (FN).

Flow Number is defined as the number of load repetitions at which constant volume shear deformation begins (Abbas, 2004). Initially FN was being considered as an alternative to DM as a potential predictor of rutting. However, some of the more stable mixes, i.e. polymer modified mixes, do not reach tertiary flow within 10,000 cycles and therefore have no FN (Myers et al., 2005). The FN would then have to be estimated, leading to inconsistent rut depth predictions. Focus has since been on the accumulated strain profiles obtained from the RLT, which have shown good agreement with field measured rut depth profiles (Myers et al., 2005).

Dynamic Modulus is currently used to predict rutting potential in the MEPDG via ε_r, which is discussed later. However, the RLT is being considered as an alternative because it takes into account the nonlinear viscoelastic region. Therefore, it may have the potential to more accurately predict rutting. This study will develop a model to represent the accumulated strain profiles obtained from the RLT. The model will capture the cyclic load behavior of the material, and therefore could be utilized to calculate permanent strain at any number of cycles prior to tertiary flow. Once the model is developed to obtain plastic strain at a given number of cycles, it could be empirically translated to determine rut depth at a given number of Estimated Single Axle Loads (ESALs). The feasibility of incorporating the model into the MEPDG will then be evaluated.

In addition, a conceptual framework for materials characterization of routinely used HMA mixtures will be developed. Many state agencies are in the process of switching to the MEPDG from the AASHTO design guide and are still investigating the different levels of required input for pavement design. The conceptual framework will aid state highway agencies in developing a mixture catalog that will help streamline the mix design process, it will also provide guidance in obtaining flexible pavement design inputs for the new MEPDG.

1.2. Hypothesis

The Hypotheses of this study are:

1. A model can be developed to accurately represent the profile of accumulated strain data obtained from the Repeated Load Test.
2. Repeated Load Test data can be incorporated into the MEPDG.
3. A Conceptual Framework for cataloging routinely used mixes can be developed for implementation in the MEPDG.

1.3. Objective

The objectives of the study are:

1. Develop a model to represent accumulated strain profiles obtained from RLT.
2. Evaluate the feasibility of incorporating the RLT into the MEPDG using the current rutting model or the new proposed model.
3. Provide a conceptual framework for development of a mix catalog to be used with Mechanistic-Empirical Flexible Pavement Design.

1.4. Research Approach

The research approach to achieve the objectives is:

1. Identify a candidate form for the model.

2. Determine the relationships between model parameters and material characterization data.
3. Develop a physical interpretation of the model, based on mechanical behavior of asphalt concrete.
4. Evaluate alternatives for incorporating the model into the MEPDG software.
5. Evaluate/Summarize current state agency efforts at material characterization for the MEPDG.
6. Develop a conceptual framework for materials characterization.
7. Present results, conclusion, and recommendations.

1.5. Scope of Study

This study will use accumulated strain data provide by the Federal Highway Administration (FHWA). The data was taken from test sections in 3 states using the FHWA mobile lab. The data will be used to develop a model that accurately represents the RLT strain profile and mechanical behavior of HMA.

This study will also utilize information from conversations with state's Department of Transportation to develop a conceptual framework.

1.6. Expected Significance

Having a model that accurately represents the accumulated strain profile of the RLT will allow for the parameters of the model to be determined for routinely used mixes. It will also allow for information from RLT to be utilized to accurately represent field rutting after calibration. Using the model in conjunction with the conceptual framework will aid in categorizing mixes and their performance in the field.

1.7. Summary

This chapter described the motivation for this study, defined objectives of the study, described the approach taken, and explained the anticipated significance. The following chapters contain detailed information on the data used and the development of the proposed model.

CHAPTER 2

Literature Review

2.1. Introduction

This chapter discusses viscoelastic behavior relevant to this study and the tests conducted in determining DM values and accumulated strain profiles. The MEPDG, binder specifications and existing models are also discussed.

2.1.1. Viscoelasticity

The binder used to make HMA is a viscoelastic material. Therefore, HMA mixtures will retain some of the viscoelastic properties of the binder. When a material is viscoelastic, it will exhibit both viscous and elastic responses. Meaning it will not behave as an elastic material, recovering all strain, nor will it behave like a Newtonian fluid, recovering no strain; rather it will recover a portion of the strain. The amount of strain accumulated or recovered during loading followed by a rest period is time dependent, as shown in Figure 2.1.

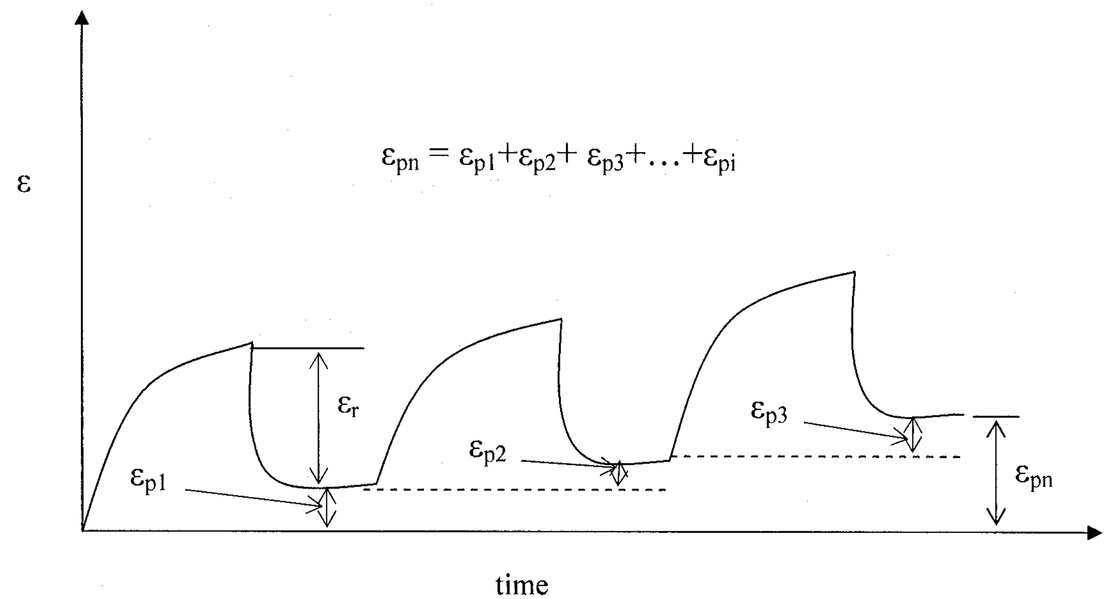

Figure 2.1: Strain accumulation and recovery versus time (3 cycles are shown).

The strain that is not recovered once the load is released is the permanent strain resulting from the load cycle. The permanent strain is a result of the dissipation of energy by the HMA specimen during the load cycle. The dissipated energy shown in Figure 2.2 consists of the energy dissipated resulting in permanent deformation and internal heat generation (Courtney, 1990). The sum of the permanent strain per cycle generates the accumulated strain profile of the RLT.

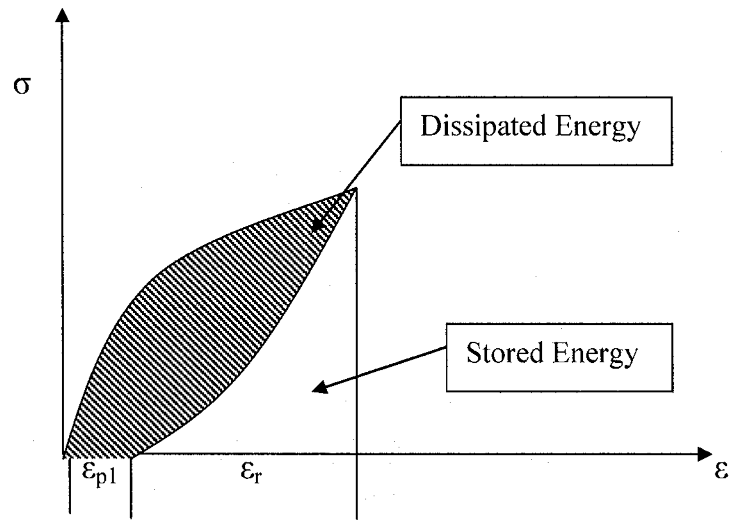

Figure 2.2: Hysteresis loop of periodic loading.

2.1.2. Dynamic Modulus, |E*|

To determine the DM of an asphalt mixture, cylindrical specimens (4 in diameter by 6 in height) are tested in accordance with test procedure ASTM D3497-79 (ASTM, 2003). The test involves the use of a servo-hydraulic testing machine, an environmental chamber, and linear variable differential transformers (LVDT) as a means of measuring axial deformation (Pellinen, 2001). During the test the specimens are subjected to a haversine loading, which is adjusted to keep the axial strains between 50 and 150 microstrain (Pellinen, 2001). If the specimen experiences accumulated strain greater than 1000 microstrain then the loading is reduced by half and the specimen is replaced with a new specimen (Pellinen, 2001). The DM is calculated using the absolute value of the maximum stress (peak-to-peak) divided by the maximum recoverable axial strain (peak-to-peak) that occurs (Dongré et al., 2005) (Figure 2.3). The recommended test series for a DM test involves 25 combinations of temperature and frequency (Pellinen, 2001). The data obtained is used to develop master curves, which are used in pavement response and performance analysis (Pellinen, 2001).

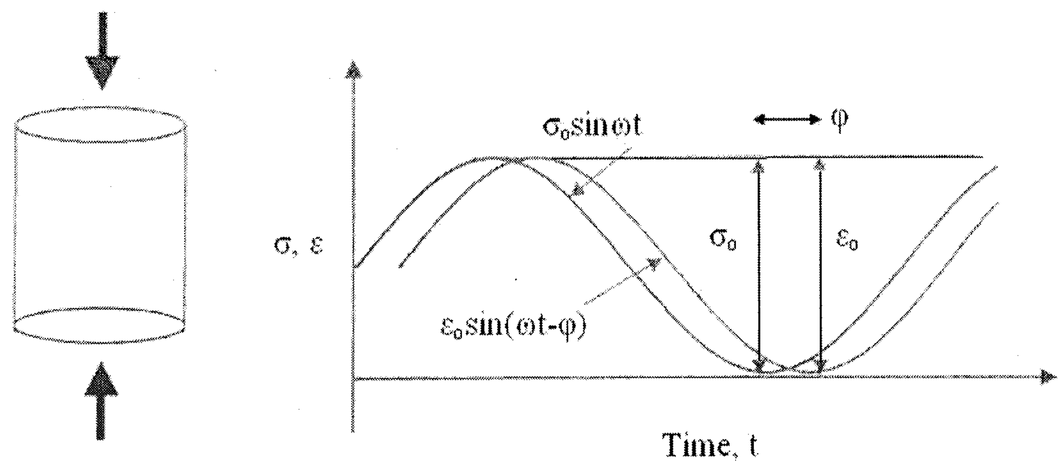

Figure 2.3: Loading pattern of Dynamic Modulus test (Myers et al., 2005).

As shown in Figure 2.3, there is a lag between the applied stress and the strain response of the specimen. This lag is represented by the phase angle, φ, and is a measurement of the specimens elasticity within the linear viscoelastic region. A completely elastic material would have a phase angle of 0 degrees, where as a completely viscous material would have a phase angle of 90 degrees.

2.1.3. Repeated Load Test

The repeated load test is conducted using similar testing equipment used during DM testing, mentioned in Section (2.1.2). During the RLT, the HMA specimen is subjected to a load cycle in which a 0.1 second 87 psi uniaxial load is applied followed by a 0.9 second rest period. For the RLT the amount of deformation experienced by the specimen is not controlled, as it is in the DM test. The cyclic loading is continued for

10,000 cycles or until the specimen undergoes shear deformation. The shear deformation is not measured, but rather implied mechanistically because there is a change in dimensions and not volume under the uniaxial load at the Flow Number. The result of the RLT is usually presented as accumulated permanent deformation versus the number of cycles, shown in Figure 2.4.

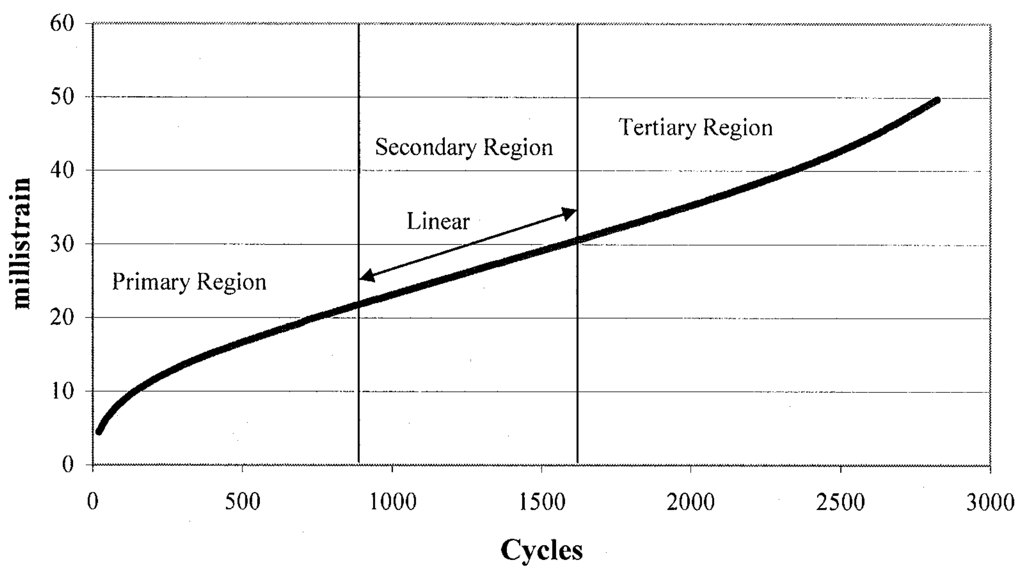

Figure 2.4: Typical accumulated strain profile.

The curve can be divided up into three regions: primary, secondary, and tertiary. The starting point of the tertiary region is generally defined as the Flow Number (FN). Flow Number is defined as the number of load repetitions at which shear deformation, under constant volume, begins (Abbas, 2004). Initially FN was being considered as an alternative to DM as a potential predictor of rutting. However, some of the more stable mixes, i.e. polymer modified mixes, do not reach tertiary flow within 10,000 cycles and therefore have no FN (Myers et al., 2005). The FN would then have to be estimated, leading to inconsistent rut depth predictions. Focus has since been on the accumulated strain profiles obtained from the RLT, which have shown good agreement with field measured rut depth profiles (Myers et al., 2005). Therefore, having the ability to model the RLT strain profile will allow for calibration of the model to field rutting performance.

2.2. Design Guide

In February 2004, a recommended mechanistic-empirical pavement design guide (Bonaquist et al. 2003, NCHRP, 2004a) was delivered to NCHRP under Project 1-37A. A research version of the MEPDG and software (NCHRP, 2005) was made available in June of 2004. The MEPDG combines advanced analytical modeling and empirical modeling to correlate field performance of in service pavements to analyze and design new, reconstructed, and rehabilitated asphalt and concrete pavements. The pavement responses to the combined effects of load and climate are computed using finite element and elastic layer computer models (NCHRP, 2004a). An incremental damage approach is used to calculate the accumulated damage in the pavement over its design life (NCHRP, 2004a).

The Design Guide has three levels of input, ranging from very specific to general input data. The level of input would depend on the needs of the design project and the resources available. The three levels of input are described below.

Level 1 input provides the highest level of accuracy of inputs. Level 1 would typically be used for obtaining inputs for designing pavements in heavy traffic areas or where premature failure has increased safety or economic consequences. Level 1 material input requires laboratory or field-testing, e.g. dynamic modulus testing and site-specific traffic data. Obtaining Level 1 input requires more resources and effort than levels 2 and 3.

Level 2 inputs provide an intermediate level of design input. Level 2 input could be used when resources for Level 1 input are not available. Level 2 inputs would typically be selected by the user, possibly from an agency database, derived from a limited testing program, or estimated through correlations, e.g. dynamic modulus estimated using the Witczak or Hirsch model.

Level 3 inputs provide the lowest level of accuracy. This level may be used for designing low volume roadways where premature failure has fewer consequences. Inputs would typically be default values or averages for the region, e.g. average HMA dynamic complex modulus values for a mixture class.

2.3. Binder Specification

The binder used in HMA is graded based on Superpave specifications given for several tests. The tests, (Rolling Thin-Film Oven, Pressure Aging Vessel, Direct Tension, etc.), are performed on the binder at various specified temperatures to determine the Performance Grade (PG) (Mamlouk et al., 1999). The grade of the binder is represented with PG followed by two numbers, which represent its design temperatures. For example, binder PG64-22 would be sufficient to use for a design high pavement temperature of 64 °C and a design low pavement temperature of -22°C. The design high pavement temperature is determined based on the average seven-day maximum pavement temperature in the field, which is calculated 20 mm below the pavement surface

(Mamlouk et al., 1999). The design low pavement temperature is determined based on the lowest pavement temperature in the field, which is calculated at the pavement surface (Mamlouk et al., 1999).

2.4. Existing Models

The RLT has been used since the mid to late 1970's to quantify the permanent deformation behavior of HMA. Since the employment of the RLT many models have been developed to represent the accumulated permanent strain curve. The most well known models are generally one of 3 forms shown below (Zhou, et al., 2004).

Semi-log model: $$\varepsilon_p = a_1 + b_1 \log(N) \tag{1}$$

where:

ε_p = permanent strain.

a_1 = positive regression constant.

b_1 = positive regression constant.

N = number of cycles.

Power model: $$\varepsilon_p = aN^b \tag{2}$$

where:

ε_p = permanent strain.

a = positive regression constant.

b = positive regression constant.

N = number of cycles.

Tseng and Lytton's model: $\varepsilon_p = \varepsilon_0 e^{-(\rho/N)^\beta}$ (3)

$$\varepsilon_{pn} = \varepsilon_0 \beta \rho^\beta \frac{\sqrt[N^\beta]{1/e^{\rho^\beta}}}{N^{(\beta+1)}} \tag{4}$$

where:

ε_p = permanent strain.

ε_{pn} = permanent strain from single cycle.

β = positive regression constant.

ρ = positive regression constant.

N = number of cycles.

By looking at the parameters of the models and the effect of an increasing number of cycles on ε_p one can see that the amount of permanent strain accumulated per cycle will decrease as *N* increases. Therefore, the models will not be able to effectively model the secondary region because the secondary region is signified by a constant permanent strain accumulation for each cycle (Zhou et al., 2004).

2.5. Summary

This chapter discussed two simple performance tests, DM and RLT, as well as the MEPDG, binder specifications and existing models. The output data from these two tests and the potential of that data to accurately represent rutting in the MEPDG is investigated throughout the following chapters.

CHAPTER 3

Data

3.1. Introduction

This chapter briefly discusses the FHWA Mobile Asphalt Testing Laboratory (MATL) as well as a description of the data and mixtures used in determining the parameters of the model that will be discusses in chapter 4.

3.2. Mobile Asphalt Testing Laboratory

The MATL contains advanced performance testing equipment as well as conventional testing equipment for asphalt mixtures. The MATL is used to collect data in the field for the purposes of validating equipment, evaluation of asphalt mixtures, validation of performance-related construction specifications, and providing input data for the MEPDG to evaluate distress models (FHWA, 2005). The MATL is also used to aid state agencies with the implementation of new pavement technologies and specification standards (FHWA, 2005).

3.3. Data

The data for this study was collected by the MATL and consists of 16 mixes from 3 states: North Carolina, Arizona, and Louisiana. The 16 mixes were felt sufficient to use for analysis because they represent a broad range of gradation and binder grade as, shown in Table 3.1. The data consisted of DM and φ values at various temperatures and frequencies, as well as accumulated strain profiles. The accumulated strain profiles used throughout the analysis were selected because they were available. The values of DM and φ used throughout the analysis were obtained at test temperatures that were the same as that of the respective RLT. The values of DM and φ at a frequency of 10 Hz were used throughout the analysis. This was done because conducting the DM test at 10 Hz results in 1 cycle being equal to 0.1 seconds, which is the duration of loading per cycle during the RLT (Section 2.1.3). The mixes and respective DM and φ values at 10 Hz are shown in Table 3.2.

Table 3.1: Gradation and Binder grade.

	State					
	North Carolina	Arizona	Louisiana			
[a]Mix ID	All	All	LA-1	LA-2	LA-3	LA-4
Binder Grade	PG70-22	PG76-16	PG64-22		PG76-22	
Gradation	Fine	Coarse	Coarse		Fine	
Nominal Max Aggregate Size (mm)	9.5	19.0	25.0		19.0	

[a]A more detailed description of the mixes is presented in Table 3.2.

Table 3.2: DM and φ values at 10 Hz for the 16 mixes

Mix ID	Mix Description	DM at 10 Hz (MPa)	φ at 10 Hz (rad)	Average FN
North Carolina-Test Temperature 45°C				
NC-1	Mix Design 4.5	1435	0.65	1121
NC-2	Mix Design 5.0	1196	0.65	801
NC-3	Mix Design 5.5	884.3	0.70	411
NC-4	Production 1	1157	0.65	1036
NC-5	Production 4	1017	0.65	326
NC-6	Production 14	1024	0.66	541
Arizona-Test Temperature 45°C				
AZ-1	Production 1	1424	0.56	1934
AZ-2	Production 2	1424	0.55	3411
AZ-3	Production 3	1575	0.55	2058
AZ-4	Production 4	1746	0.52	7361
AZ-5	Production 5	1740	0.54	4780
AZ-6	Production 6	1766	0.53	6866
Louisiana-Test Temperature 54°C				
LA-1	Mix Design 3.3	674	0.65	141
LA-2	Base Mix 4	470	0.68	141
LA-3	Binder Mix	872	0.61	1051
LA-4	Wearing Course	648	0.65	758

The test temperatures for the RLT tests are representative of the geographical location for which the mix was intended. The DM test is generally conducted at a range of temperatures. However, only the DM results from the test temperature equal to that of the RLT test were used. The temperature was not considered during the analysis because the temperature at which the DM and RLT tests were conducted was the same for the respective mixes.

3.4. Summary

This chapter presented information on the MATL and the data used throughout the analysis of the model. The next chapter discusses the proposed model and its parameters.

CHAPTER 4

Proposed Model

4.1. Introduction

This chapter will discuss the form of the proposed model and the role of each parameter in relation to an accumulated strain profile.

4.2. Model

The accumulated strain profile obtained from the RLT consists of a primary, secondary, and, in most cases, a tertiary region. The goal of this model is to represent the primary and secondary regions. The primary region is nonlinear whereas the secondary is linear, as shown in Figure 2.2. Therefore the function used to model accumulated strain profile must reduce to a linear function as the number of cycles increases. To model this behavior, a simple linear function was combined with a decaying exponential to form Equation 5.

$$\varepsilon_p = b + D_1 N - D_2 e^{-\lambda N} \tag{5}$$

where:

b	= y intercept of linear portion.
D_1	= slope of linear portion.
N	= number of cycles.
D_2	= y intercept of the exponential.

λ = dictates rate at which permanent strain accumulates in primary region.

ε_p = accumulated strain.

The slope, D_1, represents the rate at which permanent strain accumulates per cycle in the secondary region (shown in Figure 4.1).

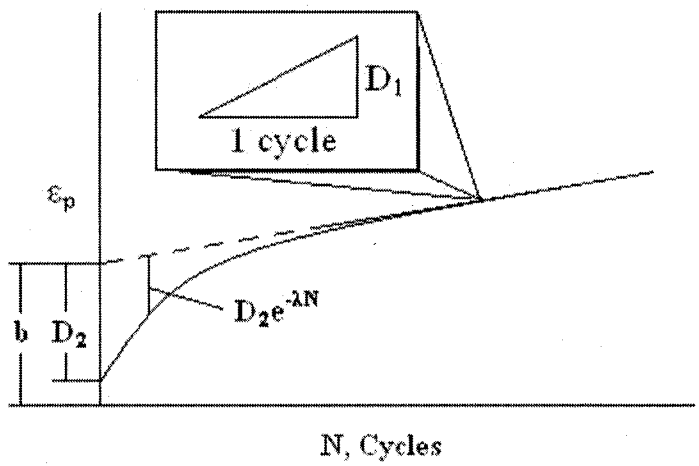

Figure 4.1: Components of the slope, D_1.

When a specimen is loaded it will display some amount of deformation. Some of this deformation will be recovered (resilient strain) and some will not (permanent strain). As discussed earlier the permanent strain is a result of dissipated energy, Figure 2.2. Therefore, the slope, D_1, can be thought to be proportional to the amount of energy being dissipated per cycle by the specimen.

In linear viscoelastic theory the loss compliance, J'', is proportional to energy dissipated by the specimen and the storage compliance, J', is proportional to energy stored. The loss compliance is a component of the complex creep compliance, |J*|, and is dependent on the phase angle, φ. The complex creep compliance is derived from the constitutive equation shown as Equation 6 (Ferry, 1980).

$$\varepsilon(t) = \int_{-\infty}^{t} J(t - t')\dot{\sigma}(t')dt' \tag{6}$$

To describe a periodic loading using the constitutive equation, (t-t') is substituted with the variable s and $\dot{\sigma}$ is substituted with $\omega\sigma_0 \cos(\omega t)$, shown as Equation 7 (Ferry, 1980).

$$\varepsilon(t) = \int_{0}^{\infty} J(s)\omega\sigma_0 \cos[\omega(t - s)]ds \tag{7}$$

where:

ε(t)	= strain as a function of time.
$\dot{\sigma}$	$= \dfrac{d\sigma}{dt}$
σ	= stress
t	= time.
t'	= initial time.

The loss compliance and complex creep compliance, shown in Equation 8 and 9 (Findley et al., 1989), respectively, are derived from Equation 7. The relationship of the loss compliance to the complex creep compliance is illustrated graphically in Figure 4.2.

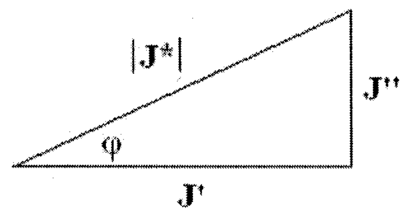

Figure 4.2: Creep compliance and its components.

$$J'' = |J^*| \sin \phi \quad (8)$$

$$|J^*| = \frac{1}{|E^*|} \quad (9)$$

where:

J'' = loss compliance, ksi^{-1}.

φ = phase angle, radians.

$|J^*|$ = dynamic complex creep compliance, ksi^{-1}.

$|E^*|$ = dynamic complex modulus, ksi.

From the form of Equation 8 it can be seen that a mix with a high dynamic modulus and small phase angle, i.e. a stable mix, would have low loss compliance. If the mix were stable, the slope of the linear portion, D_l, of the accumulated strain profile should be low as well. This, as well as both, D_l and J'', being representative of lost energy, is why J'' was input as a variable for slope. Because J'' is applicable only to the linear viscoelastic region an exponent (k_l) was introduced to capture the difference between the slope of the RLT, which is in the nonlinear viscoelastic region, and that predicted by J''. Incorporation of loss compliance and k_l into Equation 5 produces

Equation 10 and Equation 11. The subscript mix is introduced to emphasize that the relationship of the parameters in Equations 10 and 11 are unique to a mixture and the values need to be determined for each mixture.

$$(D_1)_{mix} = (J''_{mix})^{k_{1mix}} \tag{10}$$

Therefore,

$$\varepsilon_p = b + (J''_{mix})^{k_{1mix}} N - D_2 e^{-\lambda N} \tag{11}$$

The model used in the nonlinear regression analysis was of the form shown in Equation 11. However, it is acceptable to use the form of Equation 5 and determine D_1 directly from regression during implementation of the model. It should be noted that the model being developed is a mechanistic-empirical model and therefore the units from either side of the equations may not equal each other.

4.3. Summary

In this chapter the form of the proposed model used throughout the analysis, to be discussed in chapter 5, was presented. In addition, the role of each parameter within the function was discussed.

CHAPTER 5

Analysis

5.1 Introduction

This chapter discusses the process of separating the accumulated strain data into the primary and secondary regions and using nonlinear regression to determine the parameters of the new proposed model. Physical justification of the model parameters based on the mechanical behavior of HMA is also presented.

5.2. Analysis

Using the accumulated strain profiles, nonlinear regression was performed using the statistical software SPSS to determine the value of the parameters from Equation 11. To demonstrate the process of separating the accumulated strain profile data into the primary and secondary region and then determining the parameters, a typical accumulated strain profile, shown in Figure 2.3, will be analyzed.

5.2.1. Sample Analysis

A typical accumulated strain profile in RLT will consist of three regions, the primary, secondary, and tertiary. For this analysis, only data in the primary and secondary regions are needed. If a tertiary region exists, those data points were removed from the data set before statistical regression is performed. In the following sections a process of determining the transition points of the strain profile regions and the parameter values is presented for one mix (LA-3). The process was repeated for the remaining 15 mixes.

5.2.1.1. Determine End Points of Secondary Region

The point at which the tertiary region begins, or secondary region ends, is determined numerically. The initial step is to determine the difference in accumulated strain (slope) between each point and its preceding point. Then, the point with the lowest difference (lowest slope) is determined. As a result of this process some secondary region data may be dismissed. However, it should not have a significant impact on the determination of model parameters. The results of the initial step are presented in Figure 5.1.

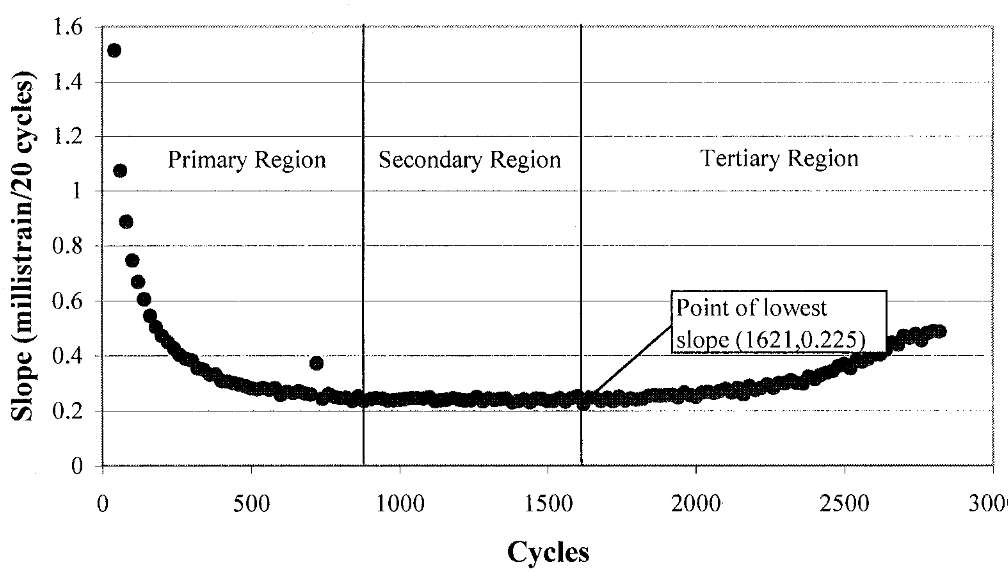

Figure 5.1: Changes in slope of accumulated strain profile (LA-3).

The next step is to determine the point at which the secondary region begins. This is also done numerically, however it is somewhat more difficult to determine precisely due to the "noise" in measure data. First, the difference in slopes is determined between each point and its preceding point, and each point and its succeeding point (Figure 5.2).

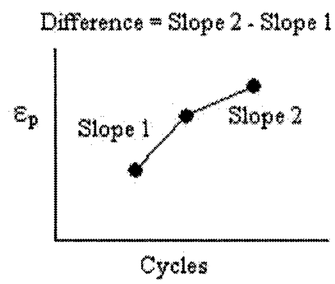

Figure 5.2: Difference in slopes between points.

Then, the point at which these differences remain small and the slope value is consistent is determined. For example, in the case of LA-3, this point was determined to be at 901 cycles because from 901 to 1001 cycles the change in slope remained between ±10 (Figure 5.3). The plots of the changes in slope and the rates of change in slope for the 16 mixes are shown in Appendix A and B, respectively.

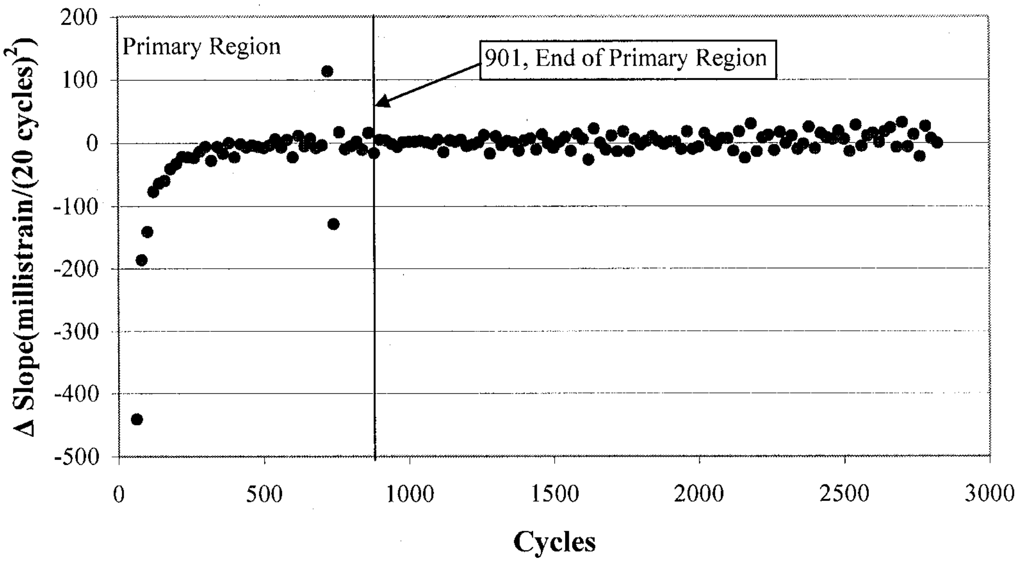

Figure 5.3: Rate of change in slope of accumulated strain profile (LA-3).

5.2.1.2. Determine Parameter Values

Once the regions are determined, the regression analysis is conducted. The data points from the secondary region only were input into the statistical software to determine the parameters b and k_1 of Equation 12. The accumulated strain of in the secondary region is denoted by ε_{ps}.

$$\varepsilon_{ps} = b + \left(J''_{mix}\right)^{k_{1mix}} N \tag{12}$$

The values of the two parameters determined by regression of the secondary region of the sample accumulated strain profile are b = 10.63 (millistrain) and k_1 = 0.81. The values determined for parameters b and k_1 are substituted into Equation 11, shown in Equation 13. The resulting slope D_1 from J''_{mix} and k_1 is 0.248 (millistrain/20 cycles), which is very close to the minimum slope in Figure 5.1.

$$\varepsilon_p = 10.63 + \left(J''_{mix}\right)^{0.81} N - D_2 e^{-\lambda N} \qquad (13)$$

The parameters D_2 and λ were then determined by nonlinear regression using the data points from the primary and secondary regions and Equation 13. The values determined by nonlinear regression for parameters D_2 and λ are $D_2 = 7.13$ (millistrain) and $\lambda = 0.0075$ (1/cycles). A table of parameter values for the 16 mixes and a typical output of SPSS are shown in Appendix C.

5.2.1.3. Results

After substituting the parameters D_2 and λ into Equation 13, the accumulated strain profile obtained from Equation 13 is compared with the measured accumulated strain profile as shown in Figure 5.4.

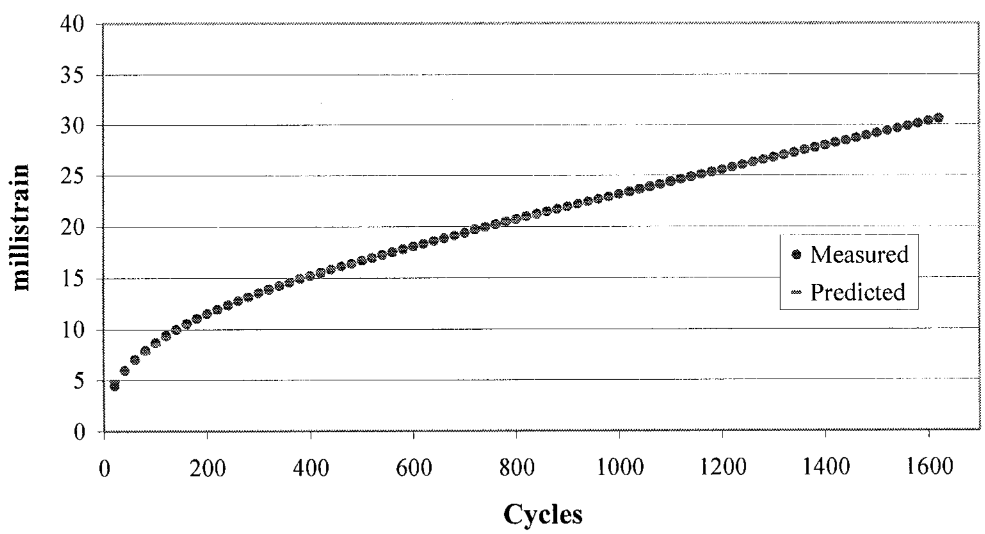

Figure 5.4: Measured accumulated strain profile and Predicted accumulated strain profile from Equation 13 for LA-3.

The measured and predicted values in the secondary region were in good agreement. However, there are slightly larger differences in the values in the primary region. The differences between the measured and predicted are plotted below in terms of residuals (Equation 14), shown in Figure 5.5.

$$residuals = \frac{(predicted - measured)}{measured} \times 100 \qquad (14)$$

The largest differences were displayed by AZ-4, AZ-5, and AZ-6. The accumulated strain profile and the profile obtained from the model for AZ-6 is shown in Figure 5.6. The residuals of the data presented for AZ-6 are shown in Figure 5.7. The remaining plots are presented in the Appendix D and E.

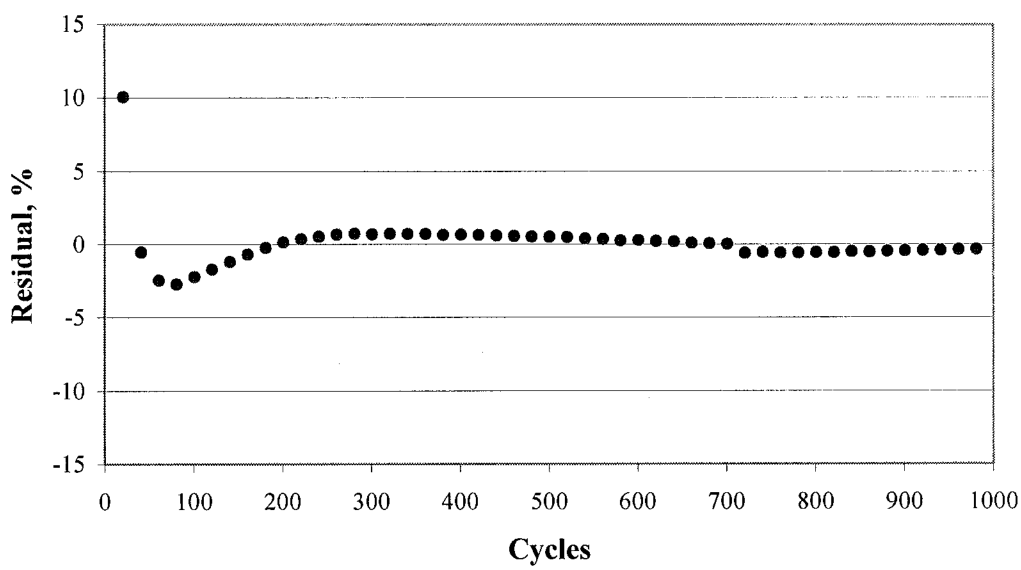

Figure 5.5: Residuals % versus cycles for LA-3.

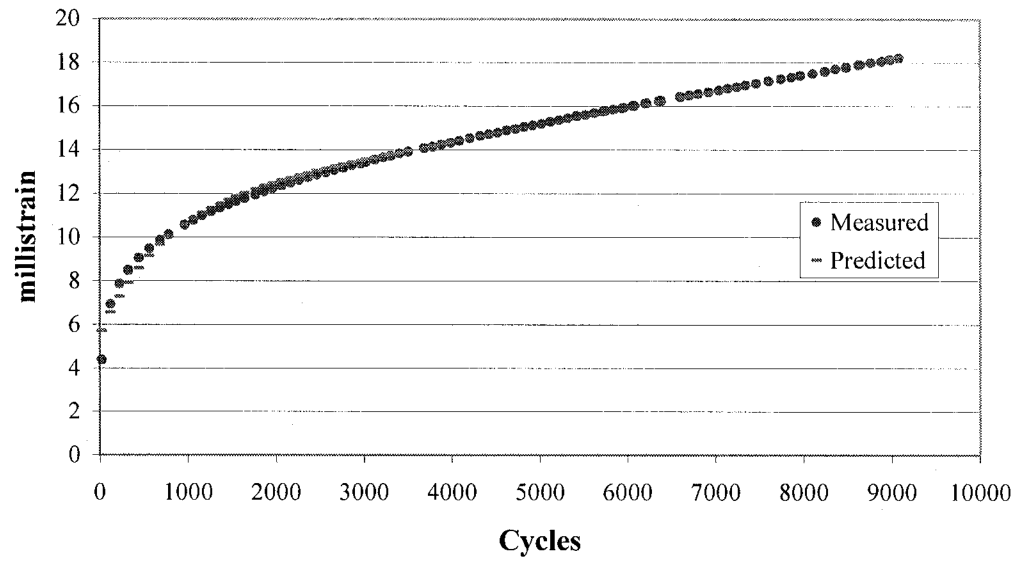

Figure 5.6: Measured and Predicted accumulated strain profiles of AZ-6.

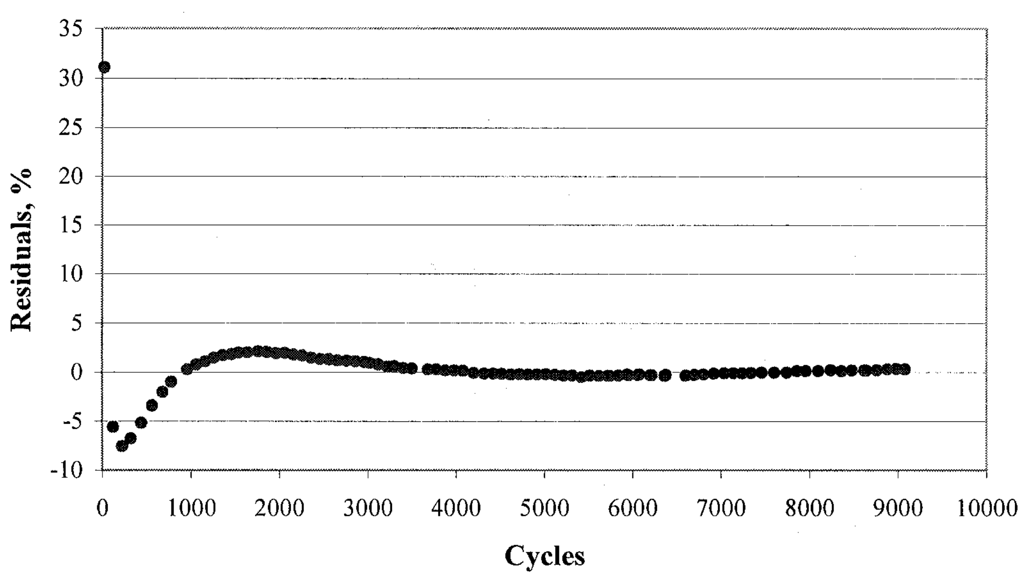

Figure 5.7: Residuals % versus cycles for AZ-6.

The high residual percentage of the initial point of the primary region is likely caused by the lack of sufficient measured data. For example, mix AZ-6 has only 2 data points in the first 121 cycles, which account for 38% of the total accumulated strain shown in Figure 5.6. The large increase in strain that occurs during the early part of RLT is represented by only a few data points, which influences the determination of D_2 and possibly λ by the regression software. This suggests that, out of the 4 fitted parameters, D_2 and λ have the largest degree of uncertainty, as shown in Figure 5.8. The residual percentage would likely decrease if the frequency of data collection were increased in the primary region. However, the data are unavailable. It is felt that the uncertainty will have only a small effect on the predictions made by the model, as most rutting lifetimes are in the thousands of cycles and the model fits the data well after several hundred cycles.

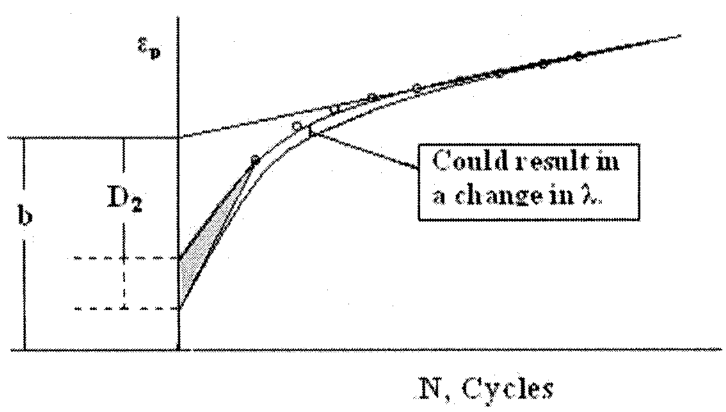

Figure 5.8: Range of possible D_2 and λ values present due to lack of measured data.

5.3. Parameters

After the parameters were determined, the model parameters were validated by evaluating their sensitivity to critical mechanical properties, such as |E*|, φ, and J". The correlations and physical justification of the parameters are described in detail in this section.

5.3.1. Parameter k_l

The parameter k_l showed a decreasing trend with φ, Figure 5.9. This suggests that specimens with smaller phase angles, i.e. more elastic, have lower slope values.

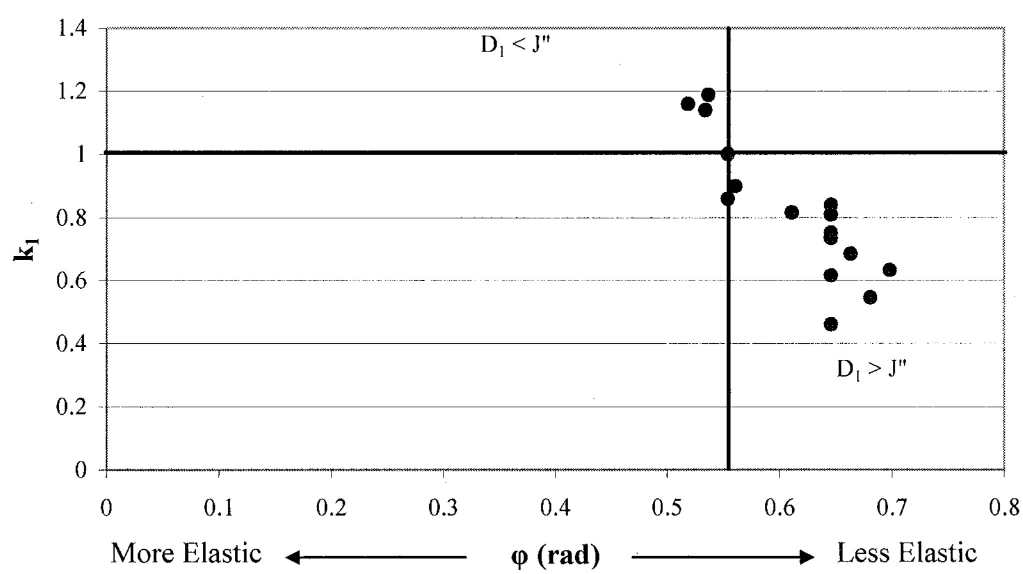

Figure 5.9: Relationship between k_l and φ.

The fact that k_1 has a range and is not equal to one for all mixes can be explained by the differences in the procedures by which the RLT and DM tests are conducted (Section 2.1.2. and 2.1.3.). The loading applied during the RLT causes HMA specimens to enter the nonlinear viscoelastic region where as the DM test does not. The less elastic specimens, which generally exhibit higher φ values and correspond to lower PG grades, would cause more strain accumulation in the nonlinear viscoelastic region than the more elastic specimens would. The differences in the behavior of the specimen between the linear and nonlinear region is accounted for by k_1.

5.3.2. Parameter D_2

The parameter D_2 sets the y intercept of the exponential function and is related to the shaded area shown in Figure 5.10. The area can be computed by integrating the exponential portion of the model from 0 to the end of the primary region (n). The integral is presented in Equation 15.

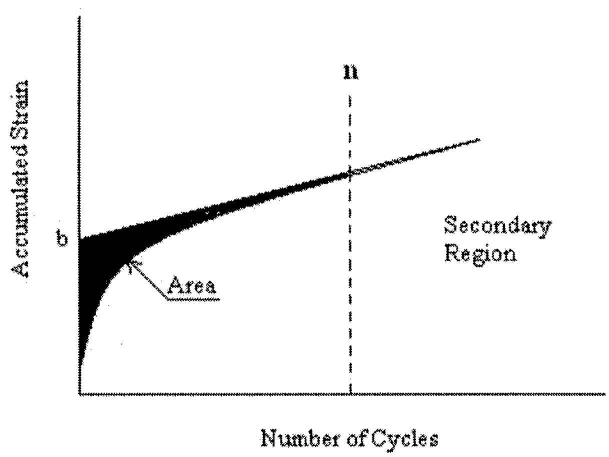

Figure 5.10: Area relating D_2 and φ

$$Area = \int_0^n D_2 e^{-\lambda N} dN = \frac{D_2}{\lambda}\left(1 - e^{-\lambda n}\right) \quad (15)$$

The areas computed using Equation 15 have an inverse relationship with φ, Figure 5.11. The relationship appears to be nonlinear, with the area approaching 0 as φ increases toward 90°. This is consistent with the observed behavior of less elastic mixes. A less elastic mix will enter the secondary region at lower N than a more elastic mix, which will decrease the shaded area in Figure 5.10. If the mix were very elastic the onset of the secondary region will take longer, resulting in a larger shaded area.

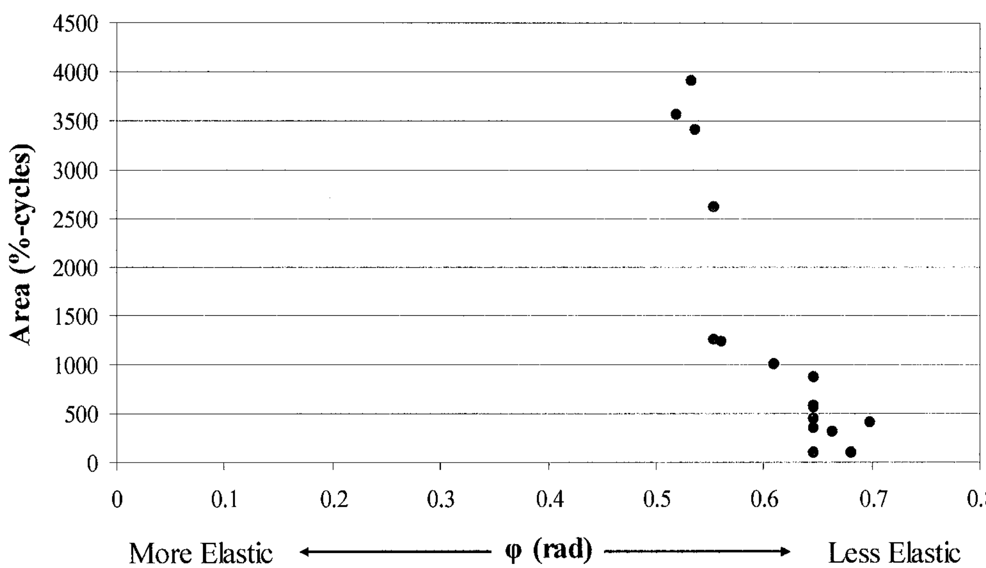

Figure 5.11: Area from Equation 15 versus the average φ of the respective mixtures.

5.3.3. Parameter λ

The shaded area, shown in Figure 5.10, is also dependent on λ. The parameter λ, which is the decay rate of the exponential, determines how many cycles the specimen will experience before it enters the secondary region. Less elastic mixes will enter the secondary region in fewer cycles than more elastic ones and will therefore have a smaller area. As a result of the smaller area, less elastic mixes will have a higher value for λ (on the positive scale). The expected increasing trend of λ with respect to φ is seen with 14 of the 16 mixtures, shown in Figure 5.12.

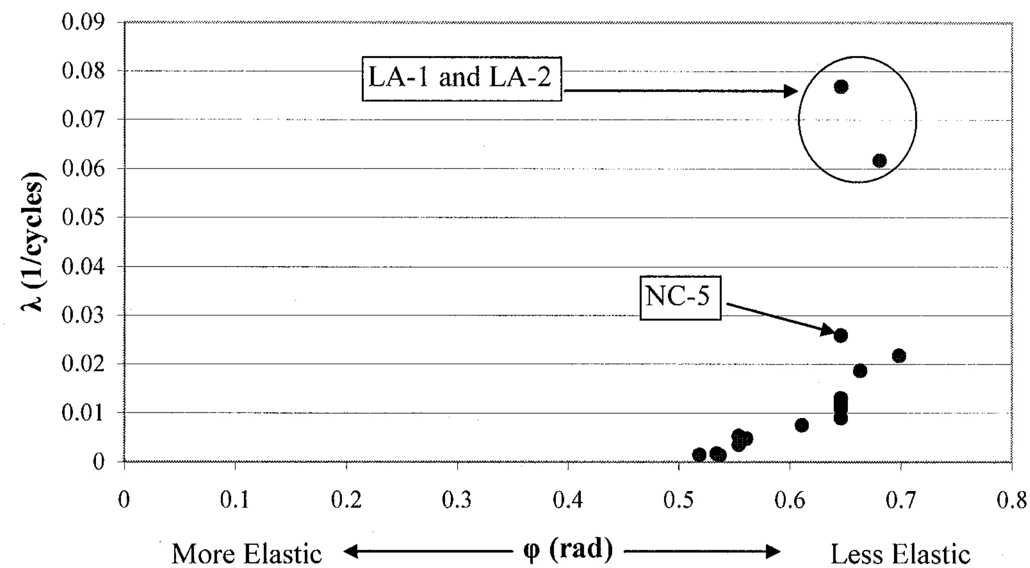

Figure 5.12: Relationship between λ and φ.

As φ becomes larger, the magnitude of λ also increases, which increases the decay rate of the exponential portion of the model and reduces the shaded area (shown in Figure 5.10). The two outliers in Figure 5.12 are two mixes from Louisiana. Theses two mixes are comprised of PG64-22 where as the other two mixes, from Louisiana, are comprised of PG76-22. In addition, the mixes from Louisiana were tested at 54 °C as opposed to 45 °C and they both exhibited a very low Flow Number of 141 (Table 3.2), which would indicate a very rapid transition from the primary region into the secondary. The next closest Flow Number was 326, which was NC-5. The two LA mixes and all of the NC mixes had a significant lack of data in the primary region. The lack of data in the primary region and the rapid transition from the primary region into the secondary region could have caused a misrepresentation of lambda from nonlinear regression. Another possibility is that the outliers could be indicative of different ranges of lambda based on a combination of temperature, FN, and PG grade. These two possibilities could not be further investigated because of insufficient data in the strain profiles and a limited number of mixes representing the behavior.

The relationships presented in this section give physical justification to the parameters of our model (Equation 11). The relationships also show that the parameter φ is an important indicator to the behavior of the accumulated strain profiles. However, these relationships are not meant to be used for extrapolation or for predicting k_1, D_2, or λ from J'' and φ because of the variability present in the trends and the number of mixes is insufficient to justify use of the trends in such a way. The variability present in the trends is likely due to the parameter φ being a linear viscoelastic parameter and the parameters k_1, D_2, and λ being nonlinear viscoelastic parameters.

5.4. Incorporation in Design Guide

Incorporation of the RLT test into the MEPDG to predict rutting potential can be accomplished using either the current form of the current MEPDG rutting model or by incorporating the model developed in this study. The sample accumulated strain profile of mix LA-3 from Section 5.1 will be used to demonstrate the two alternatives in the following section.

5.4.1. Alternative 1

Alternative 1 uses the current form of the rutting model in the MEPDG. The current model calculates the ratio of plastic strain over the resilient strain as a function of temperature and cycles, shown in Equation 16 (NCHRP, 2004b). The current form is a power model that is calibrated using the β's and k's to produce a rut profile. The β's are local calibration factors, which are determined by users of the MEPDG to calibrate the rutting models to their region or area. The k values are global calibration factors, which were determined from regression analysis of 3,476 permanent strain data points (NCHRP, 2004b).

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_{r1} 10^{k_{r1}} T^{\beta_{r2} k_{r2}} N^{\beta_{r3} k_{r3}} \tag{16}$$

where:

$\varepsilon_{rz} = \dfrac{\sigma_z - \nu\sigma_x - \nu\sigma_y}{|E^*|}$ = resilient strain, (NCHRP, 2004b)

x, y, z = denotes direction of vector in Cartesian coordinates.

ε_p = accumulated permanent strain.

T = temperature (°F)

N = number of cycles.

v = Poisson's ratio.

β_{r1}, β_{r2}, β_{r3} = local calibration factors.

k_{r1}, k_{r2}, k_{r3} = global calibration factors.

To incorporate the accumulated strain data into the MEPDG using the current form of the rutting model, an equation is presented in the form shown below (Equation 17). Nonlinear regression, using SPSS, was used to determine the calibration factors of Equation 16 for LA-3.

$$\varepsilon_p = 10^{m_{r1}} T^{m_{r2}} N^{m_{r3}} \tag{17}$$

where:

m_{r1}, m_{r2}, m_{r3} = mix calibration factors.

The mix calibration factors determined for LA-3 were m_{r1}= 0.0586, m_{r2}= -0.0565, and m_{r3}= 0.4776. The resulting accumulated strain profile obtained from Equation 17 was compared with the measured data, shown in Figure 5.13. The residual percentages between the two profiles are shown later in Figure 5.15.

Alternative 1

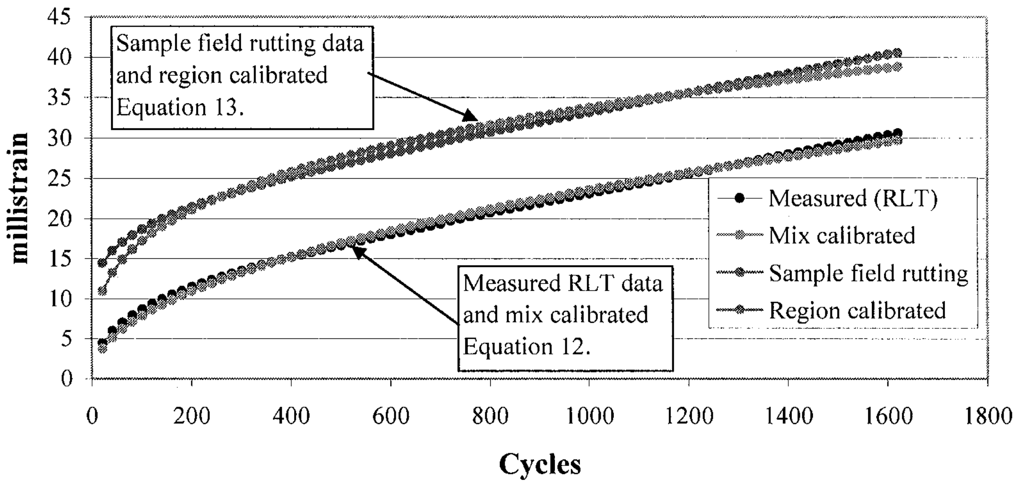

Figure 5.13: Measured and sample rut depth profiles versus predicted profiles from calibrated MEPDG models for LA-3.

The model will then have to be calibrated to field rutting data using additional regional calibration factors, as shown in Equation 18, which has the same form as Equation 16.

$$\varepsilon_p = r_{r1} 10^{m_{r1}} T^{r_{r2} m_{r2}} N^{r_{r3} m_{r3}} \tag{18}$$

where:

r_{r1}, r_{r2}, r_{r3} = regional calibration factors.

To demonstrate regional calibration to field rutting data, a rut depth profile was created by adding 10 millistrains to each accumulated strain data point of LA-3 because no actual field rutting data was available. The basis for simply scaling the accumulated strain profile was to demonstrate similar behavior to that of the accumulated strain data and field rutting data from the Florida Heavy Vehicle Simulator (HVS) shown in Figure 5.14 (Myers et al., 2005). The behavior shown in Figure 5.14 is the result of one study and is biased because the test was conducted at a constant load, in one geographical area, and for a limited duration (approximately 3 months). The agreement between the accumulated strain data and field rutting data may not always be as parallel as what is represented in Figure 5.14 due to long-term climate effects, variable traffic loading, different mixtures and pavement structures.

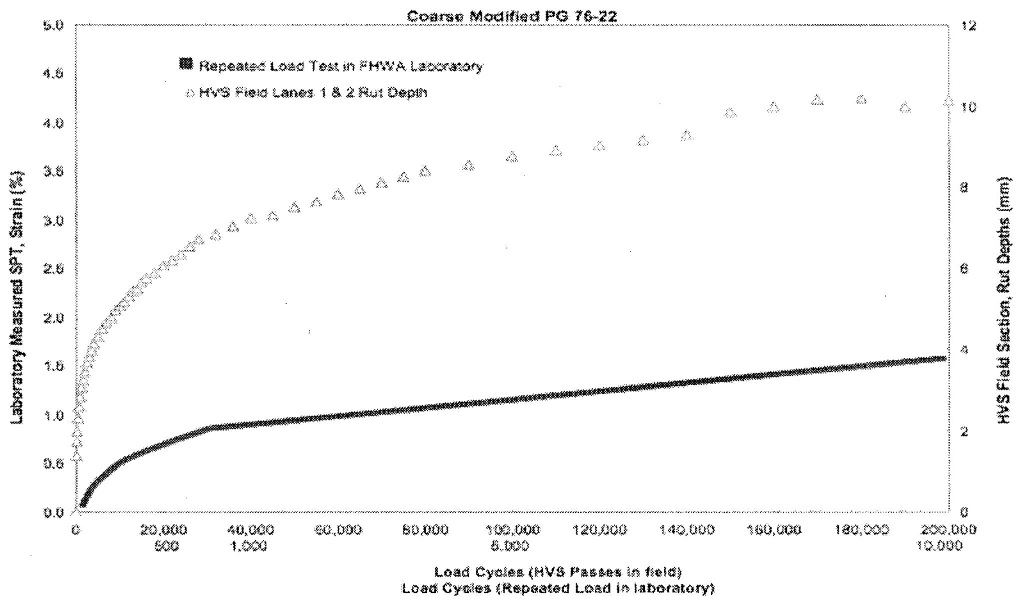

Figure 5.14: Accumulated strain profile from RLT and field rutting data from HVS (Myers et al., 2005).

The regional calibration factors determined using nonlinear regression were r_{r1}= 7.517, r_{r2}= 2.313, and r_{r3} = 0.607. The accumulated strain profile obtained from Equation 18 and the regional calibration factors determined above were compared with the field rutting data created for LA-3, shown in Figure 5.13.

To incorporate the RLT into the MEPDG, input the mix calibration factors as the global calibration factors and the regional calibration factors as the local calibration factors. It may also be necessary to scale r_1 by dividing it by ε_r before it is input into the MEPDG rutting model.

Although alternative 1 appears to be a viable one, the fit is rather crude and does not accurately capture the behavior. This can be seen more clearly when the residuals from the current model are compared with that of the proposed model, shown in Figure 5.15.

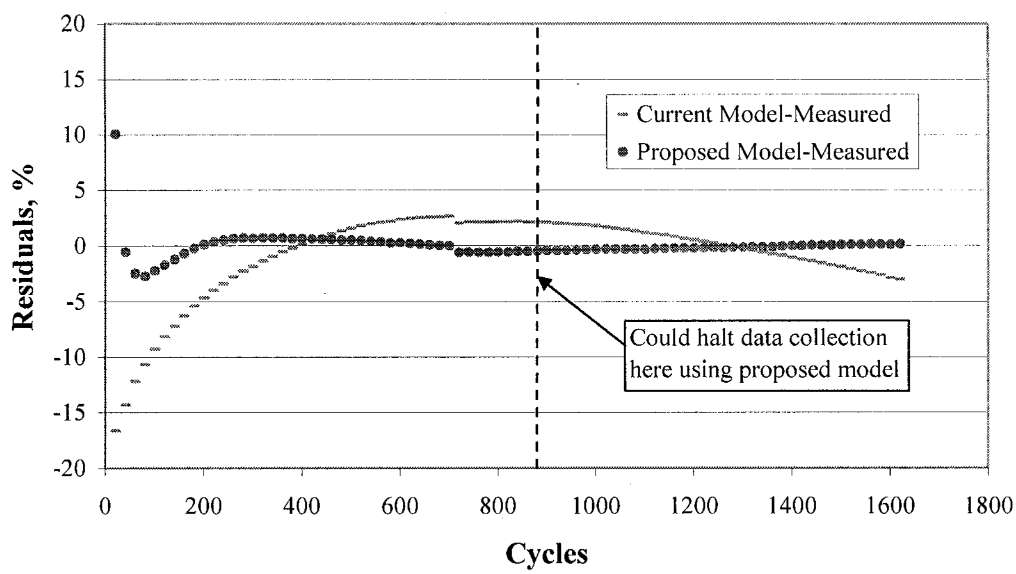

Figure 5.15: Residuals from current model and measured data and proposed model and measured data (LA-3).

It can also be seen that the model developed in this study has the potential to accurately predict the primary and secondary regions without collecting data up to the Flow Number, as shown in Figure 5.15. Therefore, it would be recommended to implement Alternative 2.

5.4.2. Alternative 2

Alternative 2 would involve removing the current rutting model from the MEPDG and replacing it with the model developed in this study. To implement the model in the MEPDG regional calibration factors will need to be added. It may also be more convenient to change the labeling of the parameters to help distinguish between mix calibration factors and regional calibration factors. A suggested format of Equation 11 with regional calibration factors added is shown as Equation 19.

$$\varepsilon_p = r_1 m_1 + r_2 m_2 N - r_3 m_3 e^{-r_4 m_4 N} \tag{19}$$

where:

m_1 = b, mix calibration factor.

m_2 = D_1, mix calibration factor.

m_3 = D_2, mix calibration factor

m_4 = λ, mix calibration factor.

r_1, r_2, r_3, r_4 = regional calibration factors.

To obtain the mix calibration factors follow the process laid out in Section 5.2.1. The results for the sample strain profile, LA-3, are shown in Figure 5.3.

To determine the regional calibration factors for the sample strain profile, LA-3, follow the process laid out in Section 5.2.1 once again, but using the form of Equation 19 and 20 rather than Equation 11 and 12 respectively. The accumulated strain in the secondary region is denoted by ε_{ps}.

$$\varepsilon_{ps} = r_1 m_1 + r_2 m_2 N \tag{20}$$

The regional calibration factors obtained by nonlinear regression for LA-3 and the sample field rutting from Section (5.4.1) were $r_1 = 1.94$, $r_2 = 1$, $r_3 = 1$, and $r_4 = 1$. The resulting accumulated strain profiles are compared with the measured and sample field rut depth profiles in Figure 5.16. Residuals of the measured RLT data and the mix-calibrated model are shown in Figure 5.5.

The model developed in this study can easily be calibrated to RLT data from a mix and then calibrated to field rut depth profiles. The model also demonstrates proper behavior within the primary and secondary regions.

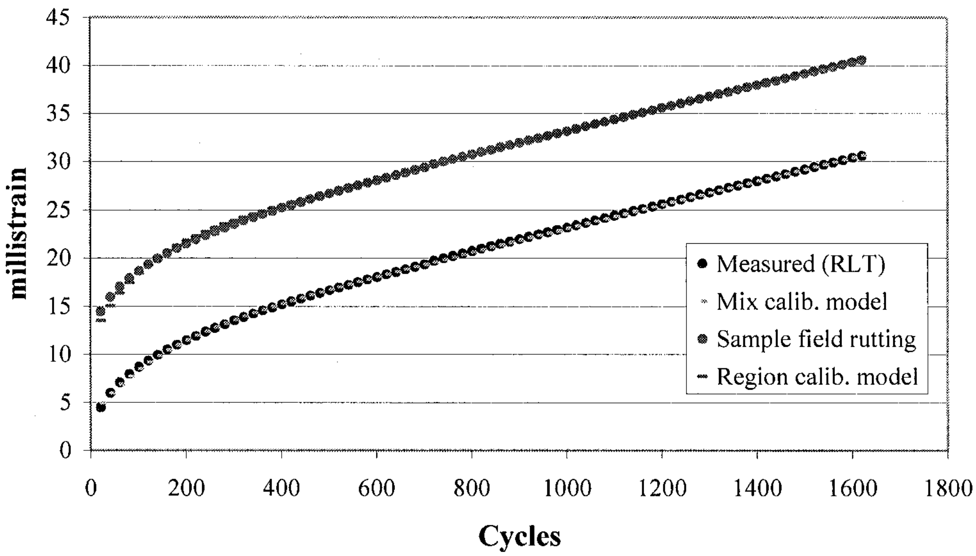

Figure 5.16: Measured and sample rut depth profiles versus predicted profiles from calibrated models for LA-3.

5.5. Application

Currently, tests are conducted up to the FN, which may require testing up to 10,000 cycles. This testing may take several hours and a significant amount of resources. The proposed model and the analysis clearly demonstrated that testing up to the FN is not necessary to determine parameters of the model. However, that brings up the question as to how many cycles are needed to accurately predict the slope of the secondary region. This section presents the results of the analysis conducted to determine the number of cycles needed to accurately predict the slope of the secondary region D_1.

Only 8 of the 16 mixes were used during this analysis because the remaining 8 mixes had short secondary regions and low frequency of measurement. In addition, the 8 mixes not used would have reduced testing times to begin with. The residual value (%)

of the predicted slope, for a given number of cycles from the inception of the secondary region, is shown in Figure 5.17. The analysis indicates that the degree of accuracy in predicting the slope increases as the number of cycles within the secondary region increases. The slope of the mixes generally decreases from left to right with the exception of some overlapping of the LA mixes with the AZ mixes in Figure 5.17.

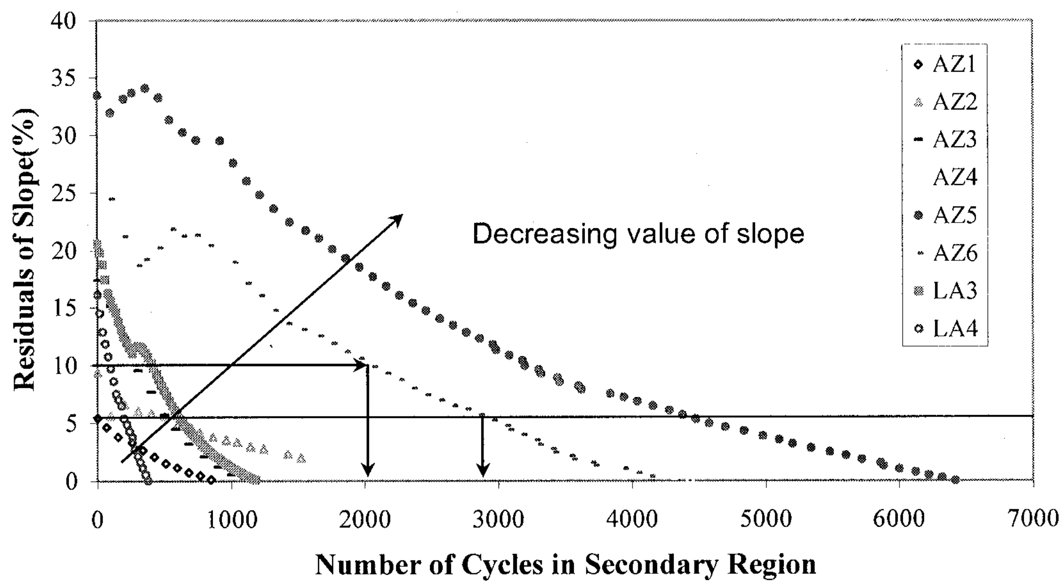

Figure 5.17: Residual (%) of the slope value versus Number of Cycles in Secondary Region.

There is a trend present between the slope of the secondary region and the number of cycles necessary to obtain the desired accuracy. The more elastic mixes have a low slope in the secondary region, so determining the slope accurately becomes more difficult and requires measurement over more cycles. The interference from the noise in the data also becomes more of an issue as the slope value decreases. In addition, very elastic mixes have large primary regions and enter the secondary region so gradually that it is difficult to determine the boundary of the primary and secondary region, requiring measurement over more cycles to accurately determine D_1.

The analysis indicates that the number of cycles necessary to obtain the desired accuracy is mixture dependent. Therefore, standard FN testing will have to be conducted on routinely used mixes to determine the number of cycles necessary to obtain D_1 within a reasonable accuracy for use with future specimens. For example, a state agency would identify a range of mixtures with various PG grades and aggregate sources. Then, FN testing would be conducted on mixture specimens to categorize the mixtures. The performance of the categorized mixtures could be used to predict the number of cycles necessary to obtain a chosen degree of accuracy in D_1 for future mixture specimen. If the desired accuracy in predicting the slope were 5% the minimum number of cycles necessary after inception of the secondary region would be 2960 (for AZ6 mix), shown in Figure 5.17. However, if the desired accuracy were 10% the minimum number of cycles after inception of the secondary region would be 2060 for the same mix, as shown in Figure 5.17. The number of cycles necessary after inception depends on the mixture and the desired accuracy, so there is no single value for all mixtures.

The example presented above is one possible solution to how a state agency could go about categorizing their mixtures. It is unclear at this time as to whether using this approach will eliminate the need for additional testing. However, it should aid in reducing the testing time of routinely used mixtures, which will be a significant economic benefit to the state agency.

5.6. Summary

This chapter presented an example of how to obtain the parameters of the proposed model, along with the physical justification of the parameters of the model. Then, incorporation of RLT data into the MEPDG was discussed using the current power model and the new proposed model, as well as the expected significance of the proposed model. The next chapter will discuss the conceptual framework.

CHAPTER 6

Conceptual Framework for Developing a Mix Catalog for Mechanistic-Empirical Flexible Pavement Design

6.1. Introduction

This chapter presents a conceptual framework for determining what level of input is needed for the MEPDG and the development of a routinely used mixture catalog. This will aid state highway agencies in controlling costs associated with determining input values for the MEPDG and it will help to streamline the mixture design process. This conceptual approach will be applicable for agencies with consistent binder and aggregate sources and may not work for agencies that import raw materials and have a large variety of paving contractors.

6.2. Existing Approaches to Material Characterization for Mechanistic-Empirical Flexible Pavement Design

There have been many activities related to the MEPDG since a research version was made available to the public in July 2005. Based on the survey of various state agencies (Mehta, 2005), it appears that the effort to implement the design guide and develop a mixture catalog is dependent on three factors:

1. The experience of personnel in state transportation agency's research division in material characterization and pavement design.

2. The confidence and the degree of involvement in applying M-E pavement design principles and the NCHRP 1-37A guide and software.
3. The relationship between the state agency and their research partners.

6.2.1. State Agency Efforts

Various state agencies, such as Maine, Maryland, Massachusetts, Missouri, New Jersey, New York, Texas, Virginia and Washington have initiated efforts to develop a mixture catalog as a step towards implementation of the Mechanistic Empirical Pavement Design Guide (MEPDG). This section summarizes the efforts of the various state agencies.

<u>Maine</u>: The Maine DOT is collecting material characterization data to get sufficient inputs for the MEPDG. They are expecting it to be a low-level design for their roadways. Standard mixtures that cover a <u>broad</u> range of gradation, aggregate sources and binder type have been selected for dynamic complex modulus testing, which is being conducted by Worcester Polytechnic Institute. In addition, resilient modulus testing is being conducted on subgrade soils. The state agency has instrumented weigh-in-motion (WIM) sites for traffic data collection. They will provide more detailed data from their database.

<u>Maryland:</u> The University of Maryland is developing a strategic plan with Maryland SHA, which began 2 years ago. The state agency is working with the FHWA to determine DM on a production basis. The FHWA is coordinating the effort to determine how well Witczak and Hirsch models are predicting DM. The state agency is currently

planning to initiate a research project to develop a mix catalog for unbound (geo-gauge) and then bound material. The next step will be to develop a good database for level 2. The priority of the agency is to evaluate performance models. They are currently using Darwin software in parallel with the new design guide. The mixture catalog will be developed based on geographical and environmental regions and the highest users of sources using typical mixtures.

Missouri: The local calibration is done by Applied Research Associates (ARA) for six months. They are simultaneously using MEPDG (from nationally calibrated models). They are anticipating thinner designs and better prediction of performance thresholds as compared to AASHTO. They are using the results of the study by Harold Von-Quintus that independently calibrated the parameters. The state agency has purchased DM testing equipment and is planning to send an RFP to other universities for low temperature characterization. The mix catalog will be developed using the MO DOT's F-gradation of smaller rocks and change gradation from the base. The sources are variable and are difficult to quantify. For very low volume, Marshall Stability will be used. The binder grades for state roads are PG 64-22, for arterials are PG 70-22 and interstates are PG 76-22. The proposed testing will be related to posted speeds. ARA will determine mixture sensitivity and make predictions from laboratory data to explain the bias with field performance.

New Jersey: The state of New Jersey has spearheaded the effort of collecting data for material characterization of both bound and unbound material in 2002 in anticipation of the pavement design guide. They initiated a project of collecting seasonal material property of various roadways in New Jersey. The final report is going to be released by the end of 2005. A database has been developed that will eventually be used for the pavement design guide.

New York: The state of New York is a lead state in various NCHRP and AASHTO studies. They are conducting local calibration using performance-monitoring sites as part of a pooled funds study. Approximately ten sites are selected annually for pavement monitoring. The sites are selected based on geographical and environmental conditions and traffic level. Among these sites, a subset is selected based on available resources and logistical issues. The plant material of these sites is then sent to Kansas State University for detailed material characterization as part of the pooled fund study.

Texas: The state agency is currently not sure if they can implement the design guide. They feel that the rehabilitation section of the design guide is weak and trenches may be necessary. They may develop their own ME design guide by refining their current semi-mechanistic design procedure. They are developing a database for volumetric, aggregate sources and mixture design process.

Virginia: The state agency of Virginia has collected a sufficient amount of pavement material characterization for rigid pavements. The state agency has initiated a research project with Virginia Transportation Research Center (VTRC) to collect dynamic complex modulus for a few mixtures consisting of a selected aggregate type, nominal maximum size and binder type. They mainly use 9.5-mm and 12.5-mm - nominal maximum size and only change the binder grade depending on the truck traffic. The state uses 9.00mm fine mixes for urban roadways.

Due to this simplified array of mixtures, the first phase is primarily focused on observing the sensitivity of the parameters to DM, the second phase may be initiated based on the results of the first phase. The state agency may not have distinct levels as maintained in the original ME Pavement design guide. Based on the results, they might have a combination of level of data requirements, i.e. elements of Level 1, Level 2 and Level 3 may be combined to create their own hierarchy for the design guide. Further information on the Virginia Department of Transportation's plan for implementation of the MEPDG is given in Appendix F.

<u>Washington</u>: As a first step towards developing a mixture catalog, typical mixtures from typical sites will be selected and samples will be selected from production sites. The mix as well as the aggregates and binder will be sent to Washington State University for Dynamic Modulus testing. The gradation will then be varied from the production mix (keeping within the Superpave gradation guidelines) and its sensitivity to DM will be determined.

If a significant change in DM is observed (in the order of magnitude), the recommendations for change in gradation may be made. These changes will not affect their structural design, since they already have thin sections and it may be uneconomical for thinner sections due to multiple passes necessary for compaction of thin layers.

6.2.2. Summary of State Agency Efforts

The state agencies are aware that developing a mixture catalog is a time consuming process and requires considerable investment. Some state agencies were willing to make this investment if it would lead to design of reduced testing effort, thinner sections, and therefore long-term savings. The various state agencies have adopted one or more of the following three broad strategies of developing a mixture catalog (Mehta, 2005):

1. Write a research proposal to determine the sensitivity of mixtures to Dynamic Modulus (DM) for eventual development of a mixture catalog for project-specific (Level 1) MEPDG input. The responsibility of developing the experimental protocol lies with the research partners, subject to approval by the state agency. This approach is employed by states such as Maine and New Jersey.
2. Collect samples from ongoing construction projects for DM testing either in-house or with research partners. Additional testing may be conducted by varying material properties from the Job Mix Formula (JMF) in the laboratory. This additional testing evaluates the sensitivity of DM to changes in gradation and volumetric parameters. Field performance monitoring will be done in-house or by research partners. This strategy is being used in New York and Washington.
3. Develop a database of material properties for site-specific (Level 1) design, in anticipation of the MEPDG software. Site selection and laboratory testing is done in-house or through a long-term research project with research partners. This approach is being utilized in New Jersey, Texas, Virginia and Wisconsin.

As various state agencies adopt one or more of the above strategies, several state agencies are also evaluating the new pavement designs using national default values from Long-Term Pavement Performance program (LTPP) Specific Pavement Studies (SPS) sections (Level 3) input for the MEPDG. The comparison of designs between Darwin software (AASHTO 1993 design guide) and the new design guide allow them to get familiarized with the input levels.

6.2.2.1. New England States

The New England states have an advantage in that the materials sources in the region are consistent and well documented. The sources do not change as often as other regions due to the lack of more land to quarry new formations. This situation provides them with a unique opportunity to pool their efforts. Even though several New England states have begun their own efforts to develop a mix catalog, a comprehensive approach would be cost effective and beneficial to all the participating states.

6.3. Conceptual Framework

The following section introduces the conceptual framework for developing a mix catalog. The framework is presented in the form of a flowchart in Figures 6.1 and 6.2 and explained in detail in the following subsections.

6.3.1. Preliminary Evaluation Prior to Site Selection

In the first phase, the state agency should conduct a preliminary evaluation of their pavement sections to determine if the agency should dedicate resources for collecting project-specific (Level 1) input. The preliminary evaluation includes selecting existing pavement sections of different pavement stiffness and layer thickness (Figure 6.1). The performance of these pavements should be evaluated using default value (Level 3) input.

If the predicted performance compares well with the measured performance (determined from routine pavement management data), as shown in Figure 6.3, the agency may not need to collect data for Level 1 input and nationally calibrated values may be sufficient to represent the material properties of the region. However, if the predicted performance does not correlate reasonably well with measured data then the agency should take measures to collect data for some site-specific material properties as input in the MEPDG. The following section provides the steps to be taken by the agency to efficiently and cost effectively collect data for Level 1 input.

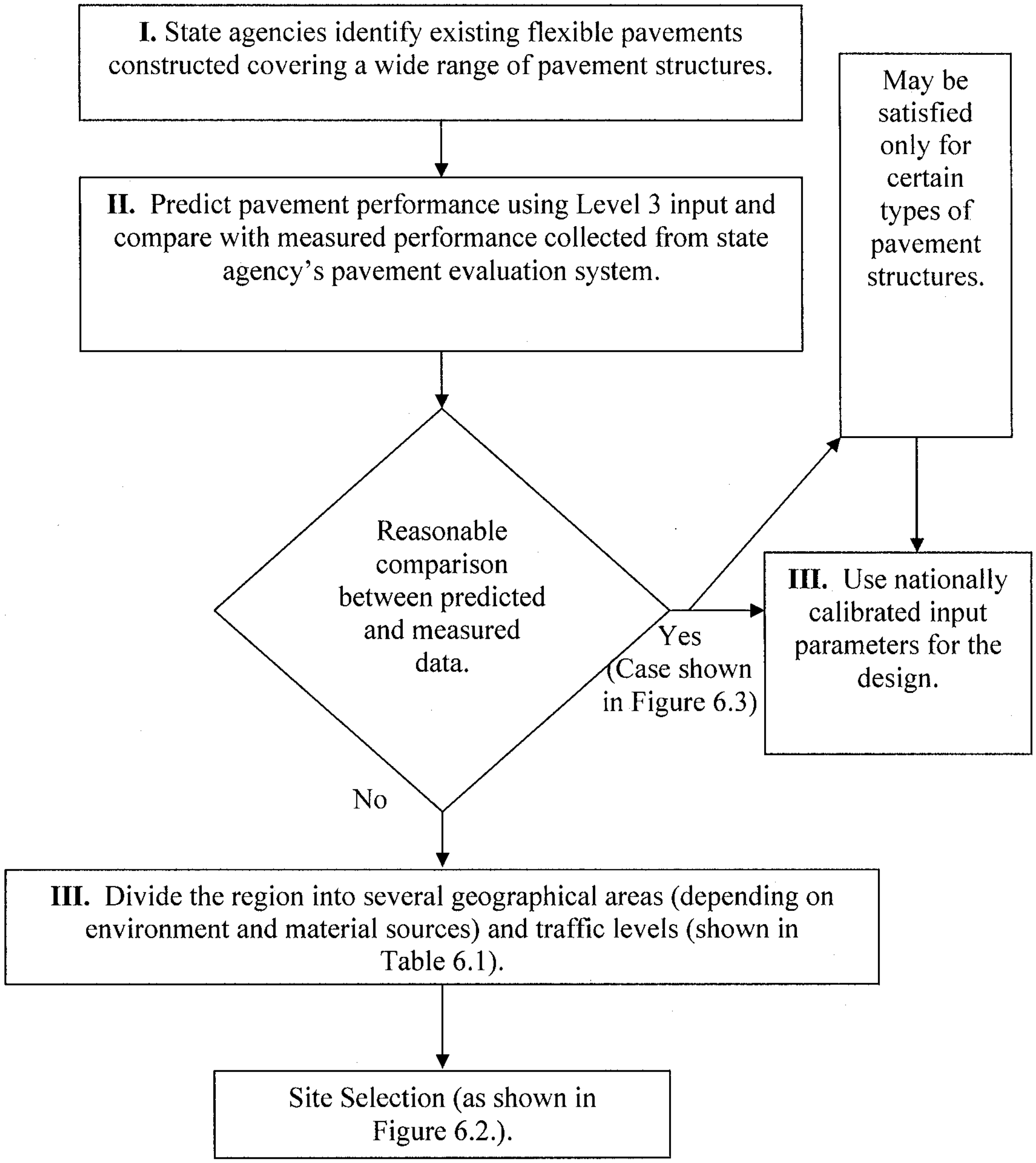

Figure 6.1: Preliminary evaluation prior to site selection.

6.3.2. Site Selection

Based on initial evaluation, should the state agency identify the need to collect data for Level 1 input, the next phase consists of dividing the region into several geographical areas, depending on environment and traffic levels (Table 6.1).

Table 6.1: Categorization of region based on environmental condition and traffic level.

	Traffic Level		
Region	TL-1 (H)	TL-2 (M)	TL-3 (L)
R-1 (Coastline)	A	B	C
R-2 (Plains)	D	E	F
R-3 (Mountain)	G	H	I
R-4 (Desert)	J	K	L

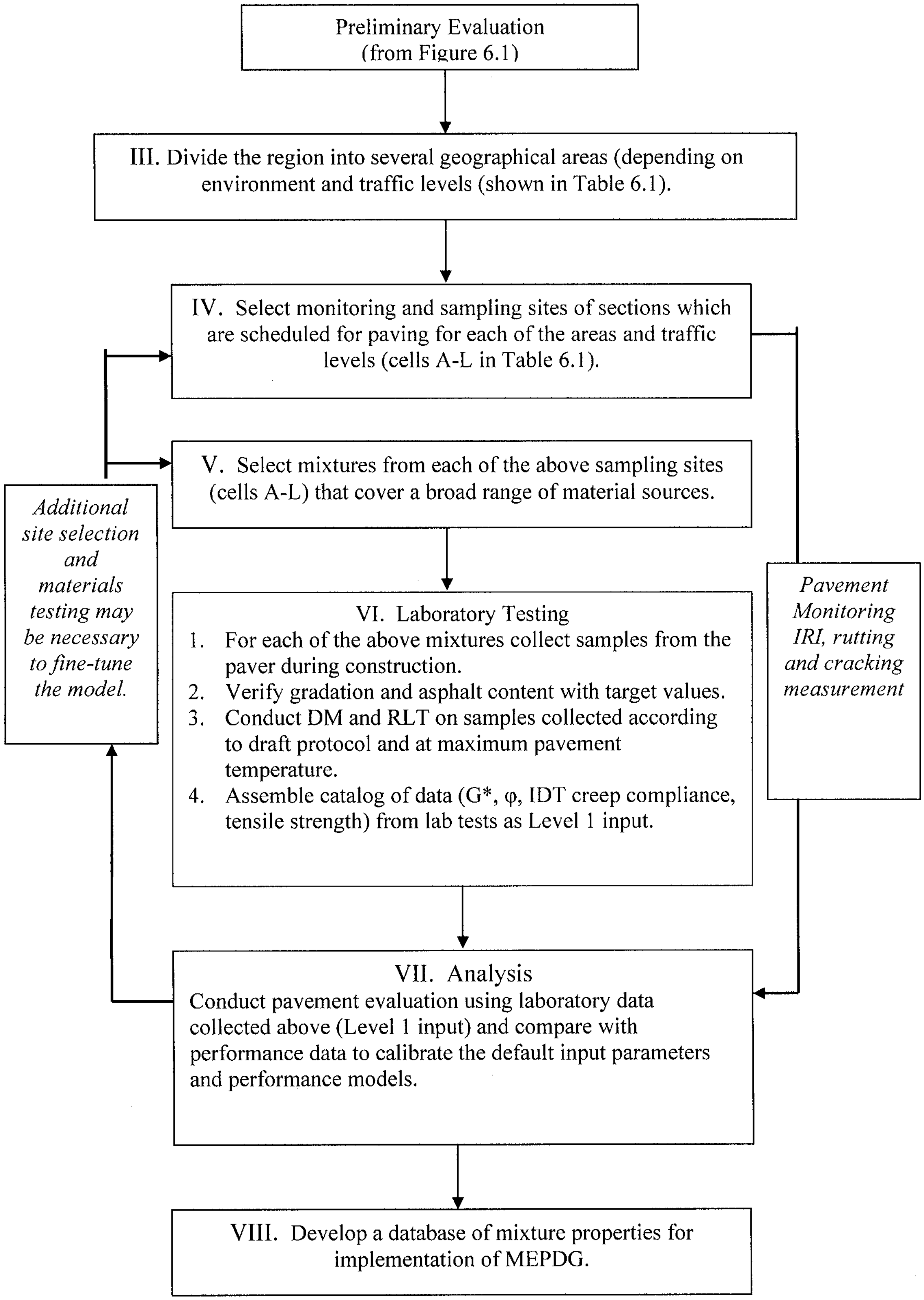

Figure 6.2: Site selection, laboratory testing and analysis.

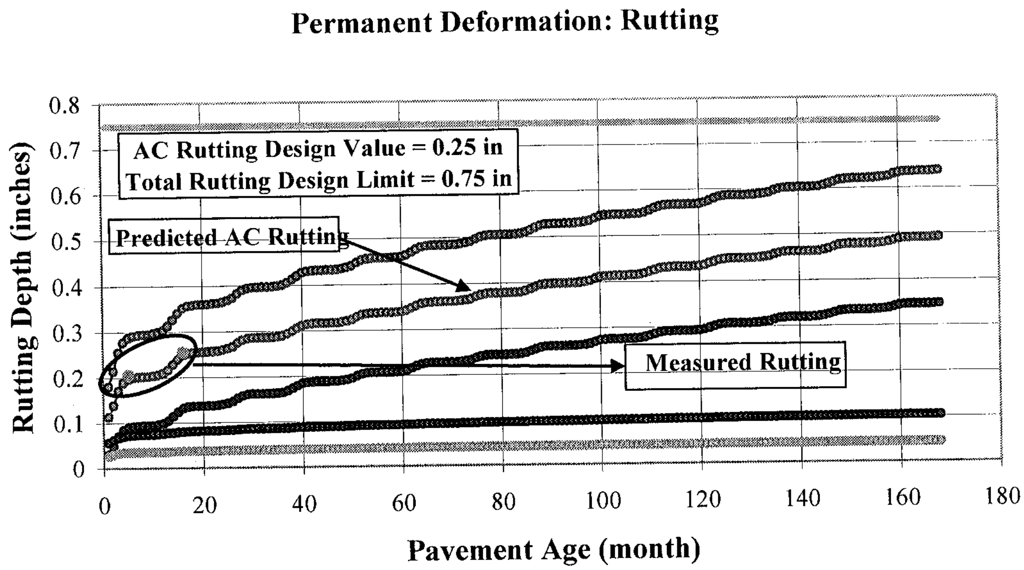

Figure 6.3: An example of good prediction of asphalt rutting using national default (Level 3) input.

For each of the areas and traffic levels (cells A-L in Table 6.1), the idea is to select monitoring and sampling sites of sections which are scheduled for paving. From each of the above sampling sites (cells A-L) mixtures should be selected that cover a broad range of aggregate sources and gradations. In the case of a state like California, where many materials and geographical regions are present, all cells in the matrix would apply. A state such as Arizona might use all but the Coastal scenario. For a region like New England, there would not be plains or desert regions to evaluate. Thus in the New England region, the state agencies can combine their effort due to consistent aggregate sources and geographic similarities. The next phase in the approach deals with laboratory testing of collected samples and is presented in the following section.

6.3.3. Laboratory Testing

For each of the representative mixtures selected, samples should be collected from the paver or mix trucks during construction. Then, binder and aggregate test data can be obtained through routine Superpave testing and followed with comparison with target values and assessment of the level of variability (if any). Measures of specific HMA properties such as the DM (|E*|) and/or repeated load test on samples are collected according to draft protocol and at maximum pavement temperature. Investment in initial regional round robin testing of binder and indirect tensile test (IDT) results will help to establish repeatability of test results on typical HMA mixes. As a final step, a catalog of data such as shear complex modulus (G*), phase angle (δ), and data from the IDT test (i.e. creep compliance and tensile strength), for region-specific (Level 1) input can be assembled.

6.4. Analysis

After the region-specific (Level 1) input data is collected, a pavement evaluation should be conducted using Level 1 input (Figure 6.1 box VII). The pavement performance should be compared with observed field performance data, which will be collected from a state's pavement management system. The default input parameters and performance models can be calibrated by comparing both short-term and long-term measured field data with predicted performance data. The final step consists of developing a catalog of mixture properties for various distress levels and routine pavement structures to assist and facilitate selection of appropriate MEPDG inputs.

6.4.1. Example Application of Framework

An example of the analysis portion of the framework is presented by selecting four asphalt mixes that are typically used in New England, in order to demonstrate the potential use of the framework for regional calibration efforts. This evaluation was done for a baseline pavement structure, taken from Interstate 95 northbound near Bangor, Maine, which was rehabilitated in August 2003. The baseline pavement structure (Structure 1) consists of a 2 inch wearing course atop a 2 inch binder course, as shown in Figure 6.4.

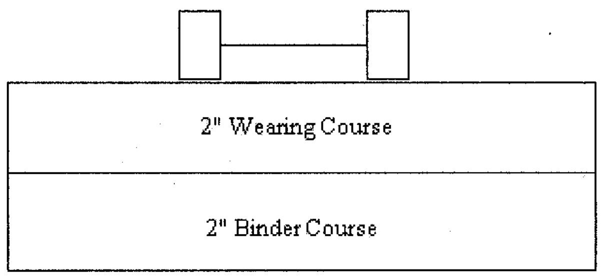

Figure 6.4: Structure 1: Diagram of baseline pavement structure (HMA lifts only).

Each of the four mixes was analyzed individually in the MEPDG as the wearing course of Structure 1. The type of gradation and nominal maximum sizes are summarized in Table 6.2. The binder grade for all mixtures was PG 64-28.

Table 6.2: The gradation of four New England mixtures.

Mix	Gradation	Nom. Max. Aggregate Size, mm
Wearing Course - Mix 1	Coarse	9.5
Wearing Course - Mix 2	Fine	12.5
Wearing Course - Mix 3	Coarse	12.5
Wearing Course - Mix 4	Fine	9.5
Binder Course (Mix 1)	Coarse	9.5

The analysis was conducted using Level 1 HMA inputs, as measured from binder and mix lab testing conducted by the Federal Highway Administration's Mobile Asphalt Laboratory, for the four mixes and binder course. The distress criteria assumed was a total rut depth of no more than 0.3 inches. The required climate data was obtained from National Oceanic and Atmospheric Administration (NOAA) data tables. The weather stations selected were those with the closest proximity to where each mix was sampled. Table 6.3 shows that the output of MEPDG analyses for the four mixes yielded predicted rut depths in the wearing course within the allowable level, but the binder course was not. Table 6.3 also contains data from structure 2, which is described in Section 6.4.1.1.

Table 6.3: MEPDG rut depth and |E*| output data for baseline structure, alternatives 1 & 2.

	Rutting Criteria: 0.3 (in)				Rutting Criteria: 0.5 (in)	
	Structure 1		Structure 2		Structure 1	
Mix 1	Wearing	Binder	Wearing	Binder	Wearing	Binder
Allowable Rut Depth (in)	0.07	0.23	0.09	0.21	0.12	0.38
Predicted Rut Depth (in)	0.06	0.25	0.06	0.19	0.06	0.25
Total Predicted Rutting (in)/ Within Failure Criteria?	0.31/No		0.25/Yes		0.31/Yes	
[1]Minimum Layer E* (ksi)	478.71	929.56	421.67	999.25	318.35	414.15
[2]Predicted Layer E* (ksi)	532.15	837.63	532.15	1244.03	532.15	837.63
Mix 2						
Allowable Rut Depth (in)	0.07	0.23	0.09	0.21	0.12	0.38
Predicted Rut Depth (in)	0.06	0.26	0.07	0.19	0.06	0.26
Total Predicted Rutting (in)/ Within Failure Criteria?	0.32/No		0.26/Yes		0.32/Yes	
[1]Minimum Layer E* (ksi)	483.95	950.50	426.01	1025.11	320.92	423.16
[2]Predicted Layer E* (ksi)	514.54	810.41	514.54	1208.20	514.54	810.41
Mix 3						
Allowable Rut Depth (in)	0.07	0.23	0.09	0.21	0.12	0.38
Predicted Rut Depth (in)	0.07	0.27	0.07	0.20	0.07	0.27
Total Predicted Rutting (in)/ Within Failure Criteria?	0.34/No		0.27/Yes		0.34/Yes	
[1]Minimum Layer E* (ksi)	485.87	958.07	427.60	1034.50	321.87	426.42
[2]Predicted Layer E* (ksi)	508.32	800.77	508.32	1195.44	508.32	800.77
Mix 4						
Allowable Rut Depth (in)	0.07	0.23	0.09	0.21	0.12	0.38
Predicted Rut Depth (in)	0.06	0.25	0.06	0.19	0.06	0.25
Total Predicted Rutting (in)/ Within Failure Criteria?	0.31/No		0.25/Yes		0.31/Yes	
[1]Minimum Layer E* (ksi)	478.71	929.56	421.67	999.25	318.35	414.15
[2]Predicted Layer E* (ksi)	532.15	837.63	532.15	1244.03	532.15	837.63

[1]Minimum stiffness of the layer (at their respective effective temperature and frequency) to prevent rutting from exceeding the criteria.
[2]Predicted stiffness of the layer at the respective effective temperature and frequency.

These results coincide with the predicted |E*| value being higher than the minimum |E*| value (at the effective temperature and frequency) for the wearing course and lower for the binder course. The effective temperature and frequency for the wearing course and the binder course are not the same for a given structure. These values are calculated by the design guide and depend on the geographical location, the speed of traffic, the pavement structure and the maximum allowable rutting. For example, for structure 1 and mix 1, the effective temperature for the wearing course and binder course was 99.14°F and 90.86°F, respectively and the effective frequency for the wearing course and binder course was 60 Hz and 45 Hz, respectively. Both the minimum and predicted |E*| values for each layer are calculated by the design guide at their respective temperatures and frequencies.

At this point the calibration of the models should be checked to ensure that the predicted rut depth compares well with the measured rut depth. If not, follow the procedures shown in Figure 6.2. In this example, it is assumed that the models are calibrated and therefore the pavement has failed. In fact, a sensitivity study begins to take form when compiling a mixture catalog. Some parameters to explore include the following:

Alternative 1: Investigate the effect of increasing thickness of the binder course layer,

Alternative 2: Investigate a different level for maximum allowable rutting,

For demonstration purposes, the sensitivity of predicted rut depth and minimum allowable |E*| to Alternatives 1 and 2 were further explored.

6.4.1.1. Sensitivity Analysis Alternative 1

A new structure (designated Structure 2) was investigated in the MEPDG and consisted of a 2 inch wearing course and a 3 inch binder course (shown in Figure 6.5).

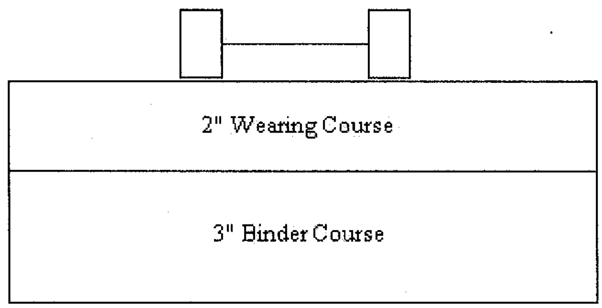

Figure 6.5: Structure 2: Diagram of Alternative 1 Pavement Structure (HMA lifts only).

Increasing the thickness of the binder course by 1 inch resulted in bringing the predicted rut depth within the allowable rut depth, and the minimum |E*| value below the predicted |E*| at their respective effective temperature and frequency (Table 6.3).

6.4.1.2. Sensitivity Analysis Alternative 2

Following the approach in Alternative 2, the maximum allowable rut depth criterion was changed for Structure 1 from 0.3 inches to 0.5 inches. Each state agency has different criteria for what they consider reasonable levels of distress that still provide a safe and smooth ride. Although one state may consider 0.3 inches the absolute level of rutting that they will accept before they perform maintenance or rehabilitation, another state in the same New England region might be comfortable with 0.5 inches as the limit. In the analysis, the predicted rut depths for the four mixes remained the same; however, increasing the allowable rut depth lead to a decrease in the minimum |E*| values. Therefore, the predicted rut depth are within allowable limits and |E*| values (at their respective effective temperature and frequency) of the HMA layer are above the minimum predicted |E*| value (Table 6.3).

From the data provided in Table 6.3, it is suggested that small changes in different gradations have little impact on the predicted values of the MEPDG. The sensitivity study proved the impact of changing the pavement structure and varying distress limits. However, increasing the thickness of a pavement layer may not always be an option. Therefore, it is important for state agencies to develop a catalog of |E*| values for routinely used mixes to streamline the design process and make comparisons such as lab measured |E*| versus minimum |E*| possible.

6.5. Summary

In this chapter a conceptual framework was presented for determining what input level is needed for the MEPDG and for developing a HMA mixture catalog. A sample application was also presented to demonstrate implementation of the framework. This next chapter presents the summary and conclusions of this study.

CHAPTER 7
Summary and Conclusions

7.1. Summary

The current rutting model in the MEPDG utilizes DM as an input parameter, which can be an inconsistent indicator of a specimen's resistance to strain accumulation. This has lead to investigating the feasibility of incorporating RLT accumulated strain data into the MEPDG in an effort to more accurately predict rutting damage.

In this study a new model was developed to accurately represent the accumulated strain data obtained from the RLT. Then, the feasibility of incorporating RLT data into the MEPDG using the existing rutting model and the newly developed model was investigated. It was found that the current rutting model obtains a crude fit and does not represent the accumulated strain profile properly. It also requires complete RLT data up to the Flow Number to obtain the proper calibration factors. The model developed in this study obtains a good fit and demonstrates the proper behavior in the primary and secondary regions. In addition, it only requires a portion of the secondary region to determine the proper calibration factors. Therefore, it is recommended that Alternative 2 (Section 5.4.2) be considered for implementation in the MEPDG.

A framework was also presented that will potentially help to reduce the amount of effort and funding required to obtain asphalt inputs for mechanistic-empirical pavement design through establishment of an asphalt materials and mixture catalog. This concept is particularly applicable for the local or regional calibration portion of implementation of mechanistic-empirical pavement design; however, an understanding of local materials limitations is critical to the success of using this approach.

7.2. Conclusion

The model developed in this study can be used to accurately represent the accumulated strain profile obtained from RLT. RLT also has the potential to predict rutting damage once calibrated and to be incorporated into the MEPDG.

7.3. Expected Significance

The parameters of the model developed in this study can be determined from accumulated strain data in the primary region and only a portion of the secondary region. This could possibly reduce testing times for state agencies. Also, once calibrated this model will be able to calculate the expected field rut depth profile for a given period of time or ESALs. In addition, all the calibration procedures for this model can be performed by a computer program using the RLT data as an input.

The proposed model can also be implemented in conjunction with the conceptual framework for materials characterization in mechanistic empirical flexible pavement design presented in Chapter 6. The framework was developed to aid state agencies in developing a catalog of data for routinely used HMA mixtures to be used with the MEPDG. The mix calibration factors discussed in Section (5.4.2) can be determined for routinely used mixes that show consistent performance during the RLT test. Then, the regional calibration factors can be determined for the regions discussed in Section (6.3.2). Using the proposed model in conjunction with the conceptual framework will aid state agencies in categorizing their mixtures based on mixture behavior and regional behavior allowing them to streamline the flexible pavement design process.

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APPENDIX A

Plots of the Changes in Slope

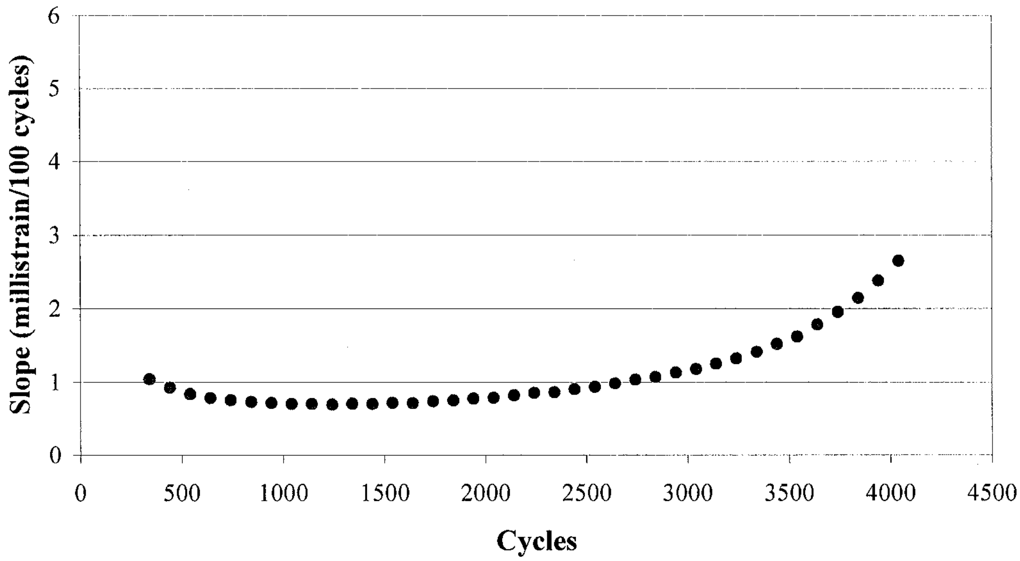

Figure A.1: Change in slope of accumulated strain profile (NC-1).

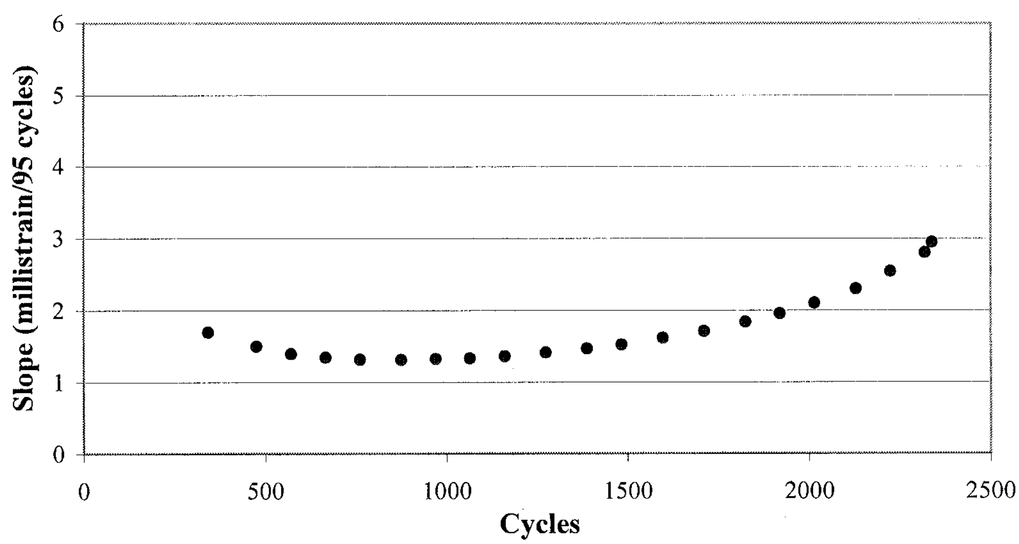

Figure A.2: Change in slope of accumulated strain profile (NC-2).

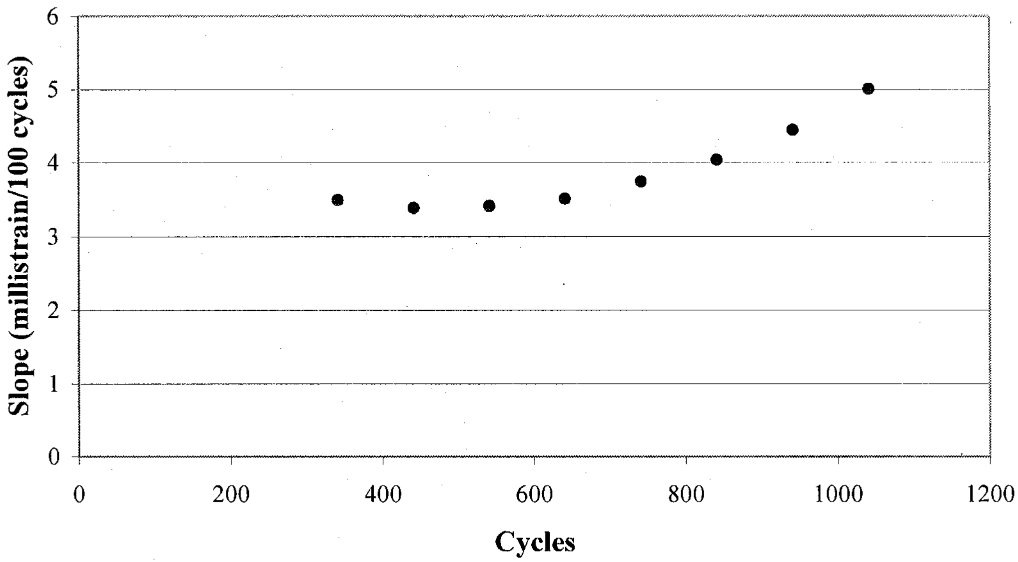

Figure A.3: Change in slope of accumulated strain profile (NC-3).

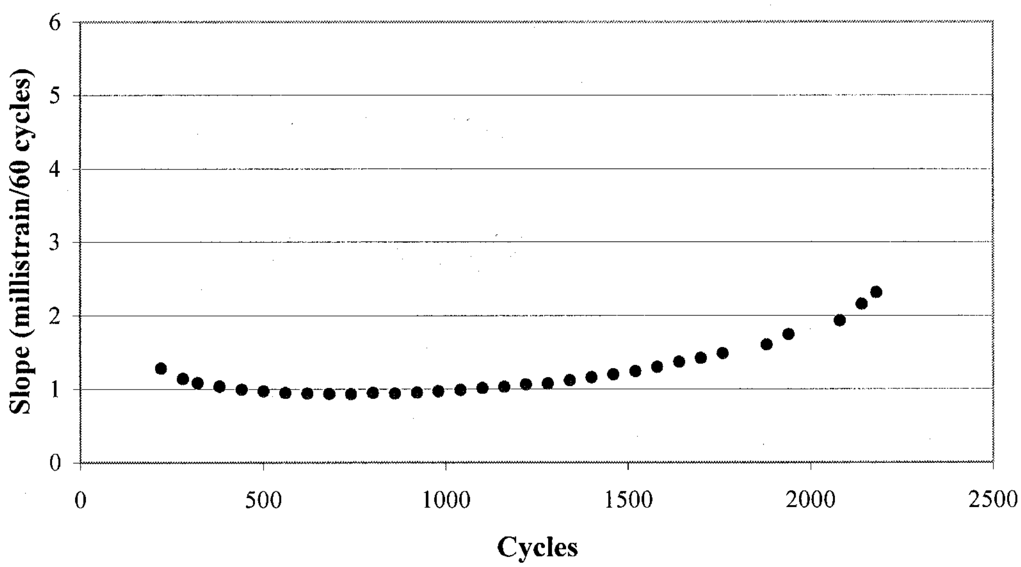

Figure A.4: Change in slope of accumulated strain profile (NC-4).

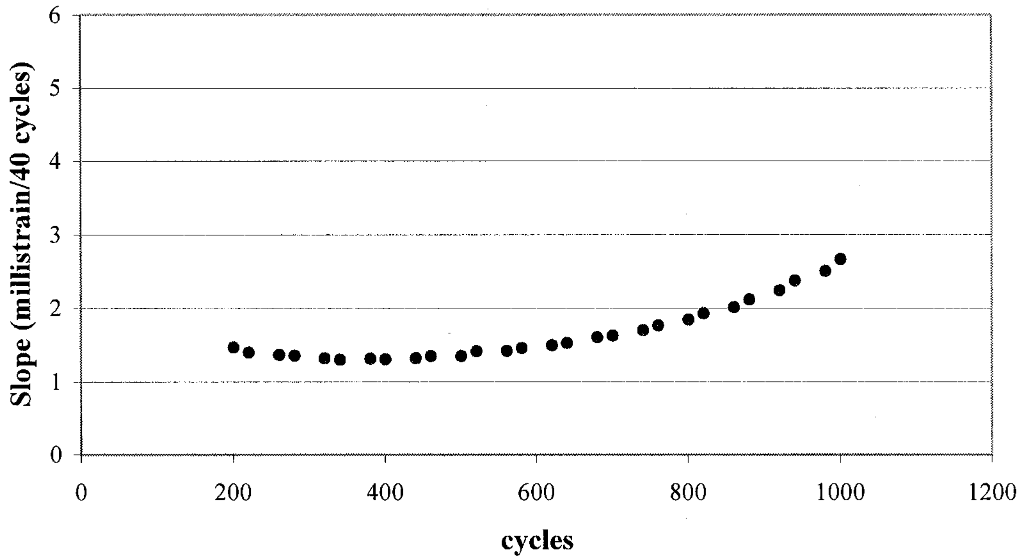

Figure A.5: Change in slope of accumulated strain profile (NC-5).

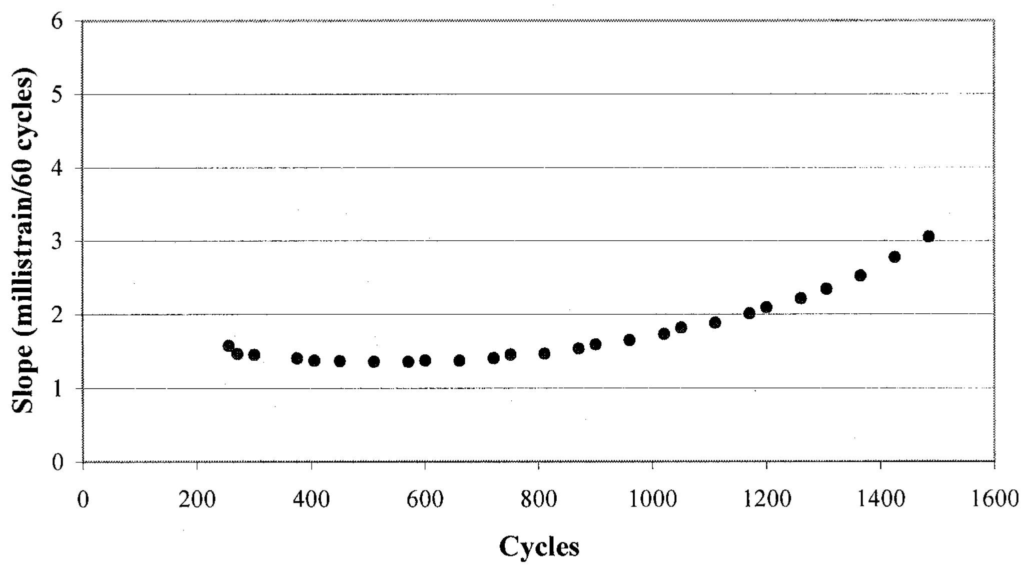

Figure A.6: Change in slope of accumulated strain profile (NC-6).

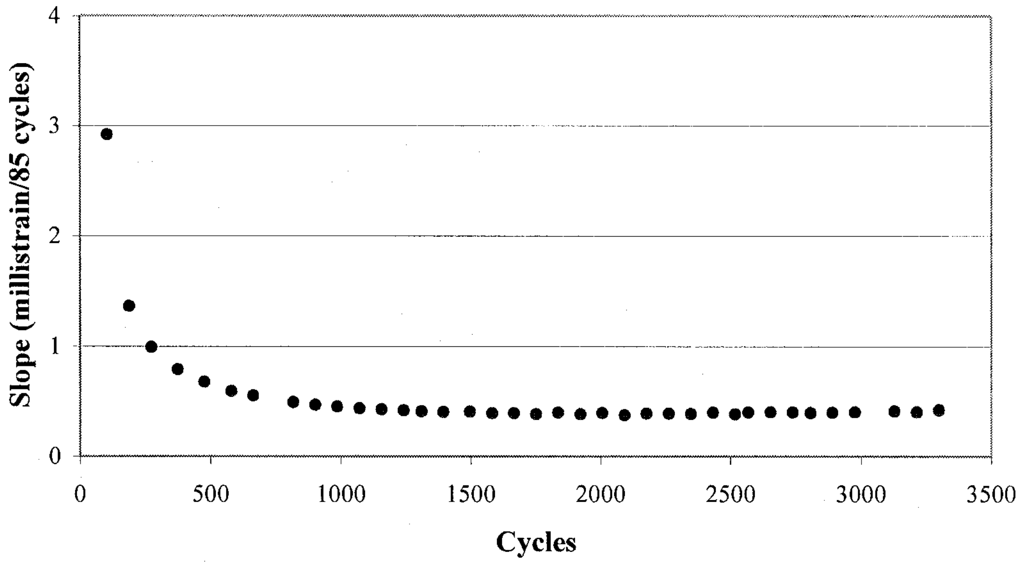

Figure A.7: Change in slope of accumulated strain profile (AZ-1).

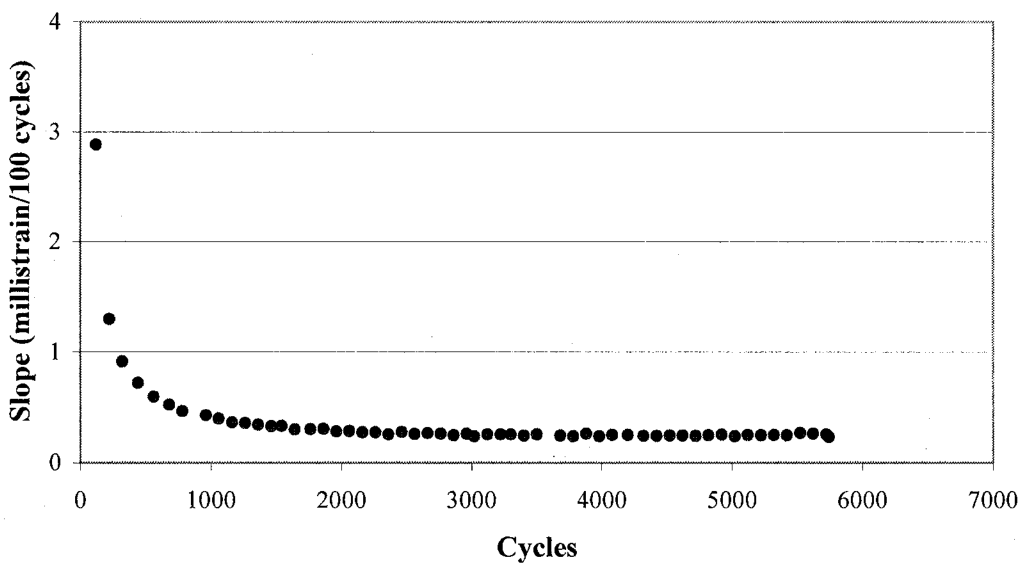

Figure A.8: Change in slope of accumulated strain profile (AZ-2).

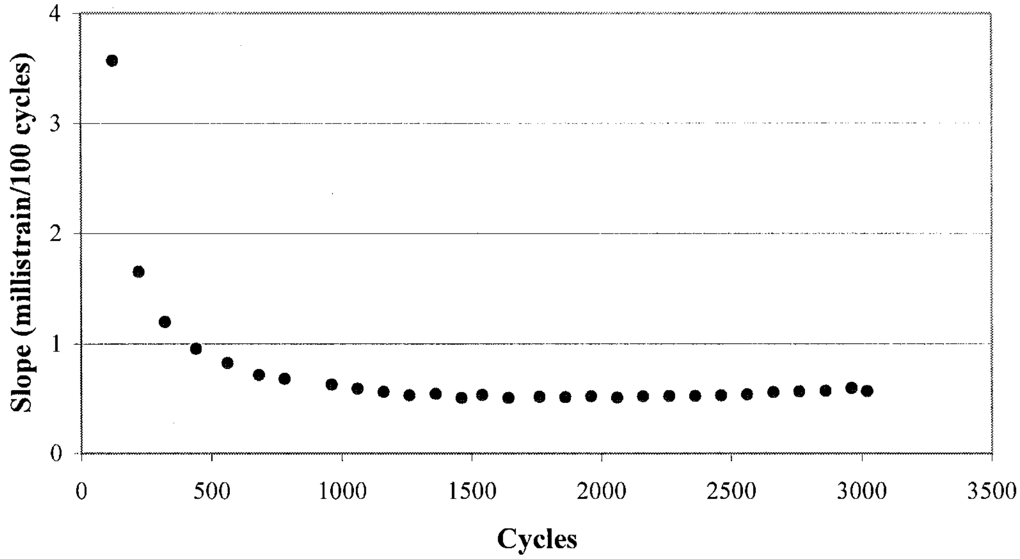

Figure A.9: Change in slope of accumulated strain profile (AZ-3).

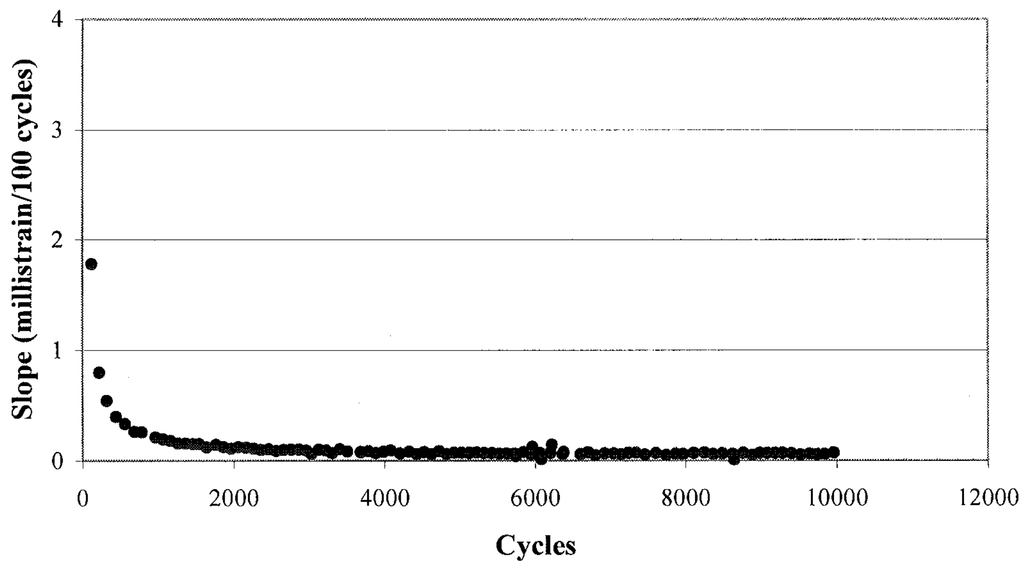

Figure A.10: Change in slope of accumulated strain profile (AZ-4).

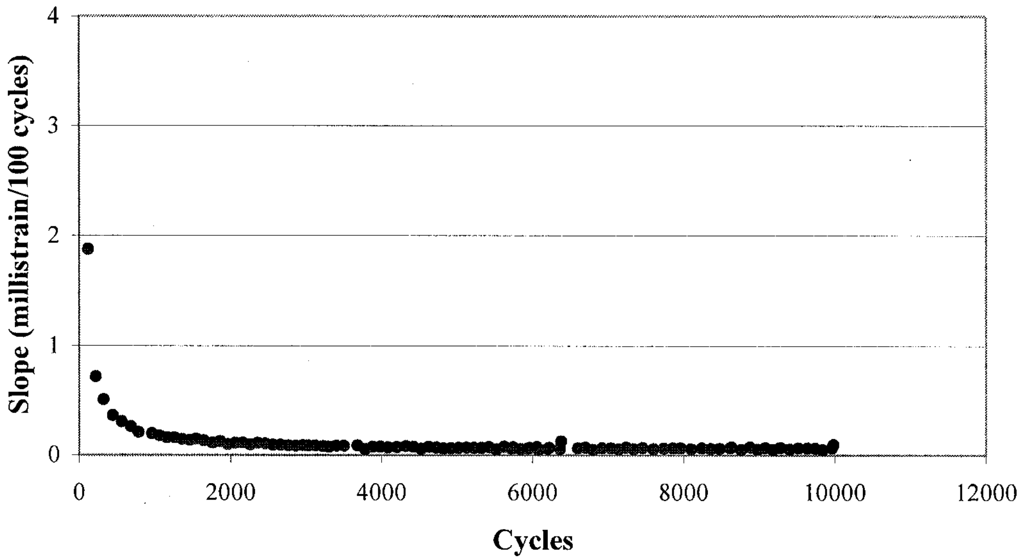

Figure A.11: Change in slope of accumulated strain profile (AZ-5).

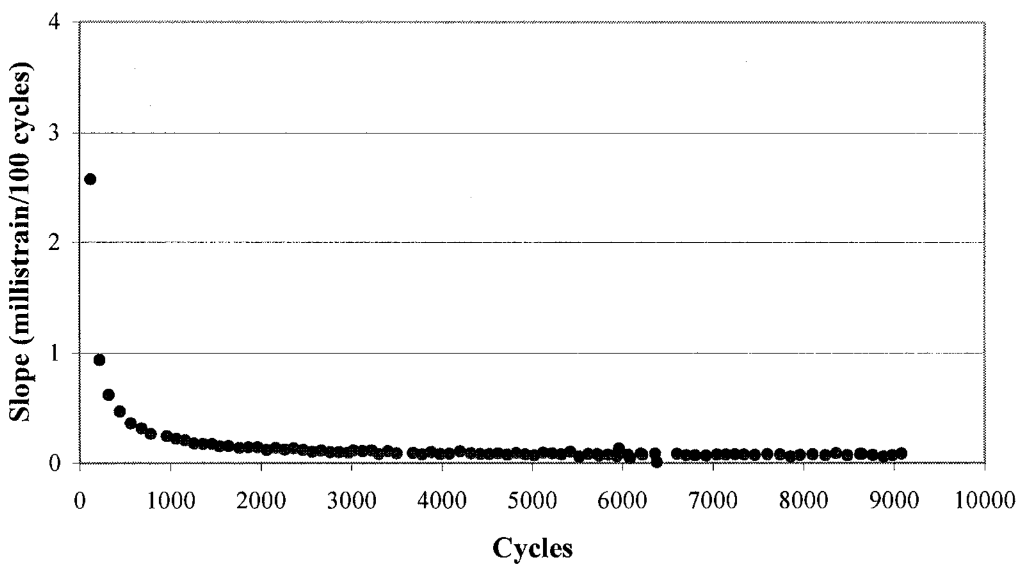

Figure A.12: Change in slope of accumulated strain profile (AZ-6).

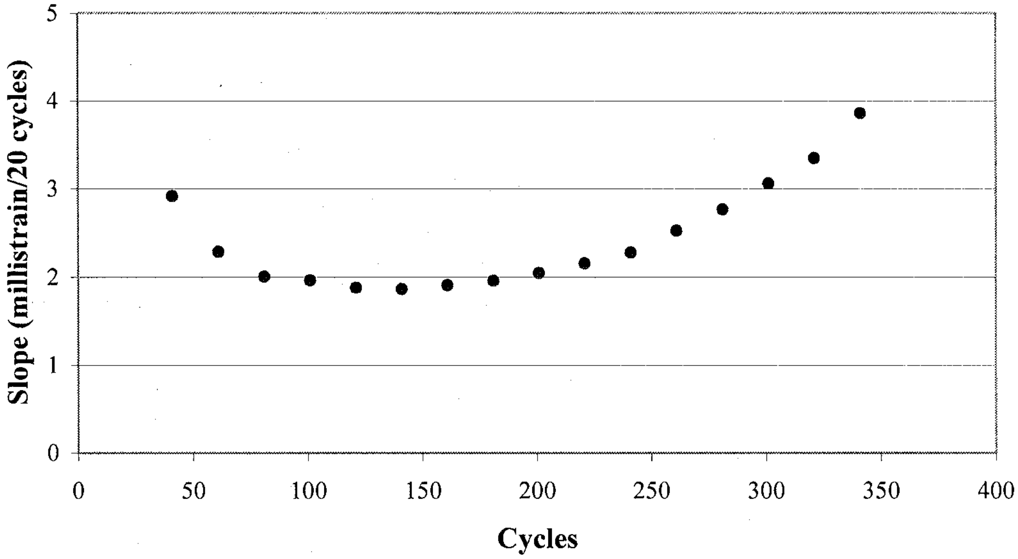

Figure A.13: Change in slope of accumulated strain profile (LA-1).

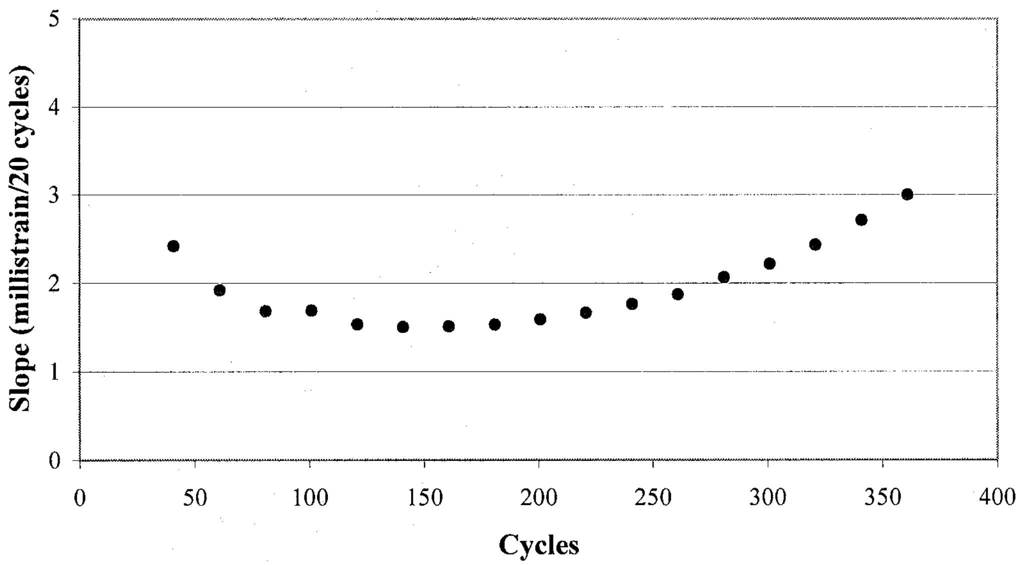

Figure A.14: Change in slope of accumulated strain profile (LA-2).

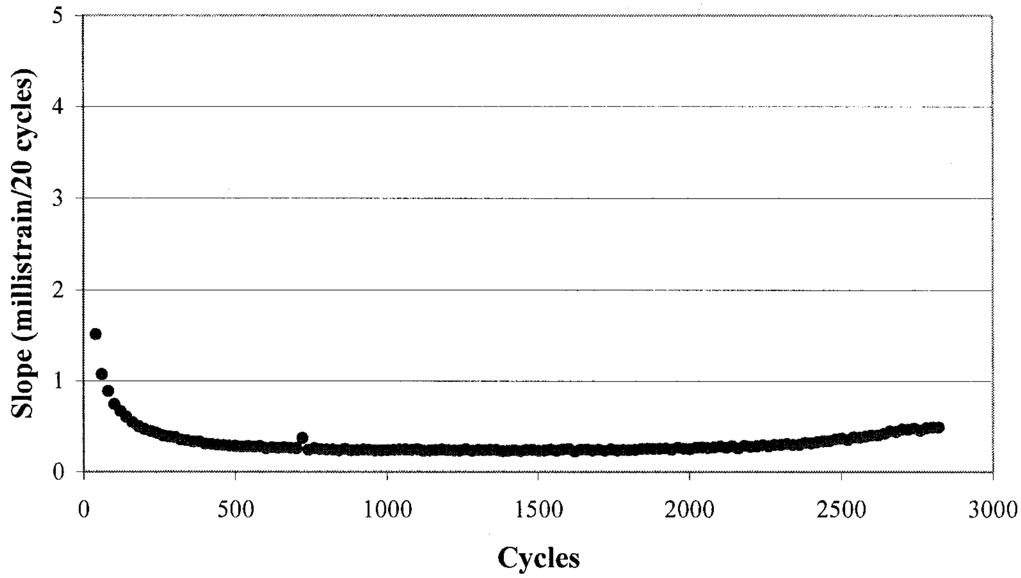

Figure A.15: Change in slope of accumulated strain profile (LA-3).

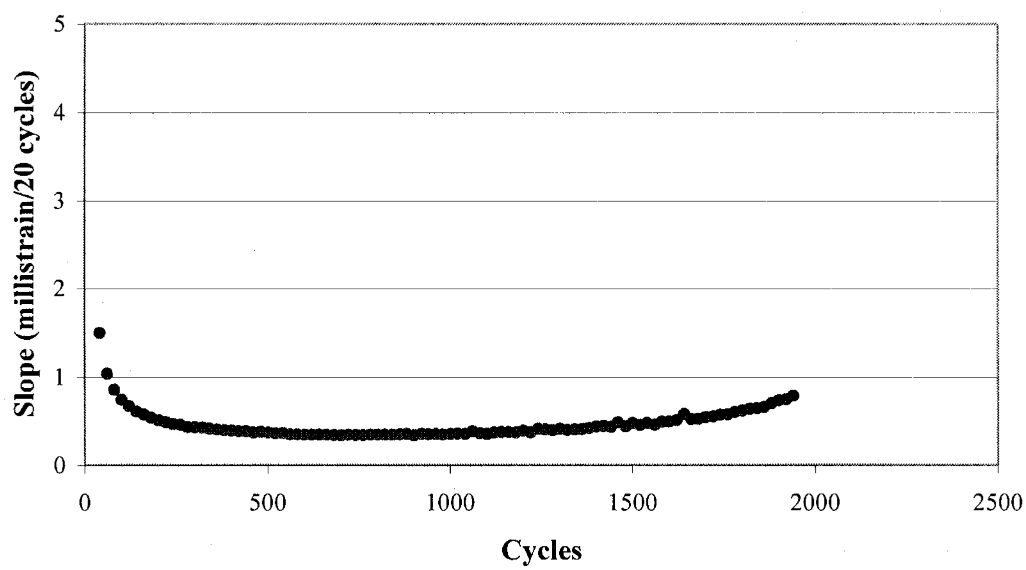

Figure A.16: Change in slope of accumulated strain profile (LA-4).

APPENDIX B

Plots of the Rate of Change in Slope

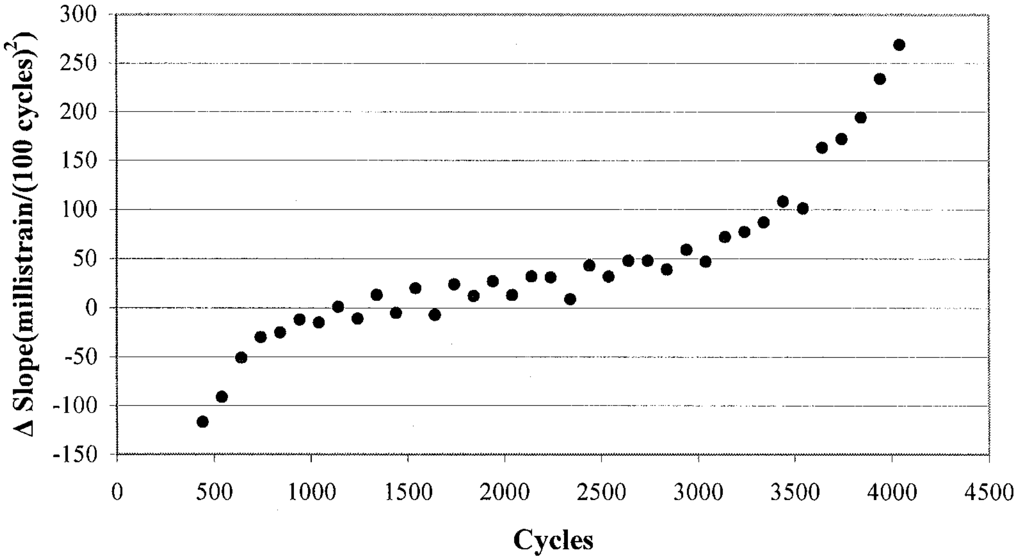

Figure B.1: Rate of change in slope of accumulated strain profile (NC-1).

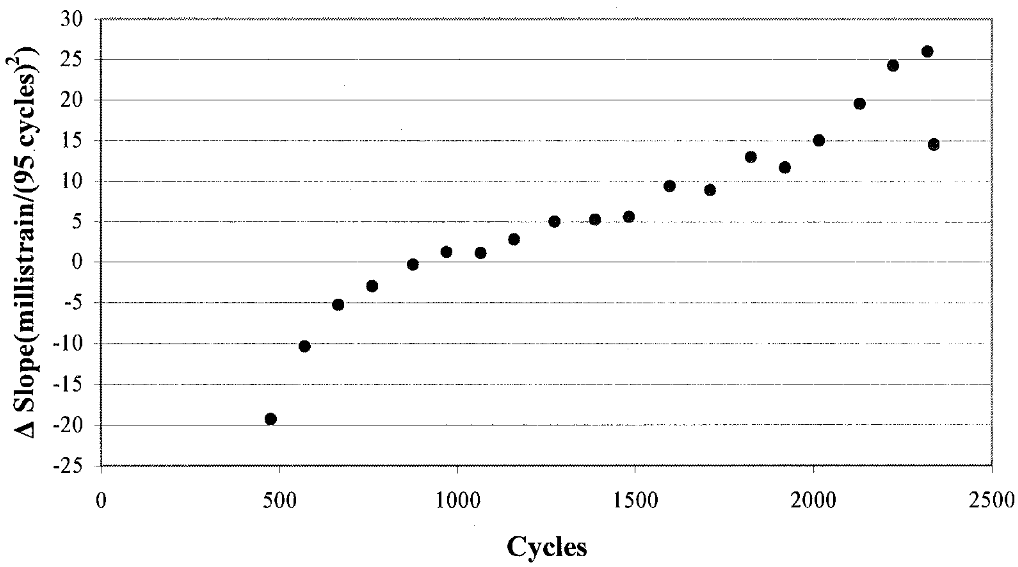

Figure B.2: Rate of change in slope of accumulated strain profile (NC-2).

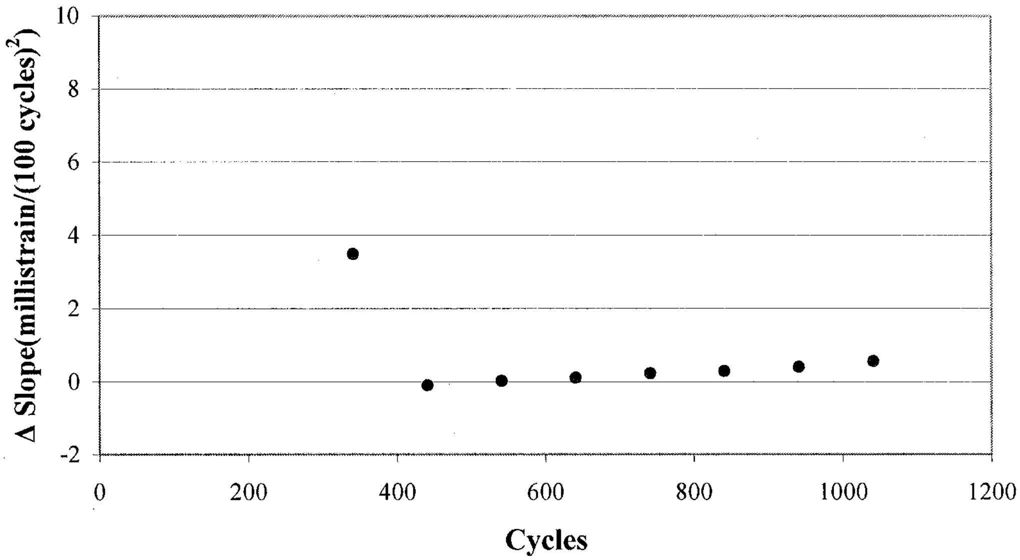

Figure B.3: Rate of change in slope of accumulated strain profile (NC-3).

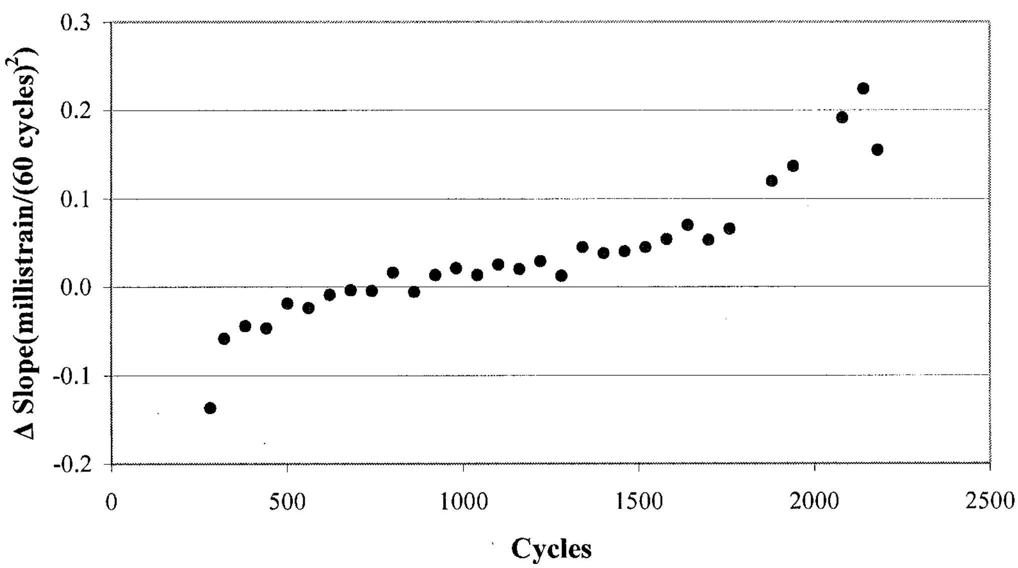

Figure B.4: Rate of change in slope of accumulated strain profile (NC-4).

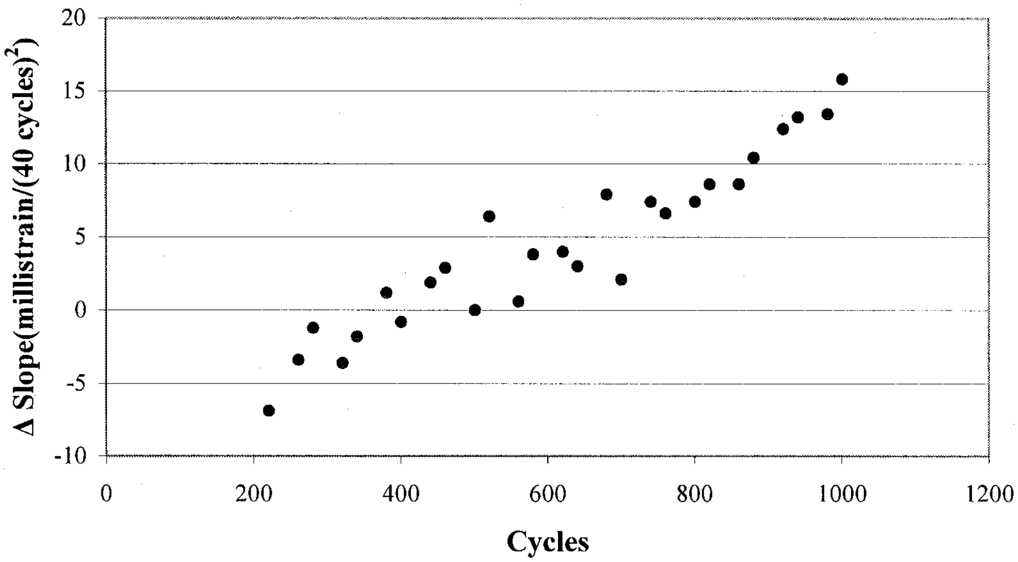

Figure B.5: Rate of change in slope of accumulated strain profile (NC-5).

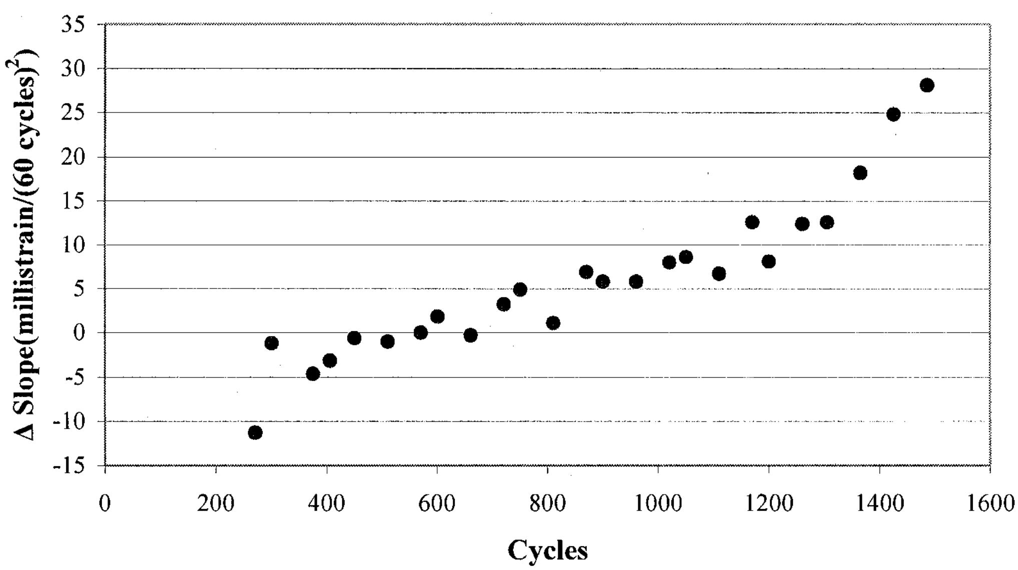

Figure B.6: Rate of change in slope of accumulated strain profile (NC-6).

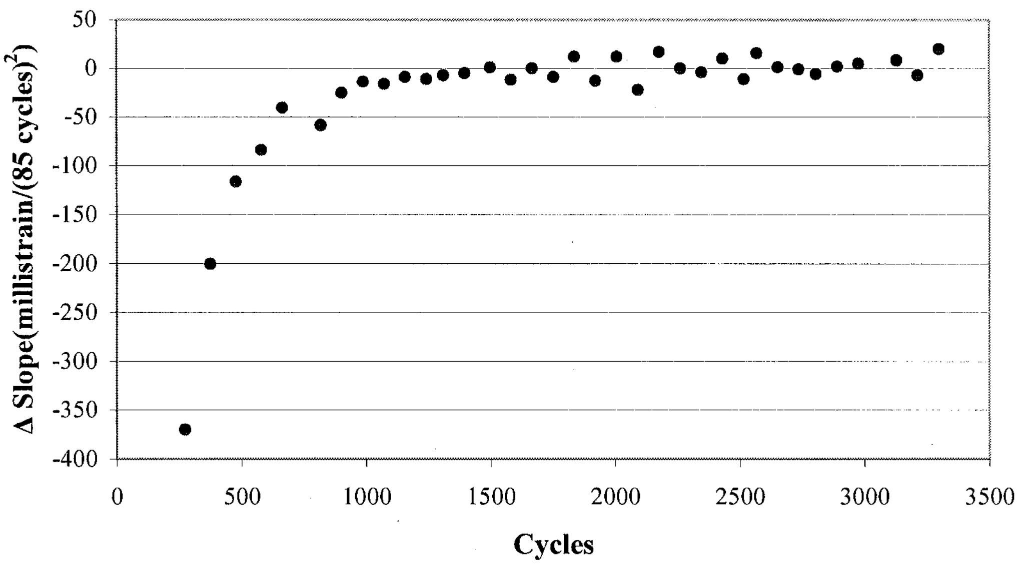

Figure B.7: Rate of change in slope of accumulated strain profile (AZ-1).

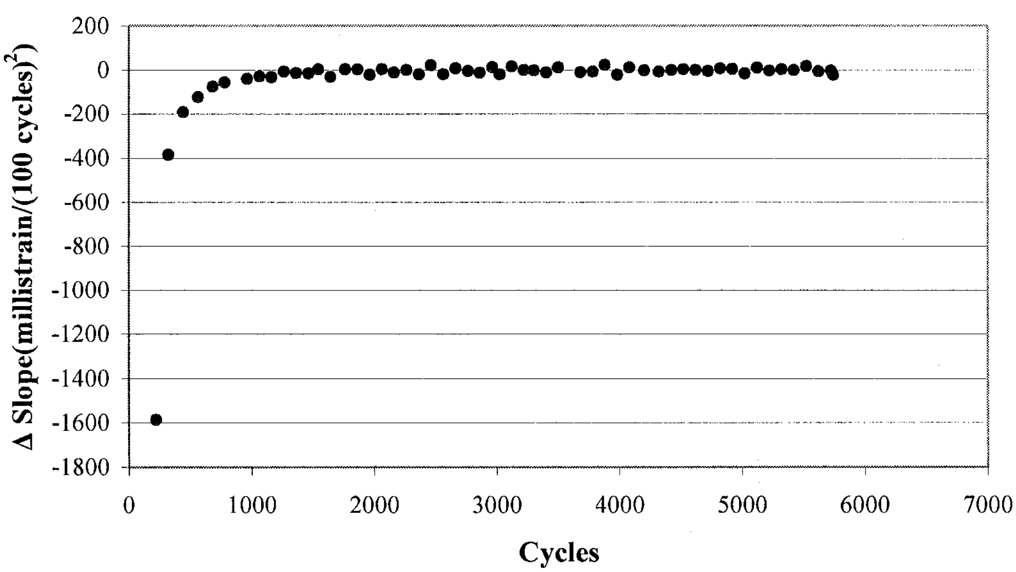

Figure B.8: Rate of change in slope of accumulated strain profile (AZ-2).

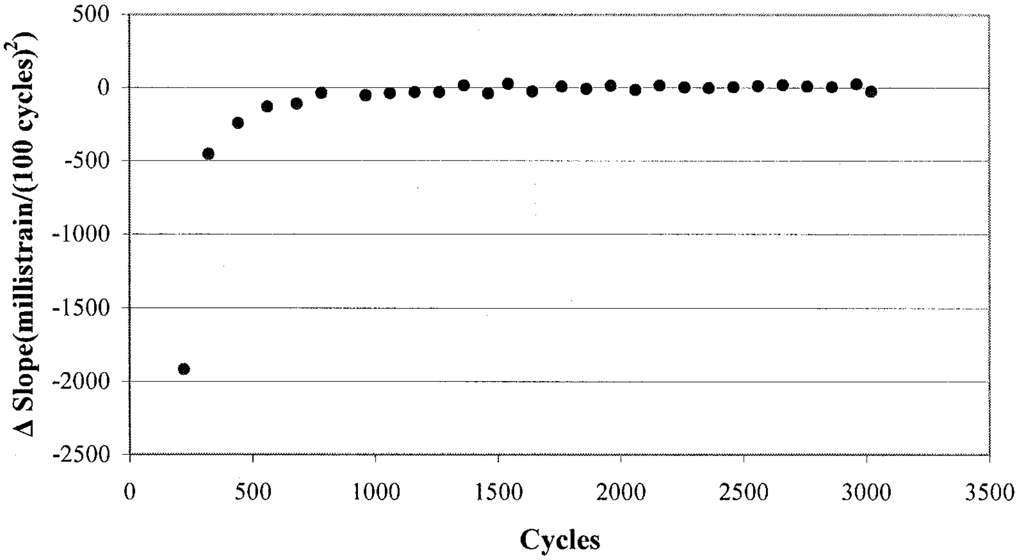

Figure B.9: Rate of change in slope of accumulated strain profile (AZ-3).

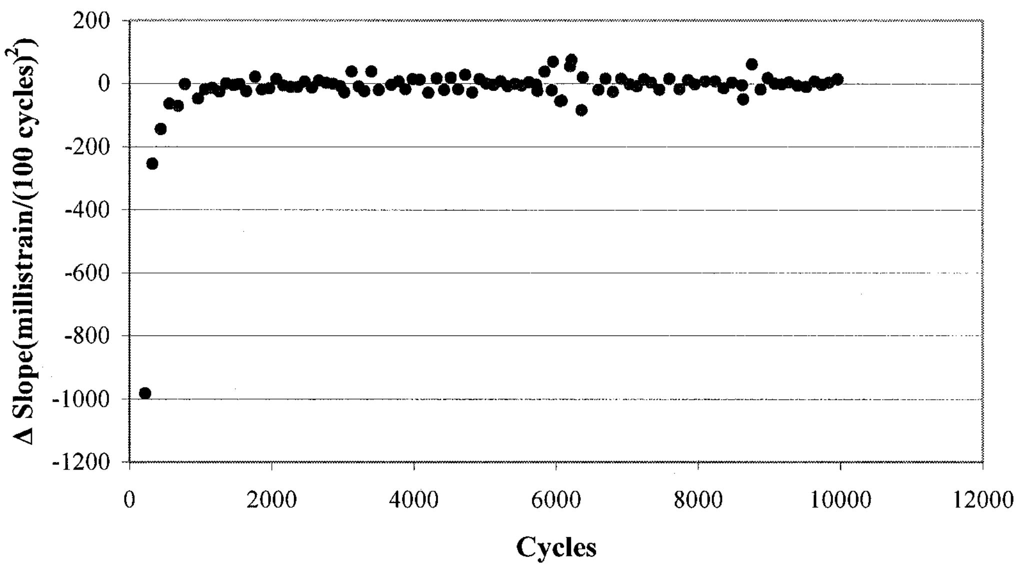

Figure B.10: Rate of change in slope of accumulated strain profile (AZ-4).

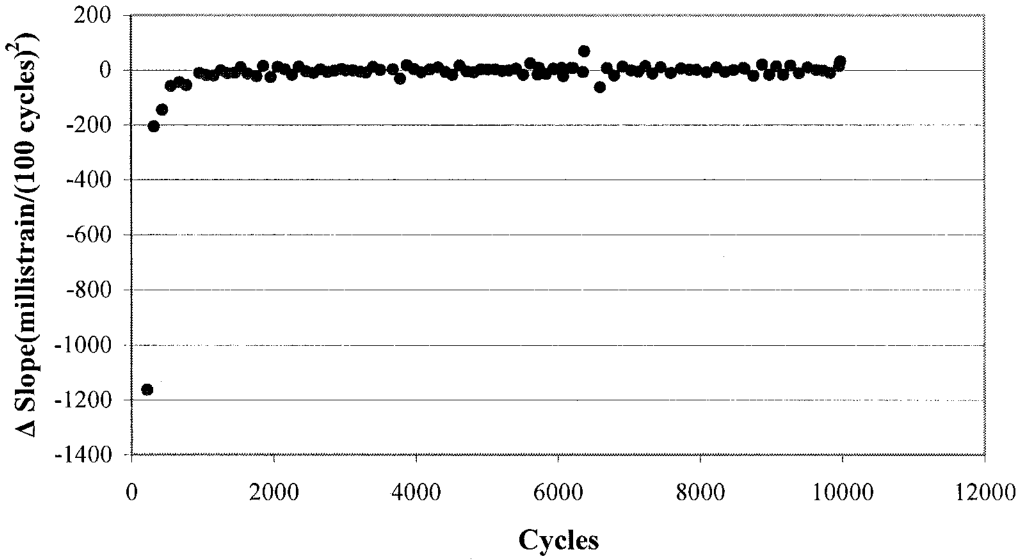

Figure B.11: Rate of change in slope of accumulated strain profile (AZ-5).

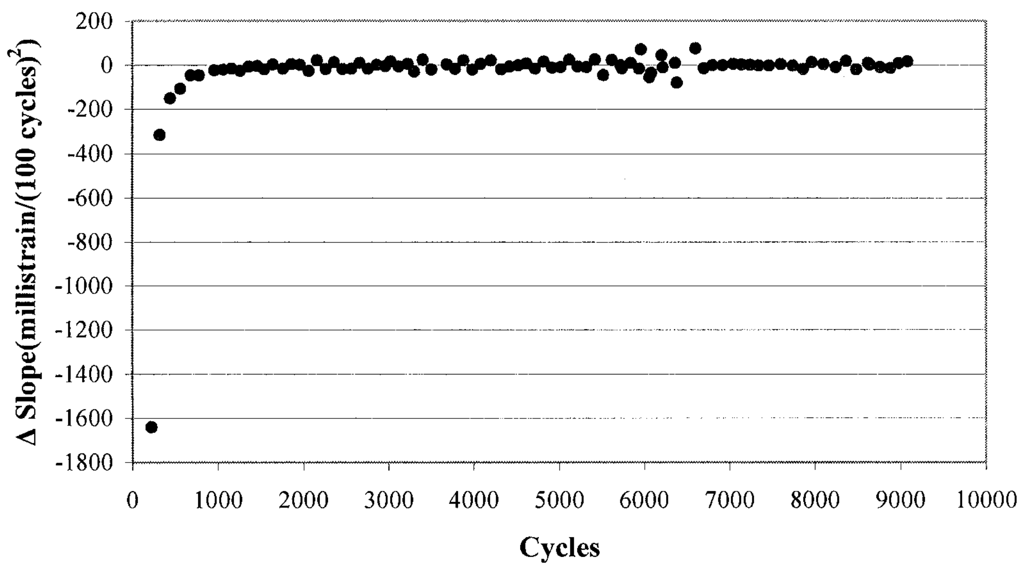

Figure B.12: Rate of change in slope of accumulated strain profile (AZ-6).

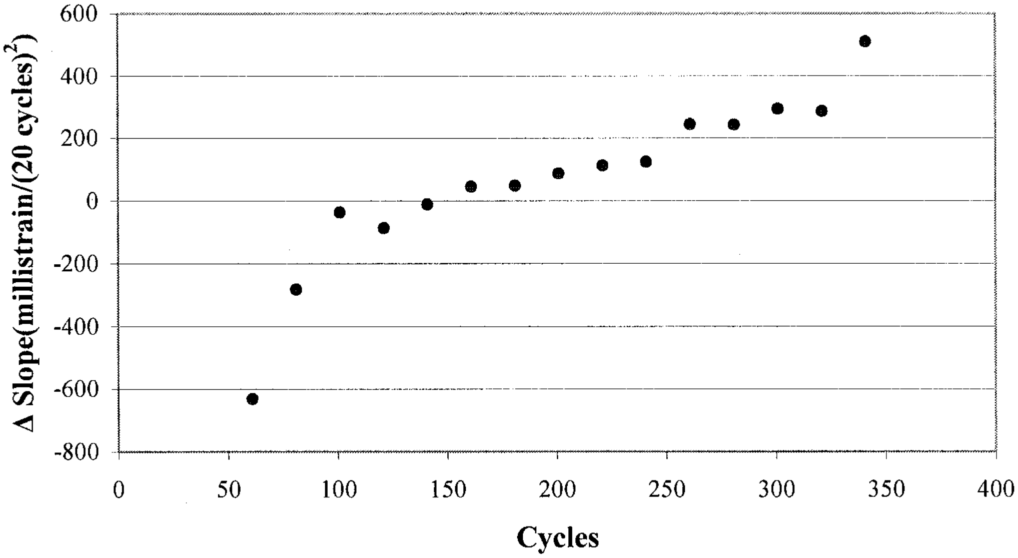

Figure B.13: Rate of change in slope of accumulated strain profile (LA-1).

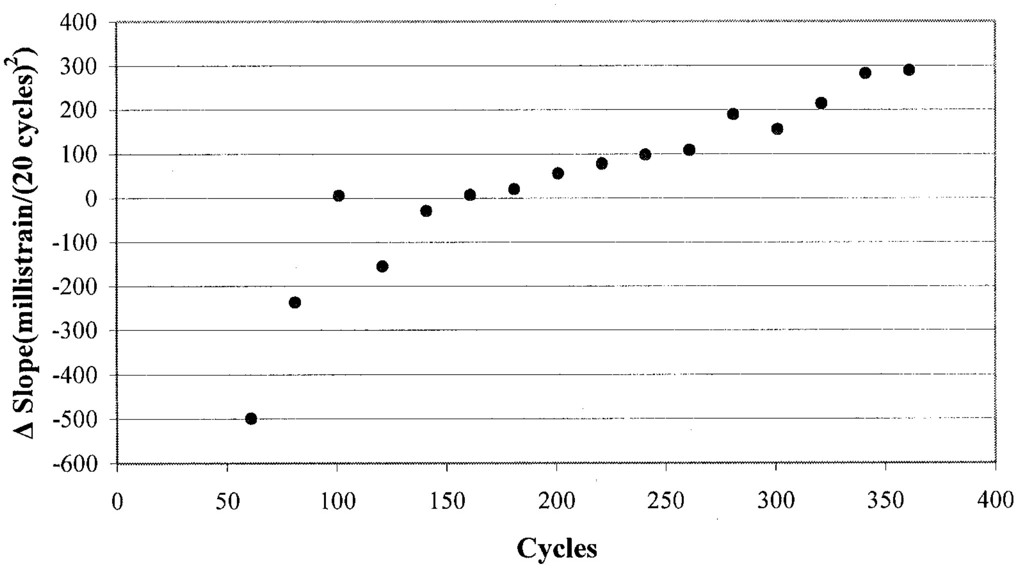

Figure B.14: Rate of change in slope of accumulated strain profile (LA-2).

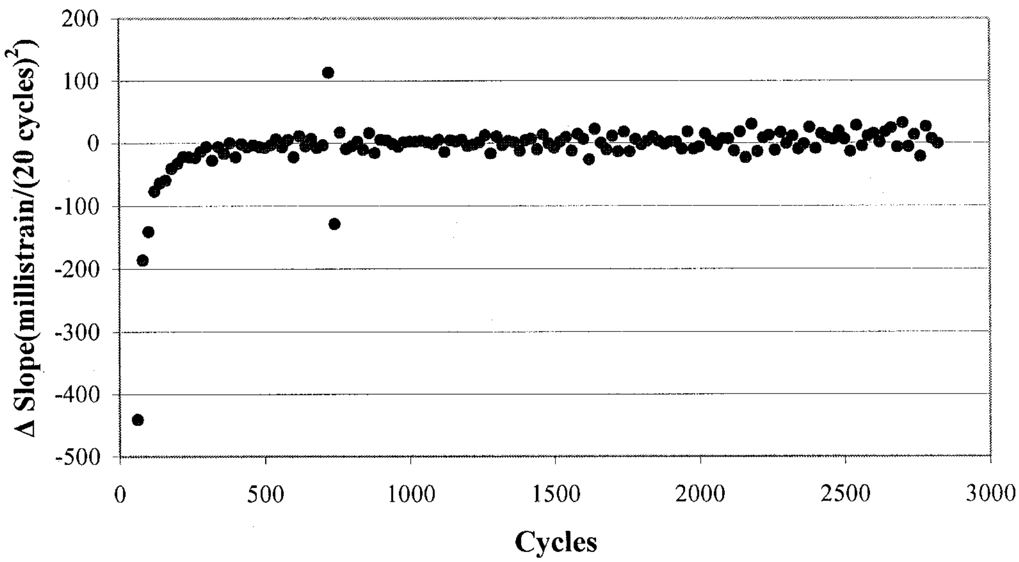

Figure B.15: Rate of change in slope of accumulated strain profile (LA-3).

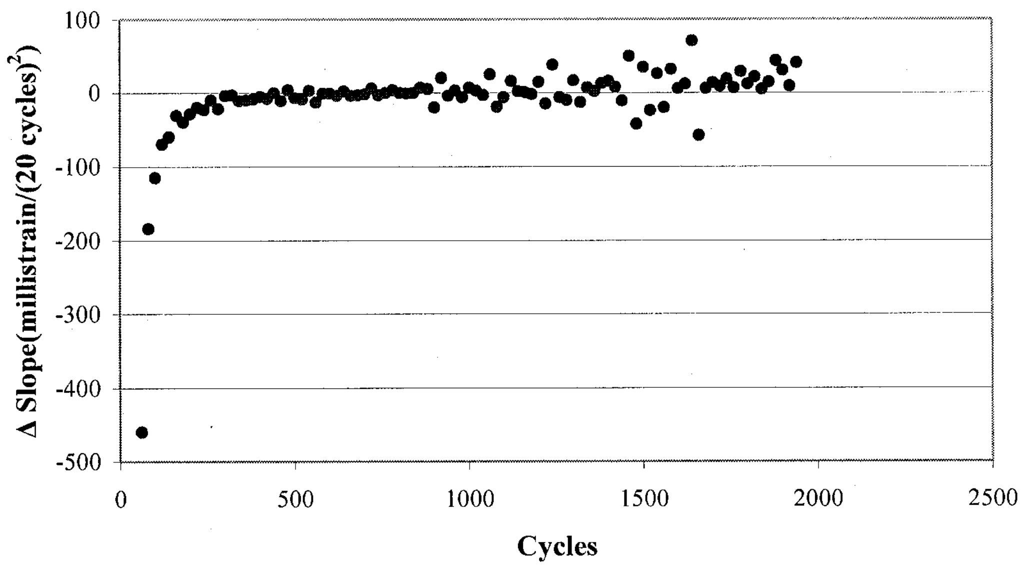

Figure B.16: Rate of change in slope of accumulated strain profile (LA-4).

APPENDIX C

Parameter Values

Table C.1: Proposed model parameter values and cycle range of the data sets used in determining the parameters.

Mix ID	b	k_1	D_1	D_2	λ	Point ending Primary Region	Point ending Secondary Region
NC-1	5.10	0.84	0.0073	5.10	-0.0090	941	1241
NC-2	9.49	0.75	0.0141	9.49	-0.0109	666	875
NC-3	8.65	0.63	0.0351	8.65	-0.0217	441	541
NC-4	5.86	0.74	0.0159	5.86	-0.0130	621	741
NC-5	8.92	0.62	0.0337	8.92	-0.0259	261	341
NC-6	5.83	0.69	0.0232	5.83	-0.0186	301	571
AZ-1	8.51	0.90	0.0047	5.65	-0.0048	1157	2092
AZ-2	10.72	1.00	0.0025	6.66	-0.0035	2161	3781
AZ-3	11.38	0.86	0.0053	6.70	-0.0053	781	2061
AZ-4	7.51	1.16	0.0007	4.78	-0.0015	2961	7461
AZ-5	8.41	1.18	0.0007	4.53	-0.0014	2661	9181
AZ-6	10.92	1.14	0.0008	5.68	-0.0017	2561	6801
LA-1	8.11	0.46	0.0962	8.11	-0.0769	101	141
LA-2	5.64	0.55	0.0778	5.64	-0.0617	101	141
LA-3	10.63	0.81	0.0124	7.13	-0.0075	901	1621
LA-4	8.41	0.79	0.0186	5.31	-0.0116	301	701

Table C.2: Typical output of nonlinear regression from SPSS

Nonlinear Regression Summary Statistics   Dependent Variable STRAIN

Source	DF	Sum of Squares	Mean Square
Regression	4	36952.63223	9238.15806
Residual	77	.95984	.01247
Uncorrected Total	81	36953.59208	
(Corrected Total)	80	3573.93668	

R squared = 1 - Residual SS / Corrected SS =   .99973

Parameter	Estimate	Asymptotic Std. Error	Asymptotic 95 % Confidence Interval Lower	Asymptotic 95 % Confidence Interval Upper
D2	7.128554772	.102839706	6.923774757	7.333334788
L	-.007508935	.000140799	-.007789301	-.007228568

APPENDIX D

Measured and Predicted Strain Profiles

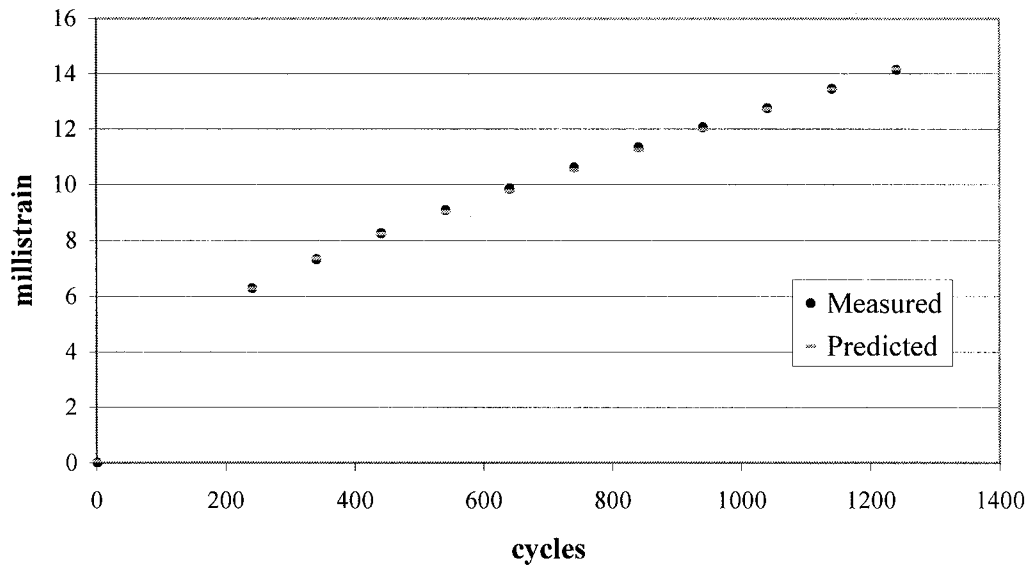

Figure D.1: Measured and predicted (proposed model) accumulated strain profiles for NC-1.

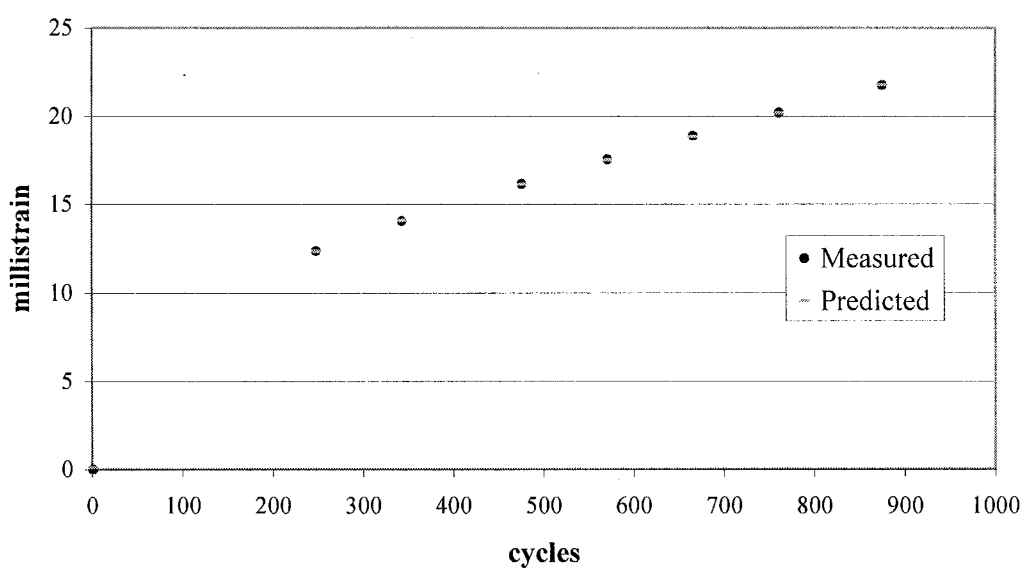

Figure D.2: Measured and predicted (proposed model) accumulated strain profiles for NC-2.

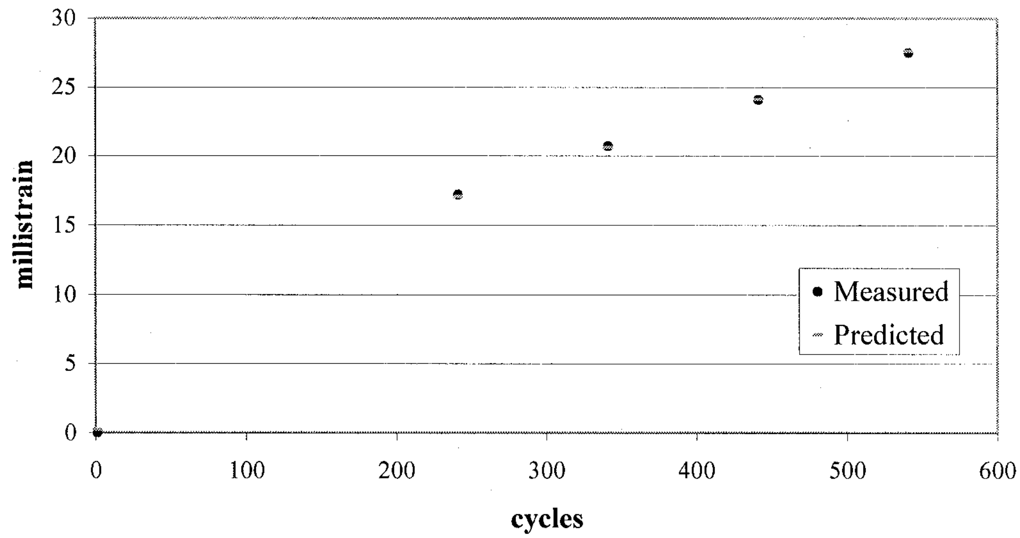

Figure D.3: Measured and predicted (proposed model) accumulated strain profiles for NC-3.

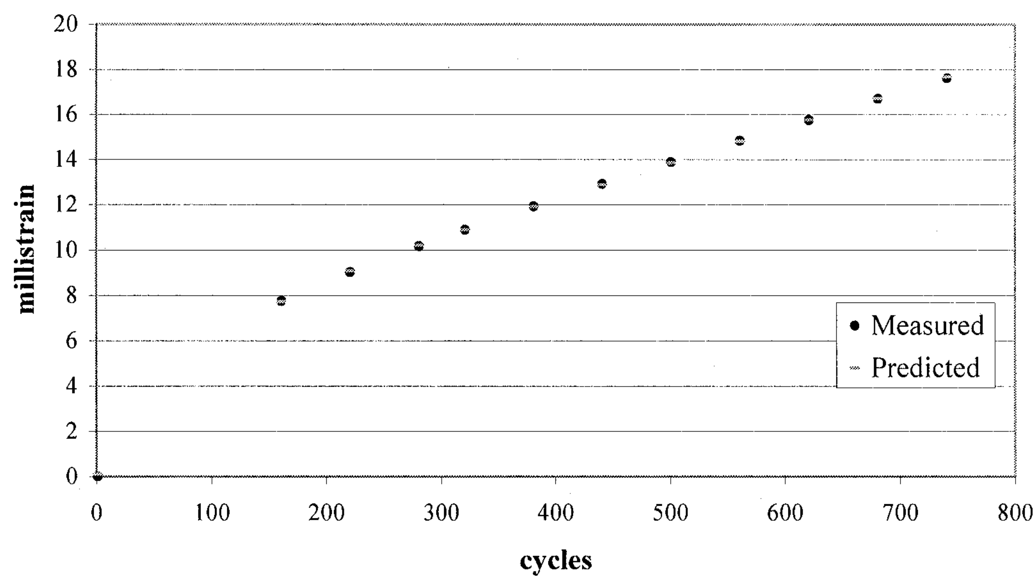

Figure D.4: Measured and predicted (proposed model) accumulated strain profiles for NC-4.

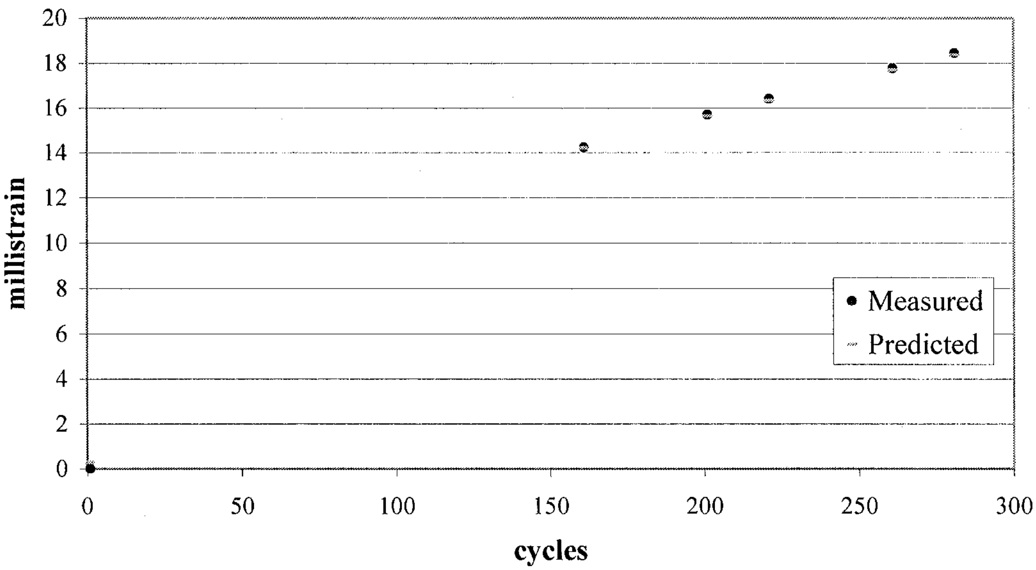

Figure D.5: Measured and predicted (proposed model) accumulated strain profiles for NC-5.

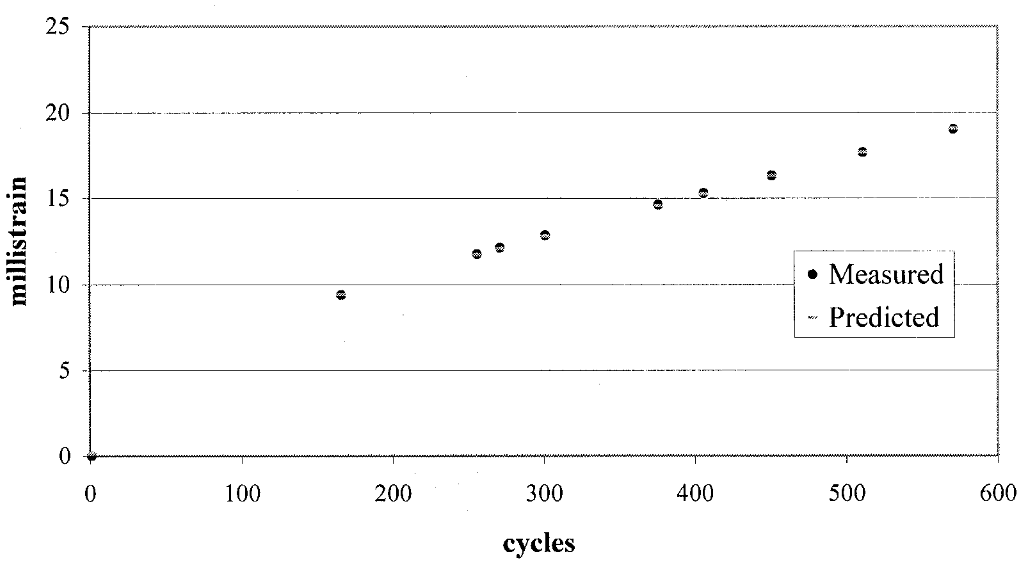

Figure D.6: Measured and predicted (proposed model) accumulated strain profiles for NC-6.

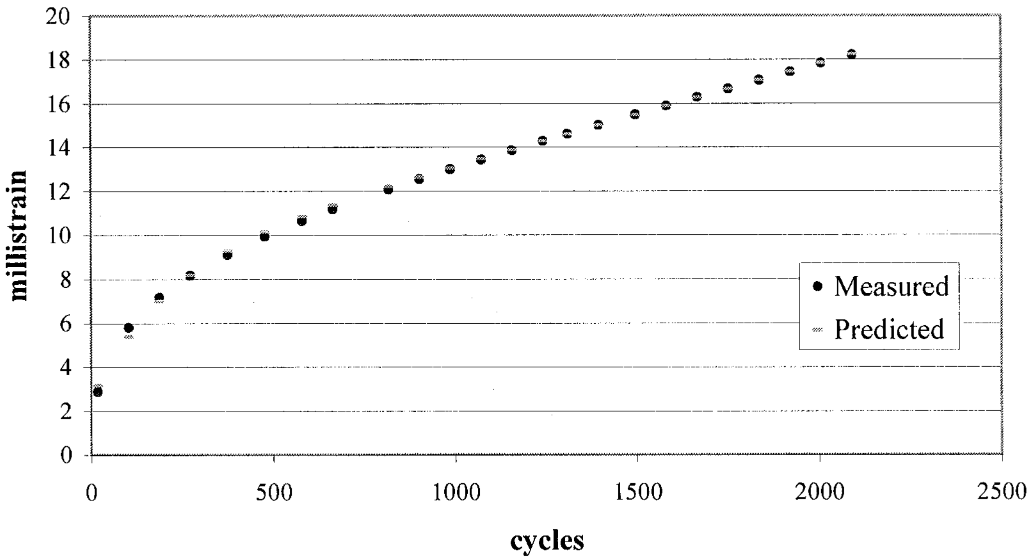

Figure D.7: Measured and predicted (proposed model) accumulated strain profiles for AZ-1.

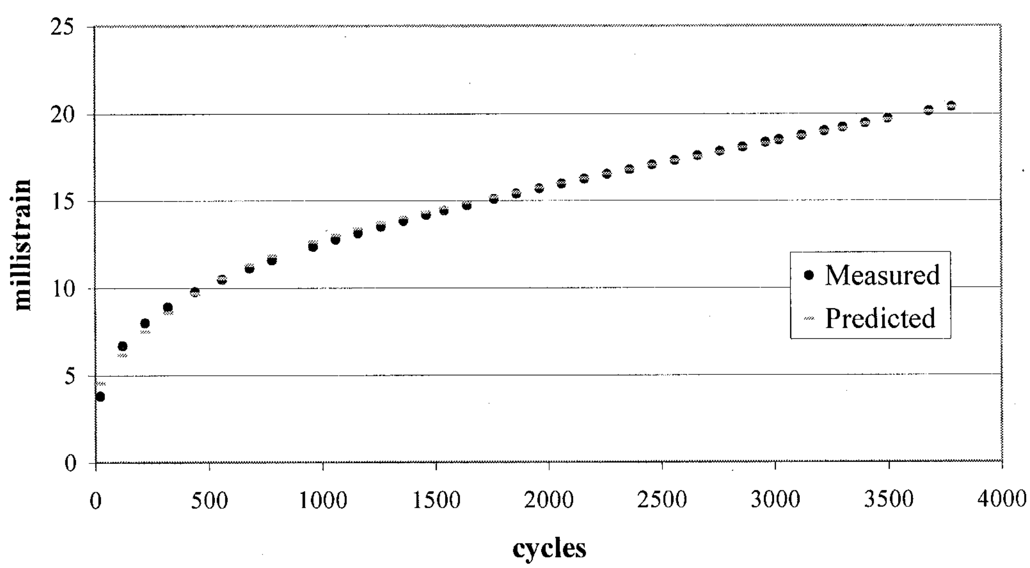

Figure D.8: Measured and predicted (proposed model) accumulated strain profiles for AZ-2.

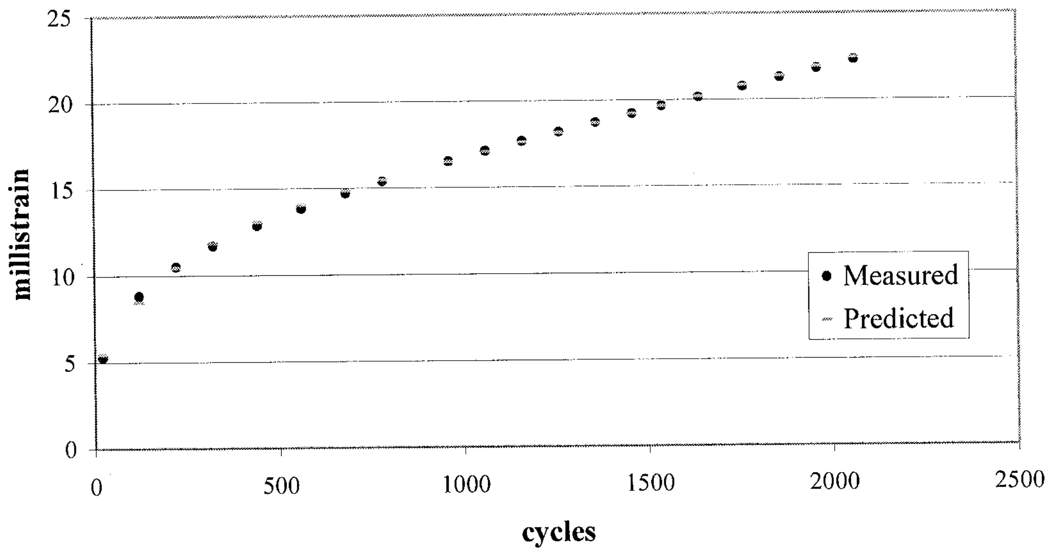

Figure D.9: Measured and predicted (proposed model) accumulated strain profiles for AZ-3.

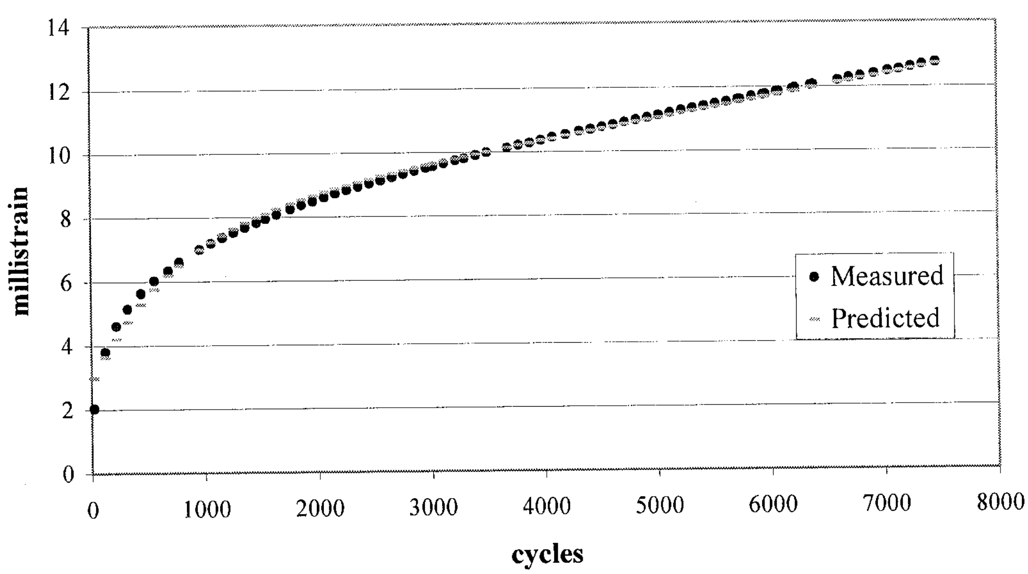

Figure D.10: Measured and predicted (proposed model) accumulated strain profiles for AZ-4.

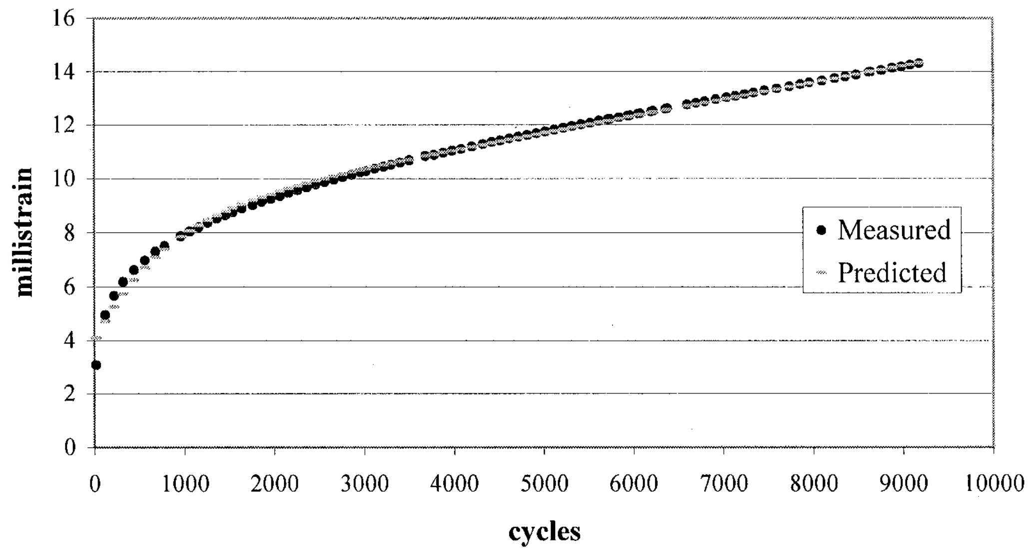

Figure D.11: Measured and predicted (proposed model) accumulated strain profiles for AZ-5.

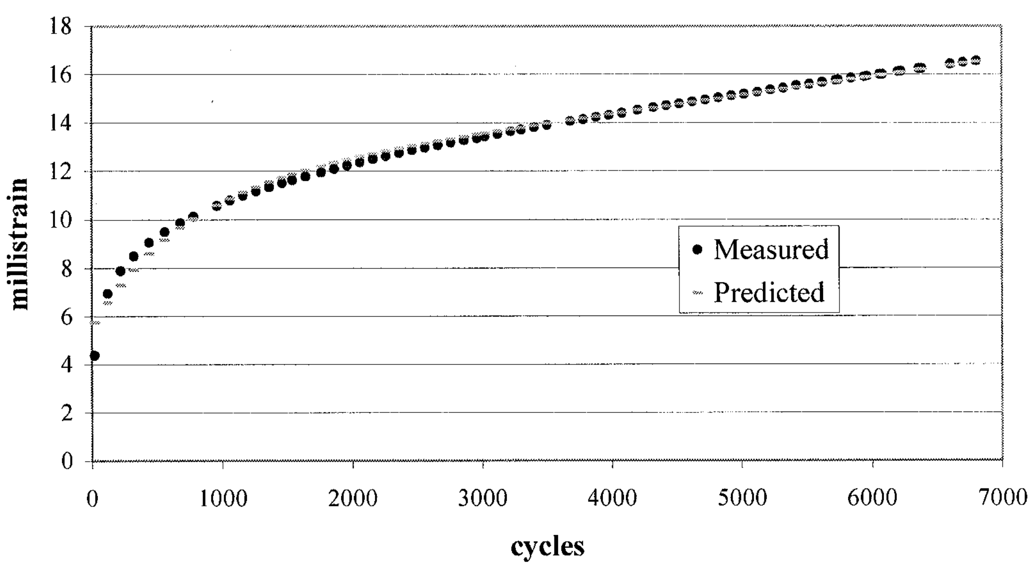

Figure D.12: Measured and predicted (proposed model) accumulated strain profiles for AZ-6.

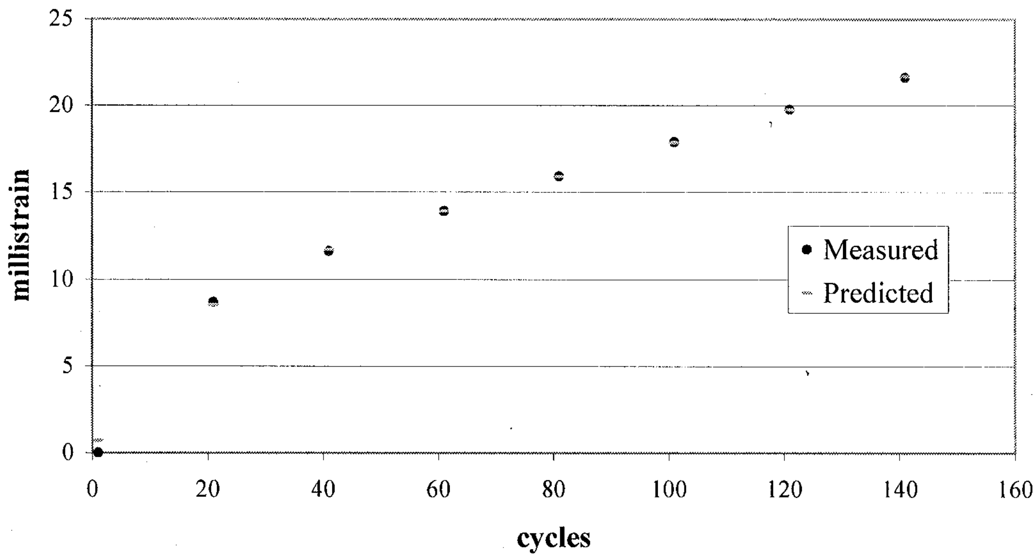

Figure D.13: Measured and predicted (proposed model) accumulated strain profiles for LA-1.

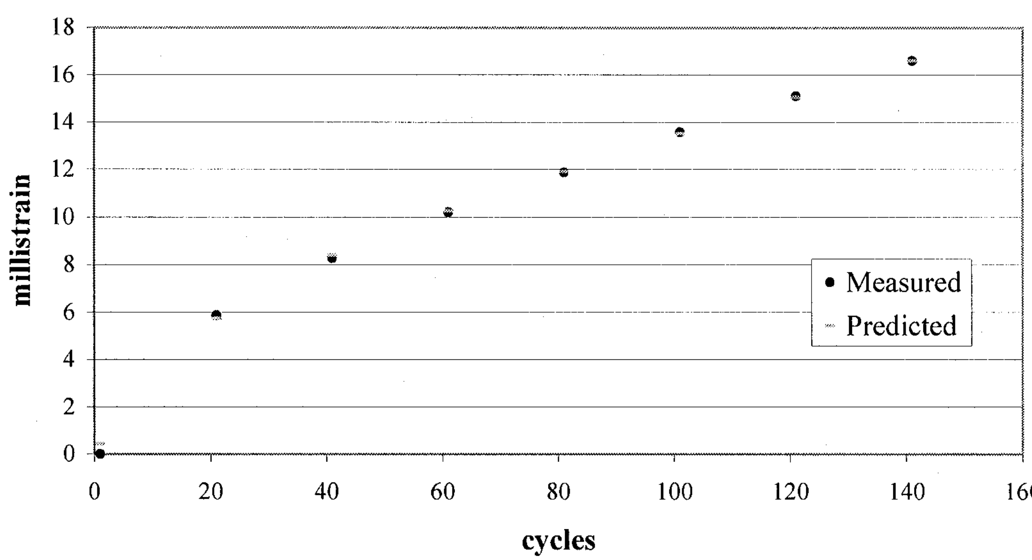

Figure D.14: Measured and predicted (proposed model) accumulated strain profiles for LA-2.

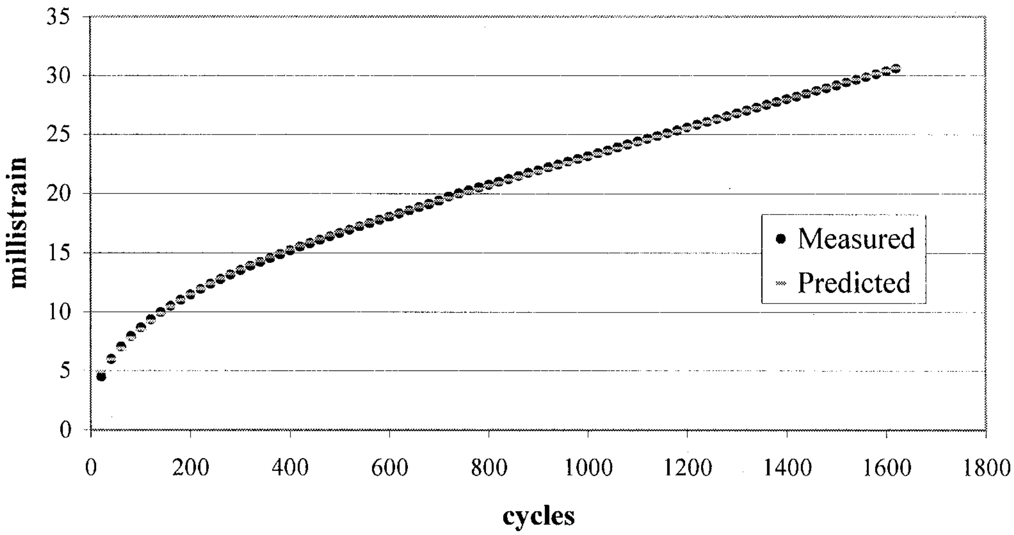

Figure D.15: Measured and predicted (proposed model) accumulated strain profiles for LA-3.

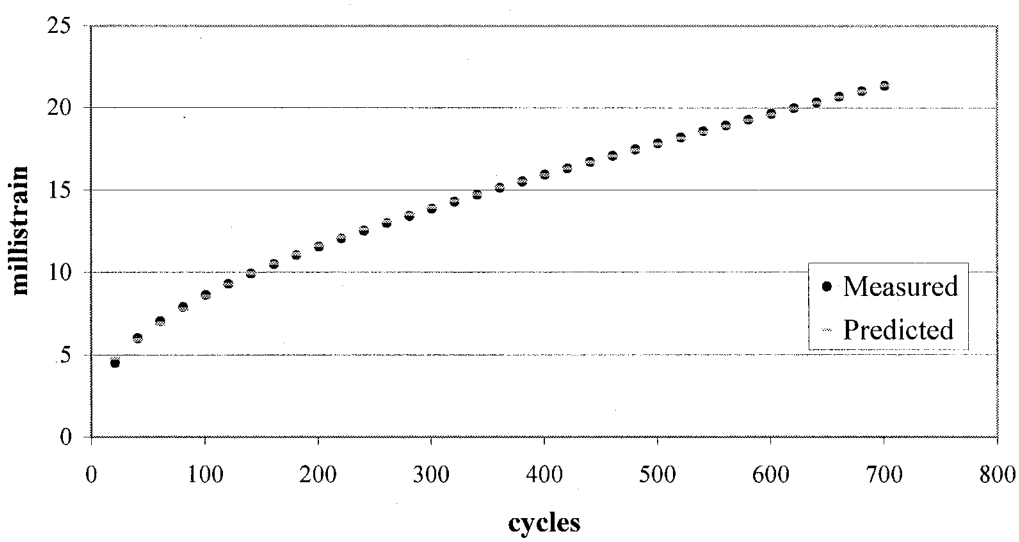

Figure D.16: Measured and predicted (proposed model) accumulated strain profiles for LA-4.

APPENDIX E

Residual (%) of Measured versus Predicted

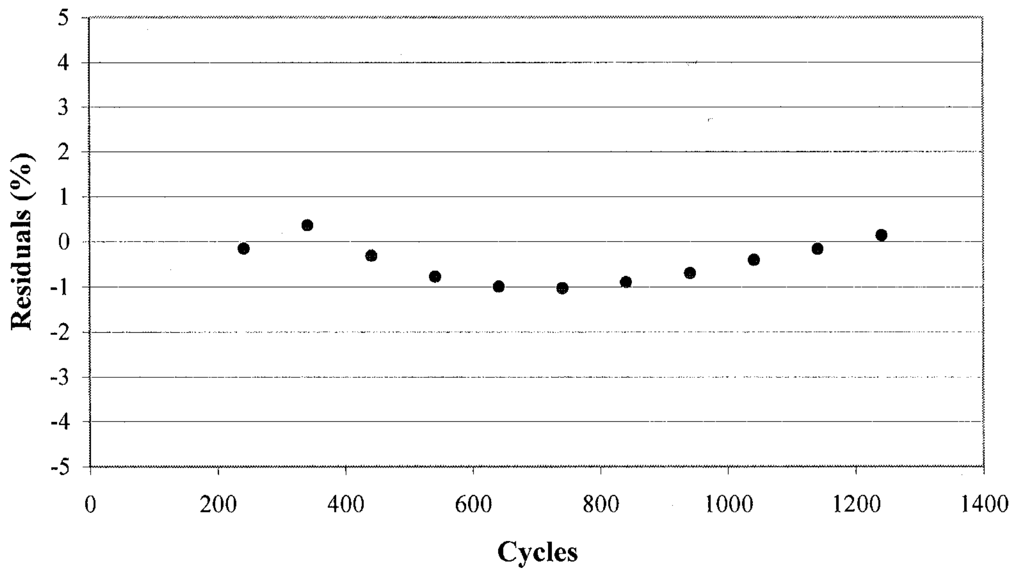

Figure E.1: Residuals (%) vs. cycles for NC-1.

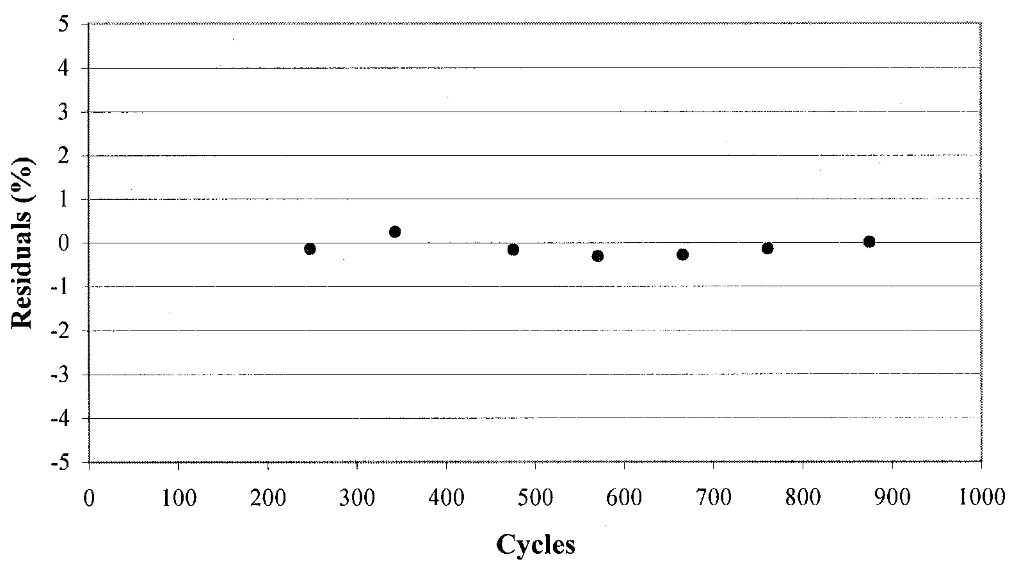

Figure E.2: Residuals (%) vs. cycles for NC-2.

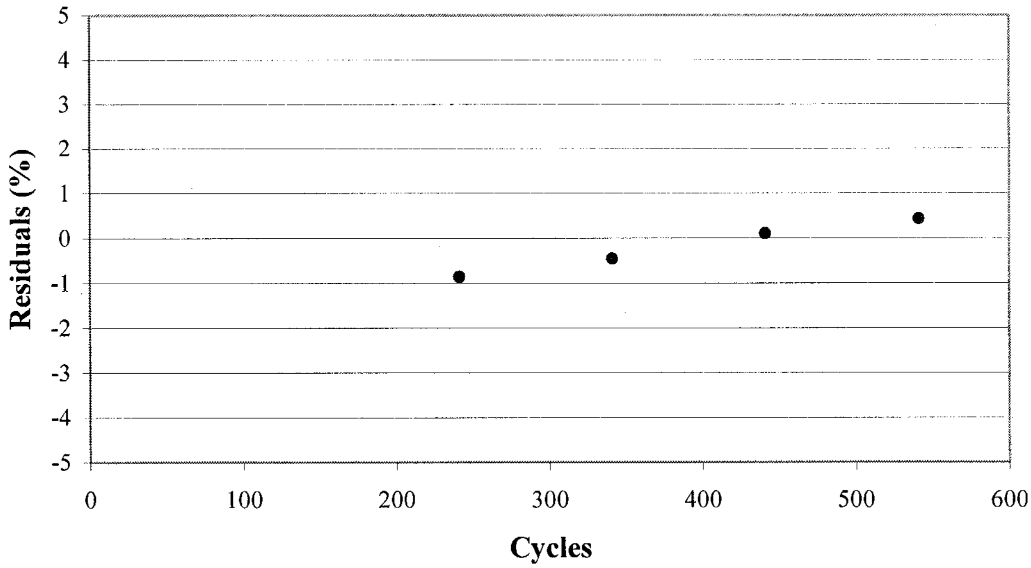

Figure E.3: Residuals (%) vs. cycles for NC-3.

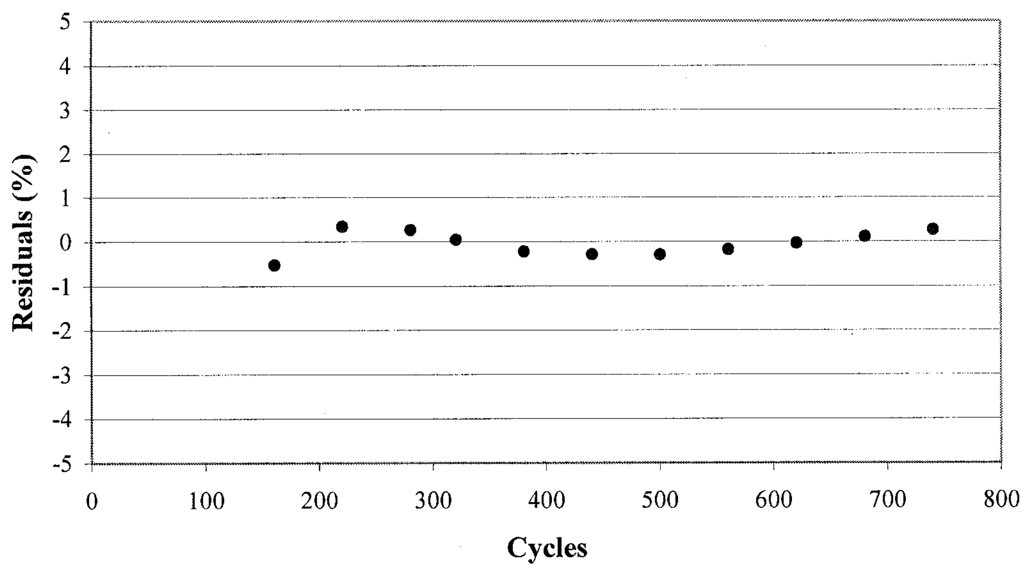

Figure E.4: Residuals (%) vs. cycles for NC-4.

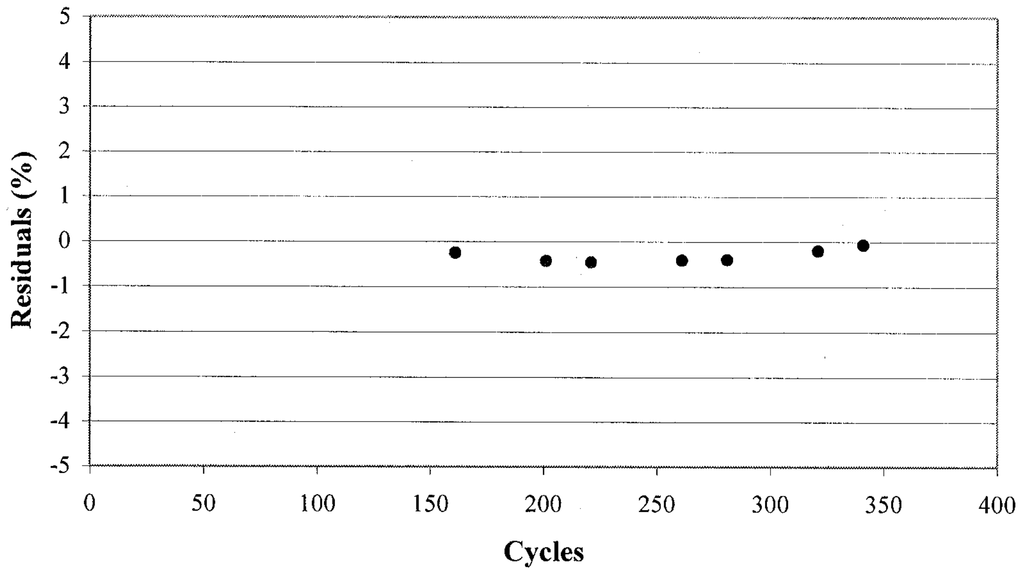

Figure E.5: Residuals (%) vs. cycles for NC-5.

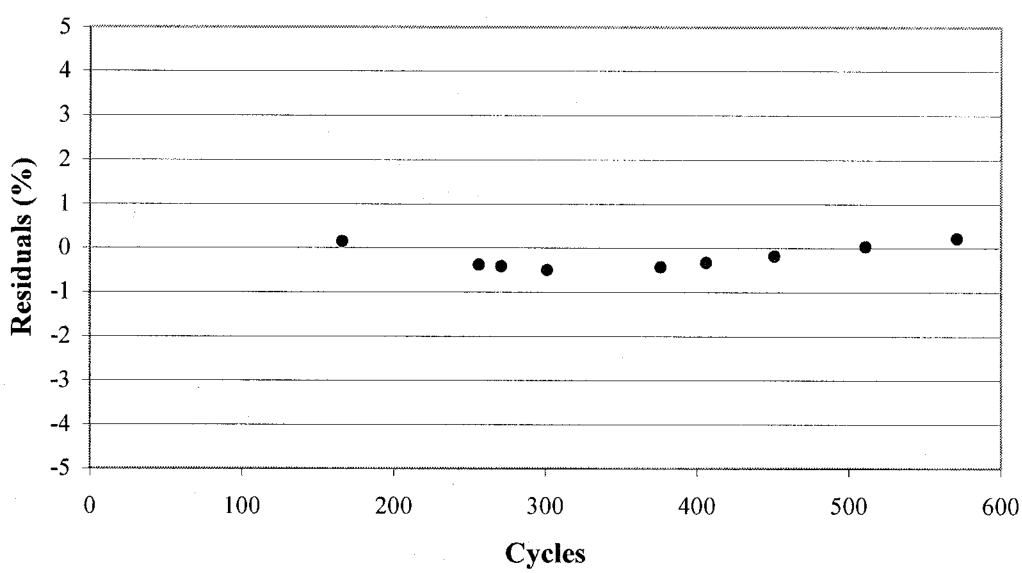

Figure E.6: Residuals (%) vs. cycles for NC-6.

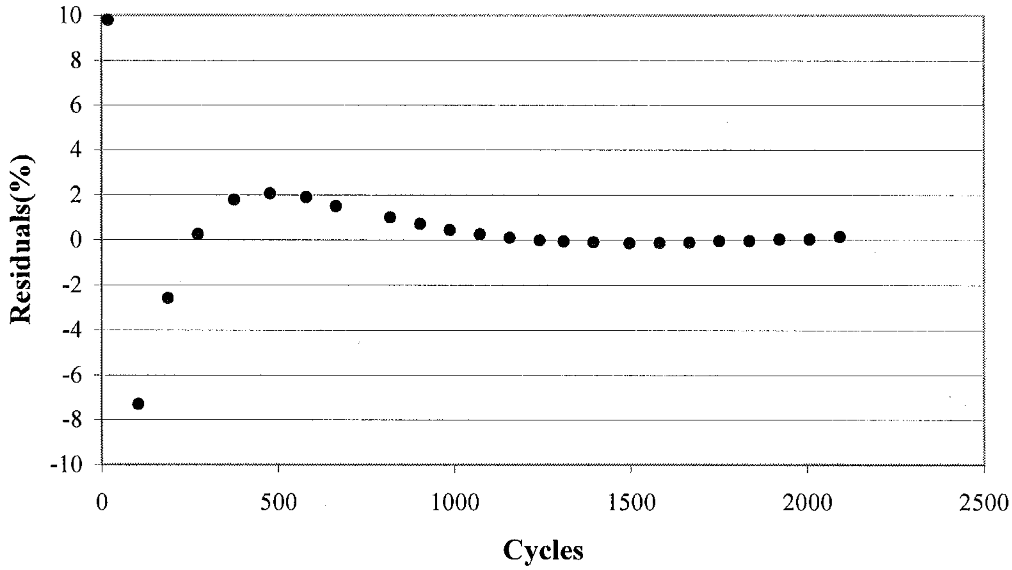

Figure E.7: Residuals (%) vs. cycles for AZ-1.

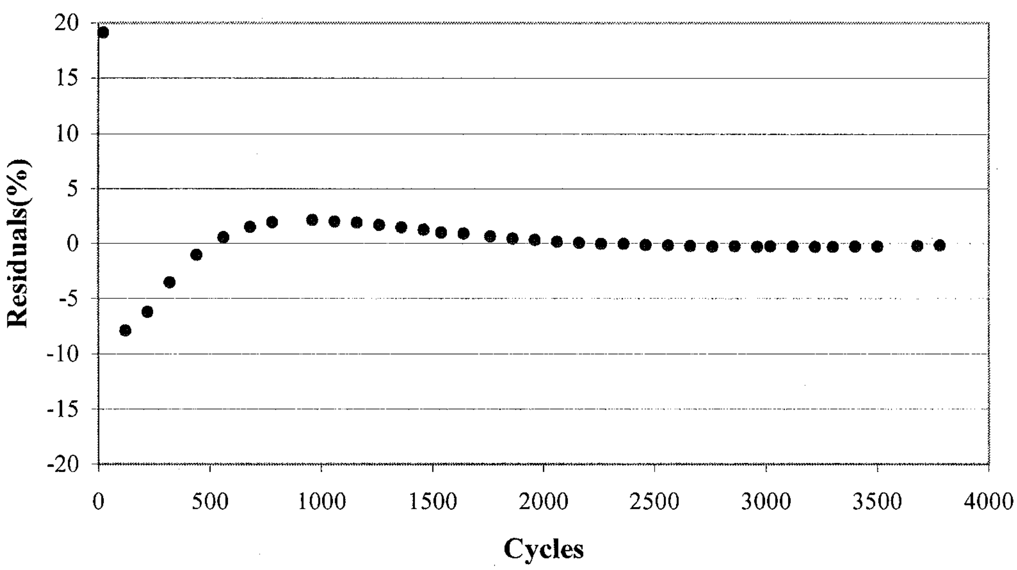

Figure E.8: Residuals (%) vs. cycles for AZ-2.

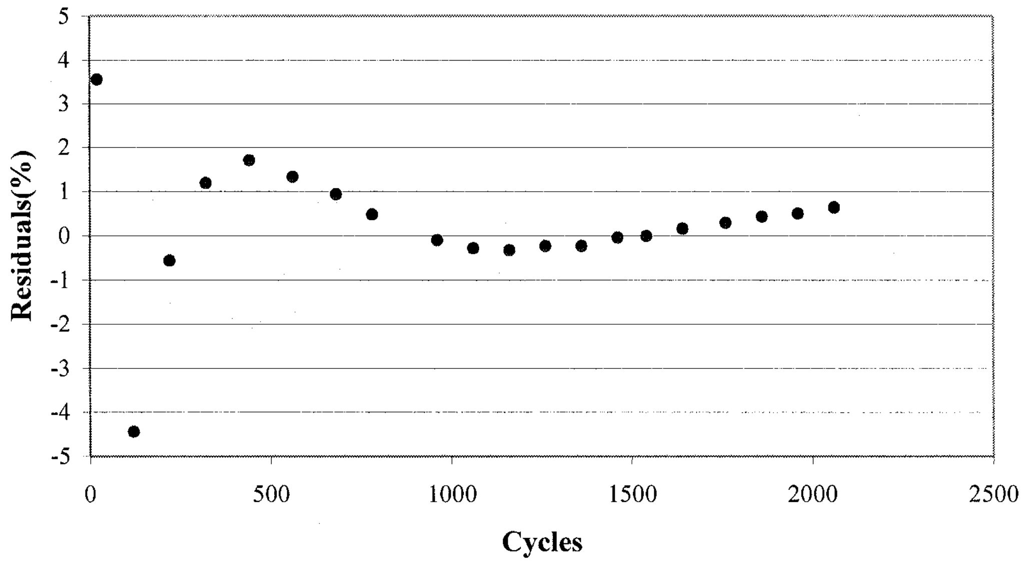

Figure E.9: Residuals (%) vs. cycles for AZ-3.

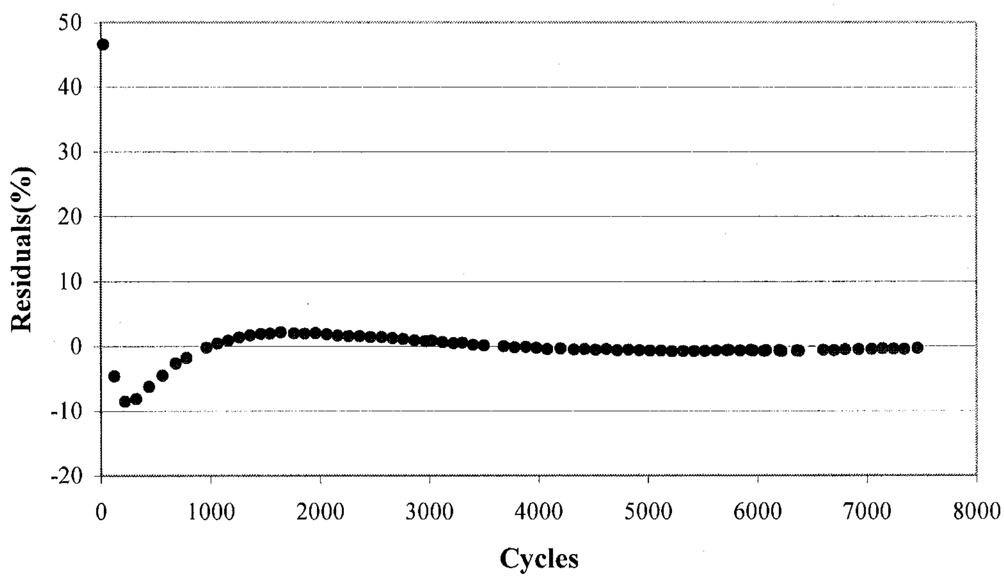

Figure E.10: Residuals (%) vs. cycles for AZ-4.

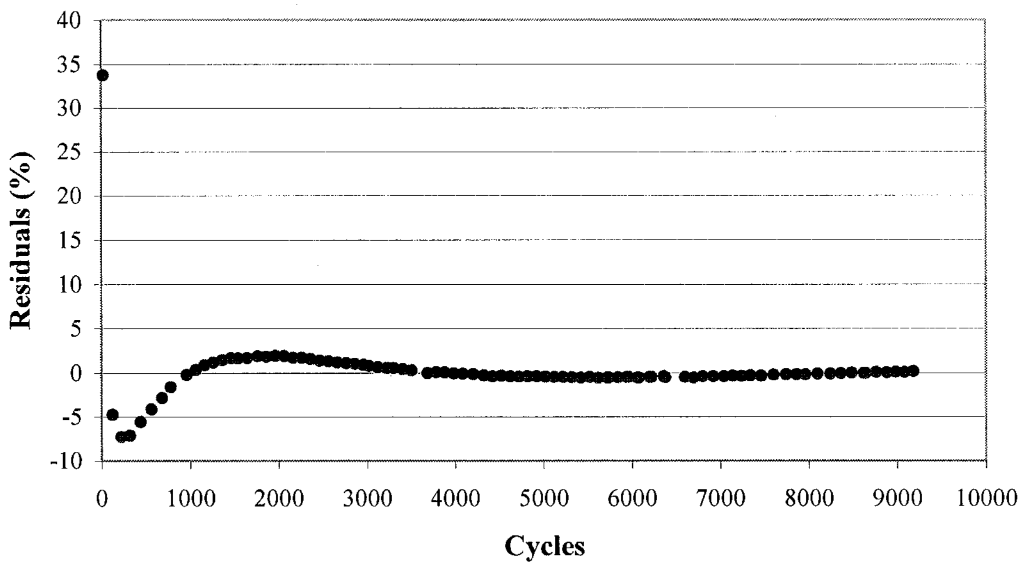

Figure E.11: Residuals (%) vs. cycles for AZ-5.

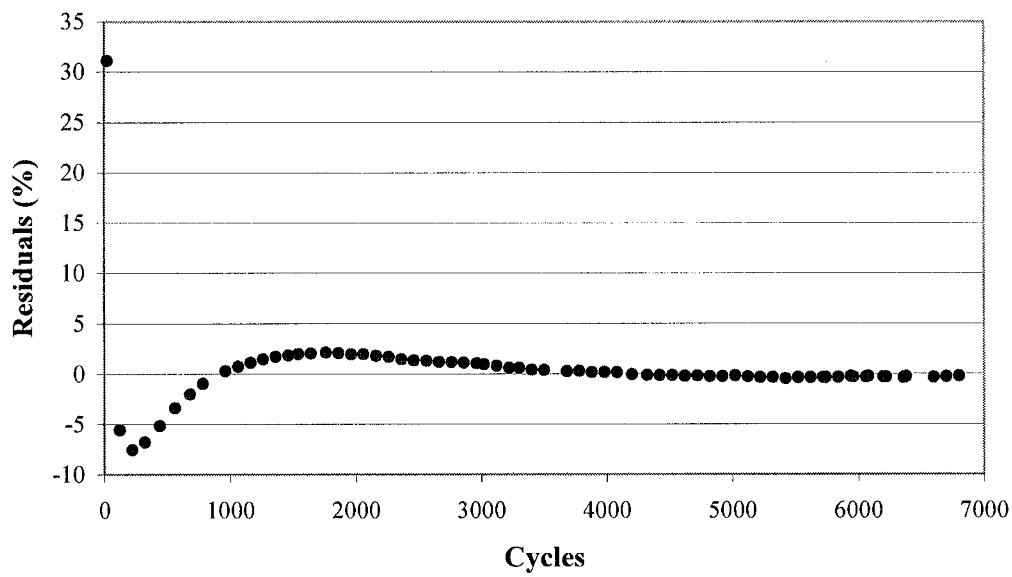

Figure E.12: Residuals (%) vs. cycles for AZ-6.

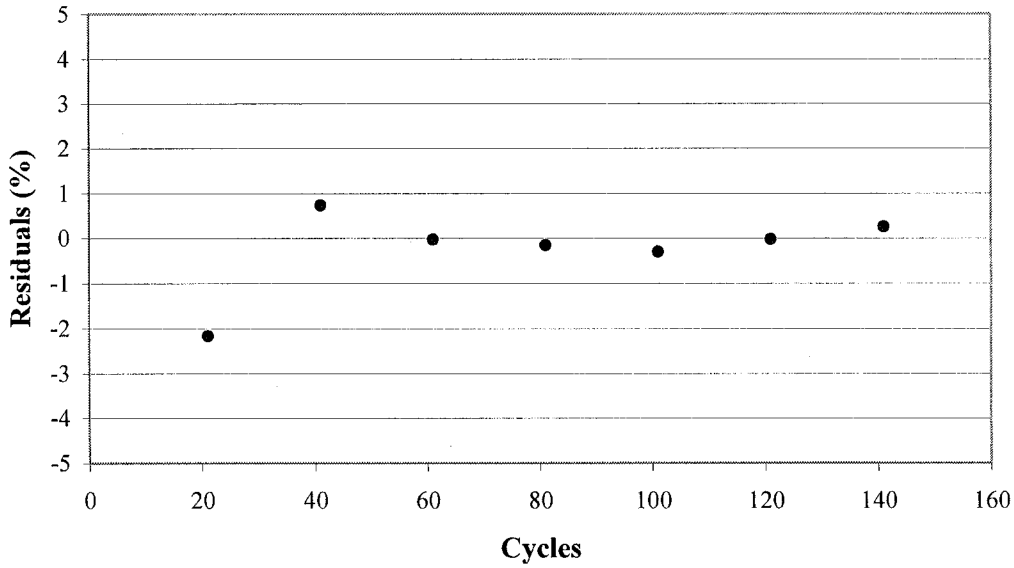

Figure E.13: Residuals (%) vs. cycles for LA-1.

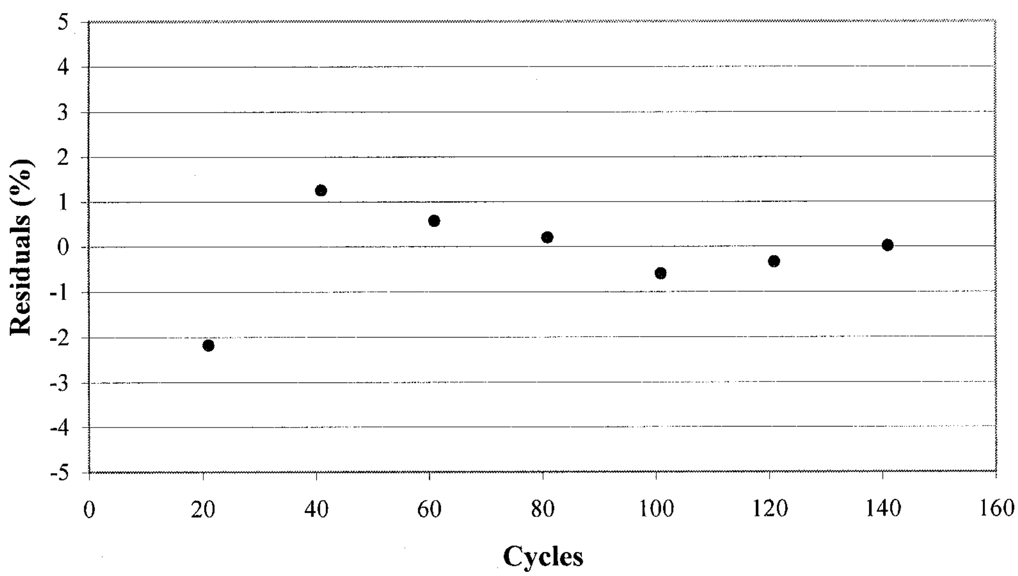

Figure E.14: Residuals (%) vs. cycles for LA-2.

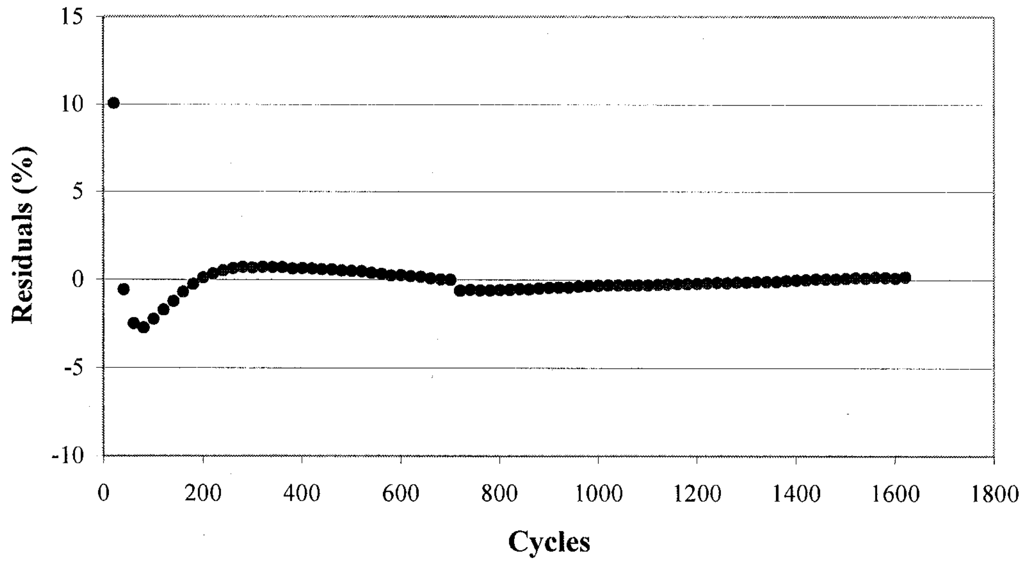

Figure E.15: Residuals (%) vs. cycles for LA-3.

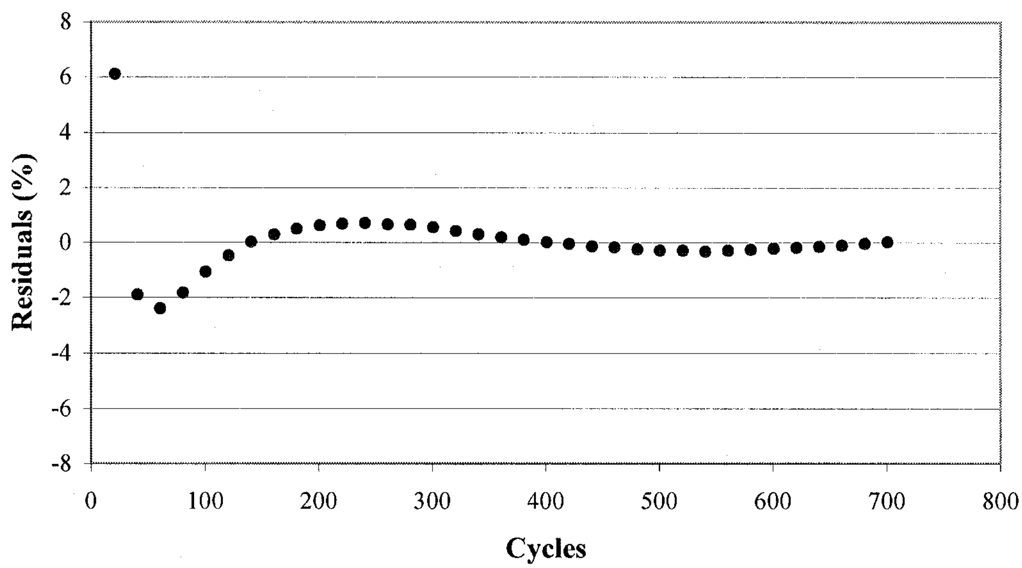

Figure E.16: Residuals (%) vs. cycles for LA-4.

APPENDIX F

VDOT Preparation Plan
for the Implementation
of the MEPDG

Introduction:

The 2002 Guide for Design of New and Rehabilitated Pavement Structures (The Guide) is a uniform and comprehensive set of procedures for the design of new and rehabilitated flexible, rigid, and composite pavements.

The Guide is based on mechanistic-empirical principles where it assumes that pavement can be modeled as a multi-layered elastic structure. This is quite a leap compared to the original AASHO Road Test, which was completely empirical (i.e. based on performance equations for the test location in Ottawa, Illinois). The Guide provides a number of new approaches for characterizing materials to be used in 21st century pavements. The mechanistic characterization of paving materials allows for the application of the principles of engineering mechanics, namely stress and strain, to the pavement analysis. Being able to input different material characteristics in the design model will allow the engineer to predict the performance of the pavement. Another improvement offered by The Guide is the use of traffic input based on the number of axles by type and weight, while eliminating the use of ESALs. The Guide also considers of the effects of temperature and moisture on a project basis using site-specific environmental factors, by implementing the FHWA's Integrated Climate Model (ICM). Additionally, The Guide offers a system of hierarchical inputs permitting the engineer to devote efforts consistent with importance of the project under consideration.

Currently The Guide is under development through the NCHRP 1-37A with target date for completion in early 2004.

It is very challenging to come up with a VDOT preparation plan of implementation, while the actual research is not yet complete, nor that The Guide is approved by AASHTO.

Nevertheless, portions of the research can be used to enhance the current 1993 AASHTO Guide for Design of Pavement Structures, while the completed and calibrated 2002 Guide will be a total replacement to the 1993 guide.

<u>Objective:</u>

To deliver an implemented 2002 Guide for Design of New and Rehabilitated Pavement Structures by December 2011, where The Guide totally replaces the current 1993 AASHTO Guide for Design of Pavement Structures in Virginia.

<u>Preparation Plan Tasks:</u>

The plan will be accomplished in two phases through the execution of the following tasks:

Phase I

Immediate utilization of applicable portions of The Guide, (as soon as The Guide is approved by AASHTO, anticipated by June 2005), to enhance the current 1993 Guide:

<u>Task 1</u>: Traffic data, Meeting the Traffic Monitoring Guide (TMG) requirements and using load spectra from Weigh in Motion (WIM) to obtain axle loading, then converting it to ESAL will provide more reliable traffic input. Traffic data will be based on 6 WIM sites, Two from VDOT and four from Department of Motor Vehicles (DMV). The level of the traffic input data is dependent on the availability the WIM equipment in the area of the project. The cost of that phase is $1,000,000, which includes installation costs, personnel costs, and operating

and maintenance costs. Personnel managing the WIM program need to be on board before this task can be accomplished. The anticipated date is June 2005.

Task2:

2.1: Subgrade resilient modulus, as input rather than using CBR correlated formulas. Equipment is operational, planned seminar on selecting resilient modulus design input value/s is being considered by the Pavement Design and Evaluation section, and the Soils section. Testing soil samples from new construction projects as part of the validation and calibration can start immediately to enhance the pavement performance evaluation.

2.2: Concrete Elastic Modulus, Use of the database generated from the actual testing of VDOT mixes. This is a level II input which is an improvement to using default values.

2.3 Concrete Modulus of Rupture, Use of the database in similar fashion to Elastic Modulus, with additional correlation to concrete Compressive Strength. This provides an advantage for new construction quality, and acceptance.

Phase II

It involves a full utilization of the completed 2002 Guide. This phase is dependent on the completion of the NCHRP and AASHTO approval of the Guide. Essentially Phase II will focus on the determination of the inputs, the use of the software, and applicability of the methods, Validation, calibration factors, and default values to Virginia's conditions and materials.

Task 3: Traffic Data:

- Using level I data for rehabilitation project, where the WIM installations are on the project site (planning for 12 sites), while level II & III are used for new alignment design, based on the project priority.
- Cost of installation, operation, and maintenance for 12 WIM sites, is three million dollars.
- It requires three full-time positions to operate/manage the WIM system. The three positions are provided by the Traffic Management, Richmond District, and the Materials Division at one position each.
- The WIM program will require five years for completion anticipated 2008.
- Steps are under way to get the personnel allocation, and sell the Operations staff on the WIM program.

Task 4: Materials Characterization:

Task 4.1: Soils Data, level I for rehabilitation projects, and to establish data base for new alignment (during construction), while level II is used for new alignment design.

- The soils Resilient Modulus testing facility is operational.
- There is no need for additional resources.

Task 4.2: Aggregate Data, Both aggregates type I, Number 21 A, and 21B will be characterized for their Resilient Modulus. Two options are under consideration, one is to modify VDOT's soils resilient modulus equipment to run the test, while the other is to farm out the testing to a private laboratory (ICAR is the first choice). This is a one time characterization to be used with all three levels of design. Anticipated completion time is Dec 2004.

Task 4.3: Asphalt Concrete Data, all VDOT mixes will be tested for the Complex Modulus (E*) and catalogued to meet level II and III for new construction. Asphalt mixes are also tested for new construction to establish database, and assist in predicting the asphalt performance, as part of the validation and calibration.

- Most of the equipment has been upgraded, with exception of the strain sensors, which is due in two months.
- The recommended procedures for asphalt testing have been provided to the Asphalt committee chairman.
- Testing is planned to start in the spring of 2004, and completion by June 2005.

Task 4.4: Concrete and Stabilized Materials Data, Current VDOT concrete mixes will be characterized for elastic Modulus, modulus of Rupture, Compressive Strength, and Thermal Coefficient of Expansion for use as level II. Additionally, mixes from new projects will be tested to enhance the database and predict concrete pavement performance, as part of the validation, and calibration process. Stabilized Materials, including Cement Treated Aggregate (CTA), Soil Cement, and Lime Stabilized Soils will be tested for Elastic Modulus. Samples obtained from newly constructed pavements will be used for testing. The obtained data is used for calibration, and establishing both correlation and default values.

- The testing facility is operational.

- No additional resources are needed.
- Database for concrete and stabilized materials to be completed by June 2005.

Task 5: Calibration and Validation:

- The challenge is to determine the validity of the design method and default values to Virginia conditions and materials. Calibration is the process of making adjustments to the theoretical models to compensate for model simplification and limitations in simulating actual pavement behavior and distress development, while validation is to determine whether the model provides a reasonable prediction of actual performance, and if the desired accuracy or correspondence exists between predicted and monitored performance.

Task 5.1: The current plan is to utilize the LTPP site in Danville, and the Smart Road in Blacksburg, in addition to the matching sites in the LTPP at large (i.e. Southeastern sites due to similar climate) to cross check the various levels of actual field distresses with the program predicted distresses. This effort should lead to either using the default values from the Guide or obtaining calibration factors.

Task 5.2: Software Applicability just started through the participation of Dot's Pavement Design Program Manager as a panel member of AASHTO's efforts to develop a new generation of Mechanistic-Empirical Design Software known as DARWinM. Finalized time line to accomplish this task still under development. It could be estimated that this effort to be completed before AASHTO would ask the DOTs to vote on the adoption of the Guide. Anticipated time is Dec 2005

Task 5.3: Data Management Work is in the preliminary stages of developing the scope for the re-write of Materials Database System. Some of the major objectives that have been identified are:

- The development of a web based system which will allow both VDOT and Contractors to input test results.
- Improved online sample tracking so that customers know the status of the testing.
- Information collected should include all data required by AMRL.
- Expand the data sources to include field testing.
- Where appropriate include GIS data.

The data is currently stored in SQL Server and we will continue to use SQL Server for storing the data. Data export will probably include comma-delimited formats and XML which is quickly becoming the de facto standard for data distribution.

This approach is very suitable for The Guide, where it ties data from the districts, and it is accessible to all committees working on the Guide.

- The Computer Technology section is charged with data management for The Guide. This is planned as internal effort, and it would take 1.5 MEL to complete, within three years time frame (June 2006).
- At present, the target time frame needed to complete the calibration and validation is 2011. It also requires two engineering positions (one senior and one entry level) to dedicate full time in conducting the verification.

Task 6: Training

VDOT personnel have been exposed to pavement design using mechanistic approach through several NHI courses since 1989. Although, that actual practice of the design procedure was not exercised, still it would provide a minimum knowledge of the methodology.

Task 6.1: The assistant State Materials Engineer (Technical) and the Pavement Design Program Manager are planning on attending the NCHRP 1-37A training anticipated to be held by September 2004. This is a five day program involving representatives from the State DOTs, and FHWA. Interaction and exchange of experiences regarding the Guide is very essential. It also provides a face to face discussion with the research team.

Task 6.2: This task is devoted to training central office and field personnel. The training will be conducted by members of the Training committee. The training of the central office personnel would be conducted first. Three-day training during March 2006 is planned. Training of field personnel (essentially at the Districts) is planned for June 2006. This is to allow for the central office personnel to gain additional expertise with the software, where they become resources to field personnel. Two districts at the time would be trained on the use of the software. It is envisioned that pavement design using both the 1993 Guide and the 2002 Guide will be performed during the time of calibration (2006-2011). This approach will provide continuous gain of expertise in using the Guide, and comparing results between the 1993 Guide and 2002 Guide.

Impact on VDOT Standards and Specification

- Level I design requires site-specific input data, but since our plans are prepared at least two years in advance, it is necessary to have close cooperation between the designer, and the contract preparation team. As an example, the Performance Prediction Model is based on the initial IRI (as built), which means that the contract has to require such initial IRI value. This may require changing the way we do business now, where the contractor is given incentive/disincentive to comply with the current rideability specs, where we may end up with initial IRI value that is different from the design value. Performance specs seem to lend itself to the goal for implementing the Guide. As an example Cement Treated Aggregate (CTA) currently has prescriptive specifications i.e., use 4% cement by weight with Type I, size 21A aggregate. This method would not be adequate for the Guide. The GRAC is preparing a research statement titled "Evaluation of the Strength of Cement Treated Aggregate for Pavement Bases". This research is to be conducted by the University of Virginia. The research would characterize the CTA in terms of its Young's Modulus, and the Unconfined Compressive Strength, which are input parameters for the Guide, and easy to specify as performance specs.
- This plan is based on the most recent available information. Several milestones in this plan are based on anticipated time for completion of the NCHRP 1-37A work, and the research panel review. The next milestone is when The Guide is completed, and the state DOTs training is done. This will provide the opportunity to ask more details questions to the Research Team, and interact with other states to benefit from their approach, and in turn making adjustment to VDOT plan.