ABSTRACT

MUTHADI, NARESH REDDY. Local Calibration of the MEPDG for Flexible Pavement Design. (Under the direction of Dr. Y. Richard Kim.)

The 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures is a mere modification of the empirical methods found in its earlier versions that are based on regression equations relating simple material and traffic inputs. Although the various editions of the AASHTO design guide have served well for several decades, they contain too many limitations to be continued as the nation's primary pavement design procedures. The Mechanistic-Empirical Pavement Design Guide (MEPDG) procedure, on the other hand, provides the tools for evaluating the effect of variations in input data on pavement performance. The design method in the MEPDG is mechanistic because it uses stresses and strains in a pavement system calculated from the pavement response model to predict the performance of the pavement. The empirical nature of the design method stems from the fact that the pavement performance predicted from laboratory-developed performance models is adjusted based on the observed performance from the field to reflect the differences between predicted and actual field performance.

The performance models used in the MEPDG are calibrated using limited national databases and, thus, it is necessary to calibrate these models for local highway agencies implementation by taking into account local materials, traffic information, and environmental conditions. Two distress models, permanent deformation and bottom-up fatigue cracking (hereafter referred to as *alligator cracking*), were employed for this effort. Fifty-three pavement sections were selected for the calibration and validation process: 30 long-term pavement performance (LTPP) pavements, which include 16 new flexible pavement sections

and 14 rehabilitated sections, and 23 North Carolina Department of Transportation (NCDOT) sections. All the necessary data were obtained from the LTPP and the NCDOT databases. To provide reasonable values in cases where data were missing, MEPDG defaults, NCDOT typical range of values, and engineering judgment were employed. Finally, an experimental matrix is developed to identify any bias resulting from the use of local materials and conditions. The NCDOT currently relies on a subjective rating or non-numeric rating system of the permanent deformation data, which presented difficulties in the conversion to the MEPDG format.

The verification runs for the LTPP sections using the parameters developed during the national calibration effort under the NCHRP (National Cooperative Highway Research Program) 1-37A project showed promising results. Microsoft Excel Solver was used to fit the predicted rut depth values to the measured values by changing the coefficients in the permanent deformation models for hot-mix asphalt (HMA) and unbound materials. This process was employed for each of the permanent deformation models separately. For the alligator cracking model, the only possibility of reducing the standard error and bias is through the transfer function. Again, Microsoft Excel Solver was used to minimize the sum of the squared errors of the measured and predicted cracking by varying the C_1 and C_2 parameters of the transfer function. It was found that there is no significant difference between the local calibrated standard error and the global standard error for the HMA permanent deformation model as well as the alligator cracking model. Therefore, it was decided to keep both the models for a more robust calibration in the future that would increase the number of sections and include more detailed inputs (mostly Level 1 inputs).

Local Calibration of the MEPDG for Flexible Pavement Design

by
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A thesis submitted to the Graduate Faculty of
North Carolina State University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Civil Engineering

Raleigh, North Carolina

2007

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DEDICATION

This work is dedicated to my parents and my brother

BIOGRAPHY

Naresh Muthadi was born on May 25, 1983, in Warangal, India. He earned his Bachelor's degree in Civil Engineering from the Indian Institute of Technology, Bombay in 2005. He then enrolled at North Carolina State University to pursue Masters Degree in Civil Engineering in fall 2005. Shortly thereafter, He began working on his thesis under the guidance of Dr. Y. Richard Kim.

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CHAPTER 1 INTRODUCTION

1.1 Research Needs and Significance

The 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures is a mere modification of the empirical methods found in its earlier versions that are based on regression equations relating simple material and traffic inputs. Although the various editions of the AASHTO design guide have served well for several decades, they contain too many limitations (NCHRP, 2004.a) to be continued as the nation's primary pavement design procedures. Besides the empirical nature of the 1993 Guide, the performance equations that are its foundation are derived from the AASHO Road Test that took place during 1958-1961 in Ottawa, Illinois. Several limitations have been noted in the procedures from then on, including 1950's traffic loading, one climatic zone, one material type for each pavement layer, and 1950's construction and drainage.

One of the major concerns regarding the previous AASHTO methods was the inability to incorporate significant material properties into the design (NCHRP, 2004.a). For flexible pavements, for example, the only parameter considered was layer coefficient 'a', which is empirical by nature. For rigid pavements, highly significant parameters such as the coefficient of thermal expansion and joint spacing were not considered at all. The mechanistic-empirical design procedure, on the other hand, provides the tools for evaluating the effect of variations in materials on pavement performance. It provides a rational relationship between the construction and material specifications and the design of the pavement structure. Because the mechanistic procedure better accounts for present day materials, aging, climate, and present day vehicle loadings, the variations in performance in relation to design life should be reduced.

Several benefits are associated with the implementation of the National Cooperative Highway Research Program (NCHRP) 1-37A Mechanistic Empirical Pavement Design Guide (MEPDG). The design method in the MEPDG is mechanistic because it uses stresses and strains in a pavement system calculated from the pavement response model to predict the performance of the pavement. The empirical nature of the design method stems from the fact that the pavement performance predicted from laboratory-developed performance models is adjusted based on the observed performance from the field to reflect the differences between predicted and actual field performance. These models require user inputs for the following four modules: materials, traffic, environment and pavement structure. The MEPDG is hierarchical in nature in that the level of sophistication for different input requirements changes according to the importance of the project in question.

The performance models used in the MEPDG are calibrated using limited national databases and, thus, it is necessary to calibrate these models for implementation by taking into account local materials, traffic information, and environmental conditions. A crucial step in ensuring that the calibration is commensurate with local conditions is the sensitivity analysis; that is, the sensitivity of the final design outcome for various input parameters must consider the material, traffic and environmental conditions in North Carolina. The term *calibration* here means to reduce or minimize the total error or difference between the measured and predicted distresses by varying the appropriate model coefficients. A successful validation means that the calibrated model produces robust and accurate predictions for pavement sections other than those pavements used in the calibration process.

The local calibration guide developed during the NCHRP 1-40B project (NCHRP, 2007) provides necessary recommendations and guidelines to ensure proper recalibration and validation to the local materials and conditions.

1.2 Research Objectives

The primary objective of the research project is to calibrate the MEPDG for flexible pavement design in North Carolina. To accomplish this objective, the following tasks are performed in this study:

1. Develop a summary of design practices outlined in the MEPDG that differ from the current design practices followed by the NCDOT;
2. Perform a sensitivity analysis on the design input parameters using realistic input ranges;
3. Develop input data collection strategy along with a set of implementation guidelines and recommendations;
4. Develop a database for the NCDOT materials, traffic and climate; and
5. Calibrate and validate the two performance prediction models (permanent deformation and alligator cracking models) in the MEPDG for local conditions and materials.

1.3 Thesis Organization

This thesis is composed of seven chapters. Chapter 1 presents the research needs and objectives. Chapter 2 summarizes the literature review of current design practices at the NCDOT and the MEPDG, as well as the design inputs required to use the MEPDG. Chapter 3 describes the data collection process of obtaining the data from LTPP database and the NCDOT. Chapter 4 describes the sensitivity analyses done by other researchers as well as the analysis performed in this project. It also provides a discussion of the input data collection strategy that is derived from the results of the sensitivity runs. Chapter 5 summarizes the literature review of the local calibration guidance developed under the NCHRP 1-40B project, distress prediction models used in the study, and the national calibration procedure used during the NCHRP 1-37A project. It also discusses the local calibration methodology employed in this study. Chapter 6 presents the results and analysis of the local calibration effort in a step by step manner. Finally, the conclusions and recommendations for future research are given in Chapter 7.

CHAPTER 2 DESIGN PRACTICES

2.1 Introduction

The main focus of this chapter is to develop a summary of design practices outlined in the MEPDG that differ from the current design practice followed by NCDOT. This study will help pavement design engineers understand the basic principles behind the design guide methodology, DG (Design Guide) 2002 software and its input requirements, thus allowing a smoother adaptation of the MEPDG by the NCDOT.

A review of the NCDOT'S current pavement design practices and existing input data constitutes the first task of the research; these practices and data include the material inputs, traffic inputs, climatic inputs, design reliability levels, performance criteria, and pavement structures. Information collected in this review includes: (1) types and formats of input data for current NCDOT design practices; (2) actual values and their variations for various input data; and (3) data collection methods.

Similarly, a summary of the study of the MEPDG design guide methodology along with the input requirements by the new MEPDG software are provided in this chapter. It also discusses the flexibility associated with the hierarchical design input levels approach in the MEPDG.

2.2 Current NCDOT Design Practice

The first task of this project is to review the NCDOT's current pavement design practices (NCDOT, 2000) and existing input data. Several engineers from key NCDOT units were interviewed for the purpose of understanding the current design practices at the

NCDOT. These units include the Pavement Management Unit (PMU), Geotechnical Engineering Unit, Materials & Tests Unit (M&T Unit), Traffic Forecasting Unit, and Traffic Survey Unit (TSU).

The current design practices follow AASHTO design equations, as presented in the 1972 AASHTO Interim Guide for Design of Pavement Structures, for the design of both flexible and rigid pavements. For flexible pavement designs, the inputs required are regional factors and soil support values, as determined from laboratory CBR tests. For rigid pavement designs, the input required is the modulus of the subgrade reaction.

The findings from the review and interviews are summarized below.

2.2.1 Pavement Management Unit

Pavement designs are created by the PMU and follow the NCDOT's Interim Pavement Design Procedure (IPDP) dated April 1, 2000. The IPDP is based on the 1972 AASHTO Interim Pavement Design Guide with some modifications. In the following, the procedures in the IPDP are summarized, along with the findings from the interviews with the PMU engineers.

2.2.1.1 Design Life

The design life for new flexible and rigid pavements is 20 and 30 years, respectively.

2.2.1.2 Traffic Inputs

The algorithm used in the current NCDOT procedure for calculating the 18 kip (80 kN) equivalent single axle loads (ESALs) for pavement design requires the following traffic inputs: initial year average daily traffic (ADT), projected year ADT, percentage of Duals, percentage of truck tractor and semi-trailer (TTST), and directional distribution percentages.

The term *Dual* represents single-unit trucks, whereas *TTST* represents multiple unit trucks including both single and twin trailer combinations.

The current NCDOT procedure is based on the following traffic growth law:

$$Total\ ESALs = \sum_{i=0}^{DL} IESAL \times (1 + \frac{GR}{100})^{i} , \tag{2-1}$$

where;

DL = design period in years;

$IESAL$ = first year ESALs; and

GR = growth rate, percent.

Eq. (2-1) can be further refined using various traffic factors, as shown below:

$$Total\ ESALs = AE + IADT \times \left\{ \left(1 + \frac{GR}{100 \times 365.25} \right)^{365.25 \times DL} - 1 \right\} \times \left(1 + \frac{GR}{365.25 \times 100} \right)^{(CY-IY)\times 365.25}$$

$$\times \left(\frac{PTT}{100} \times FTT + \frac{PD}{100} \times FD \right) \times \frac{DDP}{100} \times \frac{LD}{\ln\left(1 + \frac{GR}{100 \times 365.25} \right)} , \tag{2-2}$$

where;

AE = additional ESALs, such as military loadings;

GR = growth rate, percent;

DL = design life, year (flexible pavement = 20 and rigid pavement = 30);

$IADT$ = initial year ADT;

CY = construction year;

IY = initial year;

PTT = TTST, percent;

LD = lane distribution factor determined from Table 2-1;

FTT = truck factor for TTST (Table 2-2);

PD = Duals, percent;

FD = truck factor for Duals (Table 2-2);

DDP = directional distribution, percent; and

LN = natural logarithm.

The projected year ADT is normally for a 20-year period. The 20-year growth rate should be used to project traffic counts to the 30-year mark, unless the Traffic Forecast Unit provides 30-year projections. Loadings from automobiles are considered to be negligible. The lane distribution factors and the truck factors are tabulated in Table 2-1 and Table 2-2, respectively. A lane distribution factor of 0.5 will be used for the design of the inside (median) lane widening of existing facilities that have 2 or more lanes per direction.

Table 2-1 Lane Distribution Factor

Lane Distribution Factor	**No. of Lanes in One Direction**
1.0	1
0.9	2
0.8	3 or more

Table 2-2 Truck Loading Factors

	Truck Loading Factors			
	Flexible Pavement		**Rigid Pavement**	
Road Type	**Duals (*FD*)**	**TTST (*FTT*)**	**Duals (*RD*)**	**TTST (*RTT*)**
Rural Freeway	0.30	1.15	0.30	1.60
Rural Other	0.30	0.95	0.35	1.30
Urban Freeway	0.30	0.85	0.35	1.20
Urban Other	0.25	0.80	0.25	1.10

The following list summarizes the characteristics of traffic inputs for pavement designs at the PMU:

- ADT, %TTST, and %Dual are obtained for the construction year and for the design life based on the projection from the Traffic Forecasting Unit.
- When the project is composed of multiple segments, the highest traffic data are used for the entire project.
- Class 4 and Class 9 trucks are the major vehicle types in North Carolina.
- Eq. 2-2 is used to determine the ESALs.
- Truck factors are developed from weigh station data.
- Fifty percent is normally used to convert two-way traffic to one direction for pavement design.
- Based on the PMU's mid-day vehicle and truck counts and visual observation at project sites, it has been noticed that traffic forecast truck percentages are lower than truck percentages based on these observations.

2.2.1.3 Soil Support Value (SSV)

The SSV is determined from the following equation (NCDOT, 2000):

$$SSV = 5.32 \times \log(CBR) - 1.49 \tag{2-3}$$

The pavement design engineer will determine the final SSV value based on the laboratory CBR test results, deflection data (if any), and information from the Geotechnical Engineering Unit, including the soil classification and Dynamic Cone Penetrometer (DCP) results.

2.2.1.4 Terminal Serviceability Index

Table 2-3 shows the terminal serviceability index values used in different cases.

Table 2-3 Terminal Serviceability Index

Traffic Level	Terminal Serviceability Index
20-year traffic projection exceeds 50,000 ADT with a high heavy truck volume	2.75
20-year traffic projection exceeds 80,000 ADT with a high heavy truck volume	3.0
All other roadways	2.5

2.2.1.5 Regional Factors

The minimum values for the regional factors for different counties in North Carolina are given on page 4 of the IPDP (NCDOT, 2000). These values are used unless higher values are provided by the Geotechnical Engineering Unit.

2.2.1.6 Layer Coefficients

Layer coefficients for different materials are given on page 4 of the IPDP (NCDOT, 2000). It is noted that one structural coefficient value is used for all the surface and binder hot-mix asphalt (HMA) mixes.

2.2.1.7 Drainage

For rigid pavements, a permeable asphalt drainage layer (PADL) over a stabilized subgrade is typically used under the concrete slab to provide uniform support and positive drainage. In the past, NCDOT had specific mix types that consisted of coarse aggregate and asphalt cement only, and were not dense-graded. These mix types were identified as PADL/PADC. A separating layer of S9.5A (30 mm thick) or S9.5B (40 mm thick) is placed between the PADL and the stabilized subgrade.

Shoulder drains are always used when PADLs are used. Shoulder drains are considered for projects with an ADT greater than 15,000 and a heavy truck percentage greater than 5%. For projects with an ADT greater than 50,000 and a heavy truck percentage greater than 15%, shoulder drains are highly recommended. For flexible pavements, shoulder drains are recommended for locations that have a slope of less than 1% and poor draining soils.

2.2.1.8 Cost Analysis

According to the IPDP, the unit costs for pavement pay items are obtained from the Project Services Unit, and the request for unit costs should include the project location and description and a complete listing and total quantities of the pay items.

2.2.1.9 Asphalt Overlay on Flexible Pavements

The falling weight deflectometer (FWD) data are used to determine the overlay thicknesses. The detailed mathematical procedure is given on pages 10 and 11 of the IPDP. Traffic estimates for smaller overlay jobs and widening projects are provided by the PMU.

The modulus of an AC overlay is assumed to be 500,000 psi. Coring is often undertaken to determine layer thicknesses.

2.2.1.10 Asphalt Overlay on Rigid Pavements

The design of asphalt overlay on rigid pavements is mainly experience-based.

2.2.1.11 Concrete Overlay on Rigid Pavements

Currently, there is no design procedure for a concrete overlay on rigid pavements. Bonded overlay is not very common in North Carolina. The continuously reinforced concrete pavement (CRCP) construction is undertaken mostly in widening projects, and such projects use the design method for new pavements. Therefore, the thickness of the concrete overlay on the existing pavement is governed by the thickness of the pavement in the new lane. For

CRCP pavements that are not in good condition, patching will be done to restore them. It is the most preferred treatment which is done with concrete using mechanical couplers and is then overlaid with 5 inches of HMA.

The unbonded overlay requires a substantially thicker overlay than the bonded overlay but much lower risk. The additional thickness of an unbonded overlay often becomes a problem because of geometric constraints. Bridge clearances are the main constraint.

2.2.1.12 Rubblization

The layer coefficient for rubblized concrete is given on page 5 of the IPDP. The SSV values are obtained from the Geotechnical Engineering Unit. Rubblization projects are treated as new flexible pavement design projects.

2.2.2 Traffic Survey and Traffic Forecasting Units

2.2.2.1 Data type

The TSU has six types of traffic data products that may be used in the development of traffic forecasts. These are:

Annual Average Daily Traffic (AADT)

The unit operates a coverage count program to provide comprehensive monitoring of traffic volume on all primary and on some secondary and local routes. Data are collected in daily counts on highway segments that are factored for axle correction and seasonal adjustment to generate AADT volume estimates. These data are used to generate base year AADT estimates and for trend analyses to develop future year estimates.

Turning Movements (TMs)

Turning movements are manual volume counts collected at intersections to identify the number of turns and through movements. Truck classification data consistent with traffic forecast classification formats are collected for one leg of the intersection. Data are collected in two shifts, typically on two different days. Surveyed turning volumes and truck volumes are expanded from partial day to 24-hour estimates. These values are treated as average weekday volumes.

Manual Classification (MC)

Manual classification is similar to turning movements, but data are collected on a segment where traffic is classified in both directions. Truck percentages are generated for surveyed traffic only. These volumes are not expanded to 24-hour estimates. Surveyed truck percentages are considered representative of average weekday truck percentages. The manual classification is typically used when conditions do not allow collection of an electronic vehicle classification count.

Daily Volume Counts

Daily volume counts are the same type of count as collected for AADT. Data are provided as requested for small-scale projects, but are unfactored. Data are factored for large-scale urban transportation modeling projects (validation counts). These counts are typically collected on weekdays. Factoring means converting Short duration counts to represent the average traffic condition, which is usually done using the continuous counter data from other locations.

Hourly Volume Counts

Hourly volume counts are the same type of count as daily counts except data are collected in an hourly format. These counts are sometimes collected by direction. The hourly counts are never factored. Daily totals are sometimes factored as daily counts. These counts are typically collected on weekdays.

Hourly Vehicle Classification Counts

Hourly vehicle classification counts are electronic counts collected in the FHWA 13 vehicle classification scheme in hourly totals. These counts are collected and reported by lane. Data must be aggregated by the user for two-way totals and then aggregated by class to generate the NCDOT truck percentages. The correlation between the schemes is Duals = Class 4-7 and TTST = Class 8-13. Daily totals are sometimes factored as daily counts. These counts are typically collected on weekdays.

2.2.2.2 Data Collection Methods

In estimating the traffic volume for new roads, computer programs are used to predict the future traffic once the new road is opened to traffic. The PMU uses the default truck loading factors from Table 2-2 to determine percentage of Duals and percentage of TTSTs for different road categories. Currently, axle load information is not generated from the TSU.

2.2.2.3 Equipment

The TSU uses the following equipment to collect various traffic data:

- Daily counter – TT6 (pneumatic tube) – manual data transfer
- Vehicle classifier - PEEK ADR-1000 – hourly volume and vehicle classification – requires programming for operation
- Manual counter – Jamar's DB400
- Continuous volume monitoring site – PEEK ADR-3000 with inductance loops
- Continuous WIM – PEEK ADR-3000 with inductance loops and piezoelectric sensors

2.2.2.4 Forecasting

Currently, traffic forecasting is made once a year. Base year data, historic data for extrapolation, and land use information are used to predict the future AADTs and truck percentages. It is noted that ESALs are calculated by the PMU using Eq. (2-2).

2.2.2.5 Existing Data and Research Needs

Weigh-In-Motion (WIM) sites are classified into two types:

- TMG: These sites are compliant with the recommendations of the Traffic Monitoring Guide (TMG) where vehicle classification and truck weight data are collected in all lanes in both directions continuously. This procedure meets the requirements for generating class and load spectra.
- LTPP: Data are collected only in the lanes with test sections.

There are 55 sites in North Carolina that are equipped with a permanent piezoelectric-type WIM. Most of these sites meet the TMG standard. These sites include all 24 LTPP sites, but a few of them still have sensors in the test section lanes only, thus not meeting the TMG standard. The NCDOT has 1- to 10-year-long data from the 55 sites. For the LTPP sections, regardless of whether a site has sensors in all lanes or just in the test lanes, only data from the test lanes are reported. Additionally, most LTPP sites require the reporting of continuous vehicle class data, but only 1 week of truck weight data per quarter. Therefore, the LTPP database (maintained by the FHWA) has limited utility in developing class and load spectra for North Carolina. It is noted that the TSU collects truck weight data continuously and these data are suitable for developing load spectra. Hence, in order to use the LTPP data in the MEPDG, the traffic data in the LTPP database need to be augmented with the continuous truck weight data collected by the TSU.

2.2.3 Materials & Tests Unit

2.2.3.1 Design of New Pavements

It is common that specific information about layer materials is not known during the pavement design stage and, therefore, participation by the M&T Unit in the design of new pavements is mostly limited to the characterization of existing subgrade soils. Since the NCDOT is currently using the 1972 AASHTO Design Guide, the primary parameter that represents the strength of the subgrade soil is the Soil Support Value (SSV). This value is determined from the soil classification and/or CBR values of soils sampled from the project site.

The M&T Unit performs unconfined compressive strength tests on cement-treated base and stabilized soils to determine the amount of cement or lime in the layers. The minimum thicknesses of lime- and cement-stabilized layers are 8 and 7 in., respectively. A 6" thick cement stabilized layer can also be used, if road mixed. Currently, no testing is being conducted on HMA mixtures for the design of new pavements.

For the design of new concrete pavements, no testing is being conducted on any layer materials.

For the design of overlay, no testing is being performed on the overlay materials regardless of the material type (i.e., asphalt or concrete overlay). Cores are taken from existing concrete and asphalt pavements to determine compressive strength, layer thickness, and/or layer condition, such as honeycombing and stripping. Cores have always been part of the overlay construction QC/QA check.

2.2.3.2 Data Collection Methods

The following equipment items are available at the M&T Unit:

For HMA Mixtures:

- 2 Troxler gyratory compactors
- Asphalt Pavement Analyzer
- Pine Marshall Press for TSR testing

For Aggregate Base and Subgrade:

- 2 Humboldt HM3000s for CBR testing
- 1 Instron closed-loop servo-hydraulic machine for Resilient Modulus (M_R) testing of soil, aggregate, and stabilized materials. Acquired in 2004. Computer controlled. Uses pre-programmed software to run tests.

For Concrete:

- Reinhart loading machine for flexural strength testing
- UTM (Hydraulic) for compressive strength determination

2.2.4 Geotechnical Engineering Unit

2.2.4.1 Design of New Pavements

The following information is generated in the Geotechnical Engineering Unit for new pavement design regardless of the pavement type (i.e., flexible vs. rigid):

- Subsurface plan, including information on slopes, embankments, soil type, etc.
- County soil maps
- CBR values of soils and the SSV estimated from the CBR values
- Chemical stabilization
- Shelby tube samples for M_R determination by the M&T Unit

2.2.4.2 Overlay on Flexible Pavement

The following tasks are performed for existing pavements:

- Visual survey of pavement cores and conditions
- Determination of SSV values from:
  - ✓ Dynamic Cone Penetrometer values, obtained through the aggregate base course (ABC) and 18 in. into subgrade
  - ✓ Augering, to determine the depth of the ABC and to obtain soil samples
  - ✓ Soil classification and the moisture content of soil samples
  - ✓ CBR values
- Determination of regional factors, based on the water table, frost susceptibility, and soil type (typically 1.5 for the mountain region, 1 for the piedmont, and 0.5 for the coastal region)

2.2.4.3 Overlay on Rigid Pavement

No work is required to support the design of this pavement type.

2.2.4.4 Rubblization

The following tasks are performed for rubblization projects:

- Coring
- DCP testing
- Determining moisture content
- Augering to determine the depth of water table
- Visual condition survey

Testing with DCPs is done extensively in order to reduce the effect of excessive undercut.

2.3 MEPDG Design Practice

The MEPDG design approach (NCHRP, 2004.a) differs significantly from the earlier AASHTO design procedures which are empirically based on the performance equations developed using the 1950's AASHO Road Test data. MEPDG involves more of a mechanistic approach which refers to application of principles of engineering resulting in a more rational design process. A brief discussion of the principles and methodology involved in the new design guide is provided below.

2.3.1 Traffic

Traffic data is one of the four key input data elements for the pavement analysis and design process. Traffic data are typically collected using various data acquisition technologies i.e., Weigh-In-Motion (WIM), Automatic Vehicle Classifier (AVC), and Automatic Traffic Recorder (ATR). Traffic data that are unavailable at a particular site are usually borrowed from the other sites that exhibit similar traffic patterns. Due to differences in the data collection techniques, i.e., continuous coverage to simple 48 hr coverage, wide variation can be expected in the traffic data between pavement design sites.

NCDOT design procedure (NCDOT, 2000) aggregates the effect of traffic into Equivalent Single Axle Loads (ESAL) and then input to the regression-based performance equations in order to determine the material mix type and pavement thickness. MEPDG characterizes the truck volumes and loadings into axle numbers by type and load frequency distribution (axle load spectra). This design process requires a collection of detailed traffic data over the years to accurately characterize the future traffic for design. Since all the agencies may not have the resources to collect the data at this level, the guide adopted a hierarchical approach for developing the required traffic inputs. The hierarchical input level approach and its significance are presented towards the end of this chapter. The list of traffic inputs required for the design process is provided in Section 2.3.4.

2.3.2 Materials

In the previous AASHTO flexible pavement design procedures, the only material property included was layer coefficient ‘a’, a parameter empirical by nature. It was realized (NCHRP, 2004.a) that this lack of material property consideration leads to early or premature failures. Because some materials are time-temperature dependent (e.g., asphalt concrete modulus) and some are stress state dependent (e.g., unbound materials), it is important to include these factors in the M-E analysis. The design guide considers factors like these and many more in the design process resulting in more appropriate structural responses for different pavement distress modes. The material properties that are required to predict the stresses, strains and the displacements within the pavement structure when subjected to external loading (e.g., wheel load) include elastic modulus and Poisson’s ratio of the material.

Apart from the material properties, the other materials-related inputs that play a significant role are engineering index properties, thermal properties, gradation parameters. These properties help determine the temperature and moisture profiles throughout the pavement cross-section.

2.3.3 Climate

The two environmental variables that have a significant impact on the performance of flexible pavements (NCHRP, 2004.a) and hence its load carrying capacity are moisture and temperature. The other factors that also play a key role on the pavement performance are precipitation, temperature, freeze-thaw cycles, and ground water table.

The MEPDG uses an approach to study the effects of changing temperatures and moisture profiles in the pavement structure and subgrade using a sophisticated climatic modeling tool called the Enhanced Integrated Climatic Model (EICM). It simulates the behavior and characteristics of the pavement and subgrade under heat and moisture flow over the changing climatic conditions.

Most of the inputs to accomplish the climatic analysis are acquired from the weather station located near the project site using the software. The software requires at least 24 months of weather data to carry out the computations. In case of absence of a weather station near the project site, a virtual weather station can be identified from a combination of nearby weather stations.

2.3.4 Inputs Required by the MEPDG

In an effort to identify the types of input parameters for the MEPDG, the MEPDG report (NCHRP, 2004.a) and software program were studied. In the following, all the required input parameters that are needed to perform analyses using the MEPDG software are summarized.

2.3.4.1 General Information

1. Design life (years)
2. Existing pavement construction (month, year)
3. Pavement overlay construction (month, year)
4. Traffic open date (month, year)
5. Type of design
    a. Flexible pavement
    b. Jointed plain concrete pavement (JPCP)

    c. Continuously reinforced concrete pavement (CRCP)

6. Restoration

    a. JPCP

7. Overlay

    a. Asphalt concrete (AC)

        – AC over AC
        – AC over JPCP
        – AC over CRCP
        – AC over JPCP (fractured)
        – AC over CRCP (fractured)

    b. Portland cement concrete (PCC)

        – Bonded PCC/CRCP
        – Bonded PCC/JPCP
        – JPCP over JPCP – Unbonded
        – JPCP over CRCP – Unbonded
        – CRCP over CRCP – Unbonded
        – CRCP over JPCP – Unbonded
        – JPCP over AC
        – CRCP over AC

2.3.4.2 Site/Project Identification

1. Location
2. Project ID
3. Date

4. Section ID
5. Station/Milepost format (ft/miles/(latitude, longitude))
6. Station/Milepost begin
7. Station/Milepost end
8. Traffic direction (E/W/N/S)

2.3.4.3 Analysis Parameters

1. Initial IRI (in/miles)
2. Performance criteria

Table 2-4 Analysis Parameters for Rigid and Flexible Pavements

Pave. Type	Analysis Parameters	Limit	Reliability
AC	1. Terminal IRI (in/miles)	172	90
	2. AC Surface-Down Cracking (ft/miles) (Longitudinal Cracking)	1000	90
	3. AC Bottom-Up Cracking (Alligator Cracking) (%)	100	90
	4. AC Thermal Fracture (ft/miles)	100	90
	5. Chemically Stabilized Layer- Fatigue Fracture (%)	25	90
	6. Permanent Deformation- Total Pavement (in.)	0.75	90
	7. Permanent Deformation- AC only (in)	0.25	90
JPCP	8. Terminal IRI (in/miles)	172	90
	9. Transverse Cracking (% Slabs Cracked)	15	90
	10. Mean Joint Faulting (in.)	0.12	90
CRCP	11. Terminal IRI (in/miles)	172	90
	12. CRCP Existing Punchouts	10	90
	13. Maximum CRCP Crack Width (in)	0.02	90
	14. Minimum Crack Load Transfer Efficiency (LTE %)	75	90
	15. Minimum Crack Spacing (ft)	3	90
	16. Maximum Crack Spacing (ft)	6	90

2.3.4.4 Structure Inputs

Table 2-5 Layer Inputs

Layers	Drainage and Surface Properties	Structural Design Features
I. Subgrade/Foundation Inputs (Unbound Compacted/Uncompacted) 1. Material type 2. Thickness (in.) **Strength Properties** 3. Input level (1, 2 or 3) 4. Analysis type 5. Poisson's ratio 6. Coefficient of lateral pressure 7. Modulus (Input/Calculated) (psi) **ICM inputs** 8. Plasticity Index (PI) 9. Liquid Limit (LL) 10. Compacted layer (Yes/No) 11. Gradation (Mean/Range) 12. Derived parameters from gradation a. Specific gravity, Gs b. Saturated hydraulic conductivity (ft/hr) c. Maximum dry unit weight (pcf) d. Optimum gravimetric water content (%) e. Degree of saturation at optimum (%) 13. Soil water characteristic curve regression parameters (af, bf, cf, hr.)	N/A	N/A

Table 2-5 (Continued) Layer Inputs

II. AC Layer 1. Material type 2. Thickness (in.) 3. Input level **General Properties** 4. Reference temperature (°F) **Volumetric Properties as Built** 5. Effective binder content (%) 6. Air voids (%) 7. Total unit weight (pcf) 8. Poisson's ratio **Thermal Properties** 9. Thermal conductivity asphalt (BTU/hr-ft-°F) 10. Heat capacity (BTU/lb-°F) **Asphalt Mix** 11. Cumulative % retained on 3/4 in. sieve 12. Cumulative % retained on 3/8 in. sieve 13. Cumulative % retained on #4 14. % Passing #200 sieve **Asphalt Binder** 15. Superpave binder grading (or) Conventional viscosity grade (or) Conventional penetration grade 16. Corresponding A & VTS values	1. Surface shortwave absorptivity 2. Infiltration 3. Drainage path length (ft) 4. Pavement cross slope (%)	N/A
III. Chemically Stabilized Materials (Note: In case of overlays, existing JPCP/CRCP act as CSM.) 1. Material type 2. Layer thickness (in.) 3. Unit weight (pcf) 4. Poisson's ratio 5. Elastic resilient modulus 6. Thermal conductivity asphalt (BTU/hr-ft-°F) 7. Heat capacity (BTU/lb-°F)	N/A	N/A

Table 2-5 (Continued) Layer Inputs

IV. CRCP/JPCP Layer 1. Material type 2. Layer thickness (in.) 3. Unit weight (pcf) 4. Poisson's ratio **Thermal Properties** 5. Coefficient of thermal expansion (/°F* 10^{-6}) 6. Thermal conductivity (BTU/hr-ft-°F) 7. Heat capacity (BTU/lb-°F) **Mix Properties** 8. Cement type (I or II or III) 9. Cementitious material content (lb/yd3) 10. Water/Cement ratio 11. Aggregate type 12. PCC zero-stress temperature (°F) 13. Ultimate shrinkage at 40% R.H. (microstrain) 14. Reversible shrinkage (% of ultimate shrinkage) 15. Time to develop ultimate shrinkage (days) 16. Curing method **Strength Properties** 17. Input level	1. Surface shortwave absorptivity 2. Infiltration 3. Drainage path length (ft) 4. Pavement cross slope (%)	**I. CRCP** 1. Slab thickness 2. Shoulder type 3. Permanent curl/warp effective temperature difference **Steel Reinforcement** 4. Steel (%) 5. Bar diameter (in.) 6. Steel depth (in.) **Base Properties** 7. Base type 8. Erodibility index 9. Base/slab friction coefficient **Crack Spacing** 10. Cracking model (Enter mean crack spacing) **II. JPCP** 1. Slab thickness 2. Permanent curl/warp effective temperature difference **Joint Design** 3. Joint spacing (ft) 4. Sealant type 5. Random joint spacing 6. Doweled transverse joints 7. Dowel diameter (ft) 8. Dowel bar spacing (ft) **Edge Support** 9. Long-term LTE (%) 10. Slab width (ft) **Base Properties** 11. Base type 12. Erodibility index 13. Base/slab friction coefficient 14. PCC-base interface 15. Loss of bond age (months)

2.3.4.5 Flexible Rehabilitation

1. Rehabilitation level
2. Milled thickness (in.)
3. Geotextile present on existing surface (Yes/No)
4. Pavement rating (Very poor/ Poor/ Fair/ Good/ Excellent)
5. Total rutting (in.)

2.3.4.6 Thermal Cracking

1. Input level
2. Average tensile strength at 14°F (psi)
3. Creep compliance (1/psi)
4. Compute mix coefficient of thermal contraction
    a. Mixture VMA (%)
    b. Aggregate coefficient of thermal contraction
    c. Mix coefficient of thermal contraction (in./in./°F)

2.3.4.7 Climate Inputs

1. Latitude (degrees.minutes)
2. Longitude (degrees.minutes)
3. Elevation (ft)
4. Depth of water table (ft)

2.3.4.8 Traffic Inputs

1. General
    (a) Initial two-way AADTT

(b) Number of lanes in design direction

(c) Trucks in design direction (%)

(d) Trucks in design lane (%)

(e) Operational speed (mph)

2. Traffic volume adjustment factors

    1. Monthly adjustment factors

        i) Input level

    2. Vehicle class distribution

        i) Input level

        ii) AADTT distribution by vehicle class

    3. Hourly distribution

        i) Hourly truck distribution by period

    4. Traffic growth factors

        i) No growth/ Linear growth/ Compound growth

        ii) Default growth rate (%)

3. Axle load distributions

    1. Input level

    2. View (cumulative distribution or distribution)

    3. Axle types (Single/ Tandem/ Tridem/ Quad)

4. General traffic inputs

    1. Lateral traffic wander

        i) Mean wheel location (inches from the lane marking)

        ii) Traffic wander standard deviation (in.)

iii) Design lane width (ft)

2. Number of axles/truck

3. Axle configuration

    i) Average axle width (edge-edge, outside dimension (ft))

    ii) Dual tire spacing (in.)

    iii) Tire pressure (psi)

    iv) Axle spacing (tandem/tridem/quad)

4. Wheel spacing

    i) Average axle spacing (ft)

    ii) Trucks (%)

2.3.4.9 Rigid Rehabilitation

(In case of overlays, over JPCP/CRCP)

1. Existing distress

    a. Before restoration: percentage of slabs with transverse cracks plus percentage of previously repaired/replaced slabs

    b. After restoration: total percentage of slabs repaired/replaced

    c. CRCP Punchouts (per mi)

2. Foundation support

    a. Dynamic modulus of subgrade reaction (psi/in.)

    b. Modulus of subgrade reaction (month)

2.3.5 Input Level Hierarchy

The philosophy behind the hierarchical approach is that the level of engineering effort exerted in the design process should be consistent with the relative importance, size and cost of the project. Three levels of input systems are available in the MEPDG. This hierarchical approach (NCHRP, 2004.a) can be employed with regard to traffic, materials, and environmental inputs. The Level 1 input system involves comprehensive laboratory testing or field tests providing the highest level of accuracy (and thus lowest level of uncertainty or error) and also consuming more time and resources. Level 2 inputs are usually estimated through the correlations or derived from limited testing programs. For example, asphalt concrete modulus can be estimated from the binder, aggregate and mix properties. Level 3 allows the designer to estimate the most appropriate design value of the material property based on experience with little or no testing involved. There are several advantages with this hierarchical approach, including greater flexibility in selecting the appropriate engineering effort and cost-effective design.

2.3.6 DG 2002 Software

The new version of the software (0.9 and 1.0) released by the NCHRP incorporates many corrections, technical improvements and enhancements. Major changes (NCHRP, 2006.a) include software changes (such as changes to traffic data and other general topics), changes in the integrated climatic model, changes in flexible pavement design and analysis, and changes in rigid pavement design and analysis. Recommendations for the new version of software include a new climatic database, which is now substantially larger and enhanced, and verification of each input prior to re-running old files (i.e., ones generated with versions 0.7 and 0.8).

One of the major changes from the user perspective is the multiple file selection option for batch mode. This tool comes in handy when sensitivity runs (that involve hundreds of runs) are desired. Also, the time period required for each run is reduced to 30 minutes, compared to 40-plus minutes previously, thus saving a significant amount of time.

The integrated climatic model has been modified and enhanced and is now referred to as the Enhanced Integrated Climatic Model (EICM). The EICM is based on the results of the NCHRP 9-23 "Environmental Effects in Pavement Mix and Structural Design Systems" project. A new set of weather station files with up to 9 years of hourly data has been added to 851 stations; it is recommended that old weather station files be deleted and not used with the new software version. Several errors related to the original integrated climatic model (ICM) stability check have been fixed.

Several changes have been made to the software to enhance the design of flexible pavements. The analysis period (design life) has been increased from 25 to 50 years. Errors related to usage of level 1 |E*| characterization in the HMA analysis have been fixed. Also, errors related to Level 2 seasonal modulus analyses of unbound materials have been corrected.

All the flexible pavement distress models – bottom-up fatigue cracking, top-down fatigue cracking, permanent deformation, transverse cracking, and IRI – have been recalibrated (NCHRP, 2006.a). The new calibrated models have a low model error, reasonable sensitivity to change in inputs, and improved reliability compared to those originally developed under NCHRP 1-37A. All the existing sections have been updated with 4-5 additional years of performance data, traffic data, materials data, climatic data, and rehabilitation data. Model coefficients for all the models have been re-established using the new expanded database. Reliability model coefficients have also been updated.

2.4 Summary

From the above study it can be deduced that the NCDOT current design practice (based on earlier AASHTO design procedures) differs significantly from the MEPDG design practice (based on mechanistic and empirical principles). MEPDG is a more sophisticated design tool in that it considers the effect of inputs from four different modules, i.e., structure, climate, traffic and materials on the prediction of distresses. It is considered a significant improvement or a major change in the way the pavement design is performed. This approach will allow limiting a specific distress type to develop and help optimize the design with respect to cost and materials. The next chapter presents the data collection effort to obtain the necessary data to carry the sensitivity analysis and local calibration process.

CHAPTER 3 DATA COLLECTION

3.1 Introduction

The data collection process was the major hurdle during the research with the amount of data, number of data sources (different units of NCDOT, i.e., Traffic unit, Pavement Management unit, Geotechnical Engineering unit, Pavement Construction unit), and the time constraints involved. Also the quality of the data plays a major role in the calibration process and hence more efforts and time were expended in obtaining the data from the NCDOT and LTPP database. LTPP database (FHWA, 2007) provided complete data set for 30 flexible pavement sections (16 new and 14 rehabilitated sections) whereas NCDOT provided the data for 23 flexible pavement sections to be used in this local calibration and validation process. It was noted that there are significant differences in the data collection methods by LTPP and the NCDOT. For example, the pavement performance data collection methodology for the alligator cracking distress; LTPP directly measures (FHWA, 2003) the area cracked under the distress whereas NCDOT measures (NCDOT, 2006) the length of the crack with the appropriate severity rating. Hence NCDOT monitored performance data is converted into the MEPDG format before even the calibration is begun.

A complete set of database that has been developed and used in the calibration-validation process is appended (APPENDIX A: Local Calibration Database) to the thesis. The following sections explain the data collection efforts.

3.2 LTPP Database

LTPP program was one of the six strategic areas recommended by TRB during the 1980's study under the sponsorship of Federal Highway Administration (FHWA) to satisfy a wide range of pavement information needs. LTPP program was developed mainly to gain technical knowledge as well as to seek models that help better explain the pavement performance. It was also used to study the effects of materials, loading, environmental, specific design features on the performance of the pavement.

LTPP database was the most comprehensive and reliable source (FHWA, 2007) of the pavement data for North Carolina which includes all the material, traffic and climatic data along with the performance data. Apart from the rigid pavement sections, 30 of the flexible pavement sections were extracted from the database for use in the local calibration of MEPDG. All of these pavement sections are of standard length of 500 ft and are continuously monitored for the performance data.

3.2.1 North Carolina Data Extraction

As a part of the data collection process, the LTPP database has been examined in order to extract data relevant to North Carolina. The LTPP database called the Information Management System (IMS) contains the performance data that have been collected since 1989 on approximately 2500 test sections located across North America. Currently, the LTPP IMS (FHWA, 2004) consists of 16 general data modules with 430 tables containing more than 8,000 unique data elements.

There are 27 LTPP sections located in North Carolina with 24 GPS and 3 SPS test sections. Each test section can be identified (FHWA, 2004) in the LTPP database by a state code and the SHRP ID assigned to it. The state code is a two-digit code used to identify the state where a test section is located. The state code assigned in the LTPP database for North Carolina is 37. The SHRP ID is a four-digit code for the test section. For GPS test sections, the number has no significance other than being unique when combined with the state code.

For SPS sections, the second character represents the experiment number, shown in Table 3-1, and the third and fourth characters identify the sections at the project. Every section that enters the LTPP program is first assigned a construction number of 1. The construction number is incremented by 1 whenever maintenance or rehabilitation activity takes place, regardless of its impact on the pavement.

For example, 37-0201-1 (State code – SHRP ID – construction number) represents Section No. 01 of the SPS-2 pavement in North Carolina that has not had any maintenance or rehabilitation activity since it entered the LTPP program.

Table 3-1 List of LTPP Test Sections in North Carolina According to Pavement Type

Pavement Type	**[SHRP ID]-[Construction Number]**
JPCP over Unbound Base	0201-1, 0202-1, 0203-1, 0204-1, 3044-1
JPCP Over Non-Bituminous Treated Base	0205-1, 0206-1, 0207-1, 0208-1, 0260-1, 3008-2, 3807-1, 3816-1
JPCP Over Bituminous Treated Base	0209-1, 0210-1, 0211-1, 0212-1, 0259-1, 3011-2
AC with Granular Base	0801-1, 0802-1, 0859-1, 1006-3, 1024-2, 1028-2, 1030-1, 1040-2, 1352-3, 1801-2, 1802-2, 1803-3, 1814-2, 1817-5, 1992-2
AC with Bituminous Treated Base	0901-1, 0902-1, 0903-1
AC Overlay on AC Pavement	0960-2, 0961-2, 0962-2, 0963-2, 0964-2, 0965-2
AC with Non-Bituminous Treated Base	1645-2, 2819-3, 2824-3, 2825-1
CRCP - Over Unbound Base	5037-1, 5826-2, 5827-4

All the data relevant to the MEPDG have been extracted (FHWA, 2005) for all the GPS/SPS sections in North Carolina. The data available for North Carolina in the LTPP database are not comprehensive in the sense that much of the data are still missing, such as thermal conductivity, heat capacity, unit weight, PCC zero-stress temperature, ultimate shrinkage at 40% relative humidity, reversible shrinkage, time to develop ultimate shrinkage, drainage and surface properties, permanent curl/warp effective temperature difference, erodibility index, base/slab friction coefficient, cracking model, long-term load transfer efficiency, PCC-base interface, loss of bond age, depth of water table, etc.

Because the data are not available for these parameters, MEPDG recommendations (NCHRP, 2004.a) are followed. Parameters such as the erodibility index, base/slab friction coefficient, PCC-base interface, and drainage and surface properties are considered to be less sensitive and, hence, their default values are considered. If the specific gravity of the bitumen is not available in the LTPP database, a value of 1.01 is recommended (PCS/Law Engineering, 1993). Similarly if not enough data are available to calculate *Vbe*, a rough correlation with *Pb*, i.e., *Vbe* = 2**Pb* is recommended (NCHRP, 2003.a).

3.2.2 LTPP Section Performance Data

3.2.2.1 Fatigue Cracking

The LTPP database provides the fatigue cracking data for each LTPP section according to severity (low, medium and high). The three fatigue cracking severity values will be summed arithmetically (NCHRP, 2003.a) to obtain the total fatigue cracking, without using any weights for each category. Note: Each LTPP section has a length of 500 ft. For the bottom-up cracking, the summation of the measured alligator cracking will be divided by the total area of the lane (12 ft x 500 ft = 6000 ft^2) to calculate the percentage area cracked.

3.2.2.2 Rut Depth

Rut depth measurements are obtained at the surface of the pavement structure for each section in the LTPP database. The rut depth reported in the database was measured in the left and the right lanes. However, it is the average rut depth (NCHRP, 2003.a) that will be used for comparison to the predicted total rut depth in the analysis. The percentage from the predicted sublayer rutting will be multiplied by the average total rutting to approximate the true field rutting in each layer. This approach will be applied to estimate rutting in the base/sub-base and subgrade layers. This assumption is necessary in the rutting model development when the trench data are not available.

3.3 NCDOT Database

Data for the 12 pavement sections that were constructed in the year 1993 as well as the 11 sections that were constructed in 1999 are obtained from the NCDOT Pavement Management Unit (PMU) and reviewed. From the review of the data, it is understood that not all the required MEPDG inputs are available for the given sections and, hence, engineering judgment may have to be used during the calibration. The lack of reliable data from these pavements makes the calibration depend more heavily on the LTPP sections, as they are more consistent and reliable in terms of design and performance data.

Performance of the pavement sections obtained from the PMU is described using NCDOT defined ratings (NCDOT, 2006). For alligator cracking, the ratings given by the PMU must be converted to a single value in order to compare the predicted distresses to the measured value. Similarly for rutting, the rating needs to be converted to a single rut depth value in order to make a comparison and, hence, perform the calibration. Also from the review, it is understood that none of the pavement sections that were constructed in 1999 have undergone any distress (e.g., no rutting on any sections and light alligator cracking on only two sections) according to 2006 pavement condition survey files. Sections that have undergone significant rutting or alligator cracking are preferred ones. Hence, the data obtained from sections constructed in 1999 were supplemented by older pavement data (1993 sections) to calibrate the MEPDG for a wide range of distresses. Also, no information was obtained regarding rehabilitation – if any – of the sections. It is observed also that the sections constructed in 1999 represent the transition from Marshall mix designs to Superpave mix designs.

3.3.1 Traffic

The traffic data that are required for running the MEPDG are: Average Annual Daily Truck Traffic (AADTT) data, vehicle classification, axle load distribution, and number of axles per truck. For the NCDOT sections, the Traffic Unit has provided a list of WIM (Weigh-in-Motion) station locations on or near the project along with the necessary data (C-card and W-card) and recommendations. The raw data files (C-card and W-card) are then processed using the TrafLoad software version 1.08 (NCHRP, 2004.d) to obtain the vehicle classification, axle load distribution and number of axles per truck data. For projects where the WIM stations are located at the pavement sites, these data serve as Level 1 inputs to the MEPDG.

3.3.2 Materials

The pavement design files obtained from the pavement management unit contain structural information, i.e., layer thicknesses, HMA mix type, and subgrade type. Additionally, Job Mix Formulas (JMFs) and mix design sheets for the 11 sections let in year 1999 are obtained from the NCDOT to determine the HMA volumetric information. It is observed that several JMFs exist for the same mix type within a project for some of the sections. No other information is available to relate a particular JMF to a particular section in the project. It was decided to average the volumetric properties to obtain a single set of values for calibration purposes.

3.3.3 Climate

Climatic data (location coordinates) were missing for each of the 23 pavement sections provided by NCDOT. Google Earth software was used to estimate the latitude, longitude and the elevation data for these sections. Ground Water Table (GWT) depth values for most of the sections are provided by the NCDOT Geotechnical Unit. For those sections with missing data, the US Geological Survey (USGS, 2007) is consulted to obtain the GWT data using the latitude-longitude data.

3.3.4 NCDOT Section Performance Data

Pavement performance ratings for the 23 pavement sections are obtained from the NCDOT. The NCDOT Pavement Condition Survey Manual 2006 (NCDOT, 2006) was studied in order to understand the performance ratings given for alligator cracking and rutting. During the study, several issues are identified that need to be resolved even before the calibration process begins. One of the major issues is the difference in the methodology of collecting the distress data by LTPP and the NCDOT agencies

Therefore, it is inevitable that the NCDOT performance ratings are to be converted to a format that is easily comparable to the MEPDG predicted distresses. This conversion is necessary because the LTPP and NCDOT use different distress measurement processes. The LTPP program measures (FHWA, 2003) alligator cracking by directly measuring the area affected in the lane, whereas the NCDOT measures (NCDOT, 2006) it by assigning a rating based on the length of the section cracked.

The following facts about the pavement performance rating procedure (NCDOT, 2006) that is used by the Pavement Management Unit are to be noted:

1. Alligator cracking is measured in the outer wheel path of the most distressed lane.

2. It is observed that 90% of the time, the most distressed lane is the outer lane.
3. If the outer wheel path is continuously cracked, the section is rated as 100%, even if the inner wheel path exhibits some cracks; i.e., there is no way to quantify the cracking in the inner wheel path.
4. From 2002, the measurement of edge cracking (caused by the movement of soil beneath the pavement) has been combined with that of alligator cracking (caused by repeated traffic loading); i.e., there is no way to quantify or separate one type of cracking from the other.
5. The performance ratings do not indicate how much of each lane of a two-lane roadway (in an undivided case) contributes to the rating. For example, a rating of 8 for a section may mean:
   a. 4 (one lane of a two-lane roadway) + 4 (the other lane), or
   b. 5 + 3, or any other combination.

3.3.4.1 Converting the NCDOT Performance Rating to the MEPDG Format

Alligator Cracking

After reviewing the performance ratings for alligator cracking in the Pavement Condition Survey Manual (NCDOT, 2006), it was determined that the percentage of area that is cracked in the section should be calculated using the formula described below. This is based on the fact that a rating of 3 by NCDOT implies 30 percent of the length of the sections is cracked which is then multiplied by the standard wheel path width to obtain the area of the pavement section cracked. Here, a standard wheel path width of 2.5 ft is assumed in the calculations.

For the outer lane or most distressed lane,

% Area Cracked = (NCDOT Rating×Length of the section) × (Wheel path width) ×100/Section area

= (NCDOT Rating×Length of the section) × (Wheel path width) ×100/(Length of the section×Lane width)

= (NCDOT Rating×Wheel path width) ×100/(Lane width)

= (NCDOT Rating×2.5 ft×100)/(12 ft) [Standard wheel path width = 2.5 ft; Standard lane width = 12 ft]

= (NCDOT Rating×0.20833)

For the LTPP sections in North Carolina, it is noted that the distresses are monitored only in the outer lane. For the NCDOT sections, as observed from the NCDOT pavement condition survey files, it is difficult to say whether the distresses identified are in the most distressed lane or the outside lane, even considering the fact that the distressed lane is the outside lane 90% of the time.

Rutting

For rutting, the ratings (NCDOT, 2006) are given as *severe*, *moderate*, *light* and *none*, depending on the amount of rutting. These ratings will be assigned a numerical designation in order to make the comparisons during the calibration. The limiting value for each rating will be used for the level of rutting. For example, for ratings given as moderate (0.5 in. to 1 in.), a value of 1 inch will be used.

3.4 Summary

This chapter describes the data collection effort expended in obtaining the data from the LTPP and NCDOT databases. Thirty LTPP pavements (16 new flexible pavement sections and 14 rehabilitated sections) and 23 NCDOT pavement sections were obtained for use in the calibration process. The required data for the LTPP sections were obtained from the LTPP database, whereas data for the NCDOT sections were obtained from the Pavement Management Unit (structure, pavement condition survey files), Construction Unit (material and volumetric data – Job Mix Formulas, i.e., JMFs), Traffic Unit (AADTT, C-card and W-card data files) and the Geotechnical Unit (subgrade and ground water table depth data) of the NCDOT. The traffic data obtained from the NCDOT were processed using the TrafLoad (Version 1.08) software that was developed under the NCHRP 1-39 project. The map in Figure 3-1 below shows all the pavement sections (both LTPP and NCDOT) located in three distinct geographic areas (mountain, piedmont, and coastal regions, from left to right, respectively).

The next chapter describes the sensitivity analysis study to identify the inputs that are sensitive to the North Carolina conditions.

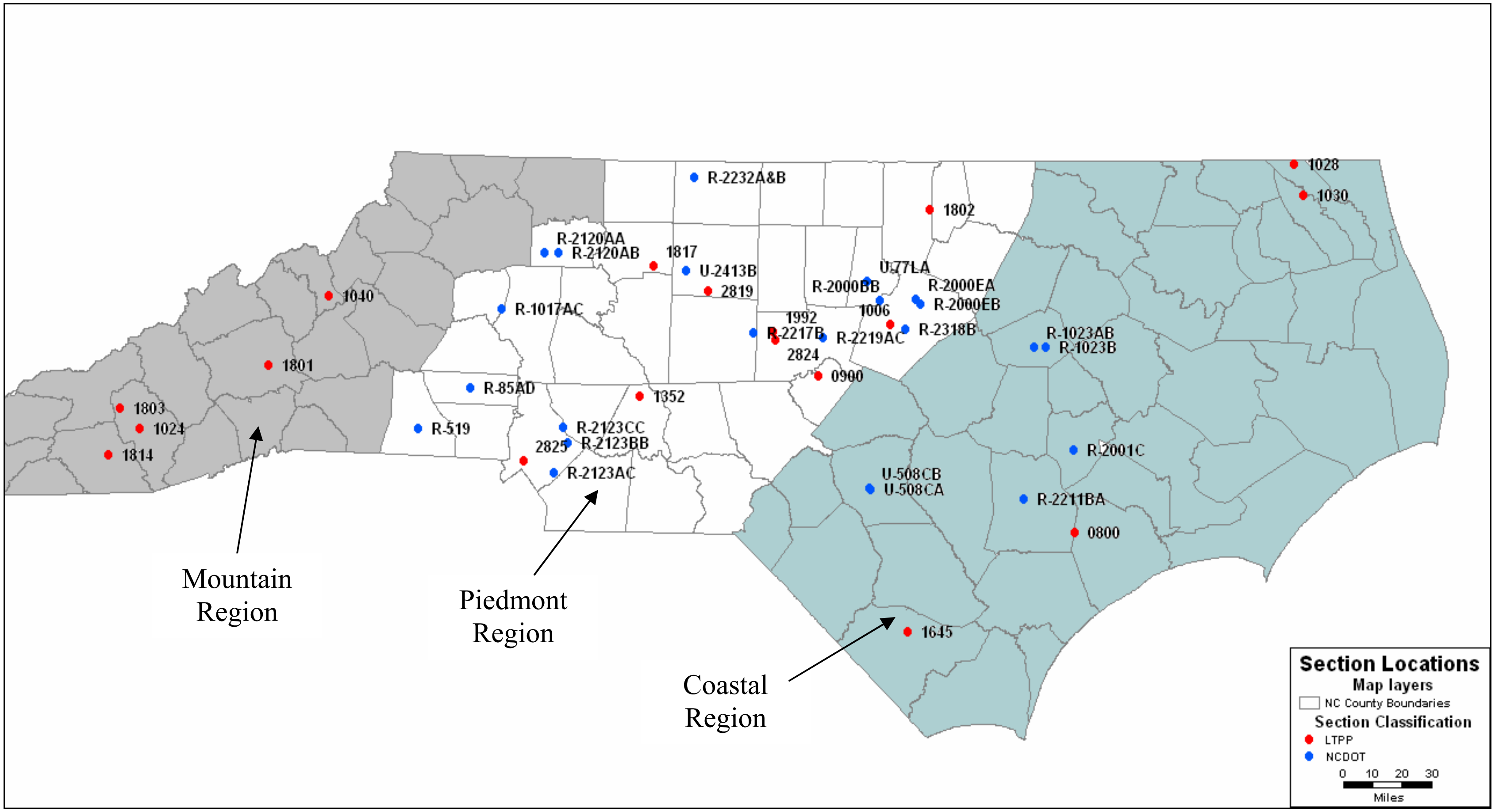

Figure 3-1 Locations of Flexible Pavement Sections Included in the Study

CHAPTER 4 SENSITIVITY ANALYSIS

4.1 Introduction

The MEPDG has been developed to be relevant to climatic and material conditions for the whole nation and, therefore, includes variables that may or may not be pertinent to North Carolina. Hence, a sensitivity analysis has been conducted in order to determine which variables are appropriate for North Carolina. Based on this analysis, an importance ranking can be assigned to each input; such a ranking significantly helps to reduce the effort and cost in obtaining the inputs that are less sensitive to the pavement performance. This study also provides a better understanding of the design parameters that affect certain pavement performances the most, thus stressing the importance of careful consideration for these parameters before the design process even begins.

Several papers have been presented at the Transportation Research Board (TRB) 2006 annual meeting regarding the MEPDG's sensitivity analysis of various input parameters for both flexible and rigid pavements. A brief summary of these papers is provided below.

4.2 Literature Review

4.2.1 Rigid Pavements

A recent study (Ceylan and Coree, 2006) employs a total of 30 input parameters. In the study, one input at a time is varied within its recommended range to study its effect on the predicted performance, while all the other inputs are assigned base case values. The study found a set of parameters that has a significant impact on various distresses. The sensitivity level of each input is categorized into one of five groups (Extremely Sensitive, Very Sensitive, Sensitive, Moderately Sensitive, Not Sensitive) based on a visual inspection of the sensitivity plots. The numerical criteria used to differentiate the sensitivity levels are not presented in the paper. In general, the curl/warp effective temperature difference, the coefficient of thermal expansion, and the thermal conductivity exhibit the greatest impacts on the distresses.

In another study (Hall and Beam, 2005), 29 inputs were evaluated one at a time. This study reports that three models (cracking, faulting, and roughness) are sensitive for only 6 out of 29 inputs and insensitive to 17 out of 29 inputs, resulting in combinations of only one or two of the distress models sensitive to 6 out of 29 inputs. However, changing only one variable at a time results in little information regarding the interaction among the variables. That is, it is not known whether the distress prediction may show sensitivity to a particular variable for all the values of the remaining variables. The sensitivity criteria used in this study for the faulting model is that the differences in total faulting after 20 years exceeding 0.1 in. were judged significant. For the cracking model, differences in the percentage of slabs cracked after 20 years exceeding 25% were judged significant. For the smoothness model, differences after 20 years exceeding 30 in./mile were judged significant. It is also mentioned that the specific numerical criteria used in this study were chosen arbitrarily, based primarily on the author's experience.

Similarly, another study (Kannekanti and Harvey, 2006) looks at the sensitivity of the models to the coefficient of thermal expansion and illustrates the sensitivity of the MEPDG faulting model to dowel diameter, slab width and edge support, built-in-temperature gradient, erodibility index of the base, and joint spacing. These researchers performed about 10,000 runs, thus enabling a study of the variable interactions as well as the single effects for the full range of projects. Overall the authors suggest that the JPCP module of the software produced reasonable predictions of the pavement performance.

4.2.2 Flexible Pavements

A sensitivity analysis study (Ceylan and Coree, 2006) evaluates 20 input parameters varying them one at a time, by using 50% reliability. As part of their study, a limited study on 2-way interactions among the inputs was also carried out by varying two inputs at a time, but no input parameter was found to be sensitive to all the performance measures. The same criteria used to determine the sensitivity level of the input parameters in rigid pavements (Section 4.2.1) was applied to flexible pavement input parameters.

Another study (Graves and Mahboub, 2006) employs global sensitivity analysis using a random sampling based technique over the entire input parameter range. A total of 100 design sections were randomly sampled using the Monte Carlo sampling routine from these input parameters, and the resulting predicted performances were analyzed using the Pearson and Spearman correlation coefficients. The results indicate that this type of sensitivity analysis may be used to identify important input parameters across the entire parameter space, utilizing a sampling-based technique where the entire input parameter space of selected input variables is sampled. These samples are then used to create a matrix of design scenarios which are then run through the MEPDG software which, in turn, predicts the various performance parameters of each design scenario. These performance outputs and the corresponding input parameters are then analyzed to evaluate the sensitivity of each input parameter with respect to each predicted distress. As the number of inputs to be evaluated increase, the number of scenarios increases significantly.

A recent study (Masad and Little, 2004) on the granular base sensitivity indicates that the base modulus and thickness have a significant influence on the IRI and longitudinal cracking. The influence of these properties on alligator cracking is approximately half of that on longitudinal cracking. The study also indicates that the granular base material properties do not seem to have an influence on the permanent deformation of the pavement. However, one study (Graves and Mahboub, 2006) shows that this type of analysis does not reflect the influence of the variation of the other parameters in the model because the inputs are varied one at a time, thus keeping the other inputs constant. Further, if predicted alligator cracking changes from 2% to 3% due to the change in a given input variable, then such a change represents a significant finding since the cracking changes by 50 percent. However, in terms of evaluating a particular design, neither 2% nor 3% is significant in selecting a pavement structure.

4.2.3 Traffic Sensitivity

A traffic sensitivity analysis study (Papagiannakis and Bracher, 2006) evaluates the potential sensitivity of the various hierarchal levels and sampling schemes for the MEPDG traffic inputs. These hierarchal levels deal with the source of traffic data that are utilized in the design, site-specific WIM, classification, volume, etc., or regional and national average values. The study illustrates that the variability of traffic data may have a significant impact on the predicted performance of the pavement system. It also shows that regional WIM data generally provide designs that overestimate the base case (continuous site-specific data) by less than 20 percent.

4.2.4 Insensitive Input Parameters

A review of the publications on sensitivity analyses conducted by different states reveals that some inputs are insensitive in all the studies (i.e. under different climates, different traffic conditions, different structures of pavements). Hence, it is reasonable to assume that these inputs do not have significant effects for North Carolina. These inputs are described below.

4.2.4.1 Rigid Pavements

The following is a list of inputs which are found to be insensitive in all the referred studies.

Table 4-1 Insensitive Parameters, from Literature Review

Faulting	Cracking	Smoothness
1. Heat capacity	1. Cement content	1. Sealant type
2. Cement content	2. Cement type	2. Dowel spacing
3. Cement type	3. Water/Cement ratio	3. PCC-base interface
4. Water/Cement ratio	4. Aggregate type	4. Erodibility index
5. Aggregate type	5. PCC zero stress temperature	5. Infiltration
6. PCC zero stress temperature	6. Reversible shrinkage	6. Drainage path length
7. Reversible shrinkage	7. Time to develop 50% shrinkage	7. Pavement cross slope
8. Time to develop 50% shrinkage	8. Ultimate shrinkage	8. PCC zero stress temperature
9. Ultimate shrinkage	9. Sealant type	9. Heat capacity
10. Sealant type	10. Dowel spacing	10. Cement type
11. Dowel spacing	11. Dowel diameter	11. Aggregate type
12. Edge support	12. Infiltration	12. Reversible shrinkage
13. PCC-base interface	13. Drainage path length	13. Time to develop 50% shrinkage
14. Erodibility index	14. Pavement cross slope	14. Ultimate shrinkage
15. Curing method	15. Curing method	15. Curing method
16. Infiltration	16. PCC-base interface	16. Traffic wander
17. Drainage path length	17. Erodibility index	17. Design lane width
18. Pavement cross slope	18. Traffic wander	18. Coefficient of lateral pressure
19. Modulus of rupture (Level 1, 2, 3)	19. Coefficient of lateral pressure	
20. Compressive strength (Level 2, 3)	20. Design lane width	
21. 20-year/28-day ratio (Level 1, 2)		
22. Modulus of elasticity (Level 1)		
23. Traffic wander		
24. Design lane width		
25. Coefficient of lateral pressure		

The inputs that are found to be common to all three distresses are cement type, aggregate type, PCC-zero stress temperature, reversible shrinkage, time to develop 50% shrinkage, ultimate shrinkage, curing method, sealant type, infiltration, drainage path length, pavement cross slope, dowel spacing, PCC-base interface, erodibility index, traffic wander, design lane width, and coefficient of lateral pressure.

4.2.4.2 Flexible Pavements

Five performance measures (alligator cracking, longitudinal cracking, rutting, transverse cracking, and IRI) are studied in all the sensitivity analyses for flexible pavements. The following parameters are found to be insensitive:

1. Aggregate thermal coefficient
2. Type of sub-base
3. Traffic wander

4.3 Sensitivity Analysis Plan

Most sensitivity analyses reported in the literature change the values of the input parameters in a systematic fashion to evaluate the effect of that change on the pavement performance. Although this approach allows a systematic evaluation of various input parameters in a straightforward manner, it may not reflect a realistic change in the various input parameters. To illustrate this point further, assume that the sensitivity analysis evaluates the effect of a ±20% change in the input values. In some inputs, a ±20% change may not be possible, whereas some other inputs may experience a much greater change than 20% under normal conditions. In order to perform the sensitivity analysis in a more realistic manner, it was decided to use LTPP pavements and data. It was found that not all the data are available in the LTPP database. The unavailability of data in the LTPP database for certain inputs does not necessarily indicate that they are not important; rather, they may turn out to be very sensitive for the pavement performance. Hence an input range has been developed based on a typical range of values as specified by the MEPDG, best guesses and values that are typically used by the NCDOT.

4.3.1 Selection of Pavements

North Carolina is divided into three distinct geographical areas – the coastal plain in the east, the piedmont in the center, and the mountains in the west – and thus experiences a wide variety of climatic conditions. All the LTPP sections are categorized according to pavement type and climatic region, as shown in Table 4-2.

Table 4-2 Pavement Classification According to Climate and Structure

No.	Pavement Type	Climatic Regions		
		Mountains	Piedmont	Coastal Plain
1	AC with Granular Base	1801-2, 1803-3, 1040-2, 1814-2, 1024-2	1817-5, 1352-3, 1992-2, 1802-2, 1006-3	0801-1, 0802-1, 0859-1
2	AC with Bituminous Treated Base		0901-1, 0902-1, 0903-1	1030-1, 1028-2
3	AC with Non-Bituminous Treated Base		2819-3, 2824-3, 2825-1	1645-2
4	JPCP over Unbound base		0201-1, 0202-1, 0203-1, 0204-1, 3044-1	
5	JPCP over Bituminous Treated Base		0209-1, 0210-1, 0211-1, 0212-1, 0259-1	3011-2
6	JPCP over Non-Bituminous Treated Base		0205-1, 0206-1, 0207-1, 0208-1, 0260-1, 3008-2, 3807-1, 3816-1	
7	CRCP over Unbound Base	5037-1, 5826-2	5827-4	
8	AC Overlay on AC Pavement		0960-2, 0961-2, 0962-2, 0963-2, 0964-2, 0965-2	

For sensitivity analysis, pavements are selected from each cell of the above table so that the structure and climatic inputs are the same among the selected pavements. Therefore, the only inputs that are left to be studied for the analysis are the traffic and material inputs.

4.3.1.1 Material Sensitivity Analysis

One LTPP section is selected from each cell in Table 4-2 above as a base case, and the variation of each material input parameter is studied for the remaining pavements in the same cell, resulting in minimum and maximum values for each input along with the base value (from the base case). Site-specific traffic data from the base pavement are used throughout the material sensitivity analysis.

For the HMA dynamic modulus, an analysis is performed to assess the use of different levels of input. Level 1 includes the input of dynamic modulus |E*| values for different temperatures and frequencies obtained from the earlier NCDOT project (Kim, King and Momen, 2004). It also includes the input of G* and phase angle data for different temperatures. Level 2 requires only the input of the mix gradation and G* values. Level 3 includes the input of the mix gradation and binder viscosity obtained from the same NCDOT project (Kim, King and Momen, 2004). Here it is assumed that the |E*|, G*, and mix gradation values represent the site-specific material properties for the LTPP sections, so that the effect of using Level 1 inputs versus the Level 3 inputs can be identified.

4.3.1.2 Traffic Sensitivity Analysis

A traffic sensitivity analysis according to the plan presented in Table 4-3 requires the estimation of regional values. To estimate regional values for a particular LTPP section, the remaining sections located all over the state are considered (irrespective of the type of pavement). There are two methods to estimate the regional values from the remaining LTPP sections: 1) classification based on functional class and 2) clustering. The Traffic Monitoring Guide (TMG) has endorsed the clustering technique as the preferred method for the estimation of regional values (FHWA, 2001). It is a mathematical approach used to establish similarities between two objects that are described by their attributes. The attributes here can be vehicle class distribution or axle load distribution. In order to identify the pattern, Euclidean distance is calculated between the attributes of the two objects (objects here refer to pavement sections). Once the Euclidean distance is calculated for the sections, clusters are identified by assuming a level of acceptable dissimilarity.

The clustering technique was applied for the 15 pavement sections falling in the piedmont region to identify the sections that exhibit similar traffic patterns as a preliminary study apart from the estimation based on functional classification. StatistiXL v 1.6, an add-on tool to Microsoft Excel was used to identify the clusters. Several clustering methods were available to determine the clusters. Here Euclidean distance was calculated using Wards Minimum Variance method which defines the distance between the two groups as the increase in the total within-group sum of squares of distances from the respective centroids.

During the main study, NCDOT preferred traffic sensitivity analysis be done based on the functional class and provide the necessary recommendations on the correctness of the method. Hence regional values were estimated by employing the functional classification method listed in the TMG. The final selections of pavements are then averaged accordingly to obtain the regional values. The default values provided in the MEPDG are used as the national values for the LTPP sections.

All sections having the most site-specific traffic data with different pavement structures and falling in one of the three regions (from Table 4-2) are considered.

Table 4-3 Traffic Sensitivity Analysis Plan

	Scenario	AADTT	Vehicle Class Distribution	MAF[b]	No. of Axles per Truck	Axle Load Distribution
WIM-SS[a] AVC-SS	1	SS	SS	SS	SS	SS
WIM-R[a] AVC-SS	2	SS	SS	SS	Regional	Regional
WIM-R AVC-R ATR-SS	3	SS	Regional	SS	Regional	Regional
WIM-N[a] AVC-R ATR-SS	4	SS	Regional	SS	National	National
WIM-N AVC-N ATR-SS	5	SS	National	SS	National	National

Note: [a]SS: Site-specific, R: Regional, N: National

[b]MAF: Monthly Adjustment Factor

Table 4-4 Pavement Classification According to Functional Class

Functional Class (FC)	FC Code	SHRP ID
Rural Principal Arterial – Interstate	1	3011, 3044, 5826
Rural Principal Arterial – Other	2	0200, 0900, 1028, 1030, 1040, 1352, 1645, 1803, 1814, 1817. 1992, 2819, 2824, 3807, 3816, 5827
Rural Minor Arterial	6	1024
Rural Major Collector	7	1802
Rural Minor Collector	8	-
Rural Local System	9	0800
Urban Principal Arterial – Interstate	11	1006, 1801, 5037
Urban Principal Arterial – Freeways or Expressways	12	-
Urban Principal Arterial – Other	14	3008
Urban Minor Collector	16	2825
Urban Collector	17	-
Urban Local System	19	-

Clustering Technique

TMG recommends several techniques to compute factor groups. Clustering technique is one of the methods recommended. The other methods include same functional class grouping or single road factor applications which are driven by professional knowledge. Some states use combination of these techniques to come up the factor groups. Once the groupings are formed, deciding where to stop is commonly determined in one of two ways. The first way is to look at the Euclidean distance between the clusters formed. There can be some major changes in the distance between two consecutive group formations which suggests that the group being formed is not very homogenous and hence is the logical point to stop the clustering process. The second common approach to ending the cluster process is to choose a predetermined number of groups to be formed. For example, an analyst might want to create no more than five factor groups.

During this study, the first approach was employed. Also, it is to be noted that only the annual tandem axle load distribution was used during the study as it is considered the most common axle load type on the pavement sections. StatistiXL v 1.6 was run on the 15 pavement sections for both axle load distribution and vehicle class distribution. The level of acceptable dissimilarity was set at 0.07 for axle load distribution and 0.14 for vehicle class distribution to identify the clusters based on the results presented in Appendix B: Clustering Analysis Technique. It is to be understood that the acceptable level of dissimilarity value varies from person to person or based on professional knowledge etc and hence the set of clusters formed may vary if another value is set. Here, three set of clusters were finally identified to estimate the regional values for both axle load and vehicle class distributions. The final sets of clusters are summarized in Table 4-5 and Table 4-6.

Table 4-5 Final Clusters for Axle Load Distribution

Final Set of Clusters for Piedmont Region, NC	Cluster 1	Cluster 2	Cluster 3
	2819	3807	1802
	1817	3008	0900
		0200	2824
		3044	1006
			1352
			1992
			2825
			5827
			3816

Table 4-6 Final Clusters for Vehicle Class Distribution

Final Set of Clusters for Piedmont Region, NC	Cluster 1	Cluster 2	Cluster 3
	2825	2824	3816
	1817	1992	2819
		5827	3008
		3807	1006
		0200	1352
		3044	0900

4.3.2 Sensitivity Runs

A total of 400-plus runs were made, including both flexible and rigid input variations (Kim and Muthadi, 2006). A preliminary set of runs were made to verify the effect of reliability on the sensitivity of the inputs. From the literature review (Ceylan and Coree, 2006), it is noted that the reliability factor incorporated in the software is questionable and is still under debate. It is also noted from the design reliability models presented in the MEPDG that the estimation of 90% reliability is just a simple reflection of 50% reliability results adjusted by a corresponding standardized normal deviate. Hence, to verify the observation, a set of runs using both 50% and 90% reliability were made. The preliminary analysis indicates that the sensitivity of the inputs has not changed at all by changing the reliability from 90%

to 50%. Hence, it was decided that a 50% reliability (based on the mean inputs) will be used in all the sensitivity runs.

4.3.2.1 Variation in the Input Data

The degree to which variation should be considered in the sensitivity analysis is determined from the variation observed in each input among the sections belonging to the same cell (seen in Table 4-2), as this indicator provides a realistic picture of the actual variation. For each input, a set of low, base and high values are prepared. Inputs such as traffic direction and design life that do not have any impact on the pavement performance are kept constant. For those inputs that are unavailable in the database, either the NCDOT typical input range or the MEPDG default range is used.

4.3.3 Determination of the Sensitivity of Input Parameters

The sensitivity of each input is then determined from the MEPDG performance measures observed from each variation. The distresses that are observed during this analysis are alligator cracking, longitudinal cracking, thermal cracking, rutting and smoothness for flexible pavements, and transverse cracking, punchouts, joint faulting and smoothness for rigid pavements. From the observed variations in the predicted distresses, each input can be categorized as *insensitive* or *sensitive* or *extremely sensitive*. Here, the change in distresses (due to the change in input from minimum to maximum) is measured with respect to the distress target, which remains constant throughout the analysis, giving rise to absolute sensitivity rather than relative sensitivity (which is measured with respect to the minimum distress predicted, i.e., (maximum-minimum)*100/minimum). The relative sensitivity keeps changing from case to case as both the minimum and maximum values change. For example, if the maximum alligator cracking predicted for a particular case is 3% and the minimum alligator cracking predicted is 2%, then the percentage change in distress turns out to be 50%. However, neither 2% nor 3% represent a significant amount of distresses. For the same prediction of distresses, according to absolute sensitivity, the percentage change in distresses turns out to be 4%, which is insensitive according to the classification given in Table 4-7. The percentage values defining the sensitivity levels in Table 4-7 are determined purely based on the visual inspection of the sensitivity tables and charts prepared during the NCDOT project (Kim and Muthadi, 2006). It is to be understood that choosing a different numerical criteria gives rise to new sensitivity classifications.

Table 4-7 Sensitivity Classification

Pavement Distresses	**Distress Target**	**Change in Predicted Distress with Respect to Distress Target**				
		<20%	**20-40%**	**40-60%**	**60-80%**	**>80%**
Terminal IRI (in./mi)	172	*I*[a]	*LS*[a]	*S*[a]	*VS*[a]	*ES*[a]
AC Surface-Down Cracking (Long. Cracking) (ft/500)	1000	*I*	*LS*	*S*	*VS*	*ES*
AC Bottom-Up Cracking (Alligator Cracking) (%)	25	*I*	*LS*	*S*	*VS*	*ES*
Permanent Deformation (AC Only) (in.)	0.25	*I*	*LS*	*S*	*VS*	*ES*
Permanent Deformation (Total Pavement) (in.)	0.75	*I*	*LS*	*S*	*VS*	*ES*
Transverse Cracking (% slabs cracked)	15	*I*	*LS*	*S*	*VS*	*ES*
Mean Joint Faulting (in.)	25	*I*	*LS*	*S*	*VS*	*ES*
CRCP Punchouts (/mi)	10	*I*	*LS*	*S*	*VS*	*ES*

Note: [a]I: Insensitive, LS: Less Sensitive, S: Sensitive, VS: Very Sensitive, ES: Extremely Sensitive.

4.4 Results and Analysis

4.4.1 Material Sensitivity

According to the proposed plan for sensitivity analysis, each input is varied within the limits (Kim and Muthadi, 2006) to observe the predicted distresses. The observed distresses are then analyzed in accordance with the sensitivity classification provided in Table 4-7. A discussion regarding the sensitivity of inputs and the steps necessary to make changes to the data collection strategy is provided in Section 4.4.5.

4.4.2 Traffic Sensitivity

Traffic inputs are varied in accordance with the input variation method (Kim and Muthadi, 2006) proposed in Table 4-3. Each scenario is then compared with the rest of the scenarios to differentiate the amount of effort needed along with the sensitivity. A discussion regarding the sensitivity of traffic scenarios along with the data collection strategy are provided in Section 4.4.5.

4.4.3 Climate Sensitivity

A study has been conducted (Kim and Muthadi, 2006) to check the sensitivity of the ground water table depth value to the distresses. In this study, a pavement section is selected such that the climatic data must be interpolated from the nearby stations and the ground water table depth value is manually input. In each case, the ground water table depth value is varied from 5 ft to 50 ft at intervals of 5 ft. The resulting performance data are studied to understand the effect of ground water table depth values on the pavement distresses.

From the study, it is observed that ground water table depth values do not influence the rigid pavement distresses whereas they have a significant impact on flexible pavement distresses, such as longitudinal cracking and alligator cracking. Alligator cracking decreases steadily with an increase in the ground water table depth value. It is supported by the fact that water table depth values closer to the surface, i.e., 5 ft to15 ft, result in an appreciable reduction of the moduli of the base and subgrade which increases the tensile strain at the bottom of the asphalt layer. For longitudinal cracking, 10 ft is found to be the critical value because a sudden rise in distress value occurs at 10 ft and then decreases thereafter. As the ground water table depth moves away from the surface, it increases foundation support which causes larger tensile strain at the surface layer.

4.4.4 Summary

The following section discusses the results of the sensitivity analysis. It also discusses the possible factors that affect the sensitivity of the distresses. International Roughness Index (IRI), which is the common distress for both types of pavements, is discussed first and the rest of the distresses are discussed under their respective pavement classification type.

International Roughness Index

For flexible pavements, neither the material inputs nor the traffic inputs are found to be sensitive towards IRI. In rigid pavements, especially JPCP, the coefficient of thermal expansion, thermal conductivity, joint spacing and dowel diameter are found to be sensitive to IRI, whereas in CRCP the percentage of steel and the bar diameter have a significant influence on IRI. The IRI model is a function of several distresses occurring in each pavement type and hence depends indirectly on the inputs that affect those distresses.

4.4.4.1 Flexible Pavements

Alligator Cracking

In general, air voids and the |E*| of asphalt concrete are the only two inputs that are found to be sensitive to alligator cracking. In particular, this distress is found to be sensitive to the percentage of air voids in base AC layers. The increase in alligator cracking can be explained by the wide range of percentage of air voids used (6-11%). Volumetric properties play an important role in governing the fatigue damage. The greater the effective volume of bitumen and the lower the air void percentage, the higher the fatigue life.

Longitudinal Cracking

Air voids, the |E*| and surface shortwave absorptivity have a significant influence on longitudinal cracking. The same explanation provided in the above section regarding the use of a wide range of air voids percentages holds here. The influence of the |E*| can be explained by the fact that the use of lower stiffness wearing course material increases the surface tensile strains, thus increasing the top-down cracking, or the use of thick or stiff layers in the upper portion of the structure of the pavement likewise increases the surface tensile strains.

AC Rut Depth

The AC rut depth is influenced by the |E*| of the AC (all layers – surface, intermediate and base) and also by surface shortwave absorptivity. The use of low stiffness layers increases the rut depth in the asphalt layer. As the surface shortwave absorptivity increases, the temperature in the top layers of the pavement structure increases, which in turn affects the stiffness of the layer. As a result, the rut depth in the asphalt layer increases.

Total Rut Depth

The total rut depth is influenced mainly by the |E*| of asphalt concrete. The use of low stiffness asphalt layers decreases the relative stiffness of the structure and, thus, increases the total rutting.

4.4.4.2 Rigid Pavements

Transverse Cracking

The inputs that are found to be sensitive to transverse cracking are the coefficient of thermal expansion, heat capacity, thermal conductivity, load transfer efficiency, and joint spacing. These inputs have significant impact within the MEPDG default range. An increase in transverse cracking can be expected by the increase in the coefficient of thermal expansion or the increase in joint spacing or decrease in load transfer efficiency.

Mean Joint Faulting

The coefficient of thermal expansion, load transfer efficiency, and dowel diameter has a significant influence on mean joint faulting, whereas thermal conductivity and joint spacing have a relatively low impact. The dowel diameter and dowel spacing are the critical design inputs and are very sensitive to mean joint faulting. An increase in dowel diameter decreases the stress that concrete has to bear and the joint faulting. Load transfer efficiency across the transverse joints is the most critical factor controlling JPCP joint faulting, which in turn affects smoothness.

Punchouts

The coefficient of thermal expansion, thermal conductivity, % of steel, and bar diameter is sensitive to punchouts. An increase in the % of steel results in fewer punchouts.

In summary, the following tables list the inputs that are sensitive in accordance with the type of distresses.

Table 4-8 Flexible Pavements – Sensitive Inputs

Terminal IRI	Longitudinal Cracking	Alligator Cracking	AC Rut Depth	Total Rut Depth
None	• Air voids • Dynamic modulus • Surface shortwave absorptivity • Subgrade modulus	• Air voids of base AC layers • Dynamic modulus	• Dynamic modulus • Surface shortwave absorptivity	• Dynamic modulus

Table 4-9 Rigid Pavements – JPCP – Sensitive Inputs

Terminal IRI	Transverse Slab Cracking	Mean Joint Faulting
• Coefficient of thermal expansion • Thermal conductivity • Joint spacing • Dowel diameter	• Coefficient of thermal expansion • Joint spacing • Heat capacity • Thermal conductivity • Load transfer efficiency	• Coefficient of thermal expansion • Dowel diameter • Thermal conductivity • Load transfer efficiency • Joint spacing

Table 4-10 Rigid Pavements – CRCP – Sensitive Inputs

Terminal IRI	CRCP Punchouts
• Percentage of steel • Bar diameter	• Coefficient of thermal expansion • Thermal conductivity • Percentage of steel • Bar diameter

4.4.5 Input Data Collection Strategy

The MEPDG requires 100-plus inputs to model traffic, climate and materials to predict the performance over the design life of pavement. This procedure differs widely from the current design practice at the NCDOT. Hence, to implement the MEPDG, the current input data collection strategy requires many changes, including equipment that allows for a smooth transition. As aforementioned, the MEPDG requires four major input data, i.e., traffic, climate, material, and structure. A set of recommendations is provided below for each of these inputs, according to the results and analysis presented in the above section.

4.4.5.1 Traffic

The axle load distribution data available in the LTPP database are given on a yearly basis, which differs from the input data format required by the MEPDG (month by month data) and, therefore, is distributed equally among the months (i.e., a MAF of 1). This type of distribution may not capture the seasonal variation in the traffic data.

Also, the data are available in Microsoft Access format in the LTPP database, which must be converted to .alf (axle load files in text form) format before inputting in the MEPDG, which is time-consuming. If the axle load distribution data collected by the NCDOT in the future are tuned to the format directly readable by the MEPDG, a significant amount of time can be saved. This can be accomplished now by using the TrafLoad software developed under NCHRP 1-39 Project (NCHRP, 2004.d) to convert the raw files generated by NCDOT WIM stations (C-card and W-card) to the MEPDG format.

4.4.5.2 Climate

The Enhanced Integration Climatic Model (EICM) incorporated in the MEPDG software reads the climatic data directly from the weather stations located all over the nation. During the design process, the designer needs to choose the weather station nearest the pavement construction site. If none of the existing weather stations are within a reasonable distance from the pavement site, then a set of weather stations are chosen so that the software can interpolate the climatic data. In such a case, the ground water table depth (GWT) values are input manually. For this study, only 23 of the several weather stations present in North

Carolina are read directly by the MEPDG; however, only one climatic station in the mountain region is built in to the software.

The MEPDG recommends that the GWT values must be characterized as accurate as possible because they play as significant role in the overall accuracy of the foundation/pavement moisture contents and hence, the equilibrium modulus values. United States Geological Survey (USGS) maintains a huge database of the ground water site inventory providing both historical and real time GWT values (USGS, 2007) for sites located all over the united states. The latitude and longitude values of a pavement section can be used to obtain the GWT value for that particular section. The accurate determination of GWT is important because, the depth to the water table controls the moisture content regime and the modulus of the layer above it. Also, the distance or radius of influence depends on the material type. The infiltration from the surface and the granular base or sub-base materials controls the moisture regime of the upper subgrade.

4.4.5.3 Materials

Asphalt Layer

Air voids in the asphalt layers (surface, intermediate and base) have turned out to be the most sensitive parameter for longitudinal cracking and alligator cracking for the input variation practiced by the NCDOT (6-11%) and, hence, needs careful consideration in achieving the proper air void content during the actual field pavement construction.

For the |E*| of the asphalt layer, Level 1 data are compared to Level 3 data to determine the sensitivity level along with Level 2 data in some cases. It can be observed from the tables presented in Section 4.4 that changing the input data from Level 1 to Level 3 is very sensitive to asphalt layer rut depth and longitudinal cracking. A dynamic modulus database (Kim et al., 2004) has already been developed for typical |E*| values for 42 various asphalt mixes in North Carolina. These values can be used as Level 2 inputs for future designs without any additional testing.

Heat capacity and thermal conductivity are found to be insignificant for asphalt layers. In order to use input Level 1, it is recommended by the MEPDG to develop values by direct measurement using the ASTM E1952 Procedure for Thermal Conductivity and the ASTM D2766 Procedure for Heat Capacity.

Surface shortwave absorptivity is found to be sensitive for three of the distresses. It is recommended that this parameter be estimated through laboratory testing. Although there are procedures in existence to measure shortwave absorptivity, no current AASHTO certified standards for paving materials exist. Hence, it will suffice to use the default values provided by the MEPDG until further notice.

Granular Base and Subgrade Layers

Level 1 analysis for granular base layers and subgrade layers is not recommended in the MEPDG until further notice because the FEM analysis model in the current version is not yet calibrated. For the present study, the MEPDG default ranges of values are used. It is recommended to develop a database of resilient modulus values for all types of bases and subgrade layers available in North Carolina.

4.4.6 Limitations

The following limitations are identified from the version 0.800 of the MEPDG during this project:

1. The MEPDG at present has a default limit of a 20-year design life for flexible pavements. The main drawback of this limitation is that the pavement life cannot be estimated

because the distresses were predicted only up to 20 years.

2. The MEPDG does not have the batch mode option enabled in this version, thus requiring a human presence to change the input after each run.
3. In the current study, a default value of 1 is used as the MAF (Monthly Adjustment Factor) due to a lack of sufficient data. This use of a default value does not capture the seasonal variation in the traffic data.
4. Two-way interaction of the input variables is not studied as part of this project. This kind of study significantly increases the number of sensitivity runs, which could not be performed within the available time limit.
5. It is to be noted that the MEPDG is still in its draft form, and many changes are expected in its final version.
6. It is recommended by the MEPDG that treated sub-base should not be analyzed using this version as the model is not yet calibrated.
7. The case with the use of Level 1 inputs in unbound layers is similar. The finite element method analysis (Level 1) is not yet calibrated in the current version of the MEPDG.
8. The computational time of each run takes at least 30-40 minutes for one analysis to finish, even on a highly configured PC.
9. The MEPDG, unlike the previous AASHTO methods, requires 100-plus inputs and concepts which were previously unknown to most of the pavement design engineers. Hence, a training program is required that summarizes the concepts used in the MEPDG.
10. Increasing the layer thickness to reduce distresses, as opposed to earlier design practices, is not the only solution for improving pavement performance. Many inputs are interrelated or interact with each other to predict the pavement performance.

11. It should be noted that even though a sensitivity analysis was carried out using the MEPDG that was nationally calibrated, the locally calibrated MEPDG may not result in the same sensitivity analysis results.

4.5 Recommended Implementation Guidelines

4.5.1 General Recommendations

1. Immediate implementation of the MEPDG as the only approach to pavement design is not feasible and, hence, the NCDOT should allow itself at least three to five years for general implementation to adjust to the new input data collection strategy. Meanwhile, the local calibration effort should take place.
2. Implementation would be unsuccessful if the MEPDG is not calibrated, verified and validated for the local conditions in North Carolina. The LTPP database will be used extensively for the verification and calibration processes. The recommended approach to the verification and calibration of the MEPDG is described in the next chapter.
3. It is now recognized, and is recommended as the preferred approach by the MEPDG, to use axle load spectra, because these can accurately characterize the axle loads better than the earlier procedures based on ESALs and AADT.
4. A local training program for the NCDOT engineers should be included as a necessary part of the implementation process to teach basic principles of the MEPDG and to provide NCDOT engineers with hands-on experience. It is estimated that four days are needed to conduct an effective local training workshop. The workshop can be given in four consecutive days or can be divided into four separate sessions with a few days to one week apart.

5. Due to an ongoing debate as to how the reliability model works, it is recommended that 50% reliability criteria are used until further notice. In the present study, both 50% and 90% reliability cases are run in order to check their sensitivity to the pavement distresses. It is found that none of the distresses are sensitive to the reliability.
6. As a general design approach, increasing the layer thickness to reduce the amount of pavement distresses is not the only solution to improve the performance in the MEPDG because many inputs are interconnected or multiple interactions exist between the inputs that provide the predicted performance measures.

4.5.2 Specific Recommendations

1. Develop a pavement design manual similar to the existing IPDG used by the NCDOT to summarize the new concepts and procedures suggested by the MEPDG.
2. A set of acceptable pavement performance criteria for both flexible and rigid pavements needs to be developed. Due to the lack of availability of performance criteria from the NCDOT, MEPDG default criteria are used in the present sensitivity study.
3. Develop the traffic database to provide accurate load spectra, project-specific truck volumes, and growth, which are critical elements to the design process due to the sensitivity of the models to these inputs.
4. Develop a set of MAFs that capture the actual seasonal variation in the traffic. The use of site-specific MAFs may have a significant effect on the distresses predicted. The MEPDG suggests that MAFs can turn out to be a significant parameter.
5. Establish realistic input values for material properties such as heat capacity, thermal conductivity etc., for HMA, PCC and CSM materials. Because these properties are very important with regard to sensitivity, it will be helpful to develop these values to be as

accurate as possible. ASTM procedures have been suggested by the MEPDG to determine these values in the laboratory.

6. During design iterations, variation of sensitive inputs results in wide variation in the prediction of pavement distresses. Hence, it is recommended to use the sensitivity analysis results presented in this chapter to identify the sensitive inputs and hence keep the predicted pavement distresses within the allowable range.

4.6 Summary

The sensitivity analysis study presented in this chapter helps identify the inputs that are sensitive to North Carolina. Since many of the inputs required by MEPDG (more than 100) are found to be sensitive during the study; which are previously unknown to the pavement design engineers, a data collection strategy is developed to obtain those inputs. Finally a set of guidelines and recommendations are developed for the implementation of MEPDG in North Carolina. The important issue in the implementation of the MEPDG in the NCDOT operation is that local conditions need to be taken into account for the calibration. The local calibration methodology implemented in this project is presented in Chapter 5.

CHAPTER 5 DEVELOPMENT OF LOCAL CALIBRATION FACTORS

5.1 Introduction

There are two NCHRP research projects (NCHRP, 2005) that are closely related to the objectives of this research. They are the NCHRP 9-30(001) and NCHRP 1-40B projects. Under the NCHRP 9-30(001) project, pre-implementation studies involving verification and recalibration have been conducted in order to quantify the bias and residual error of the flexible pavement distress models included in the MEPDG. Based on the findings from the NCHRP 9-30(001) study, the NCHRP 1-40B project focuses on the calibration refinement study of the existing load-related distress prediction models for flexible pavements and HMA overlays. The approaches taken in these two studies are reviewed and form the basis for the verification and calibration study of the MEPDG models in this research.

5.2 Literature Review

A literature review has been conducted on the MEPDG calibration procedure. For this calibration process, the local calibration approach recommended by the NCHRP 1-40B (NCHRP, 2005) project will provide an excellent framework. The following section presents the verification and calibration refinement study completed under NCHRP 1-40B project (NCHRP, 2005). It also presents the discussion of the models to be calibrated, procedure used for the development of national calibration factors, and the local calibration methodology implemented in this project.

5.2.1 Verification

For the verification study, a completely independent data set with different input levels from the sections that have at least three distress surveys (in order to avoid potential error and variability in measuring surface distress) should be used to predict the distresses for both new and rehabilitated flexible and rigid pavements. These pavements should not have been included in the NCHRP 1-37A (MEPDG prediction models) calibration process.

These data are used to create the input data files for MEPDG software for each verification section and then reviewed for reasonableness and consistency. A database is then created to store the predicted and measured distresses once the software is executed for the input data file. The accuracy of the prediction models is assessed by comparing the bias and residual error between the original calibration process (NCHRP, 2004.a) and the verification runs for different distress prediction models and then determining whether the residual error and bias are locally dependent and significantly different from the global calibration errors.

The test sections include only the newly constructed sites, thus excluding the rehabilitated test sections due to the finding that errors exist in the rehabilitation part of the software version submitted to the NCHRP in 2004 (i.e., errors related to the cumulative cracking predicted in HMA overlays and the rutting calculations in HMA overlays for Level 2 inputs).

The calibration error constitutes most of the distress measurement error and the materials input error, of which the distress measurement error is the most difficult to reduce because it is independent of the predictive capability of the model. The material input error, however, can be reduced by using the laboratory repeated load resilient modulus values for unbound materials and soils in the calibration process, including the use of a supplemental

HMA mixture characterization test. The calibration coefficients of the rut depth prediction model for each layer should be revised based on the rutting measured from the trench cuts and other destructive techniques. Global and local calibration procedures should consider estimating permanent deformation and fracture characteristics from other physical properties of the HMA mixtures, such as the effect of mixture type on the local calibration coefficients for rutting and fatigue cracking.

5.2.2 Calibration Refinement

Calibration refinement of the MEPDG distress prediction models for flexible pavements and HMA overlays can be achieved in two ways. One method is to use the mixture properties that are currently available within the LTPP database with no additional testing. The other method is to use additional mixture tests such as repeated load permanent deformation and indirect tensile strength tests to improve upon the calibration process.

Irrespective of the calibration process, the final calibrated models will have errors associated with them. Such errors are called the standard error of the estimate and can be used to establish confidence intervals for the predictive equation. This error explains the scattering of data across the 1:1 line between the predicted and measured distresses and is composed of four major components that include measurement (distress) error, input (testing) error, model lack-of-fit error and pure error (error due to replication).

Many of the test sections used for the recalibration effort include the sections that are used for verification studies. The selection of these independent pavement sections is similar to their selection for the original calibration effort (NCHRP, 2004.a); data from at least three distress surveys can be easily extracted from the existing databases. The test sections should have input Levels 1 and 2 data available for most of the input parameters.

5.2.3 Permanent Deformation Model

Permanent deformation (or rutting) is considered one of the most important load-associated distresses in flexible pavement systems. It is usually caused by the accumulation of permanent strains in all or some of the layers in the pavement structure. Rutting appears as longitudinal depressions along the roadway causing roughness (and hence serviceability or IRI), hydroplaning and other safety concerns. Figure 5-5-1 below shows the permanent deformation occurring in a flexible pavement structure.

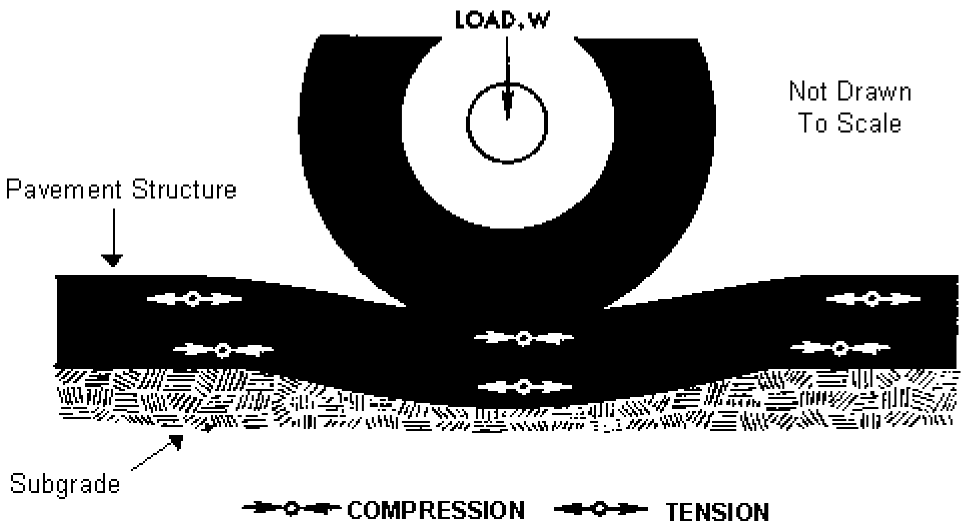

Figure 5-5-1 Permanent Deformation in the Flexible Pavement Structure

(***Source:*** http://www.ncdot.org/doh/Operations/dp_chief_eng/constructionunit/paveconst/Asphalt_Mgmt/qms_manual/2004/sect3.htm)

Design guide employs the incremental damage approach (NCHRP, 2004.a) to calculate the damage or rutting in each sub-layer. Once the material type of each sub-layer is identified, appropriate model is used by the system to calculate the accumulated plastic strains at the mid-depth of each sub-layer in each sub-season. The total permanent deformation is then calculated as the sum of permanent deformation in each sub layers and is mathematically expressed as:

$$RD = \sum_{i=1}^{nsublayers} \Delta RD_i \tag{5-1}$$

$$RD = \sum_{i=1}^{nsublayers} \varepsilon_p{}^i h^i \tag{5-2}$$

where;

RD = Pavement permanent deformation

nsublayers = Number of sublayers

$\varepsilon_p{}^i$ = Total plastic strain in sublayer i

h^i = Thickness of sublayer i

5.2.3.1 Permanent Deformation in Asphalt Mixtures

The empirical model that is used to predict rutting in the asphalt mixtures is of the form:

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_{r1} a_1 T^{a_2 \beta_{r2}} N^{a_3 \beta_{r3}} \tag{5-3}$$

The national field calibrated model (NCHRP, 2004.a) that is finally implemented in the MEPDG to calculate the permanent deformation in the asphalt layers is presented below (This equation contains the most recently re-calibrated national calibration factors):

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z * 10^{-3.35412} T^{1.5606} N^{0.479244} \tag{5-4}$$

$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth} \tag{5-5}$$

$$C_1 = -0.1039 * h_{ac}^2 + 2.4868 * h_{ac} - 17.342 \tag{5-6}$$

$$C_2 = 0.0172 * h_{ac}^2 - 1.7331 * h_{ac} + 27.428 \tag{5-7}$$

where;

ε_p = Accumulated plastic strain at N repetitions of load (in/in)

ε_r = Resilient strain of the asphalt material as a function of mix properties, temperature and time rate of loading (in/in)

N = Number of load repetitions

T = Temperature (^oF)

a_1, a_2, a_3 = Non-linear regression coefficients

k_z = Function of total asphalt layers thickness (h_{ac}, in) and depth (depth, in) to computational point, to correct for the confining pressure at different depths

$\beta_{r1}, \beta_{r2}, \beta_{r3}$ = National calibration factors

5.2.3.2 Permanent Deformation in Unbound Materials

The final calibrated model (NCHRP, 2004.a) to predict rutting in the unbound materials (unbound granular and subgrade materials) is of the form:

$$\delta_a(N) = \beta_1 \left(\frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{N}\right)^\beta} \varepsilon_v h \tag{5-8}$$

where;

δ_a = Permanent deformation for the layer/sublayer (in)

N = Number of traffic repetitions

$\varepsilon_o, \beta, \rho$ = Material properties

ε_r = Resilient strain imposed in laboratory test to obtain above listed material properties, ε_o, β, and ρ (in/in)

ε_v = Average vertical resilient strain in the layer/sublayer as obtained from the primary response model (in/in) h = Thickness of the layer/sublayer (in)

B_1 = Calibration factor for unbound granular and subgrade materials

5.2.4 Bottom-Up Fatigue Cracking Model

Load associated fatigue cracking is also one of the major distress types occurring in flexible pavement sections. It is usually caused by repeated traffic loading on the pavement structure inducing tensile and shear stresses in the bound layers. This will eventually result in loss of structural integrity and hence the development of cracks. It is believed that the fatigue cracking normally initiates at the bottom of the asphalt layer due to bending action caused by traffic loadings and propagate to the surface. Numerous world wide studies have later demonstrated that fatigue cracking may also be initiated from the top and propagate down (top-down or longitudinal cracking). It was later recommended by the NCHRP 1-40B study (NCHRP, 2005) that the longitudinal cracking model be dropped from the local calibration guide development due to lack of accuracy in the predictions. Therefore only the bottom-up cracking (Alligator cracking) model is considered in this study. Figure 5-5-2 shows the bottom-up cracking pattern that is commonly seen in the flexible pavement structure.

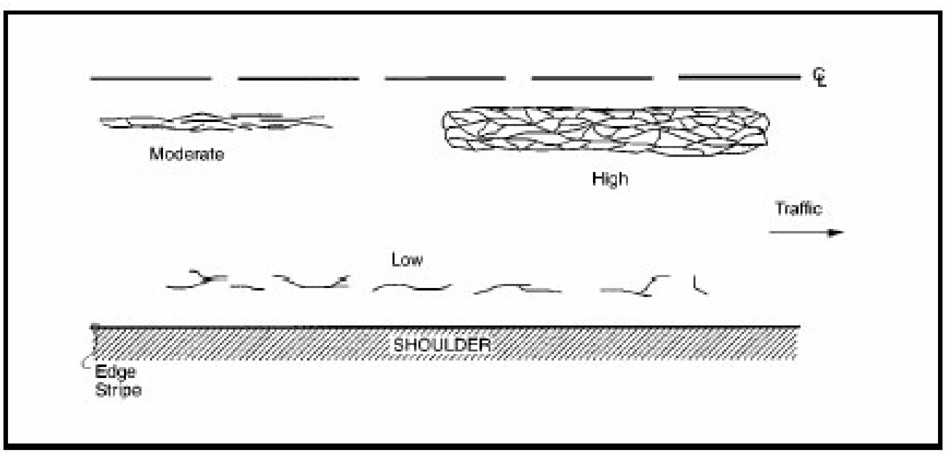

Figure 5-5-2 Bottom-up Fatigue Cracking in the Flexible Pavement Structure (***Source:*** http://www.tfhrc.gov/pavement/ltpp/reports/03031/01.htm#fatigue)

Design guide utilizes cumulative damage concept approach (NCHRP, 2004.a) based on Miner's law to predict the fatigue cracking in the flexible pavement sections. The damage is calculated as the ratio of predicted number of load repetitions to the allowable number of repetitions in that particular period.

$$D = \sum_{i=1}^{T} \frac{n_i}{N_i} \tag{5-9}$$

where;

D = Damage

T = Total number of periods

n_i = Actual traffic for period i

N_i = Traffic allowed under conditions prevailing in period i

The model form that is used to predict the number of load repetitions is a function of tensile strain (at bottom of the asphalt layer) and mix stiffness. The Equation below presents the mathematical relationship used for the fatigue characterization.

$$N_f = \beta_{f1} k_1 \left(\varepsilon_t\right)^{-\beta_{f2} k_2} \left(E\right)^{-\beta_{f3} k_3} \tag{5-10}$$

The final model form used for the prediction of number of repetitions to fatigue failure is presented below (This equation contains the most recently re-calibrated national calibration factors):

$$N_f = 0.007566 * k'_1 * C \left(\frac{1}{\varepsilon_t}\right)^{3.9492} \left(\frac{1}{E}\right)^{1.281} \tag{5-11}$$

$$C = 10^M \tag{5-12}$$

$$M = 4.84 \left(\frac{V_b}{V_a + V_b} - 0.69\right) \tag{5-13}$$

$$k'_1 = \frac{1}{0.000398 + \frac{0.003602}{1 + e^{(11.02 - 3.49 * h_{ac})}}} \quad (5\text{-}14)$$

where;

N_f = Number of repetitions to fatigue cracking

ε_t = Tensile strain at critical location

E = Stiffness of the material

k_1, k_2, k_3 = Laboratory regression coefficients

$\beta_{f1}, \beta_{f2}, \beta_{f3}$ = Calibration parameters

C = Laboratory to field adjustment factor

V_b = Effective binder content (%)

V_a = Air voids (%)

h_{ac} = Total thickness of the asphalt layers, in.

The k'_1 in the above equation is introduced to provide correction for different asphalt layer thickness effects.

$$FC_{bottom} = \left(\frac{6000}{1 + e^{\left(C_1 * C'_1 + C_2 * C'_2 * \log_{10}(D * 100)\right)}} \right) * \left(\frac{1}{60} \right) \quad (5\text{-}15)$$

$C_1, C_2 = 1.0$

$C'_1 = -2 * C'_2$

$C'_2 = -2.40874 - 39.748 * (1 + h_{ac})^{-2.856}$

where;

FC_{bottom} = Bottom-up fatigue cracking, % lane area

D = Damage (%)

The 6000 in the fatigue cracking model above is the total area of the lane (12 ft wide and 500 ft long). The (1/60) value is a conversion to obtain the cracking in percentage, not in square feet.

5.2.5 National Calibration Procedure

The calibration procedure used to develop national calibration factors during the NCHRP 1-37A project, for fatigue cracking model and permanent deformation model is presented in this section. The calibration procedure in general involved the following four important steps:

1. Collection of the calibration (performance) data for each field section from the LTPP database.
2. Simulation runs using the MEPDG software and different sets of calibration coefficients in the performance model.
3. Comparison of the predicted damage to the measured cracking observed in the field obtained from each calibration coefficient combination.
4. Correlation of the predicted damage with the measured cracking in the field by minimizing the square of errors, as the final step.

The two main requirements of calibrating the performance data models are to ensure that all the major factors that influence the development of pavement distress are included and that the selected pavements span the expected range of each factor. The approach requires the selection of a desired number of field sections as well as desirable attribute values (ranges) of key factors that follow generally accepted experimental statistical concepts. The approach emphasizes the recognition of key parameters for the factors of interest, selection of the appropriate number of levels of factors, and the selection of the number of replicates within each cell of the experiment design. These experiments are designed to statistically test the hypothesis, to determine the bias in the predictions, to determine the cause of the bias, and to determine the calibration function. The field sections were selected randomly so that a well-balanced matrix of salient pavement parameters and distresses are present in the experiment. The models were evaluated finally on the basis of bias, precision, and accuracy. Several criteria were considered in selecting and prioritizing sites for the use of calibration. Some of these criteria are listed below.

1. Test sections that have at least three distress surveys were given a high priority in the site selection process.
2. Test sections with the fewest structural layers and materials were given high priority to reduce the data collection process.
3. Field sections that were equipped with continuous WIM were given high priority.

5.2.5.1 Bottom-Up Fatigue Cracking Model National Calibration

The following section discusses the steps that are specific to the calibration of the bottom-up fatigue cracking model (NCHRP, 2004.c). One of the important steps in this calibration process is to find the most accurate transfer function, which will predict the damage relative to the observed measured cracking in the field. Before the calibration process begins, the field data must be checked for general reasonableness, and any trends from these data must be examined. The next step of the calibration process is to derive an appropriate shift function relating the asphalt thickness and the cracking. The model that has been chosen from the preliminary analysis is the MS-1 model presented below:

$$N_f = 0.00432 * C * \beta_{f1} \left(\frac{1}{\varepsilon_t} \right)^{3.291*\beta_{f2}} \left(\frac{1}{E} \right)^{0.854*\beta_{f3}} \tag{5-16}$$

where

N_f = number of repetitions to fatigue cracking;

ε_t = tensile strain at the critical location;

E = stiffness of the material;

$\beta_{f1}, \beta_{f2}, \beta_{f3}$ = calibration factors.

The three calibration factors (β_{f1}, β_{f2}, β_{f3}) must be estimated by minimizing the error in the prediction of the damage. The damage transfer function used in the calibration of the fatigue cracking (bottom-up) model in the MEPDG takes the form of a mathematical sigmoidal function shown in Equation (5-17). This model form was chosen based on two assumptions

1. Sigmoidal function is the best representative of the relationship between cracking and damage assuming the relationship is bounded by 0 ft^2 cracking as a minimum and 6000 ft^2 cracking as a maximum.
2. The alligator cracking is 50% of the total lane area when the damage is 100%.

$$F.C = \left(\frac{6000}{1 + e^{C_1 - C_2 * LogD}} \right) * \left(\frac{1}{60} \right) \tag{5-17}$$

where

F.C = fatigue cracking (% of lane area);

D = damage (%); and

C_1, C_2 = regression coefficients.

The regression coefficients C_1 and C_2 are obtained using the Microsoft Solver numerical optimization routine. The optimization is set by first predicting the damage for each alligator cracking data point, and then an initial value is assumed for C_1 and C_2. The equation presented above is then used to calculate fatigue cracking from the predicted damage percentage. The predicted damage is then compared with the measured cracking to obtain the total sum of squared errors. The obtained sum of squares is minimized using the Microsoft Solver by changing the C_1 and C_2 values for the first iteration. The new C_1 and C_2 values are then used as the input for the second iteration. This procedure is repeated until the solution converges and a minimum total sum of squares is obtained. In order to eliminate any bias in the prediction, Microsoft Solver is used again to set the arithmetic sum of errors to zero by changing the C_1 and C_2 values.

Estimation of β_{f2} and β_{f3}

Ten different simulations were run (NCHRP, 2004.c) using the MEPDG software for various combinations of calibration factors. For each coefficient, a calibration factor (β_{fi}) was introduced to eliminate the bias and scatter in the predictions; these factors are the ones used to calibrate the model to the actual field performance. Several combinations of these calibration factors (β_{f1}, β_{f2}, β_{f3}) have been used to identify the best possible combinations. The values used for the calibration factor on the strain (β_{f2}) are 0.8, 1.0 and 1.2 whereas 0.8, 1.5 and 2.5 were used for the modulus calibration factor (β_{f3}). The lab regression coefficients (k_2, k_3) used are 2.5 to 5 for strain and 0.8 to 2 for the modulus. Theses values were determined based on the fatigue models available in the literature (NCHRP, 2004.c).

To compare these runs for various β_{f2} and β_{f3} values, an optimization was done to minimize the error in the predicted damage using the fatigue cracking damage transfer function and the measured cracking. The simulation run that had a β_{f2} of 1.2 and β_{f3} of 1.5 provides a realistic prediction with minimum standard error. In order to provide a correction for the thin asphalt layer thicknesses, the calibration factor (β_{f1}) was introduced since the MS-1 model had been developed using constant stress theory (thick AC sections). It was also observed from the LTPP sections that sections with thicknesses greater than 4 in. were grouped together, whereas sections with thicknesses less than 4 in. have a higher damage percentage. These sections were eventually shifted using the factor β_{f1}.

Shifting of thin sections

To find the equation relating β_{f1} and the AC layer thickness (NCHRP, 2004.c), the sections were first divided into four groups (<2 in., between 2 in. and 3 in., between 3 in. and 4 in., and > 4 in.). β_{f1} was divided into two parameters, β'_{f1} and k'_1, where β'_{f1} is a constant and k'_1 is a function of the AC thickness. The sections with a thickness greater than 4 in. were found to have shift factors of 0.004. A shift factor was developed manually for the three other groups to match the damage from the fourth group. A sigmoidal function was then fitted for the shift factors for each AC thickness group with the AC thickness variable as an independent variable. The $k`_1$ parameter is given by

$$k`_1 = \frac{1}{0.000398 + \dfrac{0.003602}{1 + e^{(11.02 - 3.49 * h_{ac})}}} \tag{5-18}$$

where h_{ac} = the thickness of the asphalt concrete layers.

The optimization techniques used to minimize the errors constitute one form of field verification. The important component of an overall model verification approach should be that the final model provides reasonable correlations to known impacts of significant variables in the fatigue process.

5.2.5.2 Permanent Deformation Model National Calibration

The following section discusses the steps that are specific to the permanent deformation model calibration (NCHRP, 2004.b). It is to be noted that there are five calibration factors (three for asphalt layers, one for the unbound granular base and one for the subgrade materials) in the permanent deformation calibration. These factors are applied to the initial theoretical models to eliminate the bias and scatter in the predictions.

The first step involved the simulation runs using the software for a combination of β_{r2} and β_{r3} on the asphalt model only. The initial values utilized for these two calibration factors are 0.8, 1.0, and 1.2. Two additional runs for the (β_{r2}, β_{r3}) values of (0.9, 1.2) and (1.0, 1.1) were run to achieve a closer optimal solution. The optimization was done using two approaches for each combination of β_{r2} and β_{r3}.

1. The first method required to keep granular base and subgrade calibration factors at a certain value, minimize the total rut sum of squared error and then set the asphalt layer sum of error to zero varying β_{r1} only.
2. The second method required to minimize the total rut sum of squared error using all the three-calibration factors (β_{r1}, β_{GB}, and β_{SG}) at the same time, and then set the sum of error of the total rut to zero.

From the optimization results using both the methods, the best combination for β_{r2} and β_{r3} was found to be 0.9 and 1.2 respectively. The next step was to determine the calibration factors β_{GB}, and β_{SG}. The final selection of the unbound rutting model calibration was based on the unbound layer rut depth using a matrix of pavement designs generated using the 1993 AASHTO design guide. The matrix incorporated four traffic levels, four subgrade CBRs, and one climatic location. A number of simulations were run to determine the best set of calibration factors. From the results, the calibration factors of $\beta_{GB} = 1.67$ and $\beta_{SG} = 1.35$ were finally selected. The final step in the calibration process was to determine the final calibration factor for the asphalt layer rut model. The calibration factor (β_{r1}) which is a direct multiplier on the asphalt rut model posed a problem of increasing rut with increasing thickness. Hence a correction factor (k_1) was introduced as a function of asphalt layer thickness to overcome the limitation. A final optimization was run varying the β_{r1} factor, after the model is being corrected for the confining pressure-depth effect, resulting in a value of 0.509.

5.3 Local Calibration Plan

The local calibration process involves three important steps (NCHRP, 2007) for calibrating MEPDG to local conditions and materials. The first step of the local calibration plan is to perform the verification runs on the pavement sections using the calibration factors that were developed during the national calibration of the performance prediction models. A null-hypothesis test will be evaluated between the measured and predicted distresses to determine if there is a significant bias. If the test fails, it is recommended to calibrate the models to local conditions leading to the second step. The second step involves the calibration of the model coefficients, i.e., elimination of the bias and minimization of the standard error between the predicted and measured distresses. Once the bias is eliminated and the standard error is within the agency's acceptable level after the calibration, the final step, i.e., validation is performed on the models to check for the reasonableness of the performance predictions. The following sections briefly explain the steps involved in the local calibration process:

5.3.1 Verification

As a first step of the local calibration, verification runs are performed (NCHRP, 2007) on the pavement sections using the national calibration factors to check the goodness of fit between the measured and predicted distresses. The pavement sections should be selected carefully in such a way that they have at least three pavement distress surveys available and are never part of the national calibration effort that took place under NCHRP 1-37A project. Once the MEPDG is executed for these sections, the predicted distresses are compared to the measured distresses to compute the residual errors, bias, and standard error for each distress model. A null hypothesis test will be evaluated in order to assess if there is any bias (systematic difference) between the measured and predicted distress values. If the null hypothesis is rejected, then the specific models should be recalibrated to the local conditions and materials.

5.3.2 Calibration

This step constitutes the major step in the local calibration effort. The calibration process involves minimizing the sum of squared error between the measured and the predicted distresses by varying the calibration factors. For some distress models, specifically alligator cracking model; which calculates the incremental damage rather than the actual distress magnitude; the process involves calibration of the transfer function relating the measured cracked pavement area to the cumulative damage values predicted. It is suggested in the NCHRP 1-40 project report (NCHRP, 2007) to recalibrate the distress prediction models by adjusting the respective coefficients (k_1 coefficient for rutting and C_2 or k_1 coefficient for fatigue cracking) to eliminate the bias. Similarly in order to reduce the standard error, the k_2, k_3 coefficients for rutting and k_2, k_3, and C_1 for fatigue cracking are adjusted respectively.

5.3.3 Validation

This is the final step in the local calibration process (NCHRP, 2007) to check for the reasonableness of the performance predictions using the newly developed calibration factors.

A few set of pavement sections that were kept aside from the calibration process will be used to do the reasonableness check. If the standard error obtained during the validation process is not significantly different from the global standard error obtained during the national recalibration process, then the validation is considered successful. A Chi square test will be performed to check the same. The final set of calibration factors with the acceptable standard error are then entered into the MEPDG software for future pavement design use.

5.4 Summary

This chapter discusses the literature review of the two pre-implementation studies that are closely related to this project. Also, it summarizes the national calibration procedure employed to calibrate the permanent deformation and alligator cracking models during the NCHRP 1-37A project. A brief discussion of the local calibration methodology is provided towards the end of the chapter. The next chapter presents the implementation of local calibration plan in a step-by-step manner as suggested in the NCHRP 1-40B project report along with the results and analysis.

CHAPTER 6 RESULTS AND ANALYSIS

6.1 Introduction

NCHRP 1-40B project report (NCHRP, 2007) presents the recommendations and guidelines to carry out the local calibration in a step-by-step manner. These steps are followed in calibrating the permanent deformation model and alligator cracking model for local conditions and materials. The flow chart presented in Figure 6-1 shows the local calibration plan in a detailed manner.

The whole process is grouped into four stages: data extraction and evaluation, verification, calibration, and validation.

6.2 Data Extraction and Evaluation

STEP 1: This step involves the selection of hierarchical input level for the inputs to be used in the calibration process. For material, traffic and climate modules, the most detailed level of inputs were selected for all the pavement sections. For few of the inputs that are missing or unavailable in the database, either the NCDOT typical input range or the MEPDG default range is used.

STEP 2: An experimental matrix is developed as part of this step similar to the one suggested in the NCHRP 9-30 project (NCHRP, 2003), in order to observe the local dependency of bias and standard error. Table 6-1 presents the experimental matrix for this study. Each column in the table represents a change of structure and mixture. Traffic and climate are interrelated to HMA thickness and asphalt binder grade respectively.

STEP 3: The number of pavement sections selected for the rutting and alligator cracking calibration are well above the minimum requirement suggested in the NCHRP 1-40B report.

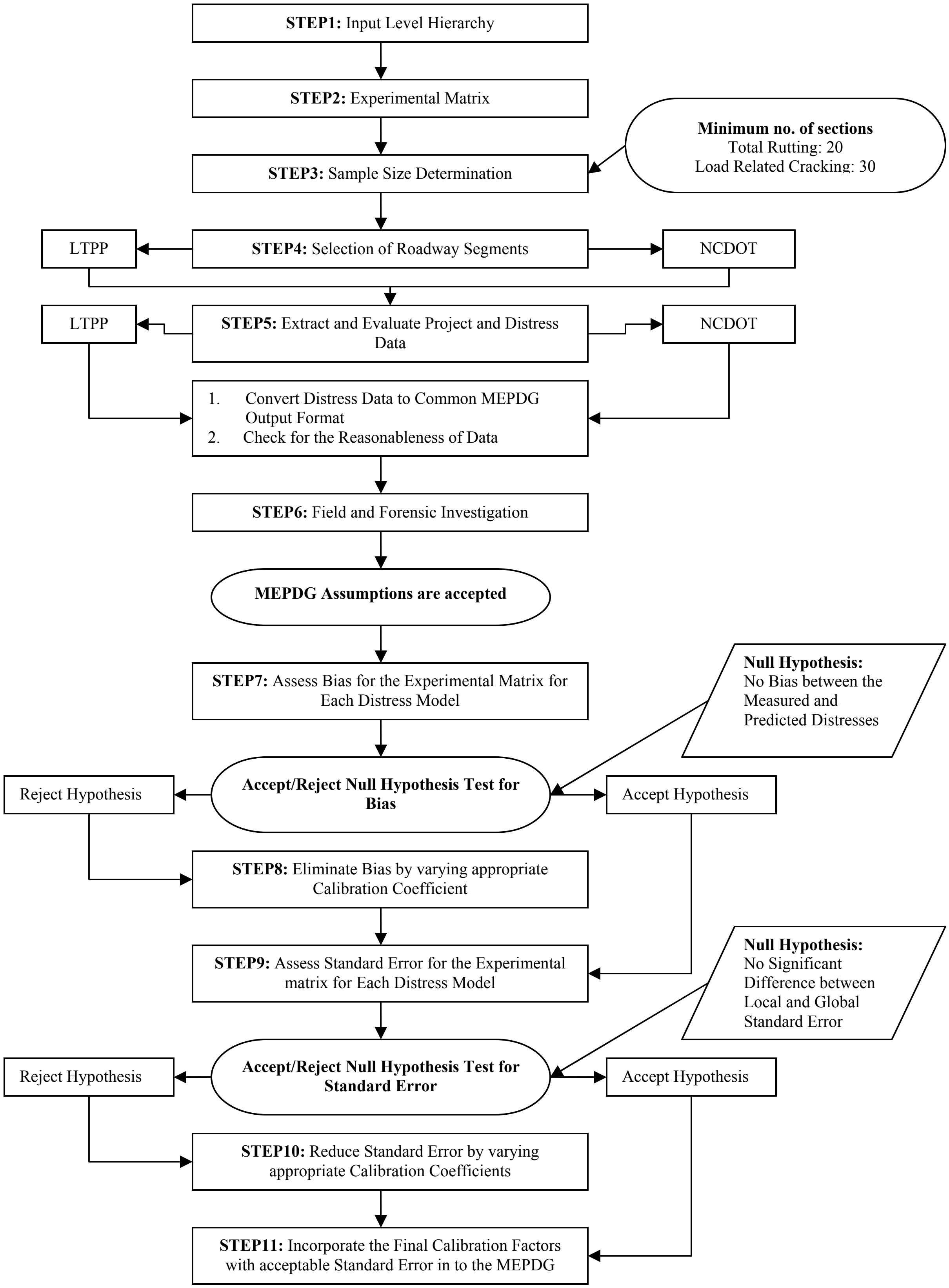

Figure 6-1 Flow Chart showing the Local Calibration Plan

Table 6-1 Experimental Matrix

Pavement Type	HMA Thickness	Mix Type	Family of Pavements					Total No. of Sections
			AC[a]-GB[b]	AC-ATB[c]	AC-Full depth	AC-CTB[d]	Rehab.	
New	Thin (<= 5 in.)	Conventional	1024,1040,1802,1803, 1817,1992	1028,1030		2819,2825		10
		PMA[e]						
		Grading						
		Drainage				2824		1
	Intermediate	Conventional	1352,1801,1814,R2120AA, R2120AB,R1023B,R1023A B,R2001C,R2318B, R2211BA			1645		11
		PMA						
		Grading						
		Drainage						
	Thick (>=8 in.)	Conventional	R2123AC,R2123BB, R2123CC,R2219AC,		R2217B,U508CB, U508CA,R1017A C,R519, R85-AD	R2000EA,R2000EB, R2000BB,U77LA, R2232A,R2232B		16
		PMA						
		Grading						
		Drainage	1006					1
Rehab.	Thin (<= 5 in.)	Conventional						
		PMA						
		Grading						
		Drainage						
	Intermediate	Conventional					1028,1040,1802, 1352,1803,1817, 2819	7
		PMA						
		Grading						
		Drainage					2824	1
	Thick (>=8 in.)	Conventional					1024,1645,1801 1814,1992	5
		PMA						
		Grading						
		Drainage					1006	1
Total No. of Sections			21	2	6	10	14	53

Note: [a]Asphalt Concrete, [b]Granular Base, [c]Asphalt Treated Base, [d]Cement Treated Base, [e]Polymer Modified Asphalt

STEP 4: There are three types of experimental test sections that can be included in the local calibration process, i.e., Accelerated Pavement Testing (APT) pads, full-scale APT experiment test tracks, and long-term full-scale roadway segments. The latter category is grouped in two types - PMS segments and research grade segments (e.g., LTPP). The pavement sections that are finally selected for the calibration process are long-term full-scale roadway segments consisting of LTPP and NCDOT sections.

STEP 5: This step involves extraction and evaluation of the data for the selected roadway segments. All the data for the LTPP sections are extracted from the LTPP database and is discussed in Chapter 3 (Data Collection). For the NCDOT sections all the data are extracted from the NCDOT database. The NCDOT section performance data is converted to the MEPDG format to maintain consistency while comparing the measured distresses to the predicted. The conversion procedure is presented again in Chapter 3. All the data are evaluated for reasonableness check and any presence of irrational trends or outliers in the data are removed from the calibration database.

STEP6: MEPDG assumptions are accepted in assigning the total surface permanent deformation to each individual layer and as well as the location of crack initiation for alligator cracking. No field and forensic investigation were undertaken during this study.

6.3 Verification

STEP7: The verification results presented in Table 6-2 and Table 6-3 include both the new and rehabilitated LTPP pavement sections that were not used in the original calibration-validation process that was completed under the NCHRP 1-37A project (NCHRP, 2004.a). The sections selected for this activity have the best data availability with all the reasonableness check done prior to the verification runs. MEPDG software (version *1.0*) was executed for all the sections selected for this process with no change in the calibration factors (i.e., national calibration factors). They also show the effect of including the NCDOT sections on the standard error and the coefficient of determination of the models. The standard error obtained during the verification runs for the LTPP sections is comparable to the global standard error for both the models. It is observed that the NCDOT sections tend to increase only the sum of the squared error between the measured and predicted values.

Table 6-2 Verification Results - Summary of Statistics for the Permanent Deformation Predictions

Comparison of Results		AC	GB	SG	Total
Verification Runs –LTPP Sections Only	***Average (in.)***	0.102	0.047	0.134	0.2828
	Percentage (%)	48.7	18.8	32.6	100.0
	R^2	0.197	0.545	0.653	0.340
	S_e^a ***(in.)***	0.057	0.037	0.081	0.111
	SS_e^b	0.514	0.219	1.044	1.962
	N^c	161	161	161	161
Verification Runs –NCDOT Sections Included	***Average (in.)***	0.086	0.025	0.121	0.232
	Percentage (%)	42.8	13.0	44.2	100.0
	R^2	0.254	0.309	0.214	0.142
	S_e ***(in.)***	0.162	0.055	0.113	0.330
	SS_e	1.380	0.387	3.822	10.387
	N	255	255	255	255
National Calibration	***Average (in.)***	0.162	0.055	0.113	0.330
	Percentage (%)	46.6	16.7	36.7	100.0
	R^2	0.648	0.677	0.136	0.399
	S_e ***(in.)***	0.063	0.023	0.045	0.121
	SS_e	1.883	0.243	0.931	6.915
	N	387	387	387	387
National Re-calibration	R^2	0.64	0.785	0.708	0.577
	S_e ***(in.)***	0.045	0.026	0.045	0.107
	S_e/S_y	0.713	0.502	0.576	0.818
	N	334	334	334	334

Note: [a]Standard Error, [b]Sum of Squared Error, [c]Number of Data Points

Table 6-3 Verification Results - Summary of Statistics for Alligator Cracking Predictions

Comparison of Results		Alligator Cracking
Verification Runs – LTPP Sections Only	***S_e*[a]*(%)***	10.7
	***SS_e*[b]**	8505.51
	***N*[c]**	76
Verification Runs – NCDOT Sections Included	***S_e(%)***	6.02
	SS_e	29487.1
	N	176
National Calibration	***S_e (%)***	6.2
	SS_e	17663.91
	N	461
National Re-calibration	***S_e(%)***	5.01
	N	405

Note: [a]Standard Error, [b]Sum of Squared Error, [c]Number of Data Points

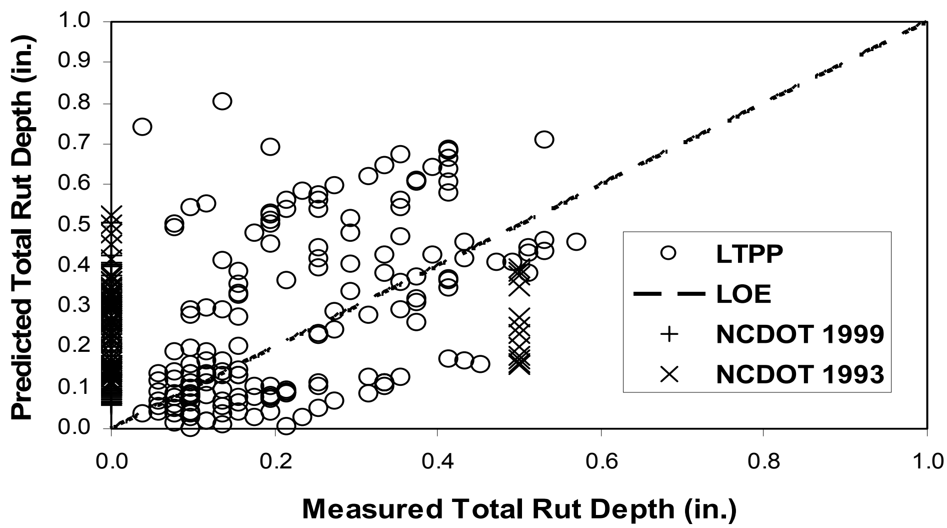

Figure 6-2 Verification Results - Total Measured Rut Depth vs. Predicted Rut Depth

From the verification results shown in Figure 6-2, it can be observed that the NCDOT measured rut depths do not match the predicted rut depths particularly well. This observation is attributed to the fact that the NCDOT rut depth measurement techniques (NCDOT, 2006) result in a single non-numeric rating (i.e., low, medium, or severe) and, therefore, unless more objective data are collected, these measurement techniques cannot be used in the calibration or fitting process of the permanent deformation model. Thus, only the LTPP sections were used for the calibration of the permanent deformation model.

A null hypothesis test is performed on the verification results to check for the presence of bias. Bias here is defined as average of the residual errors between the measured and predicted distresses. The hypothesis here is that no significant differences exist between the measured and predicted values. A p-value greater than 0.05 (alpha) signifies that no significant difference exists between the measured and predicted values and, hence, the hypothesis is accepted. From Table 6-4 and Table 6-5, it can be observed that the null hypothesis is rejected for both the distress models (permanent deformation and alligator cracking) for the verification runs and hence bias needs to be eliminated by recalibrating the models to local conditions and materials.

6.4 Calibration

STEP 8: As the null hypothesis for the verification runs is rejected (from STEP 7), bias is calculated in each block of the experimental matrix to determine if the bias is dependent on the local conditions and materials (e.g., traffic and climate). Since there are not many sections in the experimental matrix; especially that the NCDOT sections are discarded for the permanent deformation model; in-depth observations could not be made. For permanent deformation; in general, for the same mix type, bias increased and standard error decreased with increasing HMA thickness.

To eliminate the bias, the coefficients of the respective distress prediction models will be varied as suggested in the NCHRP 1-40B report (NCHRP, 2007). For the HMA permanent deformation model, a fitting process using Microsoft Excel Solver is employed to eliminate the bias between the predicted and measured HMA rut depth values by varying k_1 factor. Similarly, for the unbound permanent deformation model, β_{GB} and β_{SG} are varied to eliminate the bias between the predicted and measured granular base and subgrade values, respectively. MEPDG assumptions are used in assigning the total rutting measured at the surface to each pavement layer, because field forensic studies with trenches and cores were not performed as part of this study. The calibrated rut depths of the individual layers are then combined for comparison to the measured total rut depth.

Table 6-4 Calibration Results - Summary of Statistics for the Permanent Deformation Predictions

	AC		**GB**		**SG**		**Total**	
	Before	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**
Average (in.)	0.1178	0.1030	0.0442	0.0344	0.1551	0.1026	0.3171	0.2399
S_e^a (in.)	0.054	0.047	0.027	0.021	0.084	0.056	0.154	0.109
Bias (in.)	-0.0149	0	-0.0098	0	- 0.0525	0	- 0.0771	0
p-value	0.00622 <0.05 (alpha)	0.499 >0.05 (alpha)	0.0002 <0.05 (alpha)	0.5> 0.05 (alpha)	1.6E-9 <0.05 (alpha)	0.5 > 0.05 (alpha)	2.9E-7 <0.05 (alpha)	0.499 >0.05 (alpha)
N^b	111	111	111	111	111	111	111	111

Note: [a]Standard Error, [b]Number of Data Points

Table 6-4 presents the standard error and bias before and after the calibration effort of the permanent deformation model. Standard error is obtained by taking the positive square root of the variance of the statistic. The total standard error obtained after the calibration is comparable to the global standard error presented in Table 6-2; also, the bias is completely eliminated. A Chi-square test, which is typically used to gauge the success of validation, was performed in order to determine if the local standard error is significantly different from the global standard error. STEP 9 presents the discussion.

Figure 6-3 shows the comparison between the measured and predicted rut depths before and after calibration for each layer. The comparisons of total rut depth before and after calibration are shown in Figure 6-4(a) and Figure 6-4(b) respectively.

For the alligator cracking model, the only possibility that errors may occur lies in the distress transfer function (statistical model), assuming that the mathematical models (or conceptual models) are accurate simulations of real-world conditions. Hence, a fitting process using Microsoft Excel Solver is used to minimize the sum of the squared errors of the predicted and measured cracking values by varying the C_1 and C_2 parameters in Eq. (5-15).

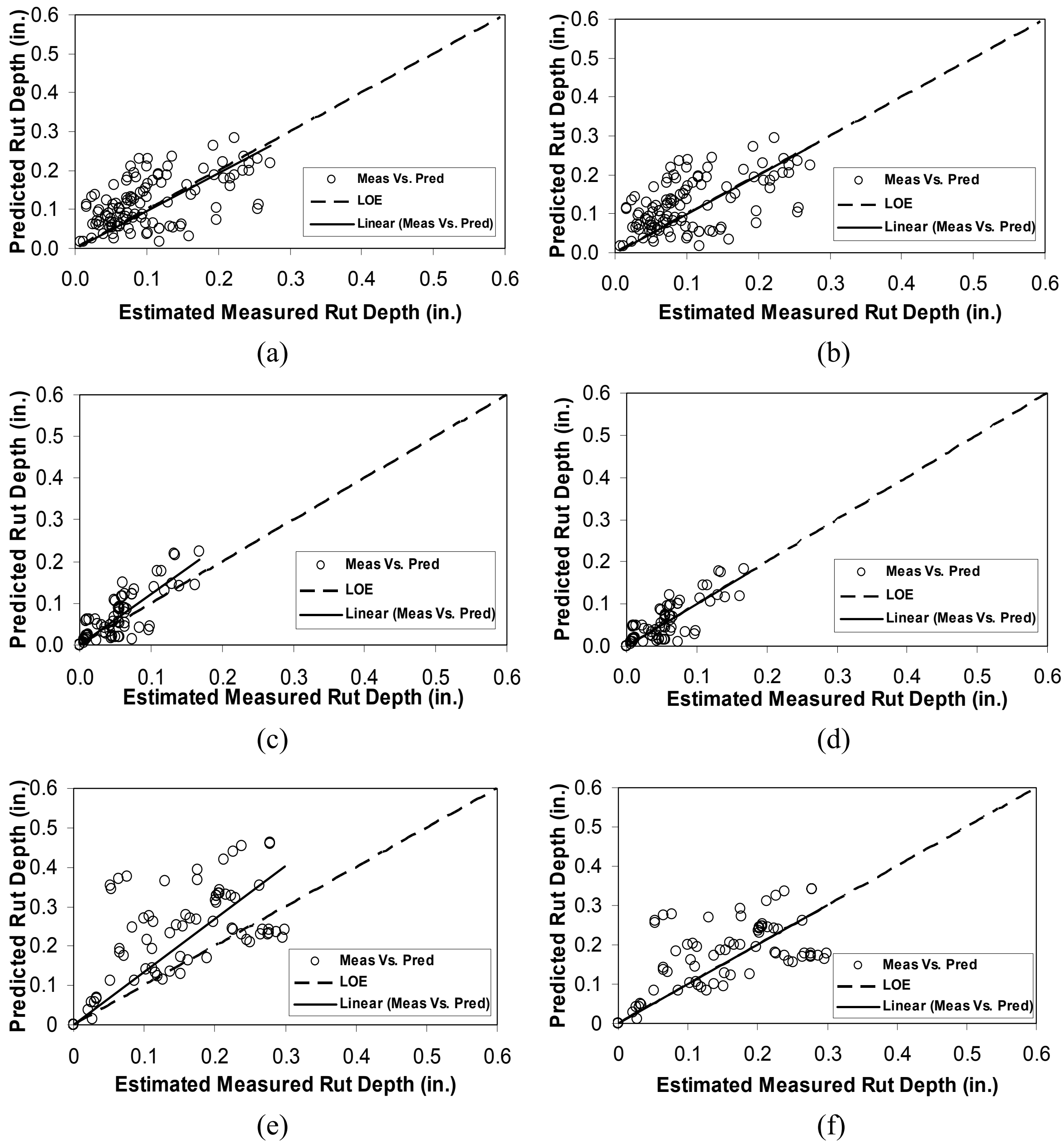

Figure 6-3 Estimated measured rut depth vs. predicted rut depth: (a) HMA before calibration; (b) HMA after calibration; (c) granular base before calibration; (d) granular base after calibration; (e) subgrade before calibration; and (f) subgrade after calibration

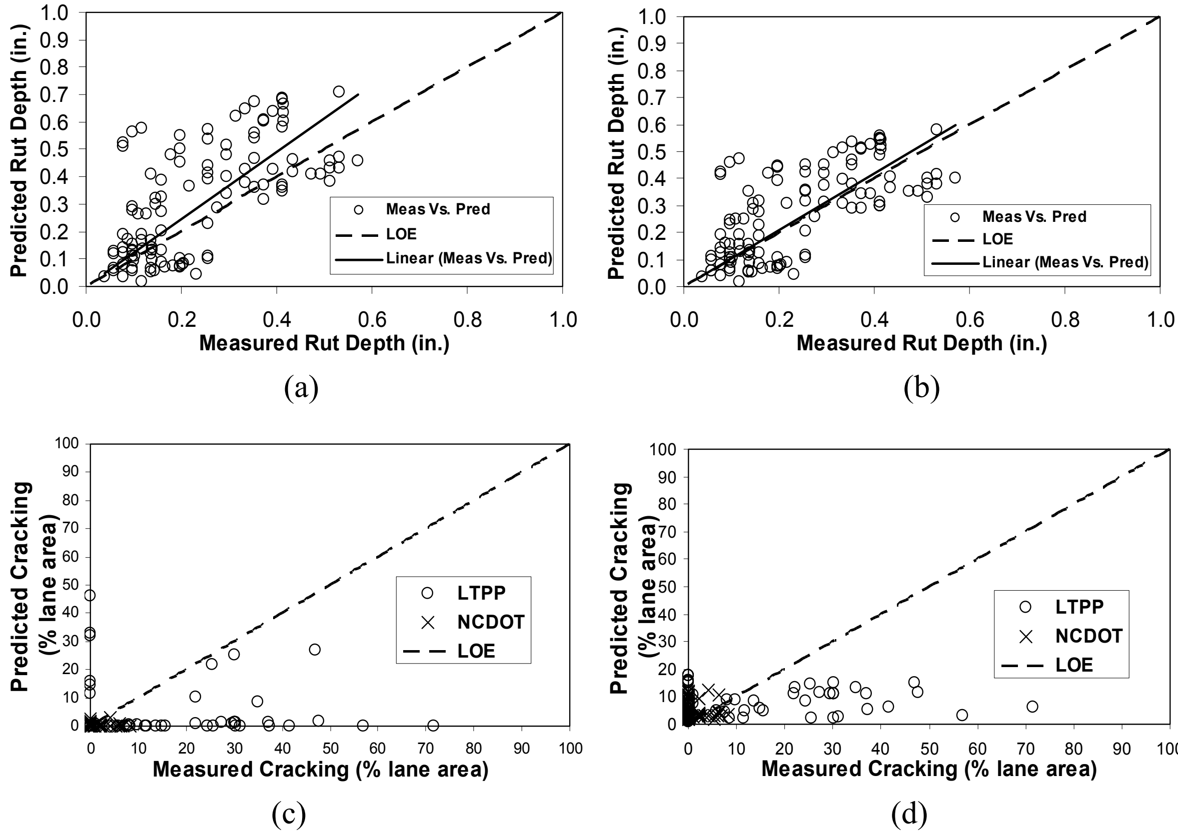

Figure 6-4 Measured vs. predicted distresses before and after calibration: (a) total rut depth before calibration; (b) total rut depth after calibration; (c) alligator cracking before calibration; and (d) alligator cracking after calibration

Table 6-5 presents the standard error and bias before and after calibration. The standard error is significantly reduced after the calibration; and the bias is completely eliminated. Also, the final standard error obtained is much less than the global standard error presented in Table 6-3. To determine if it is significantly different from the global standard error, a Chi-square test was performed as part of the STEP 9. The national re-calibrated model (NCHRP, 2006.a) has a statistics of R^2 = 0.275 and S_e = 5.01%, which includes 400 plus data points. The local calibration (R^2 = 0.11, S_e=3.64 %) of the fatigue cracking model is still considered an improvement over the verification runs (R^2 = 0.03, S_e = 6.02 %) considering 177 data points that also include NCDOT sections.

Table 6-5 Calibration Results – Summary of Statistics for Alligator Cracking Predictions

Statistical Results	Alligator Cracking	
	Before	**After**
Average (%)	1.54	5.21
R^2	0.025	0.1057
S_e^a (%)	6.02	3.636
SS_e^b	6343.814	2313.684
p-value	0.000184 < 0.05 (alpha)	0.5 >0.05 (alpha)
Bias (%)	3.67	0
N^c	177	177

Note: [a]Standard Error, [b]Sum of Squared Error, [c]Number of Data Points

Even though the standard error and bias are significantly reduced after the calibration, the visual observation of Figure 6-4(c) and Figure 6-4(d) reveals poor prediction. One possible reason for this poor prediction that can be identified at this stage is the inclusion of NCDOT sections in the calibration process. The NCDOT measurement technique (NCDOT, 2006) does not capture the cracking outside the outer wheel path and, hence, negligible cracking is measured most of the time. This measurement error can be eliminated in the future by following the LTPP Distress Identification Manual (FHWA, 2003) to measure the distresses for the local pavement sections selected for calibration. Doing so will ensure consistency between the MEPDG predicted and measured distresses. The above reason is supported from the NCHRP 1-40B (NCHRP, 2005) study that the measured area fatigue cracking was found to be one of the error components that significantly increases the standard error for this prediction model, which is model independent.

STEP 9: The total standard error for the permanent deformation model and the alligator cracking models are presented in Table 6-4 and Table 6-5. A null hypothesis test is evaluated for the experimental matrix relative to the standard error to determine if the local standard error is significantly different from the global standard error obtained during the national re-calibration effort (NCHRP, 2006.a). The null hypothesis for the initial assessment is that there is no significant difference between the standard error terms. A Chi-squared test was employed to perform this test. The following table presents the statistical results for this step:

Table 6-6 Calibration Results - Null Hypothesis Test for Standard Error

Statistical Results	Permanent Deformation	Alligator Cracking
Chi-Square Statistic	112.5505	92.17829
Degrees of Freedom	110	176
p-value	0.416 > 0.05 (alpha)	0.9999 > 0.05 (alpha)

From Table 6-6, it can be observed that the null hypothesis is accepted for both the distress models and hence the standard error resulted from the local calibration efforts and the global standard error is considered the same.

STEP 11: The local calibration coefficients with the new standard error of the estimate is thus finally entered into the MEPDG software for use in new and rehabilitated flexible pavement design in North Carolina until further and more robust calibration is carried out using more detailed inputs (mostly level-1) and increased number of sections.

6.5 Validation

For the purpose of validation, the remaining 20% of the sections that were kept aside from the calibration process are used to verify the reasonableness of the final calibrated models. Table 6-7 presents the validation results for both the models. For the permanent deformation model, the observed standard error is higher than the local calibrated standard error as well as the global standard error, which is expected. A Chi-square test is used to determine if the standard error from the validation results is significantly different from the standard error from the local calibration. From the Chi-square test results presented in Table 6-8, it can be deduced that the validation check is successful and therefore the final calibrated permanent deformation model will be used until more rigorous calibration is done in the future employing an increased number of sections.

Table 6-7 Validation Results – Summary of Statistics for the Permanent Deformation and Alligator Cracking Predictions

	Permanent Deformation Model					Alligator Cracking Model	
	AC	**GB**	**SG**	**Total**			**AC**
Average (in.)	0.0656	0.0330	0.0899	0.1885		**Average (%)**	8.99
S_e^a (in.)	0.0298	0.0077	0.0661	0.1445		***S_e (%)***	4.8594
Bias (in.)	0.012	-0.014	0.035	0.033		***Bias (%)***	-5.04
N^c	26	26	26	26		***N***	32

Note: [a]Standard Error, [b]Sum of Squared Error, [c]Number of Data Points

Table 6-8 Validation Results - Null Hypothesis Test for Standard Error

Statistical Results	Permanent Deformation	Alligator Cracking
Chi-Square Statistic	36.82374	5.664047
Degrees of Freedom	25	31
p-value	0.0599 > 0.05 (alpha)	0.9999 > 0.05 (alpha)

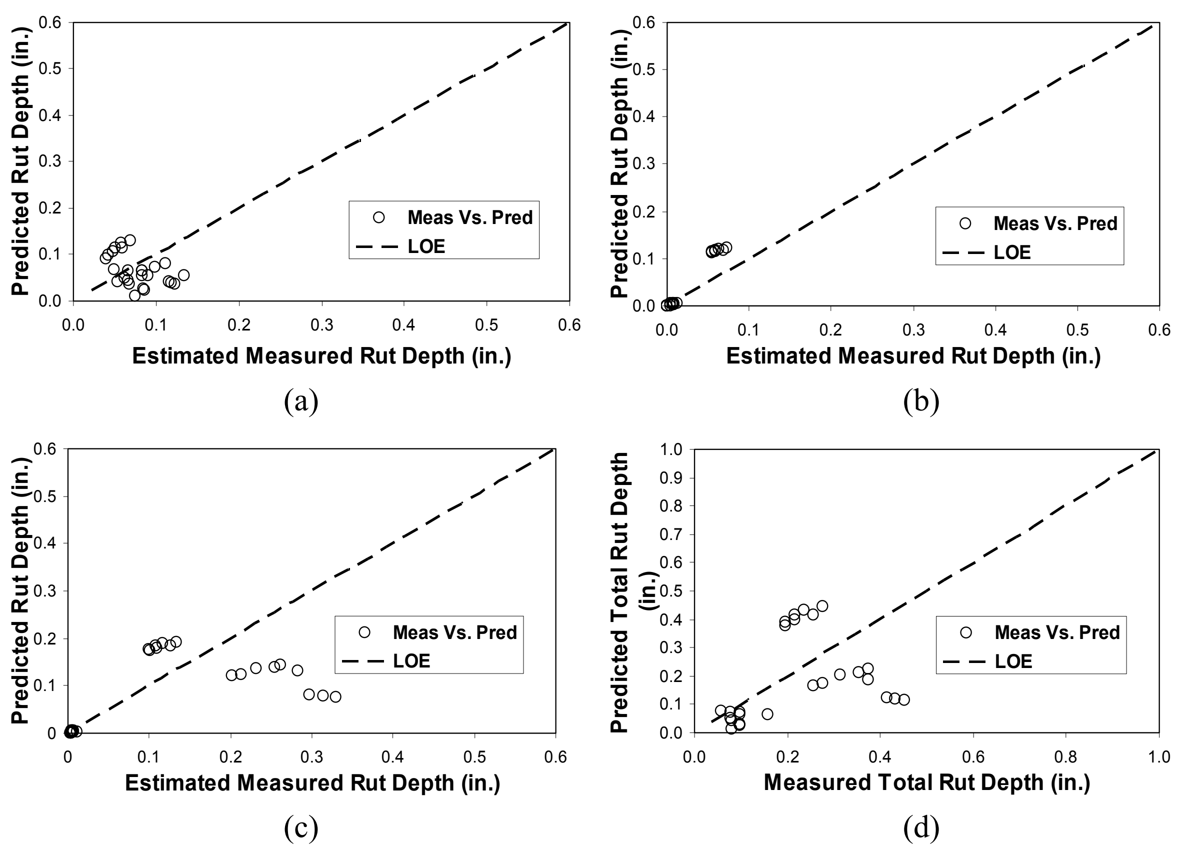

Figure 6-5 Estimated measured rut depth vs. predicted rut depth from validation: (a) HMA; (b) granular base; (c) subgrade; and (d) total

Similarly, for the alligator cracking model, it was found that the standard error from the validation runs is slightly higher than that of the local calibrated standard error. Figure 6-6 shows the validation results for this model. A chi-square test was performed to check if the standard error for the validation is significantly higher. From the Chi-square test results presented in Table 6-8, a p-value greater than 0.05 (i.e., a confidence interval of 95%) shows that the validation check is successful for the alligator cracking model. Again, it was decided to keep the alligator cracking model with the local calibrated coefficients for the flexible pavement design in North Carolina until more robust calibration is performed in the future.

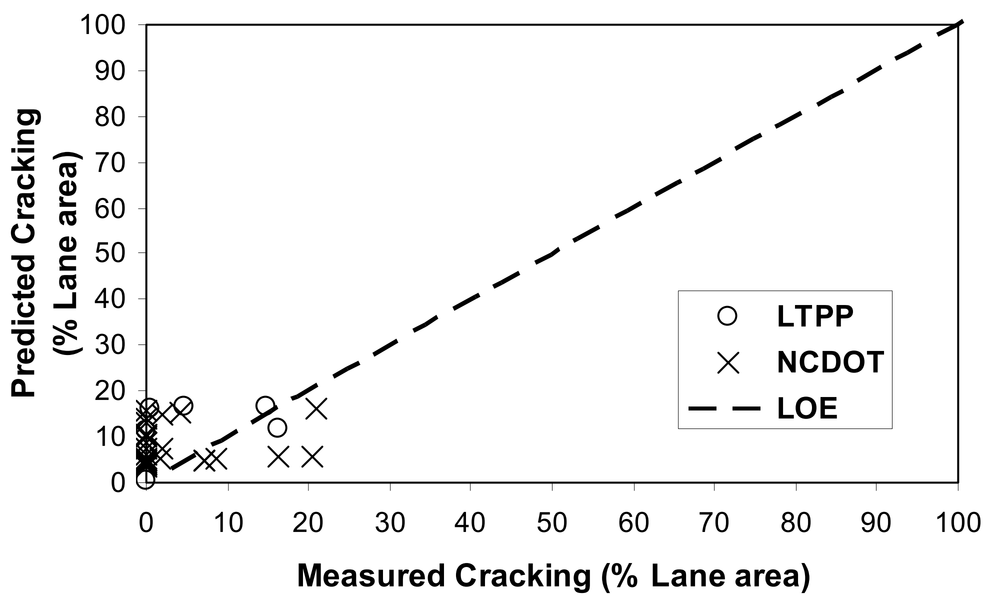

Figure 6-6 Validation Results - Measured cracking vs. predicted cracking

6.6 Summary

Table 6-9 lists the calibration factors developed from various calibration efforts. The local calibration effort was successful to an extent that the bias is eliminated and the local standard error is the same as that of the global standard error (from the null hypothesis test results for standard error, i.e., Table 6-6). Calibration of the mathematical models could not be pursued during this study as it requires a preliminary study to estimate the potential reduction in the standard error. The agency then decides whether additional cost, effort and time significantly reduce the standard error for the distress models. Until further and more robust calibrations are completed using more detailed inputs and an increased number of sections, the local calibration factors developed in this study will be used for the performance prediction of the sections located in North Carolina.

Table 6-9 Final Set of Calibration Factors

Distress Model		Calibration Factors	National Calibration	National Re-calibration	Local Calibration
Permanent Deformation	***AC*[a]**	k_1	-3.4488	-3.35412	-3.41273
		k_2	1.5606	1.5606	1.5606
		k_3	0.479244	0.479244	0.479244
	***GB*[b]**	β_{GB}	1.673	2.03	1.5803
	***SG*[c]**	β_{SG}	1.35	1.67	1.10491
Alligator	***AC***	k_1	0.00432	0.007566	0.007566
		k_2	3.9492	3.9492	3.9492
		k_3	1.281	1.281	1.281
		C_1	1	1	0.437199
		C_2	1	1	0.150494

Note: [a]Asphalt Concrete, [b]Granular Base, [c]Subgrade

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The following conclusions were made from this study:

- From the verification results, it is found that the MEPDG-predicted rut depth values match very well with the measured rut depth values for the LTPP sections.
- For the alligator cracking model, the MEPDG under-predicts the percentage of cracking for most of the sections, resulting in a significant amount of bias.
- NCDOT sections are dropped from the permanent deformation calibration effort due to the NCDOT's subjective rating approach to rutting that employs a single non-numeric rating which, in turn, presents problems with the conversion to the MEPDG format.
- From the calibration effort, it can be observed that the standard error is significantly reduced and the bias is completely eliminated. The standard error from the local calibration effort is same as the global standard error from the national calibration effort for both the distress prediction models.
- The null hypothesis test relative to the standard error to check if the local calibrated standard error is significantly different from the global standard error shows that there is no significant difference between them and hence no further calibration is pursued to reduce the standard error in this study.
- For alligator cracking, only the parameters in the transfer function (i.e., the statistical model) are varied to minimize the sum of the squared error of the measured and

predicted cracking values. It is assumed that the mathematical model used to predict damage accurately models the real-world system.

- The validation check performed on both the distress prediction models using the Chi-square test shows that the validation is successful and hence the predictions are reasonable.
- Therefore, it was decided to keep the both the calibrated permanent deformation and alligator cracking models for performance predictions until a more robust calibration is performed with an increased number of sections.
- One of the issues identified with the calibration procedure is that the MEPDG assumptions were accepted in assigning the permanent deformation observed at the surface to each pavement layer. This assumption needs to be supported with field and forensic investigations at the site locations.
- Also, it is noted that major differences are evident between the distress measurement techniques of the NCDOT and the LTPP program. If the NCDOT adopts the LTPP standard procedure, at least for the sections to be used in the future calibration effort, a significant amount of measurement error and input error can be reduced.

7.2 Recommendations

The following recommendations are resulted from this study:

- It is recommended to begin a field and forensic (trenches and cores) investigation to check for the reasonableness of the MEPDG assumptions in assigning the observed surface permanent deformation to each pavement layer.

- It is recommended to increase the number of sections and include more detailed inputs (Level 1 mostly) for future calibration efforts, which will help reduce a significant amount of input error.
- It is recommended to use the LTPP Distress Identification Manual (FHWA, 2003) to complete the distress surveys for the sections selected for future calibration purposes. Use of this manual will ensure consistency in the MEPDG predictions.
- The standard error can be reduced significantly by varying the appropriate parameters (as suggested in the NCHRP 1-40B report) in the distress prediction models. A preliminary study to check for the potential reduction in the standard error is recommended before expending any additional costs, efforts and time in pursuing the same. At present, such effort (reduction of standard error) can only be pursued using the MEPDG software. The major set back with this procedure is that the MEPDG software takes 40 minutes for each run and hence setting the time constraints.

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APPENDIX A: LOCAL CALIBRATION DATABASE

Table A - 1 Pavement Section Information

Section	Pavement Type	Construction		Traffic Opening Date	Design Life (years)	CN
		Base	Asphalt			
371006	AC-GB	5/1/1982	7/1/1982	7/1/1982	20	3
371024	AC-GB	9/1/1980	11/1/1980	11/1/1980	20	2
371028	AC-GB	3/1/1982	5/1/1982	5/1/1982	20	2
371030	AC-TB	10/1/1984	12/1/984	12/1/1984	20	1
371040	AC-GB	71/1978	9/1/1978	9/1/1978	20	2
371352	AC-GB	5/1/1980	7/1/1980	7/1/1980	20	3
371645	AC-NTB	8/1/1986	10/1/1986	10/1/1986	20	2
371801	AC-GB	3/1/1974	5/1/1974	5/1/1974	20	2
371802	AC-GB	8/1/1985	10/1/1985	10/1/1985	20	4
371803	AC-GB	10/1/1977	12/1/1977	12/1/1977	20	3
371814	AC-GB	7/1/1970	9/1/1970	9/1/1970	20	2
371817	AC-GB	10/1/1983	12/1/1983	12/1/1983	20	5
371992	AC-GB	12/1/1989	2/1/1990	2/1/1990	20	2
372819	AC-NTB	6/1/1981	8/1/1981	8/1/1981	20	3
372824	AC-NTB	8/1/1983	10/1/1983	10/1/1983	20	3
372825	AC-NTB	12/1/1986	2/1/1987	2/1/1987	20	1
R2120AA	AC-GB	2/24/2002	4/24/2002	4/24/2002	20	1
R2120AB	AC-GB	8/1/2001	10/1/2001	10/1/2001	20	1
R2123AC	AC-GB	5/1/2003	7/1/2003	7/1/2003	20	1
R2123BB	AC-GB	9/3/2003	9/3/2003	9/3/2003	20	1
R2123CC	AC-GB	8/15/2003	8/15/2003	8/15/2003	20	1
R2219AC	AC-GB	7/23/2001	7/23/2001	7/23/2001	20	1
R2217B	AC-AC	7/10/2002	7/10/2002	7/10/2002	20	1
R1023AB	AC-GB	12/31/2002	12/31/2002	12/31/2002	20	1
R1023B	AC-GB	12/20/2002	12/20/2002	12/20/2002	20	1
R2000EA	AC-NTB	8/13/2002	8/13/2002	8/13/2002	20	1
R2000EB	AC-NTB	8/23/2002	8/23/2002	8/23/2002	20	1
R2001C	AC-GB	7/1/2001	7/1/2001	7/1/2001	20	1
R2000BB	AC-NTB	12/15/1993	2/15/1994	2/15/1994	20	1
U77LA	AC-NTB	5/20/1993	7/20/1993	7/20/1993	20	1
R2318B	AC-GB	3/17/1994	5/17/1994	5/17/1994	20	1
U508CB	AC-AC	07/1/1993	09/1/1993	09/1/1993	20	1
U2413B	X	8/19/1993	10/19/1993	10/19/1993	20	1
U508CA	AC-AC	07/1/1993	09/1/1993	09/1/1993	20	1
R1017AC	AC-AC	3/18/1993	5/18/1993	5/18/1993	20	1
R2232A,B	AC-CTB	6/17/1993	8/17/1993	8/17/1993	20	1
R2211BA	AC-GB	2/15/1997	4/15/1997	4/15/1997	20	1
R519	AC-AC	5/20/1993	7/20/1993	7/20/1993	20	1
R85AD	AC-AC	9/16/1993	11/16/1993	11/16/1993	20	1

Table A - 2 Pavement Lane Properties

Section	Original No. of Lanes	Original Pavement Width	Thermal Conductivity (BTU/hr-ft-°F)	Heat Capacity (BTU/lb-°F)	Surface Shortwave Absorptivity
371006	2	12	0.67	0.22	0.85
371024	1	12	0.67	0.22	0.85
371028	2	12	0.67	0.22	0.85
371030	2	12	0.67	0.22	0.85
371040	2	12	0.67	0.22	0.85
371352	2	12	0.67	0.22	0.85
371645	2	12	0.67	0.22	0.85
371801	2	12	0.67	0.22	0.85
371802	1	12	0.67	0.22	0.85
371803	2	12	0.67	0.22	0.85
371814	2	12	0.67	0.22	0.85
371817	2	12	0.67	0.22	0.85
371992	2	12	0.67	0.22	0.85
372819	2	12	0.67	0.22	0.85
372824	2	12	0.67	0.22	0.85
372825	2	12	0.67	0.22	0.85
R2120AA	2	12	0.67	0.22	0.85
R2120AB	2	12	0.67	0.22	0.85
R2123AC	2	12	0.67	0.22	0.85
R2123BB	3	12	0.67	0.22	0.85
R2123CC	3	12	0.67	0.22	0.85
R2219AC	2	12	0.67	0.22	0.85
R2217B	2	12	0.67	0.22	0.85
R1023AB	2	12	0.67	0.22	0.85
R1023B	2	12	0.67	0.22	0.85
R2000EA	2	12	0.67	0.22	0.85
R2000EB	3	12	0.67	0.22	0.85
R2001C	2	12	0.67	0.22	0.85
R2000BB	3	12	0.67	0.22	0.85
U77LA	2	12	0.67	0.22	0.85
R2318B	1	12	0.67	0.22	0.85
U508CB	2	12	0.67	0.22	0.85
U2413B	2	12	0.67	0.22	0.85
U508CA	2	12	0.67	0.22	0.85
R1017AC	2	12	0.67	0.22	0.85
R2232A&B	2	12	0.67	0.22	0.85
R2211BA	1	12	0.67	0.22	0.85
R519	2	12	0.67	0.22	0.85
R85AD	2	12	0.67	0.22	0.85

Table A - 3 Environmental or Climatic Conditions

Section	Latitude (degrees)	Longitude (degrees)	Elevation (ft)	GWT (ft)	Location
371006	35.78	78.75	435	>20	0.75 MILES EAST OF NC 54 (RALEIGH,NC)
371024	35.30	83.18	2125	>20	2183 FEET EAST OF SR 1330 TO STA 0+00 (SYLVIA,NC)
371028	36.53	76.37	20		STA ZERO (0+00) IS 1.7 MILES NORTH OF SR 1231 (ELIZABETH CITY,NC)
371030	36.38	76.31	18		2.58 MILES FROM INTERSECTION OF US 158 & SR 1416 TO STATION 0+00 - SBL (ELIZABETH CITY,NC)
371040	35.91	82.06	2565		STA 0+00 IS 0.2 MILES EAST OF SR 1121 (ALTAPASS HWY) (INGALLS,NC)
371352	35.45	80.23	680		2.77 MILES FROM NC 49 TO STATION 0+00 SBL (RICHFIELD,NC)
371645	34.35	78.65	75		STA ZERO IS 3 MILES WEST OF SR 1001 (WHITEVILLE,NC)
371801	35.59	82.42	2270		1.3 MILES FROM SR 2740 (EXIT 59 - SWANNANOA) TO STA 0+00 WB (ASHEVILLE,NC)
371802	36.32	78.52	500	>20	4938 FEET FROM SR 1004 TO STATION 0+00, NB SR 1004 IS @ MILEPOINT 0.00; SR 1170 IS @ MILEPOINT 1.34 (OXFORD,NC)
371803	35.39	83.30	1895		3800 FEET EAST OF SR 1391 TO STA 0+00 (BRYSON CITY,NC)
371814	35.17	83.37	2060		STA 0+00 IS 0.64 MILE SOUTH OF US 64 (FRANKLIN,NC)
371817	36.05	80.15	850	35	3950 FEET FROM SR 2698 (RIDGEWOOD RD) TO STATION 0+00 SB (WINSTON-SALEM,NC)
371992	35.75	79.44	585	>20	0.75 MILES NORTH OF US 64 (EXIT 168) (SILER CITY,NC)
372819	35.93	79.83	800		3.6 MILES FROM OLD RANDLEMAN RD (SR 1104) TO STATION 0+00 SB (GREENSBORO,NC)
372824	35.71	79.43	632		1.9 MILES SOUTH OF US 64 (BEFORE EXIT 168) (SILER CITY,NC)

Table A – 3 (Continued) Environmental or Climatic Conditions

372825	35.14	80.92	610		3095 FEET FROM I-77 TO STATION 0+00 WB (CHARLOTTE,NC)
R2120AA	36.07	80.47	1086	15	RELOCATION OF US 421 FROM EAST OF I 77 TO WEST OF SOUTH DEEP CREEK IN YADKIN COUNTY
R2120AB	36.07	80.42	900	15	US 421 FROM EAST OF SR 1139 TO WEST OF US 601
R2123AC	35.05	80.44	758	15	I-485, CHARLOTTE OUTER LOOP FROM SOUTH OF NC 218 TO NORTH OF NC 51 IN MECKLENBURG COUNTY
R2123BB	35.14	80.39	712	15	ON I-485, EASTERN CHARLOTTE OUTER LOOP FROM SOUTH OF SR 2808 (CAMP STEWART RD.) TO SOUTH OF SR 2802 (ROCKY RIVER CHURCH RD.)
R2123CC	35.18	80.41	692	15	ON I-485, CHARLOTTE OUTER LOOP FROM SOUTH OF SOUTH OF SR 2802 (ROCKY RIVER CHURCH RD.) TO SOUTH OF NC 49
R2219AC	35.43	79.09	410	30	US 64 FROM WEST OF SR 1514 WEST OF PITTSBORO TO EAST OF US 15-501 IN CHATHAM COUNTY
R2217B	35.45	79.34	628	7	US 64 FROM APPROXIMATELY 220 METERS EAST OF SR 2628 (PARKS CROSSROADS RD.) TO APPROXIMATELY 200 METERS WEST OF GLENN AVENUE IN SILER CITY.
R1023AB	35.4	77.55	79	7	US 264 WILSON BYPASS FROM EAST OF SR 1136 TO EAST OF US 301 IN WILSON COUNTY
R1023B	35.4	77.5	89	7	US 264 WILSON BYPASS FROM EAST OF US 301 TO EXISTING US 264 EAST OF NC 58
R2000EA	35.54	78.36	384	20	NORTHERN WAKE EXPRESSWAY (I 540) FROM APPROXIMATELY 0.249 MILES EAST OF HONEYCUTT ROAD TO APPROXIMATELY 0.153 MILES EAST OF GRESHAM'S LAKE ROAD.
R2000EB	35.52	78.34	243	20	RALEIGH OUTER LOOP FROM WEST OF CSX RAILROAD TO EAST OF US 1

Table A – 3 (Continued) Environmental or Climatic Conditions

R2001C	35.12	77.4	77	3.5	11 FROM 1.1 MILE NORTH OF SOUTHWEST CREEK TO NC 55 AT JACKSON'S STORE
R2000BB	35.54	78.49	379	9	RALEIGH OUTER LOOP FROM 3500 FT NE OF I-40 TO 4000 FT. SW OF LUMLEY ROAD
U77LA	35.59	78.53	379	9	BUCK DEAN FREEWAY, FROM WEST OF SOUTHERN RAILROAD TO US 15-501
R2318B	35.45	78.4	252	15	US 64 FROM EAST OF NC 55 TO CSX RAILROAD AT APEX 450+00
U508CB	35.01	78.52	100	4	CBD LOOP IN FAYETTEVILLE FROM WILKES ROAD TO A POINT JUST SOUTH OF SR 2283 (EAST MOUNTAIN DRIVE)
U2413B	36.2	79.57	863		HIGH POINT-GREENSBORO WENDOVER AVENUE (SR-1541), FROM SR-1536 TO SR-1546.
U508CA	35.01	78.52	92	4	CBD LOOP IN FAYETTEVILLE, 800 FT. FROM GILLESPIE STREET TO SR 1007.
R1017AC	35.51	81.03	106		US 64/NC 90, FROM 336 FT. WEST OF THE ALEXANDER COUNTY
R2232A&B	36.28	79.54	804	20	220 FROM NORTH OF NC 704 TO 1 MILE SOUTH OF NC/VA LINE
R2211BA	34.58	77.58	146	6	NC 24, FROM KENANSVILLE TO EAST OF SR 1701 IN DUPLIN COUNTY
R519	35.18	91.32	389	17.5	SR 2104 DEKALB ST. EXTENSION SOUTH FROM INTERSECTION OF NC 150 AND SHELBY CITY SCHOOLS
R85AD	35.29	81.14	874	35	US 321 FROM 0.26 MILES NORTH OF EXISTING US 321 TO SR 1005

Table A - 4 Pavement Structure

Section	Layers		Layer No.	Layer Type	Representative Thickness (in.) CN1 | CN2	Material Type
	CN1	CN2				
370801	4	4	1	AC	1.7	Hot Mix, Hot Laid, Dense graded
			2	AC	2.3	Hot Mix, Hot Laid, Dense graded
			3	GB	8.7	Crushed Stone
			4	SS	-	A-3
370802	5	5	1	AC	1.7	Hot Mix, Hot Laid, Dense graded
			2	AC	2.5	Hot Mix, Hot Laid, Dense graded
			3	AC	2.8	Hot Mix, Hot Laid, Dense graded
			4	GB	11.5	Crushed Stone
			5	SS	-	A-3
370859	3	3	1	AC	1.4	Hot Mix, Hot Laid, Dense graded
			2	GB	6.8	Crushed Stone
			3	SS	-	A-3
370901	5	5	1	AC	2.6	Hot Mix, Hot Laid, Dense graded
			2	AC	2.8	Hot Mix, Hot Laid, Dense graded
			3	TB	3.4	Asp. bound, Dense, Hot Laid, CPM
			4	GS	7.0	SAM-Predominantly Fine Grained
			5	SS	-	A-4
370902	5	5	1	AC	2.7	Hot Mix, Hot Laid, Dense graded
			2	AC	2.6	Hot Mix, Hot Laid, Dense graded
			3	TB	3.2	Asp. bound, Dense, Hot Laid, CPM
			4	GS	7.0	SAM-Predominantly Fine Grained
			5	SS	-	A-6
370903	5	5	1	AC	2.4	Hot Mix, Hot Laid, Dense graded
			2	AC	2.9	Hot Mix, Hot Laid, Dense graded
			3	TB	3.4	Asp. bound, Dense, Hot Laid, CPM
			4	GS	7.0	SAM-Predominantly Fine Grained
			5	SS	-	A-6

Table A – 4 (Continued) Pavement Structure

370960	5	6	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	1.5	Hot Mix, Hot Laid, Dense graded
			3	AC	3.5	Hot Mix, Hot Laid, Dense graded
			4	TB	5.5	Asp. bound, Dense, Hot Laid, CPM
			5	GS	7.0	Lime Treated Subgrade Soil
			6	SS	-	A-6
370961	5	6	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	1.5	Hot Mix, Hot Laid, Dense graded
			3	AC	4.0	Hot Mix, Hot Laid, Dense graded
			4	TB	5.5	Asp. bound, Dense, Hot Laid, CPM
			5	GS	7.5	Lime Treated Subgrade Soil
			6	SS	-	A-6
370962	5	6	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	4.0	Hot Mix, Hot Laid, Dense graded
			3	AC	2.0	Hot Mix, Hot Laid, Dense graded
			4	TB	4.5	Asp. bound, Dense, Hot Laid, CPM
			5	GS	7.0	Lime Treated Subgrade Soil
			6	SS	-	A-6
370963	5	6	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	1.5	Hot Mix, Hot Laid, Dense graded
			3	AC	3.5	Hot Mix, Hot Laid, Dense graded
			4	TB	6.0	Asp. bound, Dense, Hot Laid, CPM
			5	GS	7.5	Lime Treated Subgrade Soil
			6	SS	-	A-6
370964	5	6	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	1.5	Hot Mix, Hot Laid, Dense graded
			3	AC	3.5	Hot Mix, Hot Laid, Dense graded
			4	TB	6.0	Asp. bound, Dense, Hot Laid, CPM
			5	GS	8.0	Lime Treated Subgrade Soil
			6	SS	-	A-6

Table A – 4 (Continued) Pavement Structure

370965	5	6	1	AC	2.5		Hot Mix, Hot Laid, Dense graded
			2	AC	1.5		Hot Mix, Hot Laid, Dense graded
			3	AC	4.0		Hot Mix, Hot Laid, Dense graded
			4	TB	5.5		Asp. bound, Dense, Hot Laid, CPM
			5	GS	7.5		Lime Treated Subgrade Soil
			6	SS	-		A-6
371006	5	8	1	AC		1.4	Hot Mix, Hot Laid, Dense graded
			2	AC		2.2	Hot Mix, Hot Laid, Dense graded
			3	AC		0	Slurry Seal
			4	AC	0.7	0	Hot Mix, Hot Laid, Open graded, Friction
			5	AC	2.0	0.7	Hot Mix, Hot Laid, Dense graded
			6	AC	6.5	6.5	Hot Mix, Hot Laid, Dense graded
			7	GB	9.4	9.4	Crushed Gravel
			8	SS	-	-	A-7-5
371024	4	6	1	AC		2.3	Hot Mix, Hot Laid, Dense graded
			2	AC		4.2	Hot Mix, Hot Laid, Dense graded
			3	AC	1.0	0.0	Hot Mix, Hot Laid, Open graded, Friction
			4	AC	3.8	3.8	Hot Mix, Hot Laid, Dense graded
			5	GB	12	12	Crushed Gravel
			6	SS	54	54	A-5
371028	3	5	1	AC		1.6	Hot Mix, Hot Laid, Dense graded
			2	AC		2.6	Hot Mix, Hot Laid, Dense graded
			3	AC	1.6	0.0	Hot Mix, Hot Laid, Dense graded
			4	TB	8.2	8.2	Asp. bound, Dense, Hot Laid, CPM
			5	SS	-	-	A-3
371030	3	3	1	AC	4.0		Hot Mix, Hot Laid, Dense graded
			2	TB	4.7		Asp. bound, Dense, Hot Laid, CPM
			3	SS	-		A-3

Table A – 4 (Continued) Pavement Structure

371040	5	6	1	AC		1.9	Hot Mix, Hot Laid, Dense graded
			2	AC	0.6	0.6	Hot Mix, Hot Laid, Open graded, Friction
			3	AC	2.0	2.0	Hot Mix, Hot Laid, Dense graded
			4	AC	2.6	2.6	Hot Mix, Hot Laid, Dense graded
			5	GB	14.4	14.4	SAM: Predominantly Coarse Grained
			6	SS	-	-	A-4
371352	5	8	1	AC		1.5	Recycled Asphalt Concrete Hot, CPM
			2	AC		1.4	Recycled Asphalt Concrete Hot, CPM
			3	AC		2.0	Hot Mix, Hot Laid, Dense graded
			4	AC	0.7	0.0	Hot Mix, Hot Laid, Open graded, Friction
			5	AC	2.2	1.4	Hot Mix, Hot Laid, Dense graded
			6	AC	3.4	3.4	Hot Mix, Hot Laid, Dense graded
			7	GB	6.0	6.0	Crushed Stone
			8	SS	-	-	A-4
371645	4	6	1	AC		1.6	Hot Mix, Hot Laid, Dense graded
			2	AC		0.4	Hot Mix, Hot Laid, Dense graded
			3	AC	1.9	1.9	Hot Mix, Hot Laid, Dense graded
			4	AC	6.0	6.0	Hot Mix, Hot Laid, Dense graded
			5	TB	8.2	8.2	Soil Cement
			6	SS	-	-	A-4
371801	4	6	1	AC		1.6	Hot Mix, Hot Laid, Dense graded
			2	AC		2.8	Recycled Asphalt Concrete Hot, CPM
			3	AC	2.0	0.0	Hot Mix, Hot Laid, Dense graded
			4	AC	5.2	5.3	Hot Mix, Hot Laid, Dense graded
			5	GB	12	12	SAM: Predominantly Coarse Grained
			6	SS	-	-	A-5

Table A – 4 (Continued) Pavement Structure

371802	4	5	1	AC		0.9	Hot Mix, Hot Laid, Dense graded
			2	AC	2.2	2.2	Hot Mix, Hot Laid, Dense graded
			3	AC	2.2	2.2	Hot Mix, Hot Laid, Dense graded
			4	GB	8.2	8.2	Crushed Gravel
			5	SS	-	-	A-2-6
371803	5	8	1	AC		1.5	Hot Mix, Hot Laid, Dense graded
			2	AC		0.9	Hot Mix, Hot Laid, Dense graded
			3	AC		2.4	Hot Mix, Hot Laid, Dense graded
			4	AC	0.6	0.0	Hot Mix, Hot Laid, Open graded, Friction
			5	AC	2.1	0.8	Hot Mix, Hot Laid, Dense graded
			6	AC	2.5	2.5	Hot Mix, Hot Laid, Dense graded
			7	GB	12.6	12.6	SAM: Predominantly Fine Grained
			8	SS	42	42	A-5
371814	4	7	1	AC		1.8	Recycled Asphalt Concrete Hot, CPM
			2	AC		1.4	Hot Mix, Hot Laid, Dense graded
			3	AC		1.7	Hot Mix, Hot Laid, Dense graded
			4	AC	2.4	0.9	Hot Mix, Hot Laid, Dense graded
			5	AC	2.7	2.7	Hot Mix, Hot Laid, Dense graded
			6	GB	13.5	13.5	SAM: Predominantly Coarse Grained
			7	SS	168	168	A-2-4
371817	4	8	1	AC		1.5	Hot Mix, Hot Laid, Dense graded
			2	AC		3.2	Hot Mix, Hot Laid, Dense graded
			3	AC		1.3	Recycled Asphalt Concrete Hot, CPM
			4	AC		0.3	Slurry Seal
			5	AC	2.1	2.1	Hot Mix, Hot Laid, Dense graded
			6	AC	2.2	2.5	Hot Mix, Hot Laid, Dense graded
			7	GB	12	12	SAM: Predominantly Coarse Grained
			8	SS	-	-	A-7-5

Table A – 4 (Continued) Pavement Structure

371992	4	8	1	AC		2.3	Recycled Asphalt Concrete Hot, CPM
			2	AC		1.5	Recycled Asphalt Concrete Hot, CPM
			3	AC		2.6	Recycled Asphalt Concrete Hot, CPM
			4	AC		1.9	Hot Mix, Hot Laid, Dense graded
			5	AC	2.4	2.4	Hot Mix, Hot Laid, Dense graded
			6	GB	12	12	Crushed Stone
			7	GS	24	24	SAM: Predominantly Coarse Grained
			8	SS	-	-	A-7-5
372819	6	10	1	AC		1.6	Hot Mix, Hot Laid, Dense graded
			2	AC		1.4	Hot Mix, Hot Laid, Dense graded
			3	AC		1.5	Recycled Asphalt Concrete Hot, CPM
			4	AC		0.7	Chip Seal
			5	AC	0.9	0.9	Hot Mix, Hot Laid, Open graded, Friction
			6	AC	2.0	2.0	Hot Mix, Hot Laid, Dense graded
			7	AC	2.1	2.1	Hot Mix, Hot Laid, Dense graded
			8	TB	8.2	8.2	Cement Aggregate Mixture
			9	GS	8.8	8.8	SAM: Predominantly Fine Grained
			10	SS	-	-	A-6
372824	4	6	1	AC		2.0	Hot Mix, Hot Laid, Dense graded
			2	AC		2.2	Recycled Asphalt Concrete Hot, CPM
			3	AC	1.9	1.9	Hot Mix, Hot Laid, Dense graded
			4	AC	2.8	2.8	Hot Mix, Hot Laid, Dense graded
			5	TB	6.0	6.0	Cement Aggregate Mixture
			6	SS	-	-	A-7-5
372825	4	4	1	AC	2.4		Hot Mix, Hot Laid, Dense graded
			2	AC	2.2		Hot Mix, Hot Laid, Dense graded
			3	TB	7.5		Cement Aggregate Mixture
			4	SS	-		A-6

Table A – 4 (Continued) Pavement Structure

R2120AA	5	5	1	AC	2.8	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	GB	9.8	Crushed Stone
			4	TB	7.0	Cement Aggregate Mixture
			5	SS	-	A-5
R2120AB	5	5	1	AC	2.4	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	GB	9.8	Crushed Stone
			4	TB	8.0	Lime Treated Subgrade soil
			5	SS	-	A-5
R2123AC	6	6	1	AC	3.0	Hot Mix, Hot Laid, Dense graded
			2	AC	2.5	Hot Mix, Hot Laid, Dense graded
			3	AC	3.0	Hot Mix, Hot Laid, Dense graded
			4	GB	8.0	Crushed Stone
			5	TB	8.0	Lime Treated Subgrade soil
			6	SS	-	A-7-5
R2123BB	6	6	1	AC	2.75	Hot Mix, Hot Laid, Dense graded
			2	AC	2.75	Hot Mix, Hot Laid, Dense graded
			3	AC	3.9	Hot Mix, Hot Laid, Dense graded
			4	GB	8.0	Crushed Stone
			5	TB	8.0	Lime Treated Subgrade soil
			6	SS	-	A-7-5
R2123CC	6	6	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	AC	5.5	Hot Mix, Hot Laid, Dense graded
			4	GB	8.0	Crushed Stone
			5	TB	8.0	Lime Treated Subgrade soil
			6	SS	-	A-7-5

Table A – 4 (Continued) Pavement Structure

R2219AC	6	6	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	2.5	Hot Mix, Hot Laid, Dense graded
			3	AC	3.0	Hot Mix, Hot Laid, Dense graded
			4	GB	8.0	Crushed Stone
			5	TB	8.0	Lime Treated Subgrade soil
			6	SS	-	A-7-5
R2217B	4	4	1	AC	2.8	Hot Mix, Hot Laid, Dense graded
			2	AC	2.2	Hot Mix, Hot Laid, Dense graded
			3	AC	10.8	Hot Mix, Hot Laid, Dense graded
			4	SG	-	A-7-6
R1023AB	5	5	1	AC	2.4	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	GB	8.0	Crushed Stone
			4	TB	7.0	Cement Aggregate Mixture
			5	SS	-	A-2-4
R1023B	5	5	1	AC	2.4	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	GB	7.9	Crushed Stone
			4	TB	7.0	Cement Aggregate Mixture
			5	SS	-	A-2-4
R2000EA	5	5	1	AC	2.4	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	AC	4.3	Hot Mix, Hot Laid, Dense graded
			4	TB	15.0	Cement Aggregate Mixture
			5	SG	-	A-7-5
R2000EB	5	5	1	AC	2.4	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	AC	4.3	Hot Mix, Hot Laid, Dense graded
			4	TB	15.0	Cement Aggregate Mixture
			5	SG	-	A-2-4

Table A – 4 (Continued) Pavement Structure

R2001C	5	5	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	1.75	Hot Mix, Hot Laid, Dense graded
			3	AC	3.0	Hot Mix, Hot Laid, Dense graded
			4	GB	8.0	Crushed Stone
			5	SG	-	A-6
R2000BB	5	5	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	AC	4.5	Hot Mix, Hot Laid, Dense graded
			4	GB	15.0	Crushed Stone
			5	SG	-	A-6
U77LA	5	5	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	AC	4.5	Hot Mix, Hot Laid, Dense graded
			4	GB	8.0	Crushed Stone
			5	SG	-	A-6
R2318B	5	5	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	GB	8.0	Crushed Stone
			4	TB	8.0	Lime Treated Subgrade soil
			5	SS	-	A-7-6
U508CB	4	4	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	2.5	Hot Mix, Hot Laid, Dense graded
			3	AC	3.0	Hot Mix, Hot Laid, Dense graded
			4	SG	-	A-2-6
U508CA	4	4	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	2.5	Hot Mix, Hot Laid, Dense graded
			3	AC	3.0	Hot Mix, Hot Laid, Dense graded
			4	SG	-	A-2-6

Table A – 4 (Continued) Pavement Structure

R2232A&B	5	5	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	2.5	Hot Mix, Hot Laid, Dense graded
			3	AC	3.5	Hot Mix, Hot Laid, Dense graded
			4	TB	15.0	Cement Aggregate Mixture
			5	SG	-	A-7-6
R2211BA	5	5	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	GB	8.0	Crushed Stone
			4	TB	8.0	Lime Treated Subgrade soil
			5	SS	-	A-6
R519	5	5	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	2.5	Hot Mix, Hot Laid, Dense graded
			3	AC	3.0	Hot Mix, Hot Laid, Dense graded
			4	TB	7.0	Cement Aggregate Mixture
			5	SG	-	A-6
R85AD	5	5	1	AC	2.5	Hot Mix, Hot Laid, Dense graded
			2	AC	3.5	Hot Mix, Hot Laid, Dense graded
			3	AC	5.5	Hot Mix, Hot Laid, Dense graded
			4	TB	7.0	Cement Aggregate Mixture
			5	SG	-	A-7-6

Table A - 5 Aggregate Gradation for Asphalt Mixtures

Section	No. of Layers		Layer No.	Layer Type	Gradation			
					% Retained	% Retained	% Retained	% Passing
	CN1	CN2			¾	3/8	#4	#200
371006	5	8	1	AC	0	2	29	5.9
			2	AC	2	32	44	4.0
			3	AC				
			4	AC				
			5	AC	0	2	26	6.0
			6	AC	18	50	61	4.0
371024	4	6	1	AC	0	2	16	4.8
			2	AC	0	5	24	6.0
			3	AC	2	22	39	4.1
			4	AC	5	39	53	5.2
371028	3	5	1	AC				
			2	AC				
			3	AC	0	3	23	5.0
			4	TB	1	35	53	3.0
371030	3	3	1	AC	3	22	46	1.6
			2	TB	29	58	64	2.9
371040	5	6	1	AC	0	3	26	5.2
			2	AC				
			3	AC	0	4	22	5.7
			4	AC	7	37	53	3.8
371352	5	8	1	AC	0	5	32	5.0
			2	AC	0	2	29	6.0
			3	AC	0	5	35	5.0
			4	AC				
			5	AC	2	39	58	2.1
			6	AC	0	2	19	5.1
371645	4	6	1	AC	3	17	35	6.0
			2	AC	0	0	8	8.3
			3	AC	7	40	57	3.0
			4	AC	0	6	34	5.5
371801	4	6	1	AC	0	1	25	7.0
			2	AC	3	31	48	5.7
			3	AC	0	1	15	5.3

Table A -5 (Continued) Aggregate Gradation for Asphalt Mixtures

			4	AC	7	43	58	4.0
371802	4	5	1	AC	0	5	22	5.6
			2	AC	0	14	31	6.1
			3	AC	5	34	49	5.0
371803	5	8	1	AC	0	3	27	5.9
			2	AC	0	1	32	6.4
			3	AC	3	25	25	4.0
			4	AC				
			5	AC	0	5	23	6.9
			6	AC	8	46	61	4.9
371814	4	7	1	AC				
			2	AC				
			3	AC				
			4	AC	0	1	24	10.5
			5	AC	5	39	57	5.0
371817	4	8	1	AC				
			2	AC				
			3	AC	0	7	35	4.2
			4	AC				
			5	AC	0	6	35	6.9
			6	AC	1	29	54	4.5
371992	4	8	1	AC				
			2	AC	1	18	42	6.8
			3	AC	0	4	33	5.7
			4	AC				
			5	AC	0	4	31	7.1
372819	6	10	1	AC	0	4	29	6.5
			2	AC				
			3	AC	0	11	31	4.7
			4	AC				
			5	AC				
			6	AC	0	4	23	6.4
			7	AC	5	42	58	3.9
372824	4	6	1	AC				
			2	AC	0	6	34	7.8
			3	AC	0	13	42	6.0
			4	AC	1	32	51	4.0

Table A -5 (Continued) Aggregate Gradation for Asphalt Mixtures

372825	4	4	1	AC	0	5	32	7.7
			2	AC	4	43	55	5.3
R2120AA	5	5	1	AC	0	14	39.5	4.4
			2	AC	3	42.8	60.4	3.6
R2120AB	5	5	1	AC	0	14	39.5	4.4
			2	AC	2	46	66	3.4
R2123AC	6	6	1	AC	0	4	34	4.9
			2	AC	1	27	50	4.5
			3	AC	18	45	58	4.3
R2123BB	6	6	1	AC	0	16	53	4
			2	AC	14	43	67	4
			3	AC	1	26	61	4.1
R2123CC	6	6	1	AC	1	13	34	4.8
			2	AC	11	34	58	3.4
			3	AC	2	22	43	4.3
R2219AC	6	6	1	AC	0	13	37	4.3
			2	AC	1	22	57	5.3
			3	AC	11	47	57	4.6
R2217B	4	4	1	AC	0	13	39	4.3
			2	AC	1	26	39	4
			3	AC	14	34	49	4.7
R1023AB	5	5	1	AC	0	4	25	5.4
			2	AC	1	24	52	3.5
R1023B	5	5	1	AC	0	5	25	4.8
			2	AC	24	48	59	3.2
R2000EA	5	5	1	AC	1	14	40	3.3
			2	AC	2	33	52	3.1
			3	AC	20	48	52	2.9
R2000EB	5	5	1	AC	1	14	40	3.3
			2	AC	2	33	52	3.1
			3	AC	20	48	52	2.9
R2001C	5	5	1	AC	0	13	37	4.3
			2	AC	1	22	46	5.3
			3	AC	11	47	57	4.6
R2000BB	5	5	1	AC	1	14	40	3.3
			2	AC	2	33	52	3.1
			3	AC	20	48	52	2.9
U77LA	5	5	1	AC	1	14	40	3.3

Table A -5 (Continued) Aggregate Gradation for Asphalt Mixtures

			2	AC	2	33	52	3.1
			3	AC	20	48	52	2.9
R2318B	5	5	1	AC	0	5	25	4.8
			2	AC	24	48	59	3.2
U508CB	4	4	1	AC	0	13	37	4.3
			2	AC	1	22	46	5.3
			3	AC	11	47	57	4.6
U508CA	4	4	1	AC	0	13	37	4.3
			2	AC	1	22	46	5.3
			3	AC	11	47	57	4.6
R2232A&B	5	5	1	AC	1	14	40	3.3
			2	AC	2	33	52	3.1
			3	AC	20	48	52	2.9
R2211BA	5	5	1	AC	0	5	25	4.8
			2	AC	24	48	59	3.2
R519	5	5	1	AC	0	13	37	4.3
			2	AC	1	22	46	5.3
			3	AC	11	47	57	4.6
R85AD	5	5	1	AC	0	13	37	4.3
			2	AC	1	22	46	5.3
			3	AC	11	47	57	4.6

Table A - 6 Performance Data – Rut Depth

Section	Survey Date	CN No.	LLH Mean depth (in)	RLH Mean Depth (in)	Max Mean Depth (in)	Average Rut Depth (in)
1006	13-Oct-89	1	0.079	0.079	0.079	0.079
1006	19-Mar-91	1	0.079	0.079	0.079	0.079
1006	11-Oct-92	2	0.236	0.157	0.236	0.197
1006	18-Apr-94	2	0.118	0.079	0.118	0.098
1006	20-Sep-94	2	0.157	0.079	0.157	0.118
1006	08-Feb-96	3	0.157	0.197	0.197	0.177
1006	12-Mar-98	3	0.079	0.118	0.118	0.098
1006	12-Dec-00	3	0.118	0.157	0.157	0.138
1006	14-Mar-01	3	0.118	0.118	0.118	0.118
1006	27-Jan-03	3	0.118	0.197	0.197	0.157
1006	28-Aug-03	3	0.118	0.197	0.197	0.157
1024	03-Nov-89	1	0.354	0.354	0.354	0.354
1024	09-Mar-91	1	0.433	0.394	0.433	0.413
1024	10-Apr-92	1	0.315	0.354	0.354	0.335
1024	14-Oct-92	1	0.157	0.157	0.157	0.157
1024	31-Jan-96	2	0.236	0.276	0.276	0.256
1024	29-Apr-98	2	0.157	0.157	0.157	0.157
1024	09-Mar-01	2	0.197	0.157	0.197	0.177
1024	26-Jun-02	2	0.197	0.197	0.197	0.197
1024	09-Oct-02	2	0.236	0.197	0.236	0.217
1024	17-Mar-04	2	0.197	0.236	0.236	0.217
1028	12-Oct-89	1	0.433	0.394	0.433	0.413
1028	20-Mar-91	1	0.433	0.394	0.433	0.413
1028	10-Oct-92	1	0.512	0.512	0.512	0.512
1028	09-Feb-96	1	0.433	0.512	0.512	0.472
1028	18-Apr-96	1	0.512	0.472	0.512	0.492
1028	15-Aug-96	1	0.433	0.433	0.433	0.433
1028	02-Oct-97	1	0.512	0.512	0.512	0.512
1028	17-Mar-98	1	0.551	0.512	0.551	0.531
1028	29-Sep-98	1	0.551	0.472	0.551	0.512
1028	03-Jan-01	1	0.591	0.551	0.591	0.571
1028	14-Mar-01	1	0.433	0.433	0.433	0.433
1028	22-Mar-02	1	0.551	0.512	0.551	0.531
1028	16-Jan-03	2	0.039	0.079	0.079	0.059
1028	25-Jan-03	2	0.079	0.118	0.118	0.098
1030	12-Oct-89	1	0.276	0.236	0.276	0.256
1030	20-Mar-91	1	0.315	0.236	0.315	0.276

Table A – 6 (Continued) Performance Data – Rut Depth

1030	10-Oct-92	1	0.394	0.354	0.394	0.374
1030	09-Feb-96	1	0.315	0.315	0.315	0.315
1030	09-Oct-97	1	0.394	0.315	0.394	0.354
1030	29-Aug-00	1	0.394	0.354	0.394	0.374
1040	03-Nov-89	1	0.472	0.354	0.472	0.413
1040	11-Mar-91	1	0.472	0.354	0.472	0.413
1040	15-Oct-92	1	0.551	0.512	0.551	0.531
1040	12-Dec-95	2	0.039	0.079	0.079	0.059
1040	31-Jan-96	2	0.118	0.157	0.157	0.138
1040	18-Nov-98	2	0.079	0.118	0.118	0.098
1040	15-Feb-01	2	0.118	0.157	0.157	0.138
1040	09-Mar-01	2	0.079	0.157	0.157	0.118
1040	25-Mar-04	2	0.118	0.197	0.197	0.157
1352	09-Mar-89	1	0.236	0.276	0.276	0.256
1352	18-Mar-91	2	0.079	0.079	0.079	0.079
1352	15-Oct-92	2	0.079	0.118	0.118	0.098
1352	21-Apr-94	2	0.157	0.039	0.157	0.098
1352	06-Feb-96	2	0.079	0.079	0.079	0.079
1352	23-Apr-98	2	0.118	0.039	0.118	0.079
1352	24-Jan-01	2	0.197	0.118	0.197	0.157
1352	11-Mar-01	2	0.118	0.079	0.118	0.098
1352	09-Oct-02	2	0.118	0.079	0.118	0.098
1352	28-May-03	2	0.118	0.039	0.118	0.079
1352	23-Mar-04	3	0.039	0.079	0.079	0.059
1645	15-Mar-89	1	0.197	0.315	0.315	0.256
1645	06-Mar-91	1	0.276	0.276	0.276	0.276
1645	12-Oct-92	1	0.276	0.472	0.472	0.374
1645	19-Apr-94	1	0.236	0.354	0.354	0.295
1645	29-Jan-96	1	0.315	0.433	0.433	0.374
1645	05-Feb-98	1	0.236	0.354	0.354	0.295
1645	29-Feb-00	1	0.276	0.394	0.394	0.335
1645	10-Oct-00	2	0.039	0.039	0.039	0.039
1645	10-Mar-01	2	0.079	0.079	0.079	0.079
1645	27-Jun-02	2	0.039	0.079	0.079	0.059
1645	27-Jan-03	2	0.079	0.118	0.118	0.098
1801	15-Mar-89	1	0.315	0.394	0.394	0.354
1801	10-Mar-91	1	0.354	0.354	0.354	0.354
1801	14-Oct-92	1	0.394	0.433	0.433	0.413
1801	26-Jan-96	1	0.354	0.394	0.394	0.374
1801	31-Jan-96	1	0.394	0.433	0.433	0.413
1801	25-Jul-96	1	0.354	0.394	0.394	0.374

Table A – 6 (Continued) Performance Data – Rut Depth

1801	28-Apr-98	2	0.079	0.079	0.079	0.079
1801	18-May-00	2	0.079	0.079	0.079	0.079
1801	09-Mar-01	2	0.079	0.118	0.118	0.098
1801	09-Oct-02	2	0.118	0.118	0.118	0.118
1801	12-Mar-03	2	0.079	0.079	0.079	0.079
1802	13-Oct-89	1	0.354	0.157	0.354	0.256
1802	18-Mar-91	1	0.315	0.197	0.315	0.256
1802	10-Oct-92	1	0.354	0.276	0.354	0.315
1802	15-Apr-94	1	0.433	0.236	0.433	0.335
1802	18-Jul-95	1	0.472	0.236	0.472	0.354
1802	09-Feb-96	1	0.512	0.315	0.512	0.413
1802	02-Apr-96	1	0.512	0.315	0.512	0.413
1802	11-Dec-96	2	0.118	0.157	0.157	0.138
1802	10-Oct-97	2	0.118	0.118	0.118	0.118
1802	02-Feb-00	2	0.197	0.157	0.197	0.177
1802	14-Mar-01	2	0.118	0.157	0.157	0.138
1802	15-Jan-02	2	0.197	0.197	0.197	0.197
1802	25-Jan-02	2	0.157	0.157	0.157	0.157
1803	03-Nov-89	1	0.433	0.394	0.433	0.413
1803	06-Jun-90	1	0.394	0.394	0.394	0.394
1803	14-Oct-92	2	0.157	0.236	0.236	0.197
1803	31-Jan-96	2	0.157	0.276	0.276	0.217
1803	22-Apr-96	2	0.079	0.197	0.197	0.138
1803	07-Apr-99	2	0.079	0.118	0.118	0.098
1803	17-Nov-99	2	0.039	0.079	0.079	0.059
1803	14-Nov-01	3	0.079	0.197	0.197	0.138
1803	03-Jan-02	3	0.118	0.197	0.197	0.157
1803	28-Jan-03	3	0.079	0.157	0.157	0.118
1803	06-Jan-04	3	0.079	0.197	0.197	0.138
1814	13-Mar-89	1	0.236	0.157	0.236	0.197
1814	09-Mar-91	1	0.236	0.157	0.236	0.197
1814	14-Oct-92	1	0.236	0.197	0.236	0.217
1814	30-Jan-96	1	0.276	0.157	0.276	0.217
1814	31-Jan-96	1	0.276	0.236	0.276	0.256
1814	01-Apr-99	1	0.315	0.157	0.315	0.236
1814	26-Jun-00	1	0.354	0.197	0.354	0.276
1814	12-Oct-00	2	0.079	0.039	0.079	0.059
1814	09-Mar-01	2	0.079	0.118	0.118	0.098
1814	09-Oct-02	2	0.118	0.079	0.118	0.098
1814	07-Jun-05	2	0.118	0.079	0.118	0.098
1817	15-Oct-89	1	0.276	0.512	0.512	0.394

Table A – 6 (Continued) Performance Data – Rut Depth

1817	18-Mar-91	2	0.276	0.236	0.276	0.256
1817	18-Oct-92	2	0.354	0.354	0.354	0.354
1817	15-Dec-95	4	0.079	0.118	0.118	0.098
1817	06-Feb-96	4	0.197	0.236	0.236	0.217
1817	27-Apr-99	4	0.079	0.157	0.157	0.118
1817	11-Mar-01	4	0.118	0.118	0.118	0.118
1817	13-Mar-02	4	0.079	0.157	0.157	0.118
1817	05-Feb-03	5	0.157	0.157	0.157	0.157
1992	15-Oct-92	1	0.157	0.236	0.236	0.197
1992	20-Apr-94	1	0.039	0.039	0.039	0.039
1992	06-Feb-96	1	0.118	0.157	0.157	0.138
1992	22-Apr-98	2	0.118	0.236	0.236	0.177
1992	16-May-00	2	0.236	0.315	0.315	0.276
1992	14-Mar-01	2	0.157	0.236	0.236	0.197
1992	26-Jan-03	2	0.157	0.276	0.276	0.217
1992	06-Mar-03	2	0.276	0.354	0.354	0.315
2819	13-Oct-89	1	0.433	0.472	0.472	0.453
2819	18-Mar-91	1	0.433	0.433	0.433	0.433
2819	13-Apr-92	1	0.394	0.433	0.433	0.413
2819	13-Jan-93	2	0.118	0.118	0.118	0.118
2819	06-Feb-96	2	0.118	0.157	0.157	0.138
2819	13-Aug-97	2	0.157	0.236	0.236	0.197
2819	30-Aug-00	2	0.236	0.276	0.276	0.256
2819	14-Mar-01	2	0.197	0.197	0.197	0.197
2819	28-Mar-02	2	0.236	0.276	0.276	0.256
2819	06-Feb-03	3	0.079	0.079	0.079	0.079
2824	13-Oct-89	1	0.079	0.118	0.118	0.098
2824	18-Mar-91	1	0.079	0.118	0.118	0.098
2824	15-Oct-92	2	0.236	0.236	0.236	0.236
2824	06-Feb-96	2	0.236	0.315	0.315	0.276
2824	14-Aug-97	2	0.315	0.315	0.315	0.315
2824	02-Dec-99	2	0.315	0.354	0.354	0.335
2824	31-Aug-00	2	0.315	0.354	0.354	0.335
2824	14-Mar-01	2	0.354	0.315	0.354	0.335
2824	26-Jan-03	2	0.315	0.315	0.315	0.315
2824	27-Mar-03	2	0.354	0.354	0.354	0.354
2824	24-Mar-04	3	0.236	0.079	0.236	0.157
2825	09-Mar-89	1	0.157	0.157	0.157	0.157
2825	07-Mar-91	1	0.157	0.157	0.157	0.157
2825	15-Oct-92	1	0.197	0.236	0.236	0.217
2825	19-Jul-95	1	0.157	0.118	0.157	0.138

Table A – 6 (Continued) Performance Data – Rut Depth

2825	31-Jan-96	1	0.236	0.276	0.276	0.256
2825	17-Nov-98	1	0.197	0.197	0.197	0.197
2825	14-Feb-01	1	0.315	0.276	0.315	0.295
2825	11-Mar-01	1	0.197	0.157	0.197	0.177
2825	09-Oct-02	1	0.197	0.197	0.197	0.197
2825	23-Mar-04	1	0.315	0.276	0.315	0.295
NB, R-2120AA	1/1/2004	1	-	-	0.000	0.255
	1/1/2006	1	-	-	0.000	0.302
SB, R-2120AA	1/1/2004	1	-	-	0.000	0.250
	1/1/2006	1	-	-	0.000	0.295
NB, R-2120AB	1/1/2004	1	-	-	0.000	0.261
	1/1/2006	1	-	-	0.000	0.310
SB, R-2120AB	1/1/2004	1	-	-	0.000	0.253
	1/1/2006	1	-	-	0.000	0.297
Inner,R-2123AC	6/1/2004	1	-	-	0.000	0.209
	6/1/2005	1	-	-	0.000	0.241
	6/1/2006	1	-	-	0.000	0.266
Outer, R-2123AC	6/1/2004	1	-	-	0.000	0.194
	6/1/2005	1	-	-	0.000	0.225
	6/1/2006	1	-	-	0.000	0.248
Outer, R-2123BB	6/1/2004	1	-	-	0.000	0.148
	6/1/2005	1	-	-	0.000	0.190
	6/1/2006	1	-	-	0.000	0.212
Inner, R-2123CC	6/1/2004	1	-	-	0.000	0.153
	6/1/2005	1	-	-	0.000	0.182
	6/1/2006	1	-	-	0.000	0.202
Outer, R-2123CC	6/1/2004	1	-	-	0.000	0.136
	6/1/2005	1	-	-	0.000	0.163
	6/1/2006	1	-	-	0.000	0.181
EB, R-2219AC	1/1/2002	1	-	-	0.000	0.200
	1/1/2004	1	-	-	0.000	0.268
	1/1/2006	1	-	-	0.000	0.305
WB, R-2219AC	1/1/2002	1	-	-	0.000	0.168
	1/1/2004	1	-	-	0.000	0.232
	1/1/2006	1	-	-	0.000	0.264
EB, R-2217B	1/1/2004	1	-	-	0.000	0.128
	1/1/2006	1	-	-	0.000	0.152
WB, R-2217B	1/1/2004	1	-	-	0.000	0.117
	1/1/2006	1	-	-	0.000	0.141
EB, R-1023AB	1/1/2004	1	-	-	0.000	0.293
	1/1/2006	1	-	-	0.000	0.378

Table A – 6 (Continued) Performance Data – Rut Depth

WB, R-1023AB	1/1/2004	1	-	-	0.000	0.404
	1/1/2006	1	-	-	0.000	0.508
EB, R-1023B	1/1/2004	1	-	-	0.000	0.338
	1/1/2006	1	-	-	0.000	0.426
WB, R-1023B	1/1/2004	1	-	-	0.000	0.315
	1/1/2006	1	-	-	0.000	0.398
EB/Inner, R-2000EA	6/1/2003	1	-	-	0.000	0.061
	6/1/2004	1	-	-	0.000	0.077
	6/1/2005	1	-	-	0.000	0.088
	6/1/2006	1	-	-	0.000	0.097
WB/Outer, R-2000EA	6/1/2003	1	-	-	0.000	0.059
	6/1/2004	1	-	-	0.000	0.074
	6/1/2005	1	-	-	0.000	0.084
	6/1/2006	1	-	-	0.000	0.092
EB/Inner, R-2000EB	6/1/2003	1	-	-	0.000	0.064
	6/1/2004	1	-	-	0.000	0.079
	6/1/2005	1	-	-	0.000	0.089
	6/1/2006	1	-	-	0.000	0.096
WB/Outer, R-2000EB	6/1/2003	1	-	-	0.000	0.066
	6/1/2004	1	-	-	0.000	0.082
	6/1/2005	1	-	-	0.000	0.093
	6/1/2006	1	-	-	0.000	0.102
NB,R-2001C	1/1/2004	1	-	-	0.000	0.363
	1/1/2006	1	-	-	0.000	0.410
SB, R-2001C	1/1/2006	1	-	-	0.000	0.406
NB, R-2120AA	1/1/2004	1	-	-	0.000	0.255
	1/1/2006	1	-	-	0.000	0.302
SB, R-2120AA	1/1/2004	1	-	-	0.000	0.250
EB, R-2000BB	Jun-97	1	-	-	0.000	0.106
	Jun-00	1	-	-	0.000	0.127
	Jun-01	1	-	-	0.000	0.133
	Jun-02	1	-	-	0.000	0.137
	Jun-03	1	-	-	0.000	0.142
	Jun-04	1	-	-	0.000	0.147
	Jun-05	1	-	-	0.000	0.152
	Jun-06	1	-	-	0.000	0.156
WB (Outer), R-2000BB	Jun-97	1	-	-	0.000	0.111
	Jun-98	1	-	-	0.000	0.118
	Jun-00	1	-	-	0.000	0.132

Table A – 6 (Continued) Performance Data – Rut Depth

	Jun-01	1	-	-	0.000	0.137
	Jun-02	1	-	-	0.000	0.142
	Jun-03	1	-	-	0.000	0.147
	Jun-04	1	-	-	0.000	0.152
	Jun-05	1	-	-	0.000	0.156
	Jun-06	1	-	-	0.500	0.161
WB, R-2318B	Jan-96	1	-	-	0.500	0.157
	Jan-98	1	-	-	0.500	0.181
	Jan-00	1	-	-	0.500	0.200
	Jan-02	1	-	-	0.000	0.212
	Jan-04	1	-	-	0.000	0.224
	Jan-06	1	-	-	0.500	0.233
NB, U-508CB	Jan-96	1	-	-	0.000	0.263
	Jan-98	1	-	-	0.000	0.302
	Jan-00	1	-	-	0.000	0.326
	Jan-02	1	-	-	0.000	0.346
	Jan-04	1	-	-	0.000	0.362
	Jan-06	1	-	-	0.000	0.377
SB, U-508CB	Jan-96	1	-	-	0.000	0.365
	Jan-98	1	-	-	0.000	0.419
	Jan-00	1	-	-	0.000	0.453
	Jan-02	1	-	-	0.000	0.481
	Jan-04	1	-	-	0.000	0.503
	Jan-06	1	-	-	0.000	0.524
NB, U-77LA	Jan-00	1	-	-	0.000	0.278
	Jan-02	1	-	-	0.000	0.295
	Jan-04	1	-	-	0.000	0.312
	Jan-06	1	-	-	0.000	0.330
SB, U-77LA	Jan-00	1	-	-	0.000	0.271
	Jan-02	1	-	-	0.000	0.290
	Jan-04	1	-	-	0.000	0.307
	Jan-06	1	-	-	0.000	0.324
EB, R-2211BA	Jan-98	1	-	-	0.000	0.236
	Jan-00	1	-	-	0.000	0.310
	Jan-02	1	-	-	0.000	0.344
	Jan-04	1	-	-	0.000	0.370
	Jan-06	1	-	-	0.500	0.394
WB, R-2211BA	Jan-98	1	-	-	0.000	0.232
	Jan-00	1	-	-	0.000	0.305
	Jan-02	1	-	-	0.000	0.338
	Jan-04	1	-	-	0.000	0.364

Table A – 6 (Continued) Performance Data – Rut Depth

	Jan-06	1	-	-	0.500	0.387
NB, R-2232A&B	Jan-98	1	-	-	0.000	0.196
	Jan-00	1	-	-	0.000	0.262
	Jan-02	1	-	-	0.000	0.312
	Jan-04	1	-	-	0.000	0.350
	Jan-06	1	-	-	0.500	0.376
SB, R-2232A&B	Jan-98	1	-	-	0.000	0.183
	Jan-00	1	-	-	0.000	0.245
	Jan-02	1	-	-	0.000	0.292
	Jan-04	1	-	-	0.000	0.327
	Jan-06	1	-	-	0.500	0.351
NB, R-1017AC	Jan-94	1	-	-	0.500	0.169
	Jan-96	1	-	-	0.000	0.222
	Jan-98	1	-	-	0.500	0.252
	Jan-00	1	-	-	0.500	0.274
	Jan-02	1	-	-	0.000	0.297
	Jan-04	1	-	-	0.000	0.314
	Jan-06	1	-	-	0.000	0.330
NB, R-85AD	Jan-02	1	-	-	0.000	0.164
	Jan-04	1	-	-	0.000	0.175
	Jan-06	1	-	-	0.000	0.184
SB, R-85AD	Jan-02	1	-	-	0.000	0.155
	Jan-04	1	-	-	0.000	0.165
	Jan-06	1	-	-	0.000	0.174
NB, R-519	Jan-06	1	-	-	0.500	4.635
	Jan-98	1	-	-	0.500	4.865
	Jan-00	1	-	-	0.500	5.062
	Jan-02	1	-	-	0.000	5.242
	Jan-04	1	-	-	0.000	5.378
	Jan-06	1	-	-	0.000	5.514
EB, R-2000BB	Jun-97	1	-	-	0.000	0.106
	Jun-00	1	-	-	0.000	0.127

Table A - 7 Performance Data – Alligator Cracking

Section	Survey Date	CN No.	Alligator Cracking (ft^2)			Total Cracking	% Area Cracked
			Low	Medium	High		
1006	17-Aug-91	1	0.00	0.00	0.00	0.00	0.0
1006	18-Apr-94	2	0.00	0.00	0.00	0.00	0.0
1006	20-Sep-94	2	0.00	0.00	0.00	0.00	0.0
1006	07-Jun-95	3	0.00	0.00	0.00	0.00	0.0
1006	12-Mar-98	3	0.00	0.00	0.00	0.00	0.0
1006	12-Dec-00	3	457.47	0.00	0.00	457.47	7.6
1006	28-Aug-03	3	937.54	0.00	0.00	937.54	15.6
1024	10-Apr-92	1	1442.36	307.85	342.29	2092.50	34.9
1024	29-Apr-98	2	2238.89	0.00	0.00	2238.89	37.3
1024	26-Jun-02	2	928.93	1570.45	0.00	2499.38	41.7
1024	17-Mar-04	2	924.62	3370.18	0.00	4294.80	71.6
1028	18-May-95	1	53.82	0.00	0.00	53.82	0.9
1028	18-Apr-96	1	1772.82	0.00	0.00	1772.82	29.5
1028	15-Aug-96	1	1313.20	0.00	0.00	1313.20	21.9
1028	02-Oct-97	1	1815.87	0.00	0.00	1815.87	30.3
1028	17-Mar-98	1	981.67	1243.23	0.00	2224.90	37.1
1028	29-Sep-98	1	822.36	984.90	0.00	1807.26	30.1
1028	03-Jan-01	1	818.06	818.06	0.00	1636.11	27.3
1028	22-Mar-02	1	985.97	1871.84	0.00	2857.82	47.6
1028	16-Jan-03	2	0.00	0.00	0.00	0.00	0.0
1030	09-Oct-97	1	0.00	0.00	0.00	0.00	0.0
1030	29-Aug-00	1	982.74	0.00	0.00	982.74	16.4
1040	13-Dec-95	2	0.00	0.00	0.00	0.00	0.0
1040	18-Nov-98	2	0.00	0.00	0.00	0.00	0.0
1040	15-Feb-01	2	0.00	0.00	0.00	0.00	0.0
1040	25-Mar-04	2	21.53	797.61	0.00	819.13	13.7
1352	21-Apr-94	2	0.00	0.00	0.00	0.00	0.0

Table A – 7 (Continued) Performance Data – Alligator Cracking

1352	23-Apr-98	2	502.67	0.00	0.00	502.67	8.4
1352	24-Jan-01	2	115.17	589.86	0.00	705.04	11.8
1352	28-May-03	2	282.01	304.62	0.00	586.63	9.8
1352	23-Mar-04	3	0.00	0.00	0.00	0.00	0.0
1645	19-Apr-94	1	0.00	0.00	0.00	0.00	0.0
1645	05-Feb-98	1	133.47	572.64	0.00	706.11	11.8
1645	29-Feb-00	1	219.58	89.34	38.75	347.67	5.8
1645	10-Oct-00	2	0.00	0.00	0.00	0.00	0.0
1645	27-Jun-02	2	0.00	0.00	0.00	0.00	0.0
1645	11-Apr-05	2	0.00	0.00	0.00	0.00	0.0
1801	26-Jan-96	1	0.00	0.00	0.00	0.00	0.0
1801	25-Jul-96	1	0.00	0.00	0.00	0.00	0.0
1801	28-Apr-98	2	0.00	0.00	0.00	0.00	0.0
1801	18-May-00	2	0.00	0.00	0.00	0.00	0.0
1801	12-Mar-03	2	0.00	0.00	0.00	0.00	0.0
1802	15-Apr-94	1	1520.94	0.00	0.00	1520.94	25.3
1802	18-Jul-95	1	1739.45	69.97	0.00	1809.41	30.2
1802	02-Apr-96	1	2608.10	205.59	0.00	2813.69	46.9
1802	11-Dec-96	2	504.83	0.00	0.00	504.83	8.4
1802	10-Oct-97	2	31.22	0.00	0.00	31.22	0.5
1802	15-Jan-02	2	1083.93	361.67	1977.33	3422.92	57.0
1803	22-Apr-96	2	51.67	0.00	0.00	51.67	0.9
1803	07-Apr-99	2	1337.95	123.78	0.00	1461.74	24.4
1803	14-Nov-01	3	589.86	0.00	0.00	589.86	9.8
1803	28-Jan-03	3	476.84	0.00	0.00	476.84	7.9
1814	30-Jan-96	1	33.37	0.00	0.00	33.37	0.6
1814	01-Apr-99	1	166.84	113.02	0.00	279.86	4.7
1814	26-Jun-00	1	49.51	74.27	767.47	891.25	14.9
1814	12-Oct-00	2	0.00	0.00	0.00	0.00	0.0

Table A – 7 (Continued) Performance Data – Alligator Cracking

1814	07-Jun-05	2	772.85	124.86	0.00	897.71	15.0
1817	30-Aug-90	1	1325.04	0.00	0.00	1325.04	22.1
1817	15-Dec-95	4	0.00	0.00	0.00	0.00	0.0
1817	27-Apr-99	4	0.00	0.00	0.00	0.00	0.0
1817	13-Mar-02	4	0.00	0.00	0.00	0.00	0.0
1817	05-Feb-03	5	0.00	0.00	0.00	0.00	0.0
1992	20-Apr-94	1	0.00	0.00	0.00	0.00	0.0
1992	22-Apr-98	2	0.00	0.00	0.00	0.00	0.0
1992	16-May-00	2	0.00	0.00	0.00	0.00	0.0
1992	06-Mar-03	2	0.00	0.00	0.00	0.00	0.0
2819	13-Apr-92	1	0.00	0.00	0.00	0.00	0.0
2819	13-Aug-97	2	0.00	0.00	0.00	0.00	0.0
2819	30-Aug-00	2	0.00	0.00	0.00	0.00	0.0
2819	28-Mar-02	2	0.00	0.00	0.00	0.00	0.0
2819	06-Feb-03	3	0.00	0.00	0.00	0.00	0.0
2824	14-Aug-97	2	0.00	0.00	0.00	0.00	0.0
2824	02-Dec-99	2	0.00	0.00	0.00	0.00	0.0
2824	31-Aug-00	2	5.38	0.00	0.00	5.38	0.1
2824	27-Mar-03	2	61.35	0.00	0.00	61.35	1.0
2824	24-Mar-04	3	0.00	0.00	0.00	0.00	0.0
2825	19-Jul-95	1	687.81	0.00	0.00	687.81	11.5
2825	17-Nov-98	1	294.93	1238.93	0.00	1533.86	25.6
2825	14-Feb-01	1	20.45	1796.50	0.00	1816.95	30.3
2825	23-Mar-04	1	196.98	1406.84	268.02	1871.84	31.2
NB, R-2120AA	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
SB, R-2120AA	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
NB, R-2120AB	1/1/2004	1	-	-	-	-	0.00

Table A – 7 (Continued) Performance Data – Alligator Cracking

	1/1/2006	1	-	-	-	-	0.00
SB, R-2120AB	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
Inner,R-2123AC	6/1/2004	1	-	-	-	-	0.00
	6/1/2005	1	-	-	-	-	0.00
	6/1/2006	1	-	-	-	-	0.00
Outer, R-2123AC	6/1/2004	1	-	-	-	-	0.00
	6/1/2005	1	-	-	-	-	0.00
	6/1/2006	1	-	-	-	-	0.00
Outer, R-2123BB	6/1/2004	1	-	-	-	-	0.00
	6/1/2005	1	-	-	-	-	0.00
	6/1/2006	1	-	-	-	-	0.00
Inner, R-2123CC	6/1/2004	1	-	-	-	-	0.69
	6/1/2005	1	-	-	-	-	2.08
	6/1/2006	1	-	-	-	-	2.08
Outer, R-2123CC	6/1/2004	1	-	-	-	-	0.00
	6/1/2005	1	-	-	-	-	0.00
	6/1/2006	1	-	-	-	-	0.00
EB, R-2219AC	1/1/2002	1	-	-	-	-	0.00
	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
WB, R-2219AC	1/1/2002	1	-	-	-	-	0.00
	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
EB, R-2217B	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
WB, R-2217B	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
EB, R-1023AB	1/1/2004	1	-	-	-	-	0.00

Table A – 7 (Continued) Performance Data – Alligator Cracking

	1/1/2006	1	-	-	-	-	2.08
WB, R-1023AB	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
EB, R-1023B	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	2.08
WB, R-1023B	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	2.07
EB/Inner, R-2000EA	6/1/2002	1	-	-	-	-	0.00
	6/1/2003	1	-	-	-	-	0.00
	6/1/2004	1	-	-	-	-	0.00
	6/1/2005	1	-	-	-	-	0.00
	6/1/2006	1	-	-	-	-	0.00
WB/Outer, R-2000EA	6/1/2002	1	-	-	-	-	0.00
	6/1/2003	1	-	-	-	-	0.00
	6/1/2004	1	-	-	-	-	0.00
	6/1/2005	1	-	-	-	-	0.00
	6/1/2006	1	-	-	-	-	0.00
EB/Inner, R-2000EB	6/1/2002	1	-	-	-	-	0.00
	6/1/2003	1	-	-	-	-	0.00
	6/1/2004	1	-	-	-	-	0.00
	6/1/2005	1	-	-	-	-	0.00
	6/1/2006	1	-	-	-	-	0.00
WB/Outer, R-2000EB	6/1/2002	1	-	-	-	-	0.00
	6/1/2003	1	-	-	-	-	0.00
	6/1/2004	1	-	-	-	-	0.00
	6/1/2005	1	-	-	-	-	0.00
	6/1/2006	1	-	-	-	-	0.00
NB,R-2001C	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00

Table A – 7 (Continued) Performance Data – Alligator Cracking

SB, R-2001C	1/1/2006	1	-	-	-	-	0.00
EB, R-2000BB	6/1/1997	1	-	-	-	-	0.00
	6/1/2000	1	-	-	-	-	0.00
	6/1/2001	1	-	-	-	-	0.00
	6/1/2002	1	-	-	-	-	0.00
	6/1/2003	1	-	-	-	-	2.08
	6/1/2004	1	-	-	-	-	2.08
	6/1/2005	1	-	-	-	-	2.08
	6/1/2006	1	-	-	-	-	0.00
WB (Outer), R-2000BB	6/1/1997	1	-	-	-	-	0.00
	6/1/1998	1	-	-	-	-	0.00
	6/1/2000	1	-	-	-	-	0.00
	6/1/2001	1	-	-	-	-	0.00
	6/1/2002	1	-	-	-	-	0.00
	6/1/2003	1	-	-	-	-	2.08
	6/1/2004	1	-	-	-	-	2.08
	6/1/2005	1	-	-	-	-	2.08
	6/1/2006	1	-	-	-	-	2.08
WB, R-2318B	1/1/1996	1	-	-	-	-	0.00
	1/1/1998	1	-	-	-	-	0.00
	1/1/2000	1	-	-	-	-	0.00
	1/1/2002	1	-	-	-	-	6.25
	1/1/2004	1	-	-	-	-	6.25
	1/1/2006	1	-	-	-	-	6.25
NB, U-508CB	1/1/1996	1	-	-	-	-	0.00
	1/1/1998	1	-	-	-	-	0.00
	1/1/2000	1	-	-	-	-	0.00
	1/1/2002	1	-	-	-	-	0.00
	1/1/2004	1	-	-	-	-	0.00

Table A – 7 (Continued) Performance Data – Alligator Cracking

	1/1/2006	1	-	-	-	-	6.23
SB, U-508CB	1/1/1996	1	-	-	-	-	0.00
	1/1/1998	1	-	-	-	-	0.00
	1/1/2000	1	-	-	-	-	0.00
	1/1/2002	1	-	-	-	-	0.00
	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	4.16
NB, U-77LA	1/1/2000	1	-	-	-	-	0.00
	1/1/2002	1	-	-	-	-	4.16
	1/1/2004	1	-	-	-	-	2.08
	1/1/2006	1	-	-	-	-	2.08
SB, U-77LA	1/1/2000	1	-	-	-	-	0.00
	1/1/2002	1	-	-	-	-	4.16
	1/1/2004	1	-	-	-	-	2.08
	1/1/2006	1	-	-	-	-	2.08
EB, R-2211BA	1/1/1998	1	-	-	-	-	0.00
	1/1/2000	1	-	-	-	-	7.14
	1/1/2002	1	-	-	-	-	7.14
	1/1/2004	1	-	-	-	-	8.50
	1/1/2006	1	-	-	-	-	16.15
WB, R-2211BA	1/1/1998	1	-	-	-	-	0.00
	1/1/2000	1	-	-	-	-	0.00
	1/1/2002	1	-	-	-	-	0.00
	1/1/2004	1	-	-	-	-	1.61
	1/1/2006	1	-	-	-	-	20.33
NB, R-2232A&B	1/1/1998	1	-	-	-	-	0.00
	1/1/2000	1	-	-	-	-	0.00
	1/1/2002	1	-	-	-	-	2.67
	1/1/2004	1	-	-	-	-	6.84

Table A – 7 (Continued) Performance Data – Alligator Cracking

	1/1/2006	1	-	-	-	-	8.38
SB, R-2232A&B	1/1/1998	1	-	-	-	-	0.00
	1/1/2000	1	-	-	-	-	0.13
	1/1/2002	1	-	-	-	-	3.29
	1/1/2004	1	-	-	-	-	5.99
	1/1/2006	1	-	-	-	-	5.41
NB, R-1017AC	1/1/1994	1	-	-	-	-	5.26
	1/1/1996	1	-	-	-	-	0.00
	1/1/1998	1	-	-	-	-	0.00
	1/1/2000	1	-	-	-	-	2.08
	1/1/2002	1	-	-	-	-	0.00
	1/1/2004	1	-	-	-	-	1.04
	1/1/2006	1	-	-	-	-	0.00
NB, R-85AD	1/1/2002	1	-	-	-	-	0.00
	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
SB, R-85AD	1/1/2002	1	-	-	-	-	0.00
	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	0.00
NB, R-519	1/1/2006	1	-	-	-	-	0.00
	1/1/1998	1	-	-	-	-	0.00
	1/1/2000	1	-	-	-	-	2.08
	1/1/2002	1	-	-	-	-	4.17
	1/1/2004	1	-	-	-	-	0.00
	1/1/2006	1	-	-	-	-	20.83

APPENDIX B: CLUSTERING ANALYSIS TECHNIQUE

Table B - 1 Normalized Annual Tandem Axle Load Distribution Data

Load (kN)	**0200**	**0900**	**1006**	**1352**	**1802**	**1817**	**1992**	**2819**	**2824**	**2825**	**3008**	**3044**	**3807**	**3816**	**5827**
8.9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17.79	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01
26.69	0.01	0.04	0.01	0.02	0.04	0	0.01	0	0.02	0.01	0	0	0.01	0.03	0.02
35.59	0.01	0.07	0.04	0.06	0.04	0.01	0.05	0	0.04	0.04	0.01	0.02	0.02	0.06	0.1
44.48	0.05	0.08	0.09	0.1	0.1	0.02	0.1	0.02	0.1	0.11	0.04	0.06	0.04	0.13	0.15
53.38	0.09	0.09	0.12	0.12	0.1	0.05	0.13	0.1	0.15	0.15	0.09	0.08	0.09	0.11	0.12
62.28	0.12	0.09	0.12	0.11	0.14	0.12	0.11	0.18	0.12	0.1	0.11	0.08	0.11	0.1	0.09
71.17	0.1	0.07	0.09	0.08	0.1	0.15	0.08	0.15	0.07	0.06	0.1	0.07	0.1	0.08	0.07
80.07	0.08	0.05	0.07	0.06	0.09	0.14	0.05	0.09	0.06	0.09	0.1	0.06	0.09	0.06	0.06
88.96	0.06	0.04	0.06	0.05	0.03	0.1	0.04	0.07	0.05	0.06	0.08	0.06	0.07	0.06	0.07
97.86	0.05	0.04	0.05	0.04	0.02	0.07	0.03	0.06	0.05	0.04	0.07	0.05	0.06	0.05	0.08
106.76	0.05	0.04	0.05	0.04	0.02	0.08	0.03	0.08	0.06	0.04	0.06	0.06	0.06	0.06	0.08
115.65	0.04	0.03	0.05	0.04	0.03	0.06	0.04	0.11	0.06	0.06	0.05	0.06	0.05	0.09	0.05
124.55	0.04	0.03	0.05	0.04	0.03	0.04	0.05	0.09	0.05	0.04	0.05	0.07	0.05	0.09	0.03
133.45	0.05	0.03	0.06	0.04	0.02	0.04	0.06	0.04	0.04	0.07	0.06	0.07	0.06	0.05	0.03
142.34	0.05	0.03	0.05	0.04	0.02	0.03	0.07	0.01	0.04	0.05	0.06	0.07	0.06	0.02	0.02
151.24	0.05	0.04	0.04	0.03	0.01	0.02	0.06	0	0.04	0.03	0.05	0.05	0.06	0.01	0.01
160.14	0.05	0.04	0.03	0.03	0.01	0.02	0.04	0	0.03	0.02	0.03	0.04	0.04	0	0.01
169.03	0.03	0.04	0.01	0.03	0.02	0.01	0.02	0	0.02	0.01	0.01	0.02	0.02	0	0
177.93	0.02	0.03	0.01	0.02	0.03	0.01	0.01	0	0.01	0.01	0.01	0.02	0.01	0	0
186.83	0.01	0.03	0	0.01	0.02	0.01	0	0	0	0.02	0	0.01	0	0	0
195.72	0.01	0.02	0	0.01	0.02	0.01	0	0	0	0	0	0.01	0	0	0
204.62	0	0.02	0	0.01	0.01	0	0	0	0	0.01	0	0.01	0	0	0
213.51	0	0.01	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0
222.41	0	0.01	0	0	0.02	0	0	0	0	0	0	0	0	0	0
231.31	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0
240.2	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0

Table B – 1 (Continued) Normalized Annual Tandem Axle Load Distribution Data

249.1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
258	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
266.89	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
275.79	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
284.69	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
293.58	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
302.48	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
311.38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
320.27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
329.17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
338.06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
346.96	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
355.86	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Figure B - 1 Normalized Annual Tandem Axle Load Distribution Plot

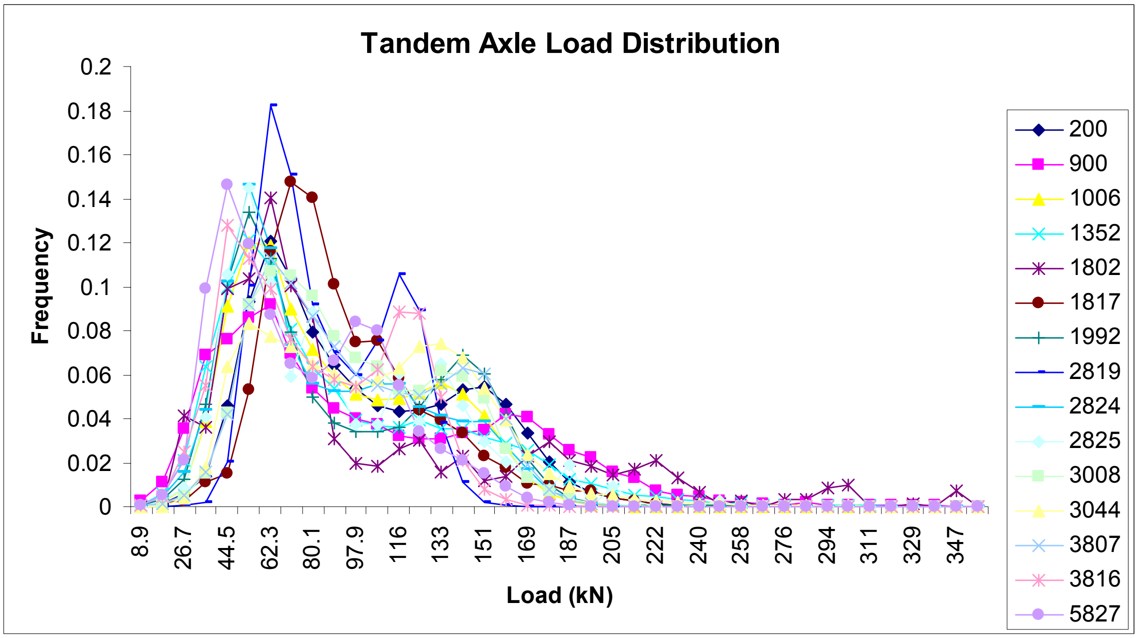

Table B - 2 Vehicle Class Distribution Data

SHRP ID	Vehicle Class									
	4	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
0200	4.4	15.5	4.2	0.1	10.6	58.7	0.5	3.9	2	0.1
0900	3.6	15.8	8.4	1.1	15.4	50.8	2.3	0.4	0	2.1
1006	7	22.5	7.5	0.6	13.5	46.7	1.1	0.7	0.3	0.1
1352	3.3	19.7	8.2	1.4	11.9	53.4	1.4	0	0	0.7
1802	4.7	2.5	18.3	2.7	12	20.2	3.8	0.4	0.9	34.6
1817	9	22.4	21.1	0.5	23.1	23.5	0.1	0.1	0.1	0
1992	5.1	15.9	4.6	0.2	7.4	64.8	1	0.9	0.1	0
2819	0.7	32.4	5.7	0.7	7.9	50.4	0.5	1.5	0.2	0.1
2824	3.6	15	10	0.2	5.9	62.3	0.9	1.9	0.1	0.1
2825	20	38.3	14.7	9	7.1	10.3	0.5	0	0	0
3008	5.6	19.5	8.6	0.2	8.7	51	0.4	4.6	1.4	0
3044	1.5	4	5.1	0.5	7.3	74.9	0.4	4.4	1	1
3807	4.7	16.7	6.7	0.3	11.3	55	0.4	3.4	1.5	0.1
3816	9.2	29.3	8.4	0.5	8.8	38.5	1.1	3.3	0.9	0.1
5827	3.9	11.4	3.2	0.1	9.5	65.8	0.4	4.9	0.6	0.1

Table B - 3 Normalized Vehicle Class Distribution Data

SHRP ID	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
0200	0.04	0.16	0.04	0	0.11	0.59	0	0.04	0.02	0
0900	0.04	0.16	0.08	0.01	0.15	0.51	0.02	0	0	0.02
1006	0.07	0.22	0.08	0.01	0.14	0.47	0.01	0.01	0	0
1352	0.03	0.2	0.08	0.01	0.12	0.53	0.01	0	0	0.01
1802	0.05	0.03	0.18	0.03	0.12	0.2	0.04	0	0.01	0.35
1817	0.09	0.22	0.21	0	0.23	0.24	0	0	0	0
1992	0.05	0.16	0.05	0	0.07	0.65	0.01	0.01	0	0
2819	0.01	0.32	0.06	0.01	0.08	0.5	0	0.02	0	0
2824	0.04	0.15	0.1	0	0.06	0.62	0.01	0.02	0	0
2825	0.2	0.38	0.15	0.09	0.07	0.1	0	0	0	0
3008	0.06	0.19	0.09	0	0.09	0.51	0	0.05	0.01	0
3044	0.02	0.04	0.05	0	0.07	0.75	0	0.04	0.01	0.01
3807	0.05	0.17	0.07	0	0.11	0.55	0	0.03	0.01	0
3816	0.09	0.29	0.08	0	0.09	0.39	0.01	0.03	0.01	0
5827	0.04	0.11	0.03	0	0.09	0.66	0	0.05	0.01	0

Table B - 4 Clustering Results for Axle Load Distribution Data

Hierarchical Clustering Results for:															
Quantitative Data Set = All Classes!$A$18:$AO$32															
Distance/Similarity Measure = Squared Euclidean Distance															
Cluster Method = Wards															
Distance Matrix (Euclidean Distance)															
	0200	0900	1006	1352	1802	1817	1992	2819	2824	2825	3008	3044	3807	3816	5827
0200		0.011	0.005	0.009	0.015	0.014	0.01	0.022	0.01	0.013	0.003	0.006	0.001	0.022	0.029
0900	0.011		0.01	0.004	0.01	0.033	0.01	0.044	0.01	0.012	0.017	0.013	0.015	0.018	0.018
1006	0.005	0.01		0.003	0.012	0.023	0.003	0.025	0.002	0.004	0.006	0.007	0.005	0.008	0.014
1352	0.009	0.004	0.003		0.007	0.032	0.004	0.035	0.003	0.005	0.013	0.012	0.012	0.01	0.011
1802	0.015	0.01	0.012	0.007		0.033	0.015	0.034	0.013	0.014	0.021	0.025	0.02	0.02	0.025
1817	0.014	0.033	0.023	0.032	0.033		0.04	0.016	0.034	0.034	0.009	0.023	0.012	0.036	0.046
1992	0.01	0.01	0.003	0.004	0.015	0.04		0.041	0.004	0.006	0.014	0.01	0.011	0.015	0.018
2819	0.022	0.044	0.025	0.035	0.034	0.016	0.041		0.031	0.036	0.019	0.032	0.021	0.029	0.05
2824	0.01	0.01	0.002	0.003	0.013	0.034	0.004	0.031		0.003	0.012	0.011	0.011	0.008	0.01
2825	0.013	0.012	0.004	0.005	0.014	0.034	0.006	0.036	0.003		0.013	0.011	0.013	0.009	0.014
3008	0.003	0.017	0.006	0.013	0.021	0.009	0.014	0.019	0.012	0.013		0.005	0.001	0.019	0.027
3044	0.006	0.013	0.007	0.012	0.025	0.023	0.01	0.032	0.011	0.011	0.005		0.005	0.015	0.026
3807	0.001	0.015	0.005	0.012	0.02	0.012	0.011	0.021	0.011	0.013	0.001	0.005		0.02	0.029
3816	0.022	0.018	0.008	0.01	0.02	0.036	0.015	0.029	0.008	0.009	0.019	0.015	0.02		0.008
5827	0.029	0.018	0.014	0.011	0.025	0.046	0.018	0.05	0.01	0.014	0.027	0.026	0.029	0.008	

Table B - 5 Clustering Strategy for Axle Load Distribution

Clustering Strategy			
Cluster	1st Item	2nd Item	Distance
1	3807	3008	0
2	2824	1006	0.001
3	Cluster 1	0200	0.003
4	Cluster 2	1352	0.004
5	Cluster 4	1992	0.006
6	Cluster 5	2825	0.009
7	Cluster 3	3044	0.012
8	5827	3816	0.016
9	1802	0900	0.021
10	2819	1817	0.03
11	Cluster 9	Cluster 6	0.039
12	Cluster 11	Cluster 8	0.053
13	Cluster 10	Cluster 7	0.072
14	Cluster 13	Cluster 12	0.114
Cophenetic Correlation			
R	DF	P	
0.584	103	0	

Figure B - 2 Cluster Identification for Axle Load Distribution

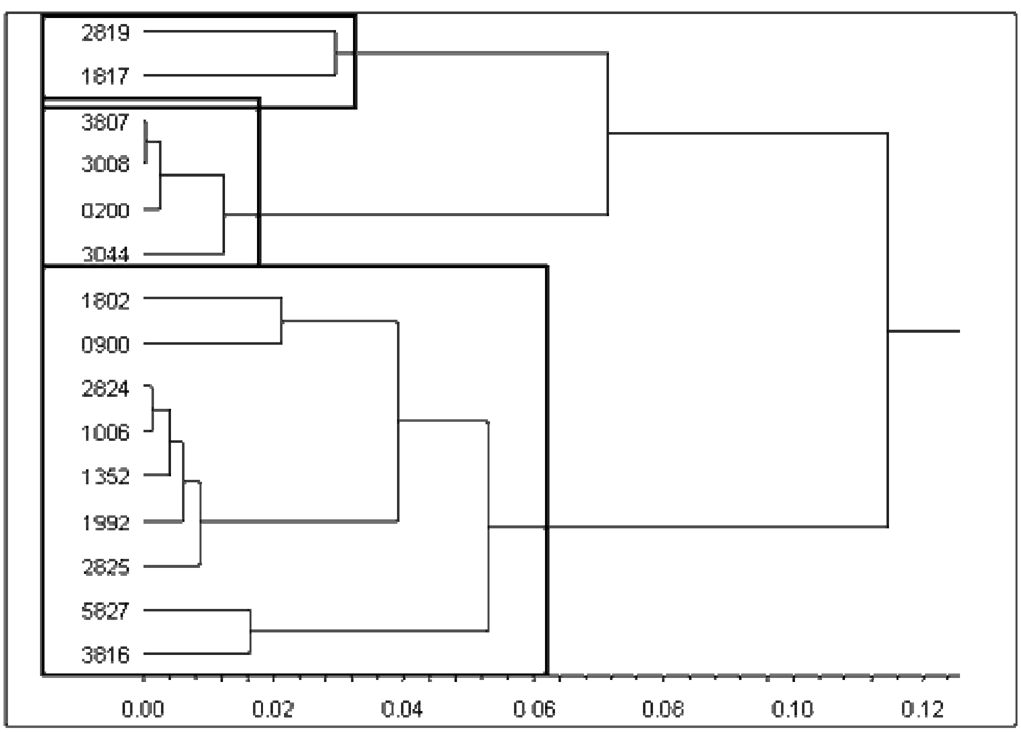

Table B - 6 Clustering Results for Vehicle Class Distribution

Hierarchical Clustering Results for:															
Quantitative Data Set = Sheet1!$A$19:$K$33															
Distance/Similarity Measure = Squared Euclidean Distance															
Cluster Method = Wards															
Distance Matrix (Euclidean Distance)															
	0200	0900	1006	1352	1802	1817	1992	2819	2824	2825	3008	3044	3807	3816	5827
0200		0.013	0.024	0.009	0.307	0.177	0.006	0.039	0.008	0.333	0.01	0.042	0.002	0.064	0.007
0900	0.013		0.008	0.004	0.228	0.105	0.028	0.036	0.024	0.259	0.009	0.082	0.006	0.042	0.034
1006	0.024	0.008		0.007	0.242	0.082	0.042	0.019	0.038	0.19	0.007	0.123	0.012	0.015	0.055
1352	0.009	0.004	0.007		0.265	0.122	0.019	0.02	0.015	0.26	0.005	0.077	0.003	0.037	0.028
1802	0.307	0.228	0.242	0.265		0.177	0.358	0.32	0.324	0.289	0.257	0.435	0.276	0.239	0.362
1817	0.177	0.105	0.082	0.122	0.177		0.228	0.136	0.201	0.092	0.116	0.356	0.14	0.065	0.246
1992	0.006	0.028	0.042	0.019	0.358	0.228		0.05	0.004	0.387	0.024	0.027	0.013	0.091	0.005
2819	0.039	0.036	0.019	0.02	0.32	0.136	0.05		0.048	0.216	0.021	0.142	0.03	0.023	0.071
2824	0.008	0.024	0.038	0.015	0.324	0.201	0.004	0.048		0.362	0.017	0.032	0.01	0.082	0.009
2825	0.333	0.259	0.19	0.26	0.289	0.092	0.387	0.216	0.362		0.235	0.587	0.286	0.112	0.43
3008	0.01	0.009	0.007	0.005	0.257	0.116	0.024	0.021	0.017	0.235		0.084	0.004	0.027	0.032
3044	0.042	0.082	0.123	0.077	0.435	0.356	0.027	0.142	0.032	0.587	0.084		0.059	0.204	0.015
3807	0.002	0.006	0.012	0.003	0.276	0.14	0.013	0.03	0.01	0.286	0.004	0.059		0.046	0.017
3816	0.064	0.042	0.015	0.037	0.239	0.065	0.091	0.023	0.082	0.112	0.027	0.204	0.046		0.113
5827	0.007	0.034	0.055	0.028	0.362	0.246	0.005	0.071	0.009	0.43	0.032	0.015	0.017	0.113	

Table B - 7 Clustering Strategy for Vehicle Class Distribution

Cluster	1st Item	2nd Item	Distance
1	3807	0200	0.001
2	1352	0900	0.003
3	2824	1992	0.005
4	3008	1006	0.009
5	Cluster 3	5827	0.013
6	Cluster 4	Cluster 2	0.017
7	Cluster 5	Cluster 1	0.026
8	3816	2819	0.038
9	Cluster 8	Cluster 6	0.063
10	Cluster 7	3044	0.089
11	2825	1817	0.135
12	Cluster 10	Cluster 9	0.239
13	Cluster 11	1802	0.379
14	Cluster 13	Cluster 12	0.808

Figure B - 3 Cluster Identification for Vehicle Class Distribution