Southwest Region University Transportation Center

Quantification of the Effect of Maintenance Activities on Texas Road Network

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16. Abstract

Pavement structures are designed for a finite life, usually referred to as performance period. This performance period is typically between 20 to 25 years for flexible pavements and between 25 and 40 years for rigid pavements. After this period, the pavement is predicted to reach a terminal level in terms of several preset criteria. This performance period can be reached by designing a structure that will withstand the effects of traffic and the environment through the design period or by planning a series of maintenance and rehabilitation activities that will keep the structure above the present terminal levels until the end of the design life is reached.

The objective of this study is to gather data on pavement performance from FHWA's Long-Term Pavement Performance (LTPP) study. The sections will be selected such that they provide enough time-series information to obtain reliable pavement performance trends. Once the data are collected, the various pavement sections will be modeled using mechanistic-empirical principles and they performance will be predicted. The Mechanistic Empirical Pavement Design Guide (MEPDG) will be used for this purpose. In addition, empirical performance models will be developed to capture the performance (and in particular the differential performance) of the various sections. Once these two types of performance models are available, we will compare the effectiveness of the three types of sections.

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QUANTIFICATION OF THE EFFECT OF MAINTENANCE ACTIVITIES ON TEXAS ROAD NETWORK

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ABSTRACT

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EXECUTIVE SUMMARY

The goal of this study was to understand how different maintenance and rehabilitation strategies affect pavement performance. The study team developed a framework for calibration of the Mechanistic Empirical Pavement Design Guide (MEPDG) and specifically the permanent deformation performance model. Following the calibration of the transfer functions, further efforts were made in determining cause and effect relationships between different maintenance strategies and bias correction factors.

The MEPDG was developed as part of the NCHRP funded studies 9-37A and 9-40D. The software program developed as part of these studies represents a paradigm shift in the design and analysis of flexible and rigid pavement structures. The program uses mechanistic principles in determining the stresses, strains and deformations that result from traffic loads on the pavement structure. The resulting deformation is then converted into visual distresses using transfer functions that require calibration to account for local site conditions such as materials, traffic characteristics, and climate among others. It is important to note that the properties of the material are age-dependent in the case of flexible pavements. This necessitates that the material responses are calculated recursively which taking into consideration the history of the material. The transfer functions in their current form in the MEPDG are calibrated to national averages which imply that the distress predictions obtained from the performance prediction model will, at best, correspond to averages for the entire country which, in reality, will not correspond to any specific region or condition. This leads to the need for determination of the bias correction factors. In the context of this study, these bias correction factors correspond to the five different climatic regions in Texas for the rutting performance prediction model. It is worth mentioning in this context that the distress predictions obtained from the model are proportional to these calibration coefficients suggesting that higher these calibration coefficients, higher the predicted rutting. Given the proportionality between predicted distresses and bias correction factors, one could potentially identify the relative effectiveness of different treatment strategies by analyzing the calibration coefficients.

This study included two primary technical objectives:

- To determine the calibration coefficients for permanent deformation performance prediction model in the MEPDG for pavements that received corrective or preventive maintenance, and
- To analyze and identify the relative effectiveness of different maintenance treatments from a rutting perspective.

Given the nature of the objectives of this study, it was essential to identify a proper dataset that would facilitate the aforementioned analysis. The study team adopted the Long-Term Pavement Performance (LTPP) database for this study, of which the SPS-3 and SPS-5 experimental studies were of special interest. The LTPP database was established in 1987 as part of the Strategic Highway Research Program (SHRP) studies.

The LTPP SPS-3 studies focused on a total of 81 locations comprising of 486 sections throughout the United States. To make the problem manageable and minimize variability, this research analyzed only 13 sections from four Texas locations. The objective of the SPS-3

experiments was to examine the optimal timing and effectiveness of each of the different pavement preservation techniques included as part of the experimental design. The analysis addressed three different strategies:

- Thin overlay
- Seal coat or surface treatment
- No treatment (control)

The SPS-5 is a controlled field experiment focused on the study of the specific variables noted below for the rehabilitation of hot mix asphalt (HMA) flexible pavements. These variables are understood to affect the performance of overlaid pavement structures. They include:

- Surface preparation
- Overlay thickness
- Overlay material
- Environment
- Condition of the original pavement at the time of construction of the overlay

The study team included six different locations from across the country in order to quantify the effect of climatic conditions on the bias correction factors. The permanent deformation distress prediction as obtained from the MEPDG is governed by the pavement temperature, thickness of the bituminous layer and the number of axle repetitions. Given the multi-dimensional non-linear nature of the prediction model, the study team developed an optimization routine that seeks to minimize the difference between the observed and predicted distresses. The authors would like to highlight the fact that the sum of squared errors was aggregated for all the sections in a given location and thus, the bias correction factors calculated correspond to the entire region rather than just a specific pavement section. In other words, these calibration coefficients correspond to Level 2 and are more robust than Level 1 bias correction factors. It should be noted, within this context, that the study team focused on calibrating the bias correction factors for the number of load repetitions and the thickness of the bituminous layer. Although past research studies have shown that the distress predictions are highly sensitive to the pavement temperature, the thermal susceptibility of the hot mix could not be determined due to lack of material to be tested. Therefore, for the purpose of this study, it was assumed that the bias in the model is not affected by the thermal susceptibility of different mixes.

The aforementioned methodology was also adopted in the determination of the bias correction factors for the SPS-3 sections. As pointed out earlier, the SPS-3 sections was formulated in order to determine the effectiveness and optimal timing of preventive maintenance technologies that included seal coats and thin overlays. The calculated bias correction factors were subsequently studied in a multiple linear regression setting with the different treatment options as the independent variables and the calibration coefficients serving as the response variable.

The bias correction factors proposed as part of this study were validated against GPS-6 sections from the LTPP database. The results showed that the calibration coefficients provided a better estimate for the rut depth when compared against the in-field observed measures of distresses as opposed to using national averages. It is recommended that more pavement sections from these regions are included and the transfer functions are re-calibrated which would help improve the level of confidence in these factors. From a practical point of view, reliable bias correction factors will enable state and local agencies to perform accurate and reliable analysis of the performance of flexible pavements, saving significant resources for highway agencies across the nation. This study also pointed out that pavements rehabilitated in "No Freeze" zones are likely to rut less during their early life compared to pavements that are constructed in freezing zones. On the other hand, roads in warmer climatic areas are likely to experience faster rate of progression of rutting with time than those in colder areas and this observation has been attributed to the heterogeneity between pavements that result from selection of different grades of binder depending on their respective geographical location.

In the second phase of this study, the study team investigated the relative influence of different maintenance and rehabilitation strategies on pavement performance from a rutting standpoint. In the case of rehabilitated pavements, it was observed that the rutting performance of a pavement is improved when using recycled asphalt (RAP) as opposed to virgin asphalt mix. However, the particular observation needs further investigation as it is known that the use of high percentages of RAP (e.g. more than 50 percent) is likely to compromise the fatigue and fracture performance of the mix. Thus a comprehensive understanding of the impact of RAP percentages on the performance of asphalt mixtures requires consideration of both rutting and fatigue performance.

In the case of the SPS-3 sections, it was observed that the use of thin overlays as a preventive maintenance measure can improve the rutting performance of the specific pavement section. On the contrary, the results from this study suggest that the use of seal coats is likely to have negative impact on the rutting performance of the pavement section. For most part, the results and conclusions derived on the effectiveness of preventive maintenance techniques are in good agreement with common knowledge of how different pavement preservation techniques affect performance characteristics. Because thin overlays are mostly designed to correct minor rutting problems, it can be expected that they exhibit reduced level of initial rutting. Seal coats are mostly used as an instrument for sealing the underlying pavement structure and retard the rate of oxidative aging and can be expected to make a positive difference in the fatigue and fracture resistance of the mix. Unfortunately, the study objectives only included analyzing the effectiveness of these maintenance strategies from a rutting perspective and therefore fatigue and fracture characteristics were excluded from this initial study. To summarize the findings of the latter half of this study, it was evident that different maintenance and rehabilitation techniques do require specific calibration coefficients. That is, the default calibration factor only applied to new pavements; for maintenance and rehabilitation a different set of calibration factors is required.

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CHAPTER 1. INTRODUCTION

1.1 THE IMPORTANCE OF PAVEMENT PRESERVATION AND MAINTENANCE

The current economic situation and a deteriorating national infrastructure has resulted in more and more emphasis being given towards pavement rehabilitation and other maintenance projects. Part of the reason can be attributed to shrinking funds for infrastructure development. Needless to say, this will require a larger focus on a systematic procedure for design of new as well as rehabilitated pavement structures.

The cumulative effects of traffic and the environment result in the deterioration of pavements with time. Routine maintenance can extend a pavement's life and significantly delay the requirement of major rehabilitation work. However, at some stage, large restoration or reconstruction will be required so that the pavement structure can adequately support ever-increasing traffic loads. Pavement rehabilitation may be executed in different ways, either through recycling of the original material or by replacing the existing material with new material. Rehabilitation can be defined as *"...a structural or functional enhancement of a pavement which produces a substantial extension in service life, by substantially improving pavement condition and ride quality"* (Hall et al., 2001). The process of pavement rehabilitation goes through a systematic phase that includes:

1) Prioritization of pavements that should undergo rehabilitation,
2) Looking for feasible rehabilitation strategies, and
3) Selecting the most cost-effective rehabilitation strategy.

Generally, the process of prioritizing the pavements that need rehabilitation starts with evaluating each of the candidate sections for structural adequacy or load carrying capacity. Falling Weight Deflectometer (FWD) measurements are often conducted to determine pavement sections that are in need of rehabilitation. It should be noted here that several other techniques to measure the stiffness of the pavement structure are also being investigated like the Rolling Wheel Deflectometer (RWD), the Rolling Dynamic Deflectometer (RDD), etc.

1.2 CORRECTIVE V/S PREVENTIVE MAINTENANCE

There are two commonly used rehabilitation strategies: asphalt overlays and concrete overlays (also known as ultrathin whitetopping). Asphalt pavement rehabilitation typically involves milling and resurfacing of the existing asphalt pavement to mitigate the effects of poor ride quality, rutting, cracking and other distresses (Tayabji et al., 2000). The thickness of the overlay or resurfacing often depends upon the expected truck traffic as well as the available budget. In general, it has been observed that a properly designed overlay may last from 8 to 12 years when subjected to high levels of truck traffic. The use of performance mixtures, such as Stone Matrix Asphalt (SMA) or Coarse-Matrix High Binder (CMHB), may further improve the life of the overlay, but, at the same time, such mixtures may prove to be costly when compared against the most popular dense graded mixes. In recent times, recycled mixes have gained popularity because of their lower cost and in-place recycling of the reclaimed material ensures savings to the highway agencies.

Currently, pavement preservation strategies, which involve a preventive maintenance program, have gained popularity because of several advantages. Preventive maintenance options require less financial resources than rehabilitating or reconstructing a pavement. They are also effective in maintaining the pavement at or above the required level of service. In situations where the pavements start showing minor signs of distress, pavement preservation strategies can help in retarding and restricting them. FHWA's Pavement Preservation Expert Task Group defined Pavement Preservation (PP) as *"a program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extend pavement life, improve safety and meet motorist expectations."* The idea behind pavement preservation lies in the fact that maintaining a road in good condition is more cost-effective than repairing a pavement that has failed.

An effective pavement preservation program treats pavements while they are still in good condition and before the onset of significant damage. By applying a cost-effective treatment at the right time, the pavement is restored almost to its original condition. The cumulative effect of systematic, successive preservation treatments is to postpone costly rehabilitation and reconstruction. Thus, it extends pavement life and arrests or retards deterioration and progressive failures. It keeps the road in good condition, which improves safety and ride quality. Perhaps the greatest advantage associated with adopting an aggressive pavement preservation policy is the financial savings, as opposed to the costs of rehabilitating the same pavement when it has completely failed.

The philosophy behind pavement preservation can be summarized as "the right treatment applied to the right candidate at the right time" (Texas Pavement Preservation Center, 2008). However, this statement underscores the importance of careful planning needed for a pavement preservation strategy. In order for any pavement preservation technique to be effective, it is required that the right strategy is selected after identifying the problem and applied at the right time. If it is applied too early, that will drive up the cost of such a measure. On the other hand, if it is applied later, the pavement would have deteriorated to an extent where pavement preservation methodology can no longer stop the pavement from deteriorating further. The choice of the PP technique is also influenced by traffic levels, climatic conditions and distress type. The best time for applying such a treatment is when the pavement starts to show minor visible signs of any kind of distress, thus indicating that the pavement is at a very early stage of the failure process (Texas Pavement Preservation Center, 2008).

Pavement preservation treatments are used for planned maintenance, actions that maintain or improve the pavement's functional condition. Some of the commonly used PP treatments include chip seals, slurry seals, fog seals, thin asphalt overlays, micro surfacing, and crack sealing.

1.3 EMPIRICAL V/S MECHANISTIC EMPIRICAL DESIGN METHODOLOGY

The standard for pavement design in most regions of the United States is empirical design methods. The leading of these design methods is the AASHTO method which is based on the AASHO road test conducted during the late 1950's. The AASHO road test involved the construction of six loops in Ottawa Illinois to test both flexible and rigid pavements under varying load types and construction practices. For the flexible portion of the test, the factors that varied from test to test were the AC thickness, Base thickness and Subbase thickness. The materials used for the base course and subbase were consistent throughout the test (Huang, 2004). Because the test involved using a single location where the base and subbase were more or less uniform, pavement design based on the ASSHTO method cannot therefore account for differences in materials, construction practices and climatic conditions which might be typical for some other geographical locations.

Although the AASHTO method is continually updated, its roots are still based on antiquated pavement designs that do not represent the materials, construction practices or traffic that is commonly seen today (Watson & Wu, 2009). This trend is true for other popular empirical design methods as well. This implies that such design methods when used for design or analysis of pavements that were not well represented in terms of their material characteristics in the AASHO Road tests will result in an extrapolation of the data which was originally used for the development of these methods; sometimes by several orders of magnitude, resulting in significant errors in the distress predictions. For this reason Mechanistic- Empirical design methods, which use probabilistic analysis, have gained increased popularity. The Mechanistic-Empirical Pavement Design Guide (MEPDG) sponsored by the AASHTO Joint Task Force on Pavements (JTFP), and developed under the National Cooperative Highway Research Program (NCHRP 1-37A and 1-40D), is a computer program that can be used to analyze new and rehabilitated pavements (Sharpe, 2004).

The new MEPDG provides a much more systematic approach, as opposed to the empirical method which is based on observations, towards the design of rehabilitated pavement structures. In the mechanistic-empirical method the fundamental pavement responses under repeated traffic loadings are calculated using a multi-layer linear elastic approach. This approach assumes that a pavement structure is a layered structure and each of the layers in the pavement structure exhibits an elastic behavior that is linear in nature. The method computes the stresses, strains and the deflections that are induced in the pavement layers due to the given traffic loadings. These pavement responses are then related to field distresses using existing empirical relationships, widely known as transfer functions (Chehab & Daniel, 2006).

The performance models used in the MEPDG have been calibrated based on various pavement test sections spread throughout the United States (Banerjee et al., 2009). Due to the variety of locations and types of test sections used, the MEPDG uses a "national average" for many parameters that need to be calibrated to local conditions to be truly accurate. Without these calibrations a pavement will be overdesigned or under designed depending on where the pavement is being built and what construction materials/practices are being used. Calibrations allow for the systematic differences in materials, climate, construction material and any other design variables found in the local area that vary from the "national average" to be accounted for in the analysis or design of a pavement structure. Recently a lot of effort has been directed at calibrating performance models to local and regional conditions, especially the transfer functions in the MEPDG for new construction projects.

CHAPTER 2. RESEARCH OBJECTIVES

2.1 OBJECTIVES

The performance models used in the MEPDG, developed under the National Cooperative Highway Research Program (NCHRP 1-37A and 1-40D), were calibrated using sections throughout the United States, including pavement sections from Texas. It should be noted in this context, there is no overlap between the datasets used for this study and that used for the national calibration of the MEPDG (ARA, 2004).

The goal of this study is two-fold. The first objective includes investigation of the influence of the selected experimental variables on the predicted pavement performance after calibrating the permanent deformation performance model in the MEPDG for each of the project locations under study. Hence, calibration coefficients or bias correction coefficient values are determined for each strategy and compared.

The latter includes determination of the Level 2 bias correction factors for the permanent deformation performance prediction models for rehabilitated flexible pavements. It should be noted here that Level 2 bias correction factors, as interpreted by the authors, refer to the region specific calibration coefficients. These bias correction factors apply to a given geographical region and are fairly accurate for predicting distresses for sections belonging to a specific region.

The first step involved the determination of Level 1 and Level 2 bias correction factors for each of the test sections included in the study. In principle, one could determine bias correction factors for Level 1, Level 2, or Level 3. The definitions of each of the design levels as interpreted by the authors are:

1) Level 1: The highest level of accuracy and reliability, implies determination of a specific set of calibration factors best suited to a given test site. Level 1 calibration factors can be very accurate while predicting pavement distresses for a specific section, but they cannot be relied upon for distress predictions at a regional or state level. These calibration factors will fit the section-specific data the best, but cannot be used for future designs unless the conditions and location are exactly the same.
2) Level 2: The intermediate level or regional level, proposes determining bias correction factors at a regional level. Calibration factors that conform to Level 2 design may not be very accurate for site specific distress predictions, but can be fairly robust for predicting distresses for sections belonging to a specific region.
3) Level 3: Level 3 has the lowest accuracy and reliability for predicting distresses for a specific site because they are most suited for predicting pavement distresses at a state level.

The first objective included studying the effectiveness of different maintenance strategies on the predicted pavement performance, Level 1 bias correction factors were considered to be the most appropriate choice.

It should be noted in the context of the second objective that it was the Level 2 bias correction factors that were determined as opposed to Level 1. The dataset used for this part of the study included sections from the Specific Pavement Studies (SPS-5) of the Long Term Pavement Performance Database (LTPP). Given the differences that exist between the sections, it will be of little use to pavement engineers and the scientific community had the Level 1 bias correction factors been determined. This is because the Level 1 bias correction factors solely relate to the conditions specific to the section that is being used for the determination of the calibration coefficients. There is little chance that another pavement section will be exactly identical to those for which the bias correction factors have been determined. On the contrary, the Level 2 bias correction factors represent the average material and structural design for a group of pavement sections from a certain geographical region which justifies its applicability towards analysis of any rehabilitated pavement structure within that region.

CHAPTER 3. LITERATURE REVIEW

3.1 MECHANISTIC EMPIRICAL DESIGN PHILOSOPHY

The MEPDG represents a major change from the way pavement design had been done in the past. The designer first considers site conditions (traffic, climate, subgrade, existing pavement conditions for rehabilitation) and construction conditions in proposing a trial design for a new pavement or rehabilitation. The trial design is then evaluated for adequacy through the prediction of key distresses (cracking and permanent deformation) and roughness. If the design does not meet desired performance criteria, it is revised and the evaluation process is repeated as necessary. Thus, the designer has the flexibility to consider different design features and materials for the prevailing site conditions. As such, the MEPDG is not a design tool but a very powerful and comprehensive pavement analysis tool.

"The MEPDG allows the designer to calibrate the performance prediction models depending on local factors such as traffic and climate. Well-calibrated prediction models result in reliable pavement design for state highway agencies. Local pavement performance data can be used to validate and adjust calibration factors integrated in the MEPDG. The procedure empirically relates damage over time to pavement distress" (Kang and Adams, 2007).

The MEPDG follows a recursive approach to pavement design where the designer begins with a trial design. The designer is expected to key in all the relevant information that is needed by the software, like climate, material, structure, traffic, some basic details that can be used to identify the project, acceptable limits of the key distresses and the reliability targets for each of these distress types. Having done this the designer, the next step is running an analysis with the input parameters that were already entered into the software and the MEPDG in return estimates the damage and the key distresses over the life of the pavement section. In the next step, the software compares the predicted distresses against the distress and reliability targets which were preset by the designer. If the trial design fails to meet the performance and reliability requirements, the designer has to revise the initial trial design and run the analysis again (see Figure 1).

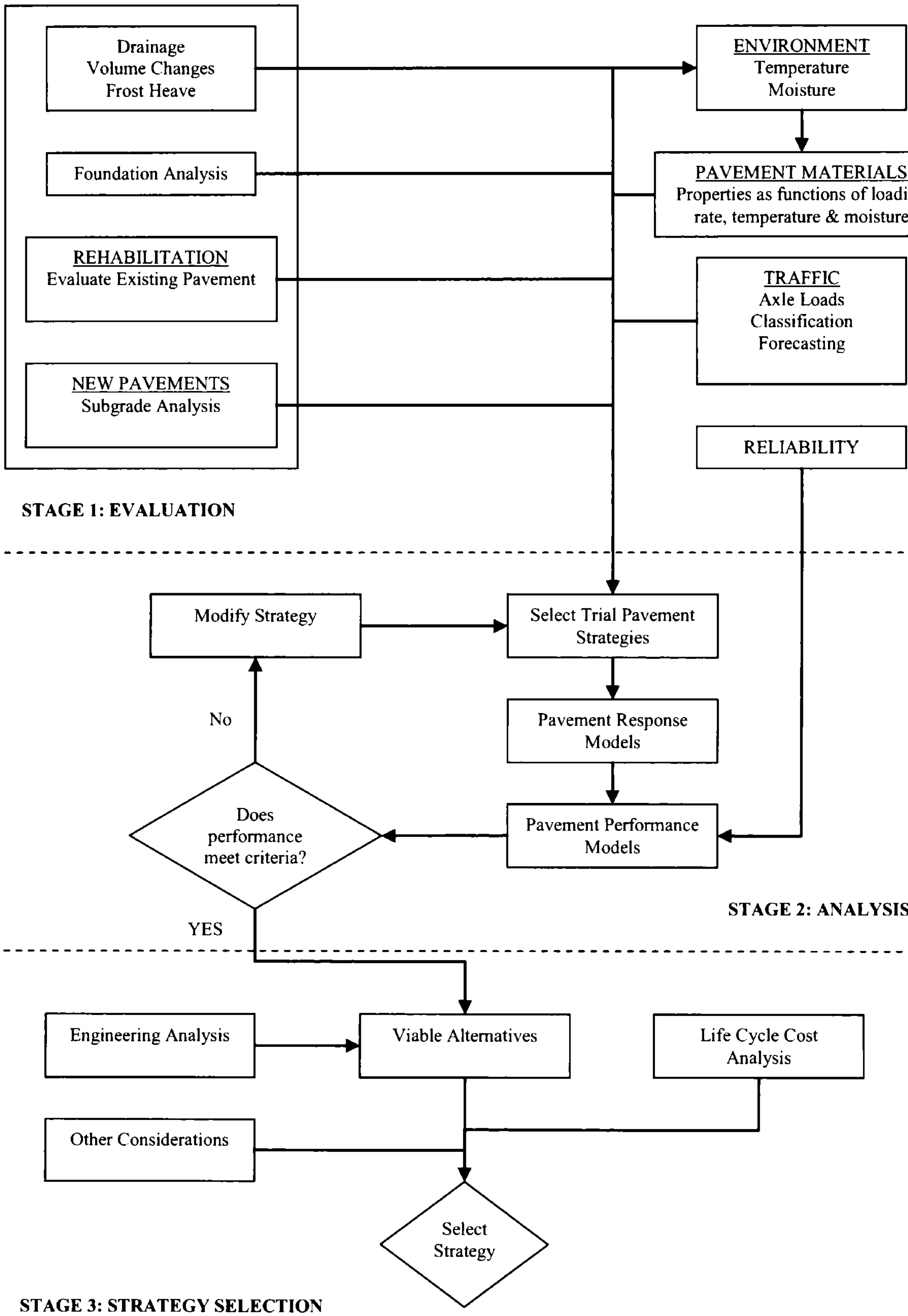

Figure 1. MEPDG Design Procedure (ARA, 2004).

3.2 MECHANISTIC EMPIRICAL PAVEMENT DESIGN GUIDE (MEPDG)

The MEPDG is not a design tool as much as it is an analysis tool that can be used for comparing different pavement designs (Manik et. al, 2009). The MEPDG needs a trial pavement structure, location, traffic type, weather conditions, existing pavement conditions (in the case of rehabilitation), reliability and failure criteria, pavement life as design inputs and analyzes the given design for various failure criteria (Banerjee et al., 2009). The MEPDG does not come up with a design but instead analyzes the adequacy of a given design against different failure mechanisms typically seen with flexible/rigid pavements, thus allowing the user to check the sufficiency of the trial design. If the design fails to meet any of the failure criteria, the design is re-evaluated. The designer gets the option of choosing between superior materials and the pavement structure in order to meet the performance thresholds as required for a given project for each of the different distress mechanisms. In this sense the MEPDG is an iterative process and can only tell a user if the preliminary design is acceptable or not.

The MEPDG has default values for most user inputs. However, these defaults are based on national averages and therefore they are not recommended where specific project information is available. The higher someone depends on these default values, it gets more and more likely that the design is less reliable because it will be influenced by these default values rather than site specific information. In general the MEPDG allows three different levels of input data – Level 1, Level 2 and Level 3. Level 1 has the highest level of accuracy and is considered site specific. Level 2 has an intermediate level of accuracy and is said to be representative of regional defaults. Level 3 has the lowest level of accuracy and is mostly characteristic of state level defaults (Banerjee et al., 2009). Level 3 requires the least amount of inputs and therefore they are least accurate. Level 3 inputs allow the user to input the very basic information for materials or use the default values set in the program for a particular parameter of interest. On the other hand, Level 1 inputs require site specific information that has been determined from in-situ and laboratory testing (Chehab & Daniel, 2006). MEPDG uses this information to determine the material properties of concern using predictive equations. Although it is easier and requires less information to input Level 3 data, the resulting distress predictions are far less accurate than those obtained from Level 1 inputs.

3.3 PAST INVESTIGATIONS ON DATA ANALYSIS OF THE SPS-3 AND SPS-5 STUDIES

Preliminary analysis of the SPS-5 data indicated that age, surface preparation, and pre-existing surface condition influenced performance of the rehabilitated sections (von Quintus et al., 2006). The study showed that more fatigue cracking occurred on test sections placed in a climate with less precipitation but higher freeze indices, while transverse cracking occurred on sections with intensive surface preparation than on sections with minimal surface preparation before overlay. A recent study on the Texas SPS-5 sections have shown that the use of recycled asphalt mix (RAP) can significantly influence the performance of flexible pavements in terms of the different distress mechanisms (Hong et al, 2009). The study suggests that the use of RAP in hot mix asphalt will be 0.47 times as effective when compared to virgin HMA from a transverse cracking perspective. It should be noted that transverse cracks are often believed to be a manifestation of reflection cracks in the underlying layers or low temperature thermal cracks. The study also reported that hot mix asphalt with 35% RAP content will most likely deteriorate at a rate of 0.7 compared to a virgin asphalt mix. In general, Texas SPS-5 sections performed extremely well and proved to be better than expected.

Previous results have indicated that chip seals performed well almost in every climatic region (Morian et al., 1998). The study suggests aggregates, emulsions, and construction practices were the keys to success in the case of chip seals. However, at the same time their performance was affected by moisture-related issues. Thin overlays were successful in improving ride quality, restraining reflective cracking, and correcting rutting.

CHAPTER 4. DATA REQUIREMENTS

As part of the first objective, two related aspects were addressed: the influence of rehabilitation strategies on bias correction factors and the effect of pavement preservation techniques on the bias correction factors. Therefore, it was necessary to look for two different datasets, each addressing one of the two requirements discussed above. The Long-Term Pavement Performance (LTPP) Specific Pavement Studies-5 (SPS5) experiment caters to the first while the SPS-3 experiment data caters to the second aspect.
However, the second objective of this study included determination of the bias correction factors for rehabilitated pavements for the rutting transfer function in the MEPDG. Thus there was an overlap between the two datasets with respect to the SPS-5 studies of the LTPP database.

4.1 LTPP SPS-5 EXPERIMENTS

The dataset used to address the second objective as well as part of the first objective of this study includes pavement sections that are part of the SPS-5 experiments from the Long Term Pavement Performance Database (LTPP). The LTPP is an international comprehensive pavement performance database, which was established in 1987 as part of the Strategic Highway Research Program (SHRP) studies. These studies included monitoring both in-use, new, or rehabilitated pavements. The information in LTPP database is broken down into two major groups of studies: the General Pavement Studies (GPS) and the Specific Pavement Studies (SPS).

The GPS is a group of studies on pre-existing in-service pavements that are common in design and use across the nation (Elkins et al., 2006). Pavements being monitored in the GPS are further classified into one of eight experiment categories based on pavement type and rehabilitation measures used, if any. The performance of these structural designs is tested against an array of climatic, geologic, maintenance, rehabilitation, traffic, and other service conditions. Each GPS site has a single test section.

The SPS test sections, on the other hand, have been specially constructed to investigate certain pavement engineering factors. Unlike the GPS, which used pre-existing pavements already in use, all SPS sections have been constructed specifically for the LTPP studies so that they can be monitored from the initial date of construction. These particular test sections allow critical design factors to be controlled or monitored. Each test site has multiple test sections where each section is unique in terms of the combination of the experimental variables that are being studied as part of the study. This makes it possible to compare the performance differential between each of these test sections introduced because of the different design factors, both within and between sites. Over time, these sections will give an insight into how different rehabilitation and maintenance procedures affect pavement performance. Pavements being monitored in the SPS are classified into one of nine major categories based on what particular factor is being tested, such as pavement rehabilitation. Multiple experiments have been conducted in each of these categories based on pavement type.

The SPS-5 is a controlled field experiment focused on the study of the specific features noted below for the rehabilitation of hot mix asphalt (HMA) flexible pavements (von Quintus et al., 2006). These are understood to affect the performance of overlaid pavement structures. These variables include:

1. Surface preparation
2. Overlay thickness
3. Overlay material
4. Environment
5. Condition of the original pavement at the time of construction of the overlay

Although the SPS-5 experiments included 14 different locations in the United States, certain practical issues like unavailability of prior distress measurements before the pavement was overlaid or lack of critical site specific information limit the scope of this study to 6 of these 14 locations. The study focuses on determining the bias correction factors for the permanent deformation performance prediction models in the MEPDG for rehabilitated pavements. Test locations from Florida, Alabama, New Mexico, and Minnesota were found to lack distress measurement data before the pavement sections were overlaid, thus providing no information on how much rutting the pavement sections were already subjected to prior to overlaying the sections. Test locations from Mississippi and Maryland had limited number of in-field distress measurements which limits the usage of such data for determination of reliable bias correction factors for the transfer functions under study. Limited traffic data for pavement sections from California and Georgia resulted in dropping the SPS-5 sections from the scope of this study. The six different locations that were considered as part of this study are shown in Table 1.

4.1.1 General Information

Table 2 highlights the basic construction information for each of the six SPS-5 sections included in this study. The analysis period for each of these locations was chosen based on the number of years of performance measurements available in the LTPP database. Although the MPEDG could be run for a longer design life, the authors would have no data to compare their outcomes, thus giving no additional benefit while trying to determine the bias correction factors.

4.1.2 Traffic

The traffic growth rate and related information were obtained from the TRF_MONITOR_LTPP_LN table (Elkins et al, 2006). A minimum of four and a maximum of fifteen years of traffic data were available for the sections under study. Annual Average Daily Truck Traffic (AADTT) is the total amount of truck traffic seen in one year, divided by the number of days in the year. As it can be seen in Table 2, the total AADTT for New Jersey was 840 vehicles per day and the total for Colorado was 799 vehicles per day. These are the initial traffic level experienced by the pavement in the year it was opened to traffic. In order to account for an increase in traffic volume over time, an annual growth rate is applied to the AADTT that the MEPDG uses to account for the total traffic load applied to the pavement over its design life.

It was assumed that the traffic growth rate is linear during the survey period. Thus, a linear trend line fitted to the available data provided the necessary information to determine the initial truck traffic (intercept of the trend line) and its growth rate (slope of the trend line). Figure 1 illustrates the procedure for determination of these two parameters. The initial traffic volume and the growth rate for each of the SPS-5 locations under study are included in Table 2. It can be observed that the initial traffic volumes as well as the growth rates are highly variable for the locations under study.

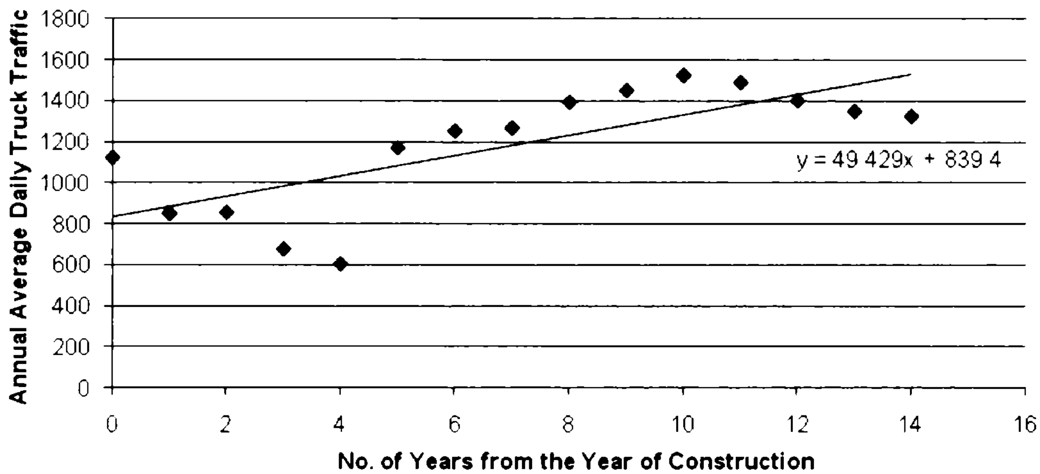

Figure 2. Determination of initial truck traffic & growth rate for New Jersey SPS-5 test location.

The vehicle classification data was obtained from the TRF_HIST_CLASS_DATA table in the Traffic database (Elkins et al, 2006). The AADTT is broken down by percent distribution per vehicle class. These data were available in the LTPP database.

The axle load distribution was obtained from the MEPDG_AXLE_DIST table of the MEPDG dataset in the LTPP database. Site-specific axle load distribution was computed for each of the two SPS-5 locations. This axle weight distribution was adapted from actual Weigh-In-Motion (WIM) stations installed at the project location. The New Jersey SPS-5 sections had axle weight information for six years between 1999 and 2005, with 2000 being the only year left out of the monitoring program. On the other hand, the Colorado test sections had axle weight information for the years 1994 and 1996. Axle spectrum was available for the years 2000, 2001, 2002, 2004, 2005, and 2006 for the Montana SPS-5 sections while for the Oklahoma sections it was only available for the year 2002. Axle load data were available for three monitoring seasons for each of the Texas and Missouri SPS-5 test locations – thus providing site-specific axle distribution data for each of the locations included in this study. For each of these project locations, the axle weight percentages were averaged and summed up for the number of years of study. Due to lack of information, the defaults were kept for the quad axles. For all other traffic related inputs, the defaults were kept unaltered. Therefore the use of defaults for the axle spectra for quad axles should not introduce any bias in the distress predictions because of the fact that none of the vehicle classes by default had a quad axle. Axle spectra for single and tandem axles from Colorado SPS-5 location for the month of January have been included in Figure 2.

The available data were averaged instead of using the first year available as a baseline and then applying a growth factor because of the high variance in the available data from one year to another that did not follow a particular pattern or growth factor. For example, in January of 1999 for the New Jersey sections, 31% of the single axles among Class 5 vehicles weighed between 0-3000 lbs, while from 2001-2005 it was less than 1% for each of the years. If only 1999 was used to compute the axle spectra, the results from the MEPDG would not have accurately reflected the actual traffic seen by that group of pavement sections. The resulting calibration factor would have been underestimated, because the axle weight group 0-3000 lbs reflect the lightest axle weight. On the other hand, if 1999 was eliminated from the dataset and only the other five years were used, a significant portion of the damage caused by the light weight single axles in Class 5 trucks would have been overlooked.

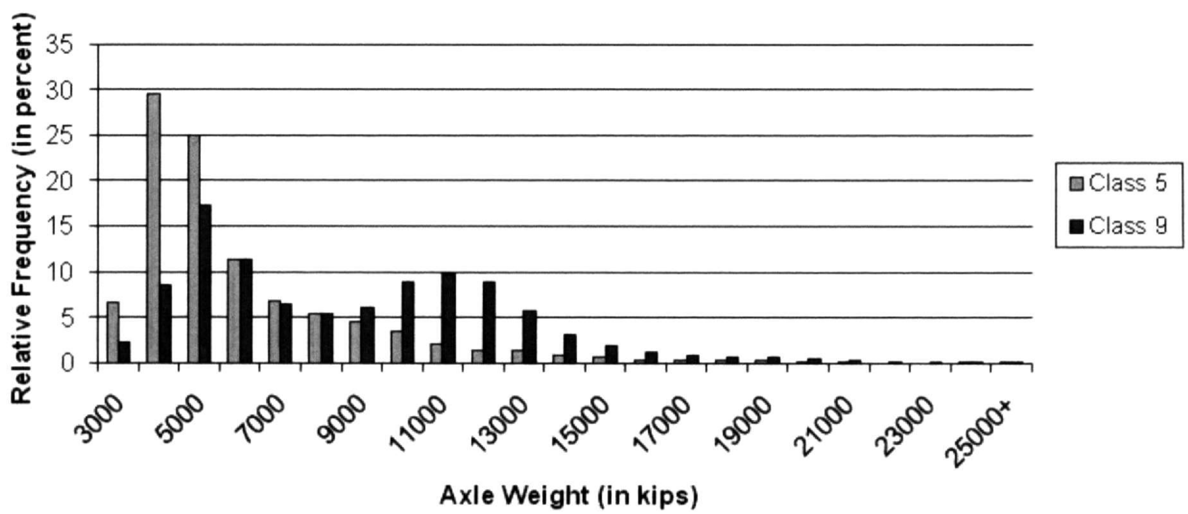

Figure 3. Axle load distribution for class 5 and class 9 single axles for January (Location: IH-70, Colorado).

Table 1. Experimental Design of the SPS-5 Experiments.

Rehabilitation Procedures			Factors for Moisture, Temperature and Pavement Condition							
			Wet				Dry			
Surface Preparation	Overlay Material	Overlay Thickness	Freeze		No-Freeze		Freeze		No-Freeze	
			Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor
Minimum	RAP	Thin	New Jersey	Missouri	Texas		Colorado, Montana		Oklahoma	
		Thick	New Jersey	Missouri	Texas		Colorado, Montana		Oklahoma	
	Virgin Mix	Thin	New Jersey	Missouri	Texas		Colorado, Montana		Oklahoma	
		Thick	New Jersey	Missouri	Texas		Colorado, Montana		Oklahoma	
Intense	RAP	Thin	New Jersey	Missouri	Texas		Colorado, Montana		Oklahoma	
		Thick	New Jersey	Missouri	Texas		Colorado, Montana		Oklahoma	
	Virgin Mix	Thin	New Jersey	Missouri	Texas		Colorado, Montana		Oklahoma	
		Thick	New Jersey	Missouri	Texas		Colorado, Montana		Oklahoma	

Table 2. General Information on SPS-5 Sites.

	Initial Monitoring	Overlay Construction	Open to Traffic	AADTT	Growth Rate (%, Linear)	% Traffic Design Lane	% Traffic Design Direction	Analysis Period (yrs)
New Jersey	Nov-91	Jul-92	Aug-92	840	5.9	100	100	14
Colorado	Jan-87	Sep-91	Oct-91	799	2.4	100	100	9
Missouri	Jan-98	Aug-98	Sep-98	569	3.1	100	100	8
Montana	Jan-87	Sep-91	Oct-91	702	4.5	100	100	10
Texas	Jan-87	Sep-91	Oct-91	301	16.1	100	100	14
Oklahoma	Jan-87	July-97	Aug-97	292	4.0	100	100	10

4.1.3 Material Information

Material-related information was obtained from the Material Test and Inventory databases. The MEPDG default values were accepted for design variables that were not available. It should be noted here that due to the unavailability of binder and mixture specific properties like dynamic modulus, the NCHRP 1-37A predictive equation was used to determine the mixture properties based on the gradation and the viscosity grade of the binders used for the specific projects. Volumetric properties like percentage of air voids and binder content by volume were available for each of the sections considered as part of this study. Project specific gradation, Atterberg's limits, and gravimetric properties were available for each of the unbound layers for all six project locations. It should be also noted that the Texas SPS-5 sections included lime-stabilized base and subbase courses. Performance prediction models for lime or cement treated base materials are yet to be calibrated in the Mechanistic Empirical Pavement Design Guide Version 1.0 and, therefore, the predicted distresses for these layers was assumed negligible. It should be also noted that lime stabilized bases, in general, do not rut significantly and, therefore, a zero rut assumption for these layers does not introduce a large error in the final predictions.

4.1.4 Distress Data

The second objective of this study focused on local calibration of the permanent deformation performance prediction model in the MEPDG for rehabilitated flexible pavements. Therefore, information on permanent deformation was obtained from the MON_T_PROF_INDEX_SECTION in the Monitoring dataset (Elkins et al, 2006) of the LTPP database. One of the key factors that were taken into consideration in the selection of pavement sections was the number of performance measurements recorded in the field. Not only is it important to have a good number of field observations as it helps in determining reliable bias correction factors, but also to have a sufficient number of early rut measurements, because rutting development is more severe during the early life of the pavement. Having in-field distress measurements during the terminal life of the pavement will most likely result in fairly even rut measurements. It is for all these reasons that a lot of emphasis was given on the choice of sections based on the number of distress measurements and how uniformly they are spread over the analysis period of the pavement.

It should be noted that there were as many as 16 distress measurements available for each of the pavement sections in the New Jersey SPS-5 site, which was the maximum, while the minimum was 5 measurements for each test section in the Colorado SPS-5 location.

4.2 LTPP SPS-3 Experiments

The LTPP SPS-3 studies focused on a total of 81 locations comprising of 486 sections throughout the United States. To make the problem manageable and minimize variability, however, this research analyzed only 13 sections from 4 Texas locations. It can be argued that the relatively small size of the dataset might drive the results to be statistically inconclusive. The reason behind restricting the dataset to a small number of sections was the time and computation effort it would otherwise require to handle a larger dataset. Even the determination of the Level 1 calibration coefficients required a minimum of a couple hundred runs on the MEPDG.

The objective of the SPS-3 experiments was to examine the optimal timing and effectiveness of each of the different pavement preservation techniques included as part of the experimental design. The analysis addressed three different strategies:

1. Thin overlay
2. Seal coat or surface treatment
3. No treatment (control)

In addition to the different treatment options listed above, the dataset also includes test sections from four different geographical locations in Texas, so that the effectiveness of these treatment options under different climatic conditions could be evaluated. A summary of the SPS-3 test sections considered for this research is given in Table 3.

Table 3. SPS-3 Experimental Sections.

County, District	Climate	Section	Const. Date	Sub-Base	Base	Binder Layer	Surface Course	Overlay
El Paso, El Paso	Dry No Freeze*	48-L310	April, 1991		8.4"		3.1"	
		48-L320	Sept, 1990		8.4"		2.3"	
		48-L330			8.8"		2.0"	
Rusk, Tyler	Wet No Freeze*	48-G310	Oct, 1990		11.3"		3.4"	
		48-G320			11.3"		2.0"	
		48-G330	Jan, 1987		11.3"		2.3"	
Mitchell, Abilene	Wet No Freeze*	48-D310	Oct, 1990	8.8"	6.8"	7.8"	2.1"	1.9"
		48-D320	Sept, 1990	8.8"	6.8"	7.3"	2.2"	1.1"
		48-D350		8.8"	6.8"	7.6"	2.4"	1.0"
Mills, Brownwood	Wet No Freeze*	48-Q310	Sept, 1990	10.0"	7.5"		1.9"	1.2"
		48-Q320		10.0"	7.5"		2.4"	
		48-Q330	Jan, 1987	10.0"	7.5"		2.1"	
		48-Q340		10.0"	7.5"		2.0"	

* The climatic classification for each region has been provided according to LTPP Information Management Systems (IMS) standards (Elkins et al, 2006).

It should be noted that sections that were structurally different were considered in the study. For example, SPS-3 test sections in Abilene are on Interstate Highway 20, which receives average daily truck traffic of 1,425 vehicles. Therefore, these sections are thicker and stronger. On the other side, the pavement sections in Tyler are located on State Highway 322 which receives an average daily truck volume of less than 100 vehicles and therefore is relatively thin. Thus, a pavement section that is part of a heavily trafficked corridor will definitely experience higher rut depth, but at the same time, sections that do receive high volumes of traffic are also built thicker to reduce the amount of rutting they might undergo. It can be argued based on the instances presented above that the effect of higher traffic is negated by the structural design of the pavement section, giving a zero net effect on the values of the bias correction factors.

CHAPTER 5. METHODOLOGY

5.1 CALIBRATION OF THE ASPHALT CONCRETE RUTTING TRANSFER FUNCTION IN THE MEPDG

The constitutive relationship used in the MEPDG for the permanent deformation model is based upon the statistical regression analysis found from repeated load permanent deformation tests conducted in the laboratory. The calibration parameters can be determined by analyzing current in-service pavement sections that are part of the SPS-5 experiments. These parameters are adjustable and known to depend upon local conditions. The calibration is done by comparing the observed pavement performance with the predicted pavement performance over time found from the MEPDG.

The MEPDG analysis is initiated with the default calibration parameters and then adjusted such that the difference between the observed and the predicted performance values are progressively reduced (Banerjee et al., 2009). The best fit minimizes the difference between the observed and the MEPDG predictions over the design life of all the pavements in a given region. It should be noted that the problem under study is a multi-dimensional non-linear minimization problem. The most direct approach to solve problems of this nature is to determine the direction of the steepest descent of the surface defined by the error term (difference of predicted and observed distresses) and the set of bias correction factors. The philosophy behind such a methodology lies in the fact that the slope of the steepest surface will eventually converge towards the local minima in the most efficient way. The MEPDG models for permanent deformation are:

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r_1} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_2 \beta_{r3}}$$

$$k_z = (C_1 + C_2 depth) 0.328196^{depth}$$

$$C_1 = -0.1039 H_{ac}^2 + 2.4868 H_{ac} - 17.342$$

$$C_2 = 0.0172 H_{ac}^2 - 1.7331 H_{ac} + 27.428$$

where,

H_{ac} = Total AC thickness (inches)

ε_p = Plastic strain (inch/inch)

ε_r = Resilient strain (inch/inch)

T = Layer Temperature (°C)

N = Number of load repetitions

k_z, k_2, k_3 = Laboratory constants

β_{r1}, β_{r2}, β_{r3} = Calibration Coefficients

A logarithmic transformation shows that the calibration coefficient β_{r1} is a shift factor that modifies the intercept term of the permanent deformation model. This shift factor primarily captures the differences in the distress predictions due to varying thicknesses of the HMA layers and other initial conditions. β_{r3} captures the differences due to the number of load repetitions. Thus, it represents the rate of permanent deformation progression. β_{r2} is the bias correction factor for temperature susceptibility of hot mix asphalt. Previous studies have shown that the distress predictions are quite sensitive to β_{r2}. However, the thermal susceptibility of the hot mix could not be determined, due to lack of material test data, so this research work will assume that the systematic differences in the predicted distress measurements do not stem from any bias in the thermal susceptibility of the mix. This assumption is reasonable because environmental condition and material characteristics are consistent within the projects and regions selected for this study.

5.2 DETERMINATION OF THE INITIAL GUESSES FOR THE CALIBRATION COEFFICIENTS IN THE RUTTING TRANSFER FUNCTION

Because of the non-linear nature of the problem, it is often difficult to determine the starting point of the optimization process. A good way to guess the starting values is to closely examine the measured and predicted distress values with the default bias correction factors. For example, if the predicted distress measurements are significantly lower than the observed values, which will require adjustments to the bias correction factors; this will be the case initially, in most occasions. The next step will involve evaluating whether the predicted and the observed values are parallel and adjust the initial predictions or whether the initial predictions and the rates of progression of rutting are different. If the first case happens, adjustments to the bias correction factor that governs the intercept of transfer function (β_{r1}) will most likely bring the predicted and observed values close to each other as the error in this case is mostly due to a significant shift in the distress values at each time point. On the other hand, if the problem resembles the latter, it will require adjustments to both the intercept (β_{r1}) and the slope (β_{r3}) of the transfer function – in this case the intercept should go higher and the slope should go lower such that the initial predictions go higher while at the same time the rate of progression of rutting slows down.

In addition to the guidelines stated above, certain generic trends were observed between the calibration coefficients that were determined and the volumetric mixture properties, local climatic conditions and the traffic volume. Table 1 summarizes these observations and recommends using these guidelines to obtain a ball park estimate of the bias correction factors.

Table 4. Recommendations for Determination of the Guessed Estimate for β_{r1} & β_{r3}.

State	Climate	(β_{r1}, β_{r3})	Traffic (AADTT)	Asphalt Grade	Asphalt Content (%)	Air Voids (%)
Colorado	Dry Freeze	238.0, 0.142	799	AC20	8.5	7.5
New Jersey	Wet Freeze	112.0, 0.122	840	AC30	7.7	3.5
Missouri	Wet Freeze	129.0, 0.140	569	AC30	7.2	9.7
Montana	Dry Freeze	320.0, 0.138	702	AC20	8.4	5.6
Texas	Dry No-freeze	80.0, 0.444	301	AC40	8.1	4.5
Oklahoma	Dry No-freeze	107.0, 0.252	292	AC40	7.8	5

Recommendations for Determining the Guessed Estimate Value for β_{r1}

Criteria: AC Content (by volume)	<7.5%, β_{r1} = 100
	For every 0.5% increase in the AC content beyond 7.5%, increase β_{r1} by 50
Criteria: AC Grade	For AC40, no adjustment is required
	For every single drop in the viscosity grade, increase β_{r1} by 50
Criteria: Air Voids	Air Void ≤ 5%, no adjustment is required
	For every 1% increase in the air void content, increase β_{r1} by 25

Recommendations for Determining the Guessed Estimate Value for the product of β_{r1} & β_{r3}

Criteria: Climatic Region	For no freeze zones, ($\beta_{r1} \times \beta_{r3}$) = 30
	For freezing zones, ($\beta_{r1} \times \beta_{r3}$) = 20
Criteria: Traffic	For traffic volume = 600, no adjustment is required
	For every single increment/decrement of 50 over 600, increase/decrease ($\beta_{r1} \times \beta_{r3}$) by 5.

Once the starting values for the optimization problem are determined, the next step will involve determining the slope of the steepest descent. The most efficient way to determine the steepest direction of descent is to run the MEPDG for a range of slope values, holding the intercept constant, and then relaxing the intercept of the transfer function. Two or three consecutive runs will help the user determine the direction of the steepest descent. One can follow this direction until it leads to the minimum for the optimization problem. However, it should be noted that the convergence point might be a local or the global minima for the problem. Therefore, it is necessary to disturb the point of convergence and recheck whether it still reaches the same solution for the problem. If so, it can be concluded that the point of convergence is indeed the global minima for the optimization problem and hence the final set of regional bias correction factors for the specified location.

5.3 CALIBRATION OF THE SUBGRADE RUTTING TRANSFER FUNCTION IN THE MEPDG

The MEPDG uses the following transfer function to relate permanent deformation in the subgrade to the response of the material.

$$\delta_a(N) = \beta_{s_1} k_1 \varepsilon_r h \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \left| e^{-\frac{\rho^\beta}{N}} \right|$$

where,

δ_a = Permanent deformation of the layer
N = Number of load repetitions
ε_v = Average vertical strain (inch/inch)
h = Thickness of the layer (inches)
$\varepsilon_0, \beta, \rho$ = Material properties
ε_r = Resilient strain (inch/inch)
β_{s1} = Calibration coefficient

In case of permanent deformation in the subgrade, the calibration coefficient β_{s1} captures the deviation in predictions from the observed distresses that may arise due to differences in the material properties. For the current study, β_{s1} was preset to regional defaults. These regional defaults were selected on the basis of type and average moisture content for the subgrade soil and the annual precipitation. This approach was adopted after investigating previous studies done on calibration of the MEPDG for the Montana Department of Transportation (von Quintus et al., 2007).

CHAPTER 6. RESULTS AND DISCUSSION

6.1 LEVEL 1 CALIBRATION FACTORS AND EFFECTIVENESS OF MAINTENANCE STRATEGIES

The Level 1 bias correction factors as obtained for each of the SPS-5 and SPS-3 test locations are given in Table 1. Looking at the results from the SPS-5 test sites, one can definitely see a trend in the data. Test sites that were constructed with recycled asphalt mixes have a lower product for the two bias correction factors under consideration. The product of the two bias correction factors was considered meaningful because the rut depth at any given point in time is directly proportional to the bias correction factors. This means a higher value for any of the bias correction factors or both will translate to a greater rut depth. Thus, their product jointly captures the influence of both calibration coefficients on the rut depth at any point in time. This trend in the data can be translated as sections that are constructed with mixes containing RAP are less susceptible to rutting. Previous research studies have also suggested that the use of recycled asphalt mix tends to improve the performance of the asphalt mixtures from a rutting perspective (Putman et al., 2005). Therefore, the initial trend in the data reinforces the fact RAP mixes are relatively more rut resistant than virgin mixes.

Table 5. Level 1 Bias Correction Factors for SPS-3 and SPS-5 Test Locations.

<table>
<tr><th colspan="7">LTPP SPS-5 Test Locations</th></tr>
<tr><th>Section Id</th><th>Milling V/s No Milling</th><th>RAP V/s Virgin Mix</th><th>Overlay Thickness (inches)</th><th>β_{r1}</th><th>β_{r3}</th><th>$\beta_{r1} \times \beta_{r3}$</th></tr>
<tr><td>48-A502</td><td>No Milling</td><td>RAP</td><td>2.2”</td><td>60.0</td><td>0.332</td><td>19.9</td></tr>
<tr><td>48-A503</td><td>No Milling</td><td>RAP</td><td>5.3” (2.1” surface + 3.2” binder course)</td><td>84.0</td><td>0.094</td><td>7.90</td></tr>
<tr><td>48-A504</td><td>No Milling</td><td>Virgin Mix</td><td>5.3” (2.2” surface + 3.1” binder course)</td><td>34.0</td><td>0.594</td><td>20.2</td></tr>
<tr><td>48-A505</td><td>No Milling</td><td>Virgin Mix</td><td>2”</td><td>56.2</td><td>0.360</td><td>20.2</td></tr>
<tr><td>48-A506</td><td>Milling</td><td>Virgin Mix</td><td>3.9” (2.3” surface + 1.6” binder course)</td><td>51.2</td><td>0.404</td><td>20.7</td></tr>
<tr><td>48-A507</td><td>Milling</td><td>Virgin Mix</td><td>7” (2” surface + 5” binder course)</td><td>35.6</td><td>0.596</td><td>21.2</td></tr>
<tr><td>48-A508</td><td>Milling</td><td>RAP</td><td>7.3” (2.1” surface + 5.2” binder course)</td><td>93.4</td><td>0.128</td><td>12.0</td></tr>
<tr><td>48-A509</td><td>Milling</td><td>RAP</td><td>4.3” (2.2” surface + 2.1” binder course)</td><td>26.1</td><td>0.460</td><td>12.0</td></tr>
<tr><th colspan="7">LTPP SPS-3 Test Locations</th></tr>
<tr><th>Section Id</th><th colspan="2">Treatment Type</th><th>Climate</th><th>β_{r1}</th><th>β_{r3}</th><th>$\beta_{r1} \times \beta_{r3}$</th></tr>
<tr><td>48-L310</td><td colspan="2">Thin Overlay</td><td>Dry-Warm</td><td>0.500</td><td>1.108</td><td>0.554</td></tr>
<tr><td>48-L320</td><td colspan="2">Seal Coat</td><td>Dry-Warm</td><td>11.1</td><td>0.498</td><td>5.53</td></tr>
<tr><td>48-L330</td><td colspan="2">No Treatment</td><td>Dry-Warm</td><td>35.6</td><td>0.278</td><td>9.90</td></tr>
<tr><td>48-G310</td><td colspan="2">Thin Overlay</td><td>Wet-Cold</td><td>14.9</td><td>0.252</td><td>3.76</td></tr>
<tr><td>48-G320</td><td colspan="2">Seal Coat</td><td>Wet-Cold</td><td>52.7</td><td>0.410</td><td>21.6</td></tr>
<tr><td>48-G330</td><td colspan="2">No Treatment</td><td>Wet-Cold</td><td>19.6</td><td>0.510</td><td>10.0</td></tr>
<tr><td>48-D310</td><td colspan="2">Thin Overlay</td><td>Dry-Cold</td><td>33.9</td><td>0.350</td><td>11.9</td></tr>
<tr><td>48-D320</td><td colspan="2">Seal Coat</td><td>Dry-Cold</td><td>47.9</td><td>0.400</td><td>19.2</td></tr>
<tr><td>48-D350</td><td colspan="2">Seal Coat</td><td>Dry-Cold</td><td>47.3</td><td>0.446</td><td>21.1</td></tr>
<tr><td>48-Q310</td><td colspan="2">Thin Overlay</td><td>Mixed</td><td>0.500</td><td>0.260</td><td>0.130</td></tr>
<tr><td>48-Q320</td><td colspan="2">Seal Coat</td><td>Mixed</td><td>26.5</td><td>0.294</td><td>7.79</td></tr>
<tr><td>48-Q330</td><td colspan="2">No Treatment</td><td>Mixed</td><td>5.20</td><td>0.522</td><td>2.71</td></tr>
<tr><td>48-Q340</td><td colspan="2">No Treatment</td><td>Mixed</td><td>14.1</td><td>0.344</td><td>4.85</td></tr>
</table>

As for the SPS-3 test locations, it seems that pavements that received a thin overlay performed better than their counterparts because they have a lower product for the two bias correction factors under study. A more detailed analysis of the results for the SPS-3 and SPS-5 locations is included in the subsequent discussion.

6.1.1 Discussion of the Results from the Texas SPS-5 Test Locations

Linear regression analysis was used in the process of identifying key experimental variables that had a significant effect on β_1. Unfortunately, the results did not show any significant evidence against the null hypothesis. The reasons for this could be attributed to the high variability of the field data and the relatively small database.

The investigation of the influence of "milling", "overlay thickness", and use of recycled asphalt mix on β3 led to some interesting findings. The results are given in Table 2.

Table 6. Influence of the Experimental Variables on β_3.

	Coefficient	t-stat	p-value
Intercept	0.495	2.7	0.054
Milling	0.069	0.5	0.667
Overlay Thickness	-0.009	-0.2	0.836
Recycle Asphalt Mix	-0.233	-1.8	0.142

The results indicate that although "milling" and "overlay thickness" have no significant effect on the bias correction factor β_3, the use of recycled asphalt mix may have a significant effect while testing the hypothesis at a confidence level of 85%. There is enough reason to believe that the use of recycled asphalt mix can retard the rate of growth in the rut depth over time (Putman et al., 2005). Since there was significant evidence that the bias correction factor β_3 was influenced by the type of mix, a decision was made in regressing β_3 against the "type of mix – recycled/virgin" variable. The results are summarized in Table 3.

Table 7. Influence of use of Recycled Asphalt Mix on β_3.

	Coefficient	t-stat	p-value
Intercept	0.489	6.5	0.001
Recycle Asphalt Mix	-0.235	-2.2	0.070

The results given above indicate that the use of recycled asphalt definitely influences the value of β_3. The negative sign of the regression coefficient indicates that a mix containing recycled asphalt is expected to perform better (less rutting) than a virgin mix because it will retard the rate of distress progression.

It was often noticed that the two bias correction factors are correlated and both of them share a direct proportionality with the predicted rut depth at any given time point. Therefore, it would be worthwhile to see if there is any influence of the input variables on the product of the two bias correction factors. The results of this particular analysis are given in Table 4.

Table 8. Influence of the Experimental Variables on the Product of β_1 and β_3.

	Coefficient	t-stat	p-value
Intercept	24.2	6.2	0.003
Milling	1.20	0.4	0.725
Overlay Thickness	-0.933	-1.1	0.342
Recycle Asphalt Mix	-7.43	-2.7	0.052

The results presented in Table 4 indicate that for the dataset evaluated, apart from the use of recycled asphalt mix (RAP), the other variables are not influential in determining the value of the product of the two bias correction factors. The negative sign of the regression coefficient for the RAP indicates that mixes designed with RAP are expected to perform better and rut less than mixes with virgin materials. However, the results as given in Table 2 suggest that section 48-A502, inspite of the fact that it had a 35% RAP mix its product of the two bias correction factors, are much higher than all the other sections that had a RAP percentage. This particular observation can be attributed to the fact that it had an overlay thickness of merely 2.2" whereas all the other RAP sections had an overlay thickness in excess of 4".

It has already been stated that β_1 and β_3 are correlated and therefore estimating either of the bias correction factors separately is not the most appropriate thing to do. Situations that require more than one response variable make use of simultaneous equations where both the equations and their parameters are estimated together. One of the major benefits of using a joint estimation approach is to improve the efficiency of the estimated coefficients. The analysis was done using SAS® (originally Statistical Analysis System) and the results are given in Table 5.

Table 9. Parameter Estimates for β_1 and β_3.

Parameter Estimates for β_1			
	Coefficient	**t-stat**	**p-value**
Intercept	102	6.6	0.007
β_3	-137	-5.4	0.012
Milling	-4.94	-0.6	0.568
Overlay Thickness	2.65	1.3	0.289
Recycle Asphalt Mix	-11.2	-1.3	0.288
Parameter Estimates for β_3			
	Coefficient	**t-stat**	**p-value**
Intercept	0.719	9.5	0.003
β_1	-0.00700	-5.4	0.012
Milling	-0.0260	-0.5	0.666
Overlay Thickness	0.0170	1.1	0.346
Recycle Asphalt Mix	-0.100	-1.9	0.159

The results given above indicate that none of the experimental variables have a statistically significant influence on β_1, while the use of recycled asphalt may have a significant effect on the value of β_3, provided the null hypothesis is tested at an 85% level of significance. It should be noted here that a level of significance higher than 85% will render all of the explanatory variables insignificant. Thus no individual variable will have a significant effect on the values for β_1. Nonetheless, a test of hypothesis with all the independent variables showed that they together have a significant effect (p-value < 0.05) on the β_1 and β_3. Further investigation reinforced this observation and the results so obtained are given in Table 6.

Table 10. Parameter Estimates for β_1 and β_3.

Parameter Estimates for β_1			
	Coefficient	**t-stat**	**p-value**
Intercept	102	6.6	0.007
β_3	-137	-5.4	0.012
Milling	-4.94	-0.6	0.568
Overlay Thickness	2.65	1.3	0.289
Recycle Asphalt Mix	-11.2	-1.3	0.288
Parameter Estimates for β_3			
	Coefficient	**t-stat**	**p-value**
Intercept	0.764	13.9	0.000
β_1	-0.00600	-5.9	0.002
Recycle Asphalt Mix	-0.100	-2.1	0.086

The results in Table 6 show that the use of recycled asphalt in the mix has an effect on the bias correction factor β_3. Also, the negative sign of the regression coefficient indicates that a mix with recycled asphalt mix is effective in retarding the rate of distress progression.

6.1.2 Discussion of the Results from the Texas SPS-3 Test Locations

Initial results indicated none of the treatment types were instrumental in determining the value of β_1. On the other hand, a warmer climate has a significant effect on its value. Results as obtained from the linear regression are given in Table 7.

Table 11. Influence of the Treatment Type and Climate on β1.

	Coefficient	t-stat	p-value
Intercept	40.1	4.4	0.002
Thin Overlay	-13.3	-1.6	0.152
Seal Coat	8.46	1.0	0.340
Warm climate	-21.0	-3.2	0.013
Wet climate	-7.64	-1.2	0.284

The results indicate that under warmer conditions, the pavement is less likely to rut to high levels during the initial period after construction, which is against what could be expected. However, mixes in warmer areas are designed with higher viscosity binders as opposed to softer used in other areas.

Thin overlays are often used as a pavement preservation technique to retard or correct minor rutting problems and, therefore, this should influence the bias correction factors. So it was decided to model β_1 as a function of the treatment types separately. However, the results indicated seal coats as a treatment option do not have influence on the value of β_1, while thin overlays were found to have a significant influence on the same. The results as obtained are given below in Table 8.

Table 12. Influence of Thin Overlay as a Treatment Type on β_1.

	Coefficient	t-stat	p-value
Intercept	28.9	5.0	0.000
Thin Overlay	-16.4	-1.6	0.140

The results indicate that thin overlays have a significant effect on the bias correction factor β_1 when testing the null hypothesis with a confidence level of 85%. The negative sign for the regression coefficient indicates that they can be used as a treatment to stop pavement from rutting too much at an early stage of its service life. On the other hand, seal coats, though quite significant in determining the value of β_1, has an adverse effect on the initial rutting for a pavement. It should be remembered that seal coats are not designed to stop or arrest rutting, but rather to seal the underlying pavement structure and stop the aging of the bituminous mix.

Analysis of the results indicated that neither any of the treatment types nor the type of climate had any impact on β_3 so no specific calibration is required in this case.

The results were analyzed to see if there was any influence of the treatment types on the product of the bias correction factors in order to identify which treatment types is more likely to retard rutting. Results suggested that a thin overlay is likely to reduce rutting at any given time. This was also previously observed while trying to investigate the influence of the treatment types on β_1. At the same time, warm weather was also found to produce a similar result, probably to the use of harder binders. Preliminary results suggested that seal coats do not have an effect on the product of β_1 and β_3. The results of the analysis as obtained with linear regression are given in Table 9.

Table 13. Influence of the Treatment Type and Climate on the Product of β_1 and β_3.

	Coefficient	t-stat	p-value
Intercept	16.0	5.9	0.000
Thin Overlay	-5.83	-2.4	0.047
Seal Coat	3.91	1.6	0.152
Warm climate	-9.32	-4.7	0.001
Wet climate	-2.86	-1.5	0.183

A simultaneous estimation approach was also adopted towards determining which of the experimental variables have a significant effect on the bias correction factors β_1 and β_3. A simultaneous estimation is more appropriate because the two bias correction factors are not independent, implying that the regression coefficients should not be estimated separately. The results as obtained have been provided in Table 10.

Table 14. Parameter Estimates for β1 and β3.

Parameter Estimates for β_1			
	Coefficient	**t-stat**	**p-value**
Intercept	46.2	5.5	0.000
β_3	-11.4	-0.7	0.511
Thin Overlay	-16.8	-2.2	0.055
Warm climate	-22.6	-3.2	0.011
Parameter Estimates for β_3			
Intercept	0.550	1.9	0.089
β_1	0.000	-0.7	0.511
Thin Overlay	0.010	0.1	0.963
Warm climate	-0.020	-0.1	0.921

The results show that using a thin overlay as well as pavement sections built in warmer climates are likely to rut less than other candidate sections. There is enough reason to believe that a thin overlay will be successful in arresting the initial rutting because it is mostly designed for correcting minor rutting problems. However, it is harder to explain why a warmer climate shows reduced initial rutting. In fact, for the same mix properties, warmer temperatures translate to higher initial rutting. Therefore, this apparently unreasonable result is attributed to the likelihood that different binders were used, as is common practice in Texas. The difference in itself is not important for this study. What is important is that a difference exists and is significant so different calibration coefficients of bias correction factors should be used.

On the other hand, none of the variables were found to influence the value of β_3. Also, seal coats as a treatment type were found to have no impact on the values of the bias correction factors.

6.2 CALIBRATION OF THE PERMANENT DEFORMATION PERFORMANCE MODELS FOR REHABILITATED PAVEMENTS

As stated, the second objective of this study focuses on obtaining a set of regional calibration coefficients (Level 2) for the AC permanent deformation model for each of the regions. The Level 2 calibration that was conducted for each of the six regions involved determining a set of calibration coefficients that jointly minimizes the sum of squared errors (the square of the difference of the observed and the predicted distress measurements = SSE) for all the sections (within a specific region) taken together. Although this method is mathematically sound and statistically efficient, it requires a significant amount of computation effort when the number of sections increases. The final set of bias correction factors is included in Table 11.

Table 15. Final Bias Correction Factors & Standard Error of the MEPDG Permanent Deformation Performance Model for each of the Calibrated Regions.

County	State	Climate/Region	β_{r1}	β_{r3}	β_{s1} (Fine & Coarse Grained)	Standard Error of Predicted Permanent Deformations	% Reduction in Standard Error
Lincoln	Colorado	Dry Freeze/Western	238.0	0.142	0.3	0.055	61.7
Monmouth	New Jersey	Wet Freeze/North Atlantic	112.0	0.122	0.7	0.055	25.2
Taney	Missouri	Wet Freeze/North Central	129.0	0.140	0.7	0.083	41.1
Sweet Grass	Montana	Dry Freeze/Western	320.0	0.138	0.3	0.105	61.7
Kaufman	Texas	Wet No Freeze/Southern	80.0	0.444	0.5	0.075	59.8
Comanche	Oklahoma	Wet No Freeze/ Southern	107.0	0.252	0.4	0.081	50.2

The calibration coefficients as presented in Table 11 show that pavements rehabilitated in "No Freeze" areas tend to rut less in their early life but continue to rut more as they are subjected to high traffic volumes. It should be kept in mind that these mixes were designed with different binders. On the other hand, roads that undergo rehabilitation in freezing zones tend to rut much more in their early life but are less affected by traffic loading. It should be also noted that the predicted rutting is directly proportional to both β_{r1} and β_{r3}. Therefore, the product of the two bias correction factors gives an idea about the total amount of rutting a pavement is likely to undergo over its design life. This parameter is higher in case of pavements constructed in "No Freeze" zones. This observation happens to fall in line with our normal expectation as pavements built in warmer climatic areas generally have relatively higher levels of rutting. It should be noted in this context that the pavements constructed in Missouri, Montana, Colorado, and New Jersey used a softer binder grade (AC-10 and AC-20) to address issues related to low temperature thermal cracking. On the other hand, a stiffer binder grade, mostly AC-40, was used in the SPS-5 test sections in Texas and Oklahoma which were successful in resisting high levels of initial rutting. This contrasts with pavements in Montana, Missouri or Colorado, which had higher levels of initial rutting. However after the first few seasons during the pavement's service life, the binder undergoes long term aging which results in a stiffer binder grade irrespective of their initial binder grades. Under such conditions, pavements that are now exposed to warmer climatic areas will naturally deform more under heavy traffic loads compared to their counterparts that are in service in colder areas. This explains the difference in the bias correction factors between "Freeze" and "No Freeze" areas.

6.2.1 Comparison of Calibrated Versus Uncalibrated MEPDG Predictions

Figure 1 shows the predictions as obtained from the permanent deformation prediction model in the MEPDG before and after the calibration were done. It can be seen that there is a significant improvement in the performance predictions from the Design Guide after each of these six geographical areas were calibrated to local conditions.

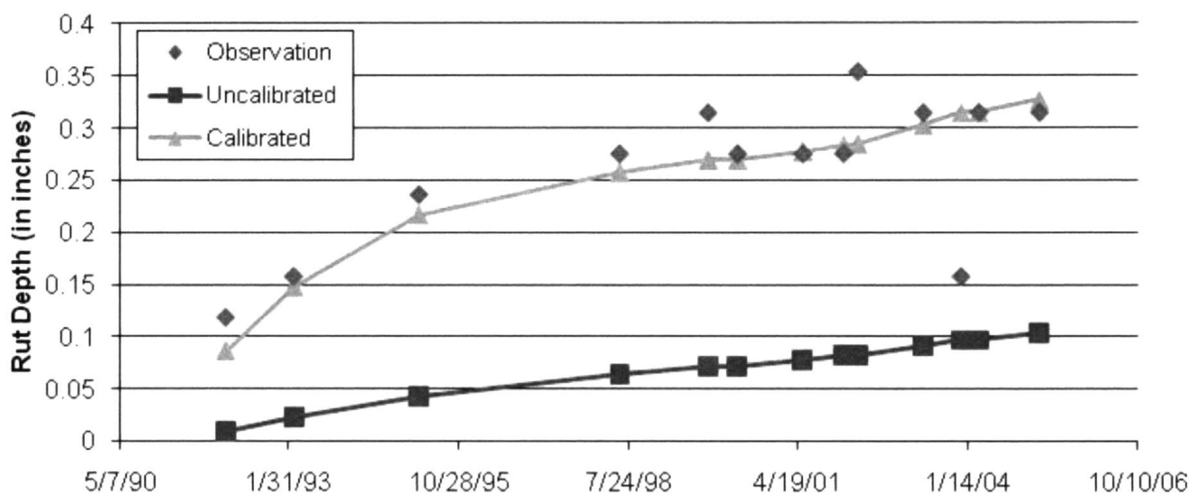

Figure 4. Calibrated Versus Uncalibrated Predictions For Section 48-A502 (Kaufman, Texas).

6.2.2 Validation

In order to have confidence in the calibration coefficients, it is necessary to validate the proposed values against pavement sections from the same region. Table 12 summarizes the results of the validation study from each of the six geographical regions that were included in this study. It should be noted here that since the bias correction factors that are being proposed related to rehabilitated pavement sections, it was therefore necessary to identify sections that has also undergone serious maintenance work. In addition, information related to their structure prior to rehabilitation and historical information on the rate of rutting was required. It is due to all these reasons the following sections were selected for the validation exercise. Unfortunately the constraints highlighted above led to the selection of a single section from the states of Missouri and Montana. Overall the results were positive in the sense that the standard error of prediction was considerably lower for the predicted performance with the calibrated bias correction factors. However, further validation of the bias correction factors for the states of Missouri and Montana is recommended.

Table 16. Standard Error of Prediction for the Validation Dataset.

StateCode-SHRPID-ConstructionID	Standard Error of Prediction (in inches) (Uncalibrated)	Standard Error of Prediction (in inches) (Calibrated)	% Reduction in Standard Error
Colorado			
8-6013-5	0.1374	0.0366	73.3
8-1053-2	0.1946	0.0662	66.0
8-1047-2	0.1994	0.0596	70.1
8-1029-5	0.1243	0.0515	58.6
New Jersey			
34-6057-1	0.0912	0.0426	53.3
34-1003-4	0.0794	0.0437	45.0
34-1011-2	0.0909	0.0412	54.7
34-1030-4	0.1214	0.0464	61.8
34-1031-2	0.1261	0.0381	69.7
34-1010-2	0.1493	0.0527	64.7
Missouri			
29-6067-1	0.0779	0.0356	54.3
Montana			
30-8129-2	0.1108	0.0303	72.6
Texas			
48-3835-3	0.1095	0.0608	44.4
48-1113-2	0.1305	0.0439	66.4
48-1119-2	0.1584	0.0370	76.7
48-1068-5	0.1448	0.0311	78.5
48-1111-2	0.1157	0.0342	70.4
Oklahoma			
40-4163-6	0.2017	0.0876	56.6
40-6010-1	0.1891	0.0424	77.6
40-4154-2	0.2064	0.0889	57.0
40-4087-2	0.2179	0.0390	82.1
40-4164-3	0.2271	0.0352	84.5

CHAPTER 7. CONCLUSIONS

7.1 EFFECTIVENESS OF MAINTENANCE STRATEGIES ON PERFORMANCE OF PAVEMENTS FROM A RUTTING PERSPECTIVE

The first objective of this study led the researchers to some interesting conclusions on how various rehabilitation and maintenance strategies influence the bias correction factors. The most important conclusion is that different maintenance and rehabilitation techniques do require specific calibration coefficients or bias correction factors: "one size does not fit all." Besides this, other more specific findings were realized.

Firstly, it was observed and tested that using a mix containing recycled asphalt pavement (RAP) is more likely to retard the rate of rutting development. The trends were similar when the regression parameters were estimated separately as well as simultaneously. However, a simultaneous estimation showed the effect of using RAP was less severe on the rate of distress progression. Nonetheless, none of the input variables were found to be influential in determining the initial rut depth. It was also found that pavement sections that were built with RAP can be expected to rut less at any given time.

On the other hand, after scrutinizing the bias correction factors obtained for each of the SPS-3 test sections, it was found that none of the explanatory variables had any effect on the rate of distress progression. However, both seal coats and thin overlays governed the values of $\beta1$. Thin overlay sections were found to resist initial rutting while seal coats as a treatment option showed the opposite trend. Because thin overlays are mostly designed to correct minor rutting problems, it can be expected that they show lower amount of initial rutting. It was also seen that pavement sections built in warmer climate deformed less than their counterparts. This indicates that mixes in warmer weather are designed to be more rut resistant.

When the regression coefficients were estimated simultaneously, seal coats appeared to have no effect on the value of the bias correction factors. It was also found that pavement sections that had thin overlays or if they were built in and designed for a warmer climatic region, can be expected to rut comparatively less.

7.2 CALIBRATION OF THE MEPDG RUTTING TRANSFER FUNCTIONS FOR REHABILITATED PAVEMENTS

The second objective focuses on calibration of the permanent deformation performance model in the MEPDG for rehabilitated pavements. The methodology proposed as part of this study uses a joint optimization procedure that seeks to determine the direction of the steepest slope for the surface, defined by the bias correction factors in the asphalt concrete permanent deformation transfer function and the sum of squared errors between the observed and predicted distress measurements. The study proposes a set of guidelines to determine the guessed estimate for the bias correction factor for the AC rutting transfer function which will then be optimized to determine the set of regional bias correction factors.

The study looked into six different geographical regions and developed bias correction factors for the asphalt and subgrade rutting transfer functions. The dataset for each of these geographical areas was comprised of eight different pavement sections, though there are certain differences in the rehabilitation work on each section. The bias correction factors that have been developed in this study, therefore, apply to a pavement that represents an average of these eight sections for that given region. The bias correction factors proposed as part of this study were validated against GPS-6 sections from the LTPP database. The results showed that the calibration coefficients provided a much better fit for the performance predictions when compared against the in-field observed measures of distresses.

Given that the behavior of pavements differs significantly from one region to another, it is recommended that more pavement sections in more regions be included in the determination of these bias correction factors in order to develop a higher level of confidence in these factors and to develop a set of factor to address the entire nation. Reliable bias correction factors will enable state and local agencies to perform accurate and reliable analysis of the performance of flexible pavements, thus saving significant resources in terms of time and money.

This study also pointed out that pavements rehabilitated in "No Freeze" zones are likely to rut less during their early life compared to pavements that are constructed in freezing zones. On the other hand, roads in warmer climatic areas are likely to experience faster rate of progression of rutting with time than those in colder areas and this observation has been attributed to the heterogeneity between pavements due to softer and stiffer binder grades that are used during their construction as well as how the mixture properties change due to effects of long term aging.

By comparison with bias correction factors developed for new pavements, the ones developed in this study indicate that we should not mix calibration factors for new and rehabilitated pavements. They are significantly different and they seem to indicate that for rehabilitated pavements, $\beta r1$ tend to be larger and $\beta r3$ smaller as to those developed for new pavements.

It is also emphasized that until more efforts are channelized in this direction, the bias correction factors proposed as part of this study can be adopted rather than using national averages for more reliable analysis of pavement performance.

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