CENTER FOR TRANSPORTATION RESEARCH
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PROJECT REPORT ON THE CONTINUED DEVELOPMENT AND ANALYSIS OF THE FLEXIBLE PAVEMENTS DATABASE

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16. Abstract

The original scope of TxDOT Research Project 0-6275 included continuing the development of the Texas Flexible Pavement database (TFPD) developed as part of Research Project 0-5513. The objective of 0-5513 was to deliver a database capable of hosting the data necessary for new pavement design models that would result in more reliable and economical pavement design. The objective of 0-6275 was to continue the data collection and data population of the flexible pavement databases and to calibrate the models (transfer functions) used in the new MEPDG as well as TxDOT's mechanistic design checks. In addition, this new project was to review the development conducted in 0-5513 to evaluate whether the database created covered the inference space of pavements in Texas and to make the necessary additions and changes. Therefore, the efforts of 0-6275 were primarily twofold. One aspect was geared towards extensive performance monitoring and material testing for better characterization of the experimental sections included as part of 0-5513 as well as new sections that were to be incorporated in the enhanced version of the TFPD. The other aspect of the study was to look into the calibration of the transfer functions in the newly developed MEPDG as well as Texas ME Design procedure being sponsored by TXDOT through the 0-5798 research study. It was understood that an effort to determine reliable bias correction factors would necessitate a large volume of project-specific information for a number of pavement sections that are well spread out within the state such that differences in materials, traffic, construction practices and climate are captured. The TFPD supplemented by other databases like the DCIS and the LIMS will ensure that the data needs for the calibration exercise are met and fulfilled to the extent possible. When time series data were not available, the researchers utilized relevant data from the FHWA's Long Term Pavement Performance Studies (LTPP). This interim report summarizes the activities performed during the first 7 months of the project, when the current project was terminated. Therefore, this document reports on the work performed in Tasks 1 through 7.

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Engineering Disclaimer

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Acknowledgments

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Chapter 1. Introduction

1.1 Background

For more than 30 years, in an almost continuous effort that began in 1972, the Texas Transportation Institute (TTI) maintained a Texas Flexible Pavement Database (TFPD). This database originally comprised 350 pavement sections that were selected following a stratified random sampling approach. The number of sections selected in each TxDOT District was proportional to the number of total miles in each district for each type of facility (e.g. IH, US and State Highways, FM and RM roads). The TFPD has gone through several changes and updates from the time it came into existence, the latest as part of the TxDOT funded project 0-5513 (Hong et al., 2008; Banerjee et al., 2010).

The 0-5513 research project had two primary goals: development of the TFPD and local calibration of the performance models in the Mechanistic Empirical Pavement Design Guide (MEPDG) developed under the sponsorship of the National Cooperative Highway Research Program (NCHRP). The TFPD developed under this project can be accessed online (http://pavements.ce.utexas.edu) and is open to all registered users; anyone with access to the internet can register. The database currently houses nearly 160 flexible pavement sections of which only 73 sections are part of the experimental design proposed for this research project. Of these 73 pavement sections, 32 have been adopted from the Long Term Pavement Performance (LTPP) database that was established as part of the Strategic Highway Research Program (SHRP) in 1987. The other 41 sections are the Texas sections included by the research team as part of the TFPD. Another 18 pavement sections from the Specific Pavement Studies (SPS) experiment of the LTPP database has been also included in the TFPD because these 18 formed the dataset for the local calibration of the permanent deformation performance model in the MEPDG. To date, the LTPP sections have provided the bulk of the data for calibration because of the amount of times series data.

The web interface provides the user an ability to view as well as download structural and traffic details, performance and deflection data and material information for each of these pavement sections. There are more than 130 users registered including many states (Arizona, California, Illinois, Iowa, Georgia, Louisiana, Minnesota, New Mexico, North Carolina, Texas, Washington DC) in the USA as well as many other countries, including Argentina, Australia, Brazil, Canada, Chile, China, France, and South Africa.

1.2 Project Objectives & Scope

The scope of the 0-6275 Research Project included continuing the development of the TFPD developed as part of the 0-5513 Research Project. The database was to support new pavement design models that would result in more reliable and economical pavement designs. The objective of this new project was to continue the data collection and data population of the flexible pavement databases and to calibrate the models (transfer functions) used in the new MEPDG as well as TxDOT's mechanistic design checks. In addition, this new project would review and update the developments of 0-5513 to evaluate the database created covers the inference space of pavements in Texas and to make the necessary additions and changes required. Therefore, the efforts of 0-6275 were primarily twofold. The first aspect was extensive performance monitoring and material testing for better characterization of the experimental sections included as part of 0-5513 as well as new sections that would be incorporated in the enhanced version of the TFPD during 0-6275. The other aspect of the study was to address the calibration of the transfer functions in the newly developed MEPDG as well as the Texas ME Design procedure, which is being sponsored by TXDOT through the 0-5798 Research Project.

It is understood that an effort to determine reliable bias correction factors would necessitate a large volume of project-specific information for a number of pavement sections that are well spread out within the state such that differences in materials, traffic, construction practices, and climate are captured. The TFPD, supplemented by other databases such as the DCIS (Design and Construction Information System) and the LIMS (Laboratory Information Management System), would ensure that the data needs for the calibration exercise are met and fulfilled to the extent possible. When data available in the TFPD were not sufficient (in particular, time series observations of pavement performance), the research team made use of data from the Federal Highway Administration's (FHWA) LTPP. To date, the LTPP database is the only database available that provides up to 20 years of performance information, which is essential for the calibration of transfer functions.

The project kick off meeting took place on March 13, 2009, and the last project progress meeting took place on October 5, 2009. Soon after this second meeting, the project was terminated because the research team was not progressing at the anticipated rate. Therefore, this interim report presents the progress made during this 7-month period, including progress on Tasks 1 though 7.

Although several of the tasks were moving as planned, the setting up of laboratory testing equipment was significantly delayed due to a multitude of factors. In particular, instrumentation for performing triaxial testing of soils and resilient modulus of hot-mix asphalt (HMA) was not set up and operational as anticipated.

1.3 Outline and Organization of the Report

The report is organized into seven chapters including this one, which introduces the objective and goal of the research project. The second chapter gives an overview of the experimental design that was conceived for this research study and the previous study. The third chapter highlights the testing of materials as well as in-field performance monitoring that was one of the main objectives of this study. The fourth chapter focuses on the new sections that were added as part of this database during the summer of 2009. Details related to their location, alignment, and design features in terms of geometry and structure are also included for each of these sections. The fifth chapter focuses on the review of project 0-5513 and further database development and necessary upgrades that were considered important for the successful implementation of the project. The sixth chapter presents the efforts in terms of calibration as an update of the calibration efforts initiated during 0-5513. Finally, the last chapter summarizes the recommendations for the research study.

Chapter 2. Experimental Design

2.1 Experimental Design

To optimize the use of the resources allocated to Research Project 0-5513, a reduced set of experimental variables (experimental design) were considered. The variables included:

- Pavement type (4 levels)
  - HMA surface on top of HMA base
  - HMA surface on top of untreated granular base
  - Two course surface treatment on top of untreated granular base
  - Flexible pavement containing lime- and cement-treated bases or subbases
- Traffic levels (3 levels)
  - Heavier traffic
  - Medium
  - Lighter traffic
- Environmental conditions (5 levels)
  - Wet No Freeze
  - Wet Freeze
  - Dry No Freeze
  - Dry Freeze
  - Mixed
- Aging (2 levels)
  - Two aging levels were considered (relatively new and relatively old pavements).
- Section replicates (3 levels)
  - Whenever available, replicates were included.

This experimental design consisted of 360 sections. However, due to budget constraints, it is believed that a partial factorial consisting of approximately 100 sections, if designed statistically, has the potential to deliver the information required at a reasonable cost. Preliminary calibration of the rutting transfer functions has demonstrated the high degree of variability of these sections (Banerjee et al., 2010); therefore, for making the experiment statistically valid, replicate sections were consider essential and should be incorporated when available. Thus, the new experimental variables to be included as part of 0-6275 are:

- Pavement type (3 levels)
  - HMA surface on top of HMA base

- HMA surface on top of untreated granular base (flexbase)
- Two course surface treatment on top of untreated granular base (flexbase)

- Traffic levels (2 levels)
  - Heavier traffic
  - Lighter traffic
- Environmental conditions (5 levels)
  - Wet No Freeze
  - Wet Freeze
  - Dry No Freeze
  - Dry Freeze
  - Mixed
- Age and Section replicates (2+1 levels)
  - The target was set for two new sections and one old section to capture the effect of age as well as the in-section variability.

Whenever replicate sections are available, there were going to be included. In most cases, this was achieved by selecting more than one experimental section in the same road project with a typical spacing between 1 or 2 miles so that both the test sections are exposed to identical conditions.

Thus, the complete main factorial consisted of 60 sections (3 x 2 x 5 x 2). In order to include a limited number of other variables, the originally proposed number of sections was 64. TxDOT research has demonstrated the importance of aging the HMA on flexible pavement performance, so the database has to include aged and un-aged sections. Therefore, a much larger number of sections were also considered to account for aging (old and new sections) and other variables not explicitly accounted for. It was decided that 67% of the total number of sections would be constituted of "new" pavement sections while the remaining 33% coming from the existing TFPD would be deemed the "old" sections. Thus, a mix of old and new sections will provide the opportunity to monitor performance trends against aging of asphalt mixes. For this reason the third replicate was added so as to result in 90 sections.

One of the main shortcomings, already identified during 0-5513, was the reduced number of sections in the wet-warm and dry-cold areas and the abundance of sections in Central Texas. Thus, the new, more balanced experimental design incorporated sections in Beaumont, Lubbock, and Wichita Falls, and other Districts that are closer to Austin (Central Texas). In addition, 10 sections outside the experimental design were also identified at the discretion of the Project Monitoring Committee (PMC) to include unconventional materials like warm-mix asphalt (WMA), crack attenuating mixes (CAM), and crumb rubber mixes into the performance monitoring as well as material testing program.

During this project (0-6275), the researchers started upgrading the 41 new TxDOT-recommended sections by collecting and testing site-specific materials. In addition, some 20 sections, on average, were to be added each year to reach the total of 100 new sections. The research team demonstrated a total of 73 test sections during the progress meeting held during September 2009, which consisted of:

- 41 TxFlex sections, which were part of the Research Project 0-5513 TFPD database.
- 32 TxFlex Miscellaneous sections that were also monitored as part of 0-5513 research study but were later on dropped from the field monitoring program.

These 73 test sections were considered the baseline for the selection of the 33% "old" pavement sections. The district-wise composition of these 73 test sections are as given here:

- Bryan: 11 TxFlex sections + 4 TxFlex Miscellaneous sections
- El Paso: 12 TxFlex sections + 14 TxFlex Miscellaneous sections
- Tyler: 6 TxFlex sections + 14 TxFlex Miscellaneous sections
- Waco: 12 TxFlex sections

These existing sections had a blend of the different facility types including Interstate Highways, US Highways, State Highways, and Farm to Market Roads. Thus, this partial experiment includes three different pavement structures, five different environments, and two levels of traffic that can be easily accommodated within the choice of sections that the research team had to work with. However, certain cells in the factorial were left empty due to absence of sections in Wet No Freeze and Dry Freeze climatic regions. Thus, it was agreed that a greater emphasis will be given in these geographical zones in the selection process for "new" pavement sections.

Chapter 3. Material Testing and Performance Monitoring

This chapter presents the testing of materials that was performed on samples collected from the various sections. It also includes the different types of distress measurements that were performed during the summer of 2009 on each of these test sections.

3.1 Performance Monitoring

Performance monitoring of the experimental sections included automated and manual distress surveys as well as deflection testing to determine the remaining life of the pavements using the TxDOT Falling Weight Deflectometer. The following section further illustrates the frequency and the level of effort involved during data collection. It should be noted that all data collected has been submitted to TxDOT's Research and Implementation (RTI) office as part of this project's deliverables.

3.1.1 Deflection Testing

The pavement sections contained in the TFPD are typically 500 feet in length. The length of these sections was determined based on the length of the LTPP database, a study that was initiated in 1987 as part of the SHRP. Deflection testing was completed at each of these test sections with stations spaced at 50-feet intervals, thus providing 11 stations in total over the length of the section. Deflections were recorded for three different load levels with full load history along the right wheel path of the lane. Figure 3.1 shows a typical data display over a half-mile section. It should be noted that, during the first survey, deflection testing was performed over 0.5-mile long sections. This information was then used to select a homogeneous location for the 500-foot database sections.

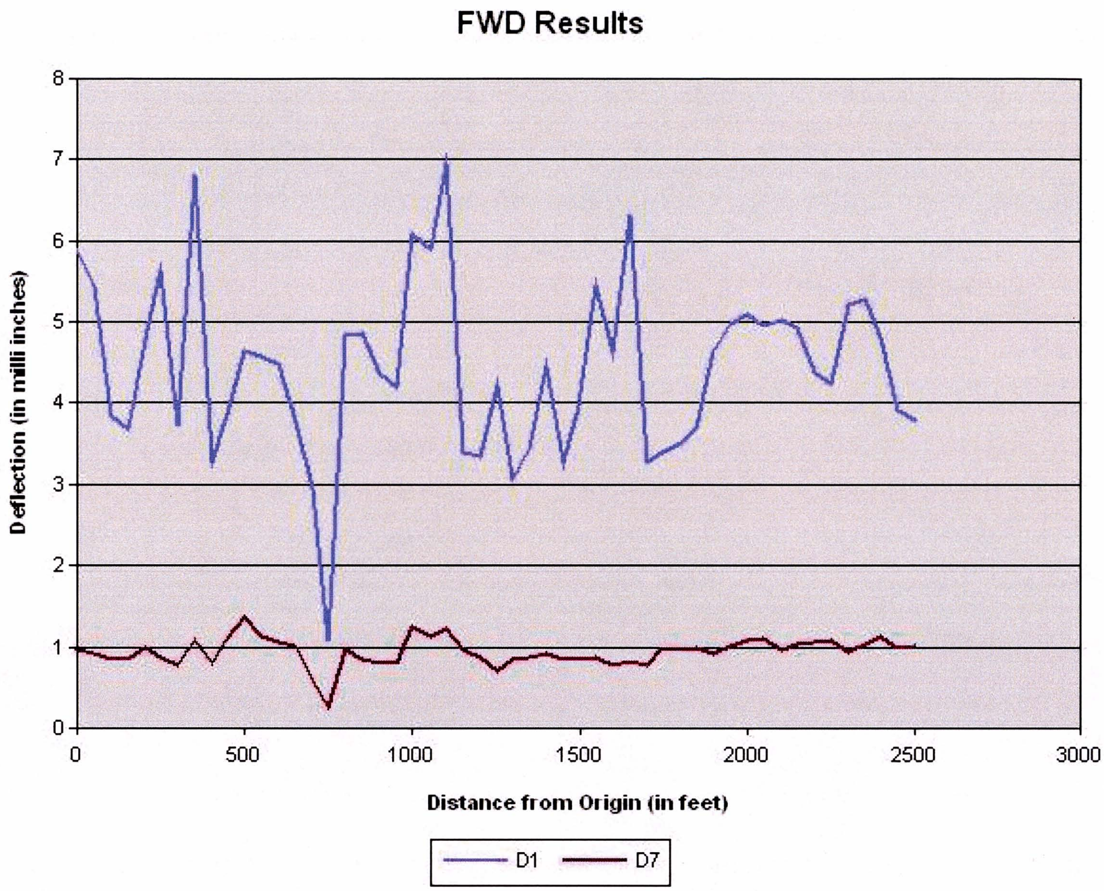


Figure 3.1: Deflection Measures on Sensors D1 and D7 Performed on TXTF17001 on 5/2/2007

3.1.2 Manual Distress Surveys

Manual rutting and cracking measurements were performed during the data collection cycle in the 2009 summer season. The manual rut depth measurement procedure included the recording of the vertical distance between the road surface and a 6-feet straight edge bar at 1-foot intervals using a triangular wedge with a precision of 1/16th of an inch, thus giving 10 surface rut depth measurements over the transverse profile of the road. This procedure was performed every 50 feet over the length of the test section. The advantage of this method lies in the fact that it will not only help the agency know the maximum rut depth over the length of the section but can also generate a three-dimensional transverse profile of the road for the entire length of the section. As far as the manual crack measurements are concerned, the length of the longitudinal cracks was recorded using a moving wheel that was guided by one of the personnel in the field. Manual counts of transverse cracks were also recorded during visual condition surveys. Figure 3.2 shows how rutting data can be displayed.

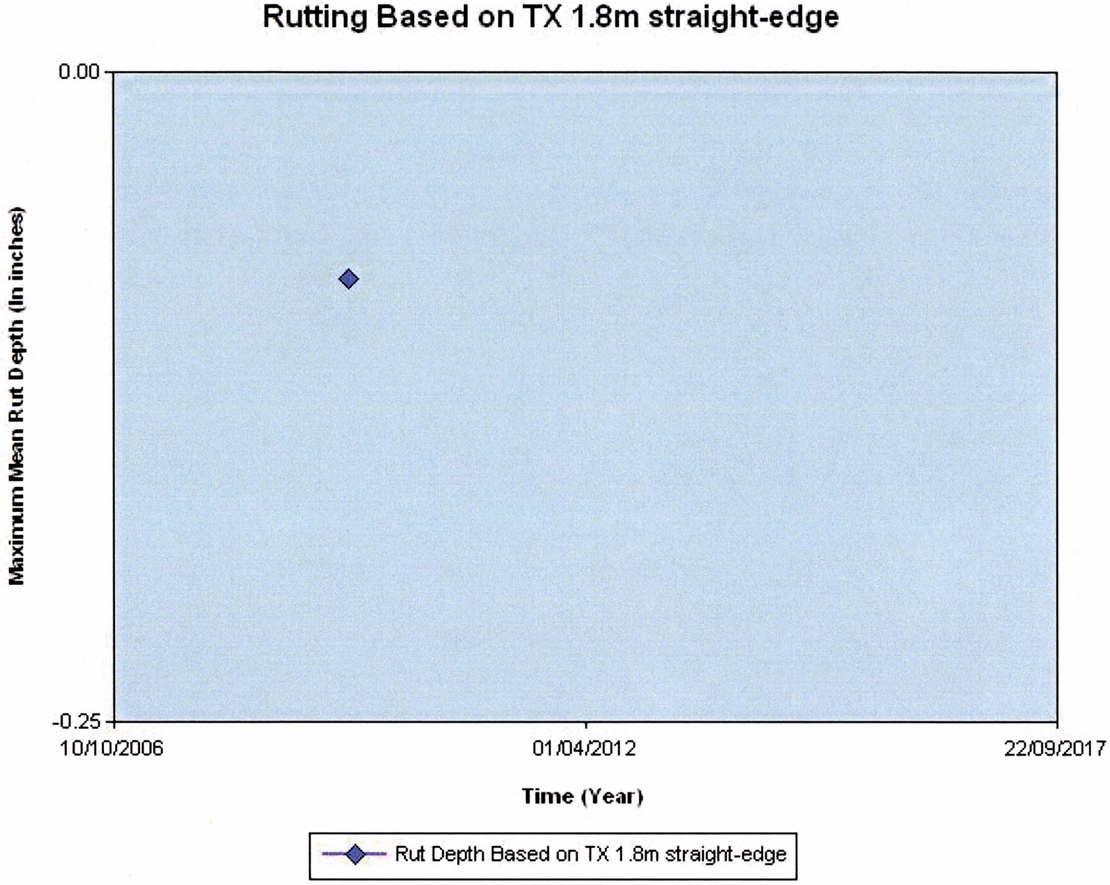


Figure 3.2: Manual Rut Measurement Using the 6-foot Straight Edge on TXTF17001

3.1.3 Automated Distress Surveys

Automated distress surveys using the TxDOT Profiler were performed at each of these experimental test sections. Although the system is capable of measuring the number and the length of cracks, rut depth, and the longitudinal profile of the road, the only parameter of interest to the research team was the roughness measurement in terms of the International Roughness Index (IRI). Figure 3.3 shows roughness (in IRI) measured at a given section in 2 consecutive years.

The research team also saved digitized images of the road surface using the VCRACK system should there be any need for post-processing of the files to develop a crack map for each of these test sections. Ultimately, automated distress should be incorporated into the database; however, to date the equipment available was not of the required accuracy for project level investigations.

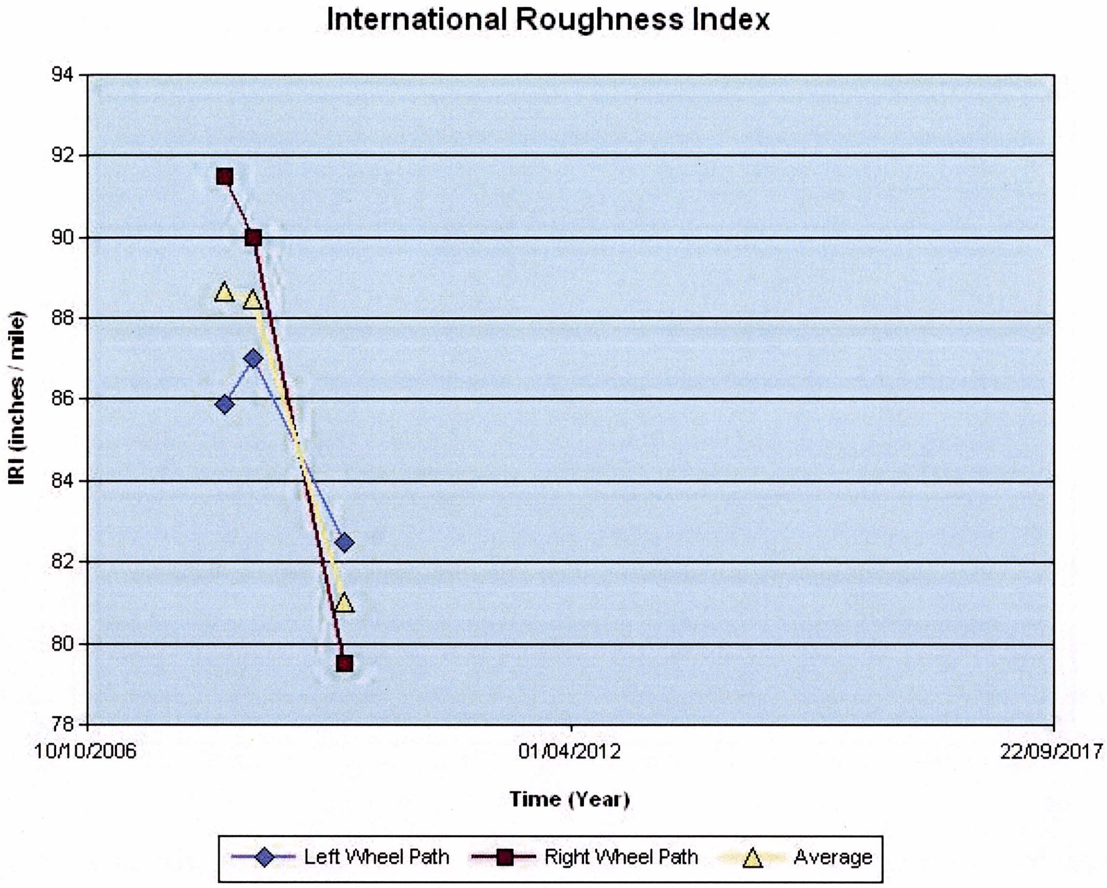


Figure 3.3: Roughness Measurement Using the TxDOT Inertial Profiler on TXTF17001

3.2 Material Testing

3.2.1 Hamburg Wheel Tracking

Wheel tracking tests are empirical tests that attempt to evaluate the mixture behavior with respect to permanent deformation under a rolling wheel. In the case of the Hamburg Wheel Tracking Device (HWTD), when it is performed in a water bath, an assessment of the moisture sensitivity of the mixture can also be obtained.

Distress: Permanent Deformation (Rutting)

HMA should be stable enough to minimize permanent deformation under repeated traffic loads. Permanent deformation occurs as a combination of two distinct mechanisms (see Figure 3.4): densification (volume change) and shear deformation (shape change) (SHRP, 1994).

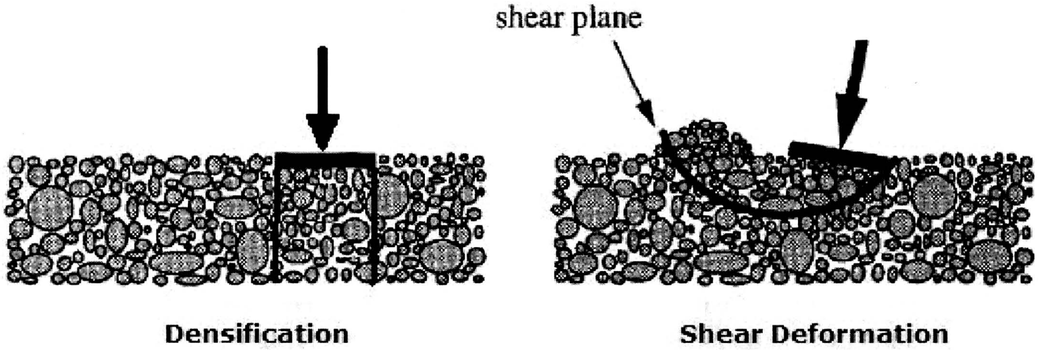


Figure 3.4: Permanent Deformation Mechanisms (McGennis et al., 1995)

Permanent deformation of the mixture (or rutting) is directly related to internal friction, provided by the aggregate, and to cohesion, provided by the asphalt binder. The primary factors affecting rutting are mix properties, temperature, number of load applications, load frequency, and state of stress. The critical condition for permanent deformation accumulation is found at elevated temperatures and lower load frequencies (i.e., slow moving traffic), given that these parameters decrease the viscosity of the binder and, consequently, the material deforms to the extent that traffic loads are carried predominantly by the aggregate structure (aggregate skeleton).

Rutting can be minimized by the selection of materials with appropriate characteristics, such as aggregate maximum size, shape, surface texture, and binder stiffness (e.g., asphalt binder grade); and by the selection of the adequate combination of materials in terms of gradation characteristics, asphalt content, and compaction effort. These factors influence the overall stiffness and the air-void content of the mixture. Permanent deformation is affected by shape, gradation, durability, and toughness of the aggregate (Dukatz, 1989).

Permanent deformation can be evaluated by using predictive models (performance models and transfer functions) along with laboratory tests to determine the model parameters influencing rutting. These models or methods can be empirical regression equations, typical plastic strain laws, and functional equations directly based on laboratory test results as those currently incorporated into the MEPDG. There are several laboratory tests that attempt to characterize this distress and transfer to field performance; for example: uniaxial and triaxial creep load tests (e.g., flow time), uniaxial and triaxial repeated load tests, dynamic tests (e.g., flow number), diametral tests, torsion shear tests, simple shear tests, wheel-tracking tests, accelerated pavement tests (APT), etc.

Distress: Resistance to moisture induced damage (stripping)

Moisture has a direct impact on the loss of adhesion between the asphalt binder and the aggregate as a result of stripping, and may accelerate the effect of other distresses such as permanent deformation and fatigue cracking. The negative effects of water can be controlled by making the mix as impermeable as possible, and by ensuring the appropriate adhesion between the aggregate surface and asphalt binder. This characteristic is an inherent property of the aggregate and the binder surface energy, and the affinity between both components can be described by the laws of thermodynamics.

Moisture damage can be evaluated qualitatively by the use of laboratory tests—wheel-tracking tests (including but not limited to HWTD), boiling water test, ultrasonic method, and chemical immersion test—or by comparison of tests performed on dry and moisture conditioned specimens by indirect tensile strength (Tensile Strength Ratio [TSR]), and static and dynamic creep tests. Moisture damage appears to be more affected by surface chemistry (composition, solubility, and surface charge) than aggregate shape (Dukatz, 1989).

Laboratory wheel-tracking devices are intended to determine a set of parameters in the asphalt mixture by repeatedly rolling a reduced-scale loaded wheel device across a prepared specimen. The results of the test could be somehow correlated to actual in-service material performance in order to try to predict rutting, fatigue, moisture susceptibility, and stripping distresses. A more practical application is to compare two or more mixture designs to determine the one that will present the better characteristics to rutting and moisture-susceptibility.

Test Description: Hamburg Wheel-Tracking Device

As it is used in Texas, the HWTD measures the combined effects of rutting and moisture damage, related to stripping potential, on HMA mixtures. The apparatus is shown in Figure 3.5.

The test's principle consists of rolling a normalized steel wheel (8-inch diameter, 1.85-inch width, and 154-pound weight) across the surface of a specimen immersed in hot water (122 ± 2°F or 50±1°C) for a maximum of 20,000 cycles (or when specific deformation depending on the binder grade occurs). The tests are typically conducted on slabs or compacted samples with the Superpave Gyratory Compactor (SGC) to 7% air voids. The average rut depth is measured and registered by the mid and two adjacent linear variable differential transformer (LVDT) sensors. The test is performed in accordance with TxDOT standard procedure Tex-242-F: "Hamburg Wheel Tracking Test."

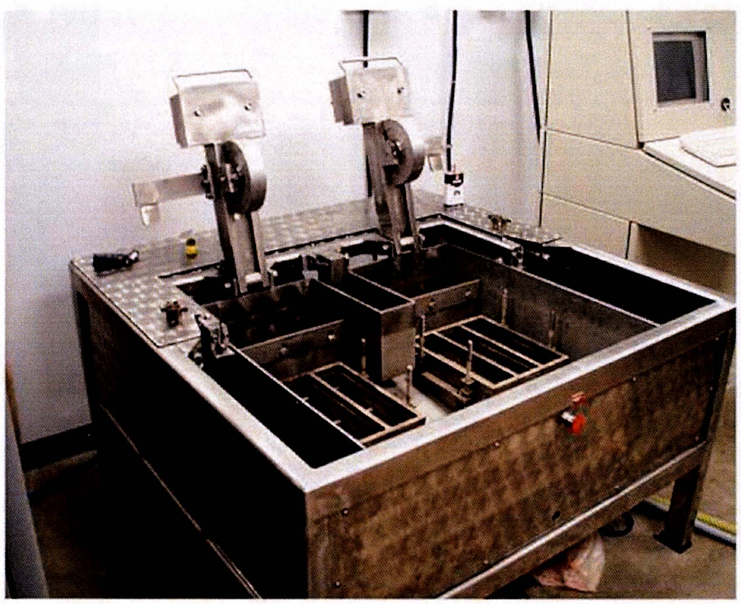


Figure 3.5: Hamburg Wheel-Tracking Device.

3.2.2 TxDOT Overlay Tester

The overlay tester (OT) is a performance-related test designed in the late 1970s to simulate the reflective cracking process of HMA placed over jointed concrete pavements (Zhou et al. 2007). Originally, the test attempted to replicate the effects of the opening and closing of concrete joints, which were considered the main driving force inducing reflective crack initiation and propagation.

The apparatus consists of two steel plates, one fixed and the other movable horizontally, to simulate the opening and closing of joints or cracks in the old pavements beneath an HMA overlay (Figure 3.6). The specimens can be trimmed from 150-mm-diameter (5.9 inch) standard field cores or Superpave Gyratory Compactor (SGC) specimens.

The test is performed in accordance with TxDOT standard procedure Tex–248–F: "Overlay Test." According to the specification, the test is performed at a constant temperature of 77 ±3°F (25 ±2°C). The sliding block applies the necessary tension in a 10-second length cyclic triangular waveform to achieve a constant maximum displacement of 0.025 in. (0.06 cm), as shown in Figure 3.7. The test measures the number of cycles to failure, defined as the point where the pulling force drops 93% of the initial value.

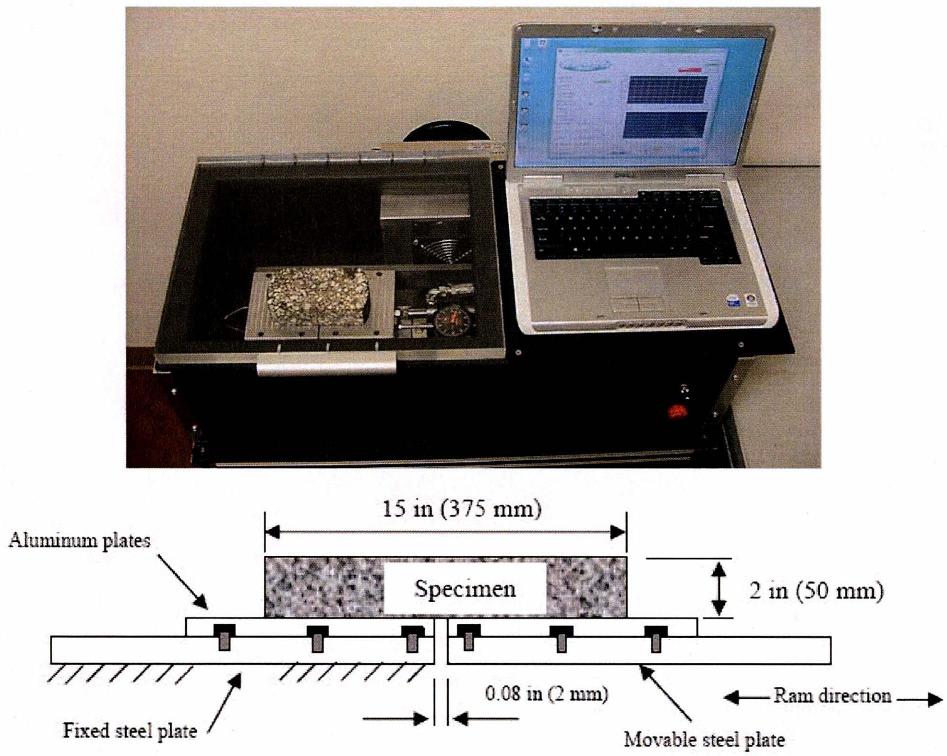


Figure 3.6: OT Apparatus

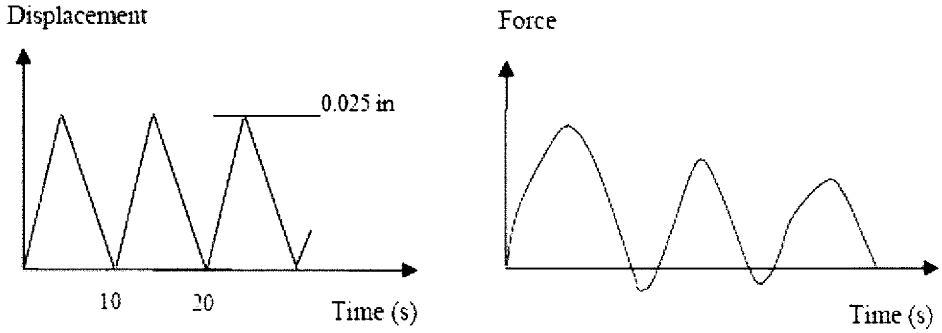


Figure 3.7: OT Load Scheme

In the study performed by Zhou et al. (2007) to determine the sensitivity of the OT to capture variations in HMA properties, it was found that the test was able to detect the effects of temperature, aggregate type (related to absorption), asphalt content, and air-void content. The higher the asphalt content, the better the reflective cracking resistance of the asphalt mixture.

For comparative purposes between OT and bending fatigue testing (BFT), it should be noted that the OT was developed to assess reflective cracking resistance; consequently, higher strain levels are applied to the specimen. Another significant difference is the loading time. The loading time in the OT is 100 times longer than the BFT and is not quite consistent with highway speed. Frequencies in the order of 10 Hz are typically used to represent highway speed. The parameters for the BFT are selected to assess fatigue cracking as opposed to reflective cracking. Given that the asphalt mixtures can be considered a visco-elasto-plastic material, the results of the OT and the Beam Fatigue Tests should not be expected to be similar. Each piece of equipment has its area of application, but both test types may show similar trends for fatigue cracking potential.

3.2.3 Asphalt Resilient Modulus

As part of the project, testing to determine the resilient modulus of HMA material was performed. The testing was performed following the ASTM Designation D 7369—09 specifications (Resilient Modulus by Dynamic Indirect Tension Test), which were developed as a result of NCHRP 1-28A. The previous specification replaced the previous AASHTO (American Association of State Highway and Transportation Officials) specification for determination of resilient modulus in HMA mixes (AASHTO T 322 – 07).

The testing consists of applying a diametral haversine load ($P_{max} - P_{contact}$) at 1 Hz and 25°C/77°F (0.1 s loading and 0.9 s resting). The previous low and high temperature repetitions are optional. The principles behind the test are summarized in Figure 3.8. After the loading pulse is applied, the sample is subjected to a given amount of deformation, part of which is instantaneous and recoverable (component used to estimate the resilient modulus), and a given percentage that is permanent.

Therefore, estimation of the resilient modulus requires quantifying the instantaneous deformation on each loading cycle. Based on the methodology, the instantaneous deformation is defined as the difference between the displacement peak and the intercept between a line extending from the straight portion of the unloading path and a line tangent to the deformation at 55% of the rest period (Figure 3.8). Then, based on the horizontal and vertical instantaneous deformation, the Poisson's modulus (μ) and the resilient modulus (M_R) can be estimated as per Figure 3.9. The $I_1 - I_4$ are dimensionless factors related to the state of stresses and to the location of the LVDTs in the face of the specimen.

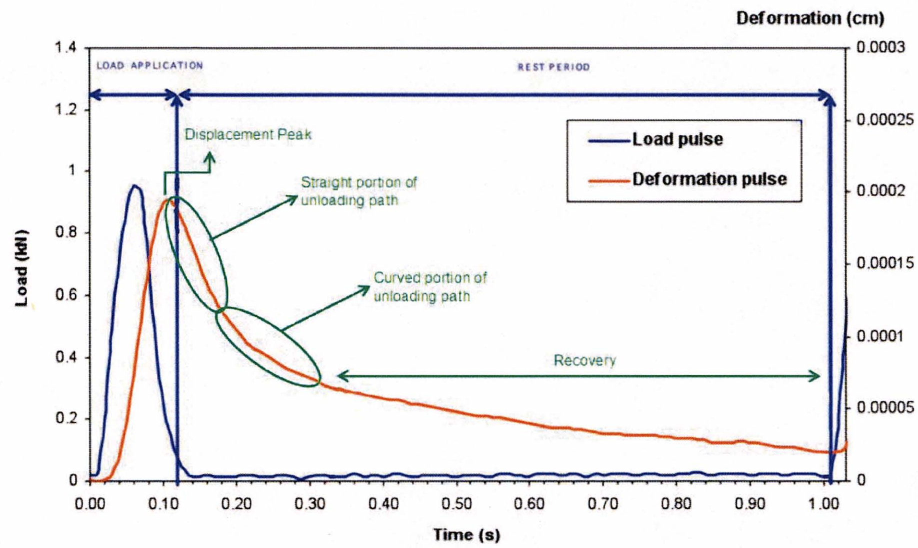


Figure 3.8: Displacement of the HMA Sample at Different Stages of the Loading Cycle

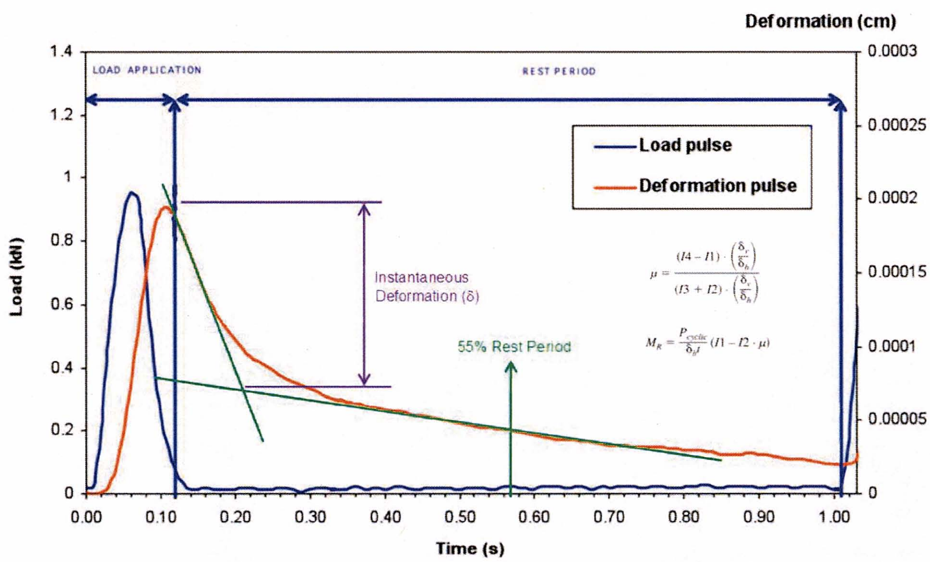


Figure 3.9: Estimation of the Resilient Modulus Based on the Instantaneous Deformation

The purpose of the test was to determine the resilient modulus at 0.5 Hz, 1 Hz, 2 Hz and 5 °C, 15 °C, and 25 °C (41 °F, 59 °F, and 77 °F), and evaluate the possibility of developing master curves based on these data. The LVDT configuration on the face of the specimen and the testing apparatus are shown in Figures 3.10 and 3.11, respectively.

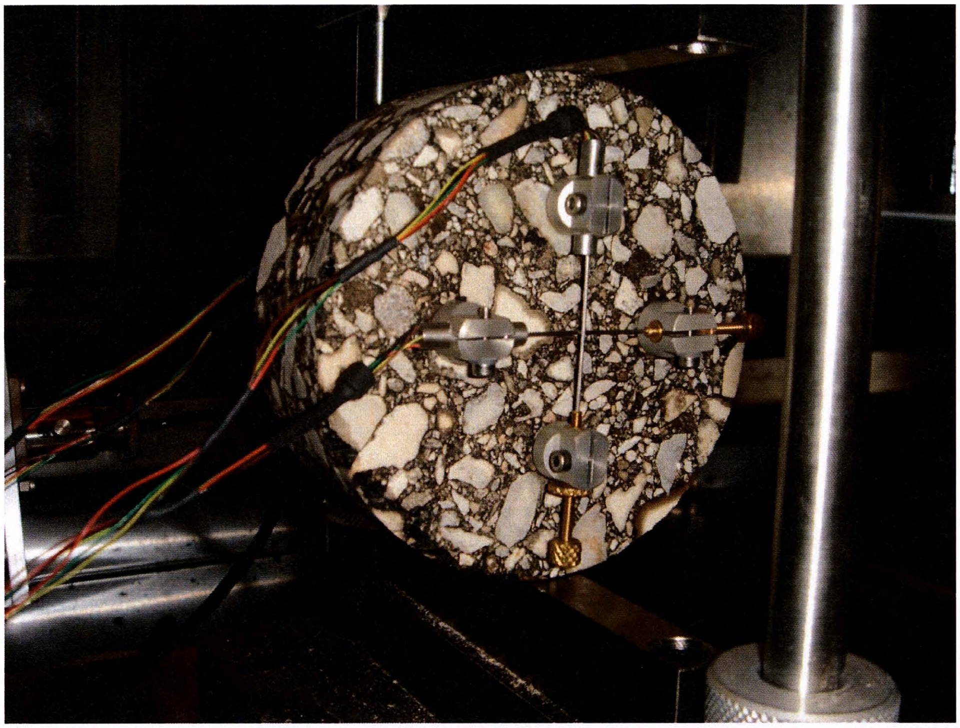
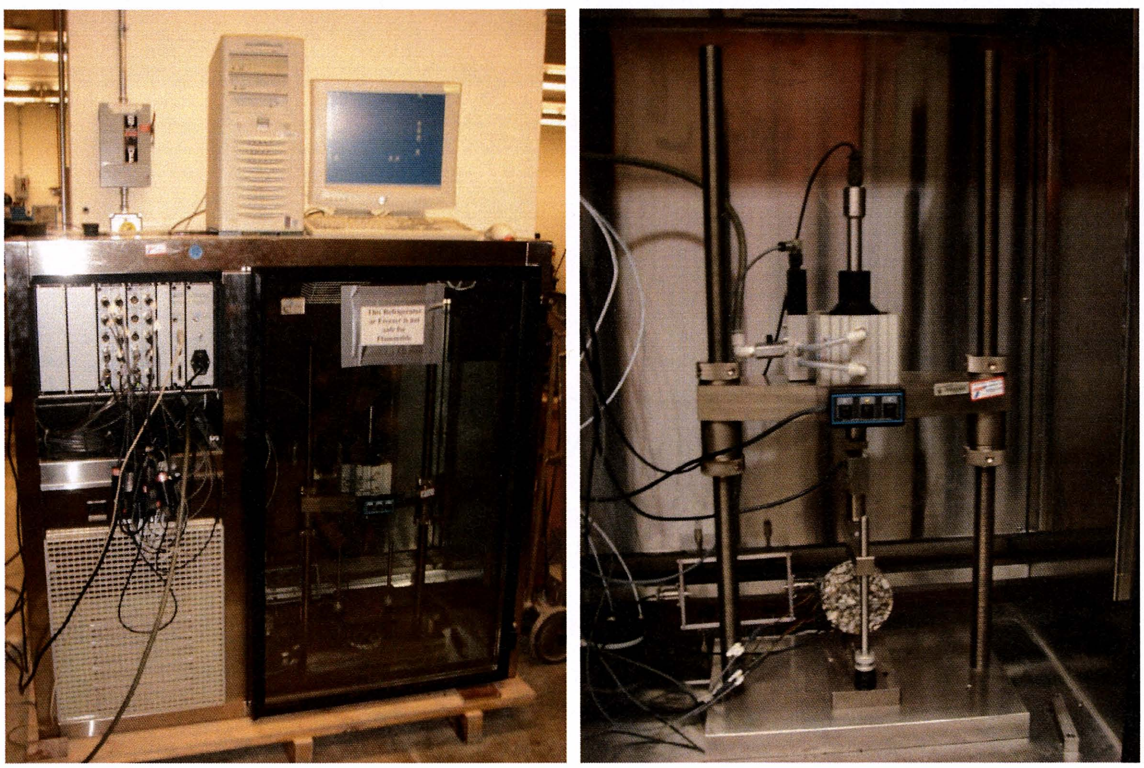


Figure 3.10: LVDT Setup Used in Resilient Modulus Testing



(a) (b)

Figure 3.11: (a) Environmental Chamber and (b) Loading Frame Used in Resilient Modulus Testing

Chapter 4. Experimental Sections

This chapter gives a brief summary of the newly added pavement sections included in the TFPD as part of the 0-6275 research project. The summary of the section includes the district, county, location coordinates, and structure of the pavement section. For all other information on any specific pavement section, please refer to the web interface available at the following location: http://pavements.ce.utexas.edu/. The detailed information is also available as part as Product 1 of this project (0-6275-P1).

4.1 Experimental Sections

4.1.1 US90, Hondo, Texas

- County: Medina
- Functional Class: US
- Route Number: 90
- Starting Point Latitude: 29.34604°
- Starting Point Longitude: -99.15696°
- Ending Point Latitude: 29.34626°
- Ending Point Longitude: -99.15541°
- Climatic Region: Dry No Freeze

Asphalt Layer: 2 inches thick (Dense Graded Type C)
Asphalt Layer: 2 inches thick (Dense Graded Type C)
One Course Surface Treatment
Base Course: 10 inch thick (Type B)
Natural Subgrade

4.1.2 US79, Taylor, Texas

- County: Williamson
- Functional Class: US
- Route Number: 79
- Starting Point Latitude: 30.57472°
- Starting Point Longitude: -97.37532°
- Ending Point Latitude: 30.57527°
- Ending Point Longitude: -97.37386°
- Climatic Region: Mixed

Asphalt Layer: 2.5 inches thick (SMA Type C)
Asphalt Layer: 2 inches thick (Dense Graded Type C)
Asphalt Layer: 3.5 inches thick (Dense Graded Type B)
One Course Surface Treatment (Prime Coat MC30 or AEP)
Base Course: 16 inch thick (Flex Base Type A Grade 4)
Treated Subgrade: 8 inch thick (Lime Treated)
Natural Subgrade

4.1.3 IH37, San Antonio, Texas

- County: Bexar
- Functional Class: IH
- Route Number: 37
- Starting Point Latitude, deg: 29.38164°
- Starting Point Longitude, deg: -98.46643°
- Ending Point Latitude, deg: 29.38066°
- Ending Point Longitude, deg: -98.46529°
- Climatic Region: Dry No Freeze

Asphalt Layer: 2 inches thick (Dense Graded Type C + Evotherm/Sasobit/Control) Asphalt Layer: 1.5 inches thick (Leveling Course Type D)
Base Course: 16 inch thick
Subbase Course: 8 inch thick
Treated Subgrade: 6 inch thick
Natural Subgrade

4.1.4 SH71, Spicewood, Texas

- Length of Roadway: 2.65 miles, 4 lane divided highway
- Full Depth Rehabilitation
- Uniform Traffic Flow

Porous Friction Course
One Course Surface Treatment (Seal Coat)
Asphalt Layer: 2 inches thick (Dense Graded Type C)
Base Course: 6 inch thick (Type A) Base Course: 8 inch thick (Crushed Stone)
Natural Subgrade

4.1.5 Existing Sections

In addition to the new sections, 22 pavement sections proposed as part of the 0-5513 research study were also brought forward and included in the list of sections proposed as part of the 0-6275 study. These 22 sections are well distributed within Texas, with six in El Paso, six more in Bryan, two in Tyler and the remaining eight in Waco. For details on the sections, please see the online version of the database (http://pavements.ce.utexas.edu) or the PC version, which constitutes a research product (0-6275-P1).

These sections not only capture the inherent variability in terms of climatic conditions in these geographical areas, but also capture the effect of traffic on pavement performance. These existing sections are exposed to different levels of traffic volumes and therefore the performance trend of these sections can be used to determine the effect of traffic volume and axle loads on pavement performance. In addition, the fact that these sections are relatively older can also help in capturing the effect of aging on pavement distresses, as these sections have been monitored since 2008. The sections also provide the added advantage of a higher number of in-field distress measurements when compared with the relatively newer sections, which is particularly beneficial when these sections are used for calibration of mechanistic empirical design procedures.

Chapter 5. Database Updates

5.1 Database Updates

The TFPD, in its current form, is home to nearly 200 pavement sections from throughout Texas, each with varying amounts of data. It is primarily organized into four different sections, namely (i) pavement performance, (ii) traffic, (iii) pavement structure, and (iv) material and inventory information. In addition, the website also has an administration feature built into it with limited capabilities like managing user-roles as well as approving or discarding pavement sections from the TFPD. A high-level conceptual model of the system is given in Figure 5.1. This conceptual model was the backbone behind developing the different system components.

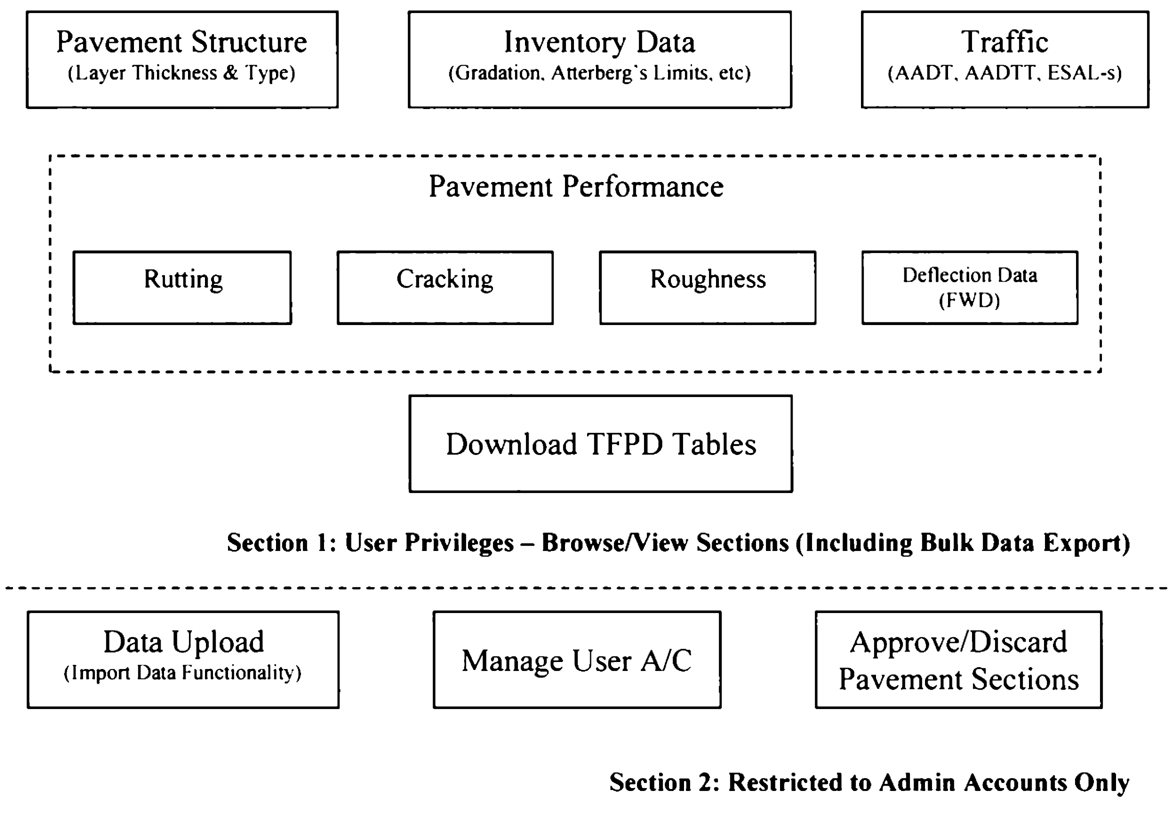


Figure 5.1: High Level Conceptual Data Model for the Texas Flexible Pavements Database

The database architectures of existing pavement-related databases were reviewed as references in the early stages when the database structure was being developed. The databases that were reviewed included the TxDOT Pavement Management Information Systems (PMIS) and the FHWA's LTPP database. The database design finalized for the TFPD was based on this review; most of it has been adapted from the LTPP database. The decision on the list of database fields to include was made after carefully studying the list of inputs essential to run a calibration with Level 1 data for each of the four modules (Traffic, Structure, Materials, and Climate) given in the MEPDG (Version 1.0). Comments and recommendations received at several meetings with several TxDOT personnel from the Information Systems Division (ISD) and the Construction Division (CST) were considered in defining the database design.

5.1 QC/QA Checks and Database Population

The TFPD, in its current form, is home to nearly 200 pavement sections from throughout Texas. Information related to pavement distresses, structure, traffic, and materials are stored in the database with the purpose of supporting the data needs for calibrating any mechanistic empirical design procedures with site-specific data. However, it should be noted that is of utmost importance that this significant volume of data is true to the best of knowledge. Therefore, detailed quality control and quality assurance (QC/QA) checks are imposed on the data being uploaded. To this effect, sufficient numbers of QC checks in the form of valid values and range checks have been imposed on the spreadsheet being used for uploading new data (http://pavements.ce.utexas.edu). On the database side, certain restrictions in terms of accepted data types have been also imposed—for example, a numeric field does not accept textual data or vice versa. Constraints like auto increment fields on the database tables are also in effect in order to differentiate between records where a set of unique keys serve as the primary key for the record. In addition to these checks, QA tests are performed from time to time by the research team on data that is uploaded in bulk in to the database, especially on deflection data.

Chapter 6. Calibration Updates

6.1 Introduction

During Project 0-5513, initial calibration of the permanent deformation models was performed. This calibration focused on new designs (Banerjee et al., 2010). During this study, the calibration of the deformation models of rehabilitated pavements was investigated to determine differences with the previously reported work.

Routine maintenance can extend a pavement's life and significantly delay the requirement of major rehabilitation work. This type of work does not increase the structural capacity of the pavement. At some stage, heavy rehabilitation or full 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) prioritizing 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.

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 to 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. Texas has been one of the champion states in terms of preventive maintenance with the implementation a very extensive and well established Seal Coat Program.

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 pavement preservation treatments include chip seals, slurry seals, fog seals, thin asphalt overlays, microsurfacing, and crack sealing.

During Research Project 0-5513 and due to the lack of times-series data for Texas's pavements, the performance models used in the MEPDG, developed under the NCHRP (NCHRP 1-37A and 1-40D), were calibrated using sections throughout the United States, including pavement sections from Texas. It should be noted that, in this context, there is no overlap between the non-LTPP datasets used for this study and those used for the national calibration of the MEPDG (ARA, 2004).

During this project, the researchers focused on analyzing 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 first step involved the determination of Level 1 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. Following are the definitions of each of the design levels as interpreted by the authors.

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 it is most suited for predicting pavement distresses at a state level.

Because the focus of this study was on analyzing the calibration coefficients for a given section subjected to a particular type of treatment, Level 1 bias correction factors were considered to be the most appropriate choice.

The calibration performed during this task of the project was performed with data that are not necessarily part of the TFPD but were intended to determine significant variables that affect the calibration process. The calibration methodology, however, is the same no matter whether the data is from Texas or not. As during Research Project 0-5513, the researchers started by calibrating the permanent deformation models of the MEPDG with data from LTPP.

6.2 Adopted Methodology

The empirical performance model or transfer function for the permanent deformation in the asphalt layer, as shown in the MEPDG, is:

AC Rutting Transfer Function

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r_1} 10^{k_1} T^{k_2 \beta_{r_2}} N^{k_3 \beta_{r_3}}$$

$$\log \varepsilon_p - \log \varepsilon_r = \log k_z + \log \beta_{r_1} + (\ldots) + k_3 \beta_{r_3} \log N$$

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

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

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

where,

H_{ac} = total AC thickness (in.)

ε_p = plastic strain (in/in)

ε_r = resilient strain (in/in)

T = layer temperature

N = number of load repetitions

k_z, k_2, k_3 = laboratory constants

$\beta_{r1}, \beta_{r2}, \beta_{r3}$ = calibration coefficients

It can be seen from the logarithmic transformation of the rutting transfer function, that log (β_{r1}) serves as an intercept of the transfer function, while β_{r3} represents the slope of the transfer function. In other words, the bias correction factor β_{r1} can be interpreted as the propensity of the pavement towards rutting during the early stages of its service life. On the other hand, β_{r3} could be interpreted as the rate at which the rut depth develops with time. The logarithmic transformation of the transfer function also indicates that the predicted rut depth at any time point bears a direct a relationship to the value of the bias correction factors. Thus, a lower value for the product of the bias correction factors implies that the pavement is less likely to rut than other pavement structures under identical traffic and climatic conditions because it captures both the intercept and the slope of the transfer function.

Once the calibration factors were determined, the second step focused on analyzing the effect of various pavement preservation strategies on the bias correction factors. The goal was to establish whether different calibration coefficients should be used for different pavement preservation strategies. The bias correction factors, as estimated, were analyzed to determine which of the experimental variables have a significant influence on the values of the bias correction factors. It should be taken into consideration that because the dataset contained pavement sections from a variety of climatic zones, the influence of pavement preservation options were also studied under different weather patterns.

6.3 Data Used

This task of the research project addressed two related aspects: one that looks into the influence of maintenance and rehabilitation strategies on bias correction factors and the other one that looks into the effect of pavement preservation techniques on the bias correction factors. Therefore, it is necessary to look for two different datasets. The LTPP Specific Pavement Studies-5 (SPS-5) experiment caters to the first aspect, while the SPS-3 experiment data caters to the second aspect.

6.3.1 LTPP SPS-5 Experiments

The primary objective of the SPS-5 experiment (rehabilitation of flexible pavements) was to identify the relative influence and long-term effectiveness of factors that influence the performance of overlaid flexible pavements. The SPS-5 study includes three key variables: (i) type of material used for the overlay (virgin asphalt mix versus recycled asphalt pavement (RAP) in the mix), (ii) thickness of the overlay (thin versus thick), and (iii) type of surface preparation (milling versus no milling) undertaken before overlaying the pavement section. Because preliminary analysis results have shown that pavement performance is influenced by surface preparation, it can be expected that the bias correction factors should be significantly different for test sections with intense surface preparation than those with minimum surface preparation. Thus, the relative values of the bias correction factors provide grounds for establishing the influence of experimental variables on the bias correction factors. To compare these factors with those determined during 0-5513, the researchers followed the next steps:

1) Comparison and development of empirical prediction models for performance of HMA pavements with different intensities of surface preparation, with thin and thick HMA overlays and with virgin and RAP materials.
2) Evaluation and field verification of the MEPDG design procedures for rehabilitation of existing HMA pavements.
3) Determination of appropriate timing for rehabilitation of HMA pavements.
4) Development of procedures to verify and update the pavement management and life cycle cost concepts in the MEPDG for rehabilitated HMA pavements.

The experimental design of the SPS-5 experiments included the following variables:

1) Effects of climatic factors (Dry versus Wet, Freeze versus No-freeze)
2) Pavement Condition (Fair versus Poor)
3) Amount and type of surface preparation before overlaying: Minimum (i.e., sections where only patching was done prior to application of overlay) versus Intensive (i.e., sections where 2 inches (51 mm) of the existing pavement was milled off)
4) Type of material used for overlay: Virgin versus Recycled
5) Variations in overlay thickness: 2 inches (51 mm) versus 5 inches (127 mm)

The Texas SPS-5 experimental sections correspond to the Wet No-freeze climatic region with fair pavement condition rating. Each of the test sections in the SPS-5 experiments is designated with a specific number that identifies some of its specific properties. The details are given here:

1) 501: Control, no treatment
2) 502: Thin overlay (2 in.), recycled HMA mix
3) 503: Thick overlay (5 in.), recycled HMA mix
4) 504: Thick overlay, virgin mix
5) 505: Thin overlay, virgin mix
6) 506: Thin overlay, virgin mix, with milling
7) 507: Thick overlay, virgin mix, with milling
8) 508: Thick overlay, recycled mix, with milling
9) 509: Thin overlay, recycled mix, with milling

6.3.2 LTPP SPS-3 Experiments

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, the dataset also includes test sections from four different geographical locations in Texas (part of 0-5513), 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 6.1.

Table 6.1: 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 during the calibration. 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 concluded that the effect of higher traffic is negated by the increasing thickness of the pavement structure, giving a zero net effect on the values of the bias correction factors.

6.4 Previous Analyses of SPS-3 and SPS-5

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 more 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 has shown that the use of 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 HMA 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 HMA 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.

The LTPP SPS-3 studies focused on a total of 81 locations comprising 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. Once enough time-series information is available in the TFPD, this exercise should be repeated.

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.

6.5 Our Results

The Level 1 bias correction factors as obtained for each of the SPS-5 and SPS-3 test locations are given in Table 6.2. Looking at the results from the SPS-5 test sites reveals a definite trend in the data. Test sites that were constructed with RAP 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 to mean that sections 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 that RAP mixes are relatively more rut resistant than virgin mixes.

Table 6.2: Level 1 Bias Correction Factors for SPS-3 and SPS-5 Test Locations

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

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.

6.5.1 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 β1. 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 6.3.

Table 6.3: Influence of the Experimental Variables on β3

	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 β3, 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). Because there was significant evidence that the bias correction factor β3 was influenced by the type of mix, a decision was made in regressing β3 against the "type of mix – recycled/virgin" variable. The results are summarized in Table 6.4.

Table 6.4: Influence of Use of Recycled Asphalt Mix on β3

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

The results given indicate that the use of recycled asphalt definitely influences the value of β3. 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 seen during this study that the two bias correction factors are correlated and both of them share a direct proportionality with the predicted rut depth at any given point in time. 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 6.5.

Table 6.5: Influence of the Experimental Variables on the Product of β1 and β3

	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 bear enough evidence to conclude that for the dataset evaluated, apart from the use of 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 given in Table 6.2 suggest that the product of the two bias correction factors for section 48-A502, in spite its 35% RAP mix, is 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 inches whereas all the other RAP sections had an overlay thickness in excess of 4 inches.

It has already been stated that β1 and β3 are correlated and therefore estimating either of the bias correction factors separately is not the most appropriate course. 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 conducted using SAS® (originally Statistical Analysis System) and the results are given in Table 6.6.

Table 6.6: Parameter Estimates for β1 and β3

Parameter Estimates for β1			
	Coefficient	**t-stat**	**p-value**
Intercept	102	6.6	0.007
β3	-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 β3			
	Coefficient	**t-stat**	**p-value**
Intercept	0.719	9.5	0.003
β1	-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 indicate that none of the experimental variables have a statistically significant influence on β1, while the use of recycled asphalt may have a significant effect on the value of β3, 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 β1. Nonetheless, a test of hypothesis with all the independent variables showed that they together have a significant effect ($p\text{-value} < 0.05$) on the β1 and β3. Further investigation reinforced this observation and the results so obtained are given in Table 6.7.

Table 6.7: Parameter Estimates for β1 and β3

Parameter Estimates for β1			
	Coefficient	**t-stat**	**p-value**
Intercept	102	6.6	0.007
β3	-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 β3			
	Coefficient	**t-stat**	**p-value**
Intercept	0.764	13.9	0.000
β1	-0.00600	-5.9	0.002
Recycle Asphalt Mix	-0.100	-2.1	0.086

The results in Table 6.7 show that the use of recycled asphalt in the mix has an effect on the bias correction factor β3. 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.

6.5.2 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 β1. 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 6.8.

Table 6.8: 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 contrary to what could be expected. However, mixes in warmer areas are designed with higher viscosity binders as opposed to the softer ones 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. Thus, it was decided to model β1 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 6.9.

Table 6.9: Influence of Thin Overlay as a Treatment Type on β1

	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 β1 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 β1, have 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 the treatment types nor the climate type had any impact on β3, so no specific calibration is required in this case.

The results were also analyzed to see whether there was any influence of the treatment types on the product of the bias correction factors in order to identify which treatment types are 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 β1. At the same time, warm weather was also found to produce a similar result, probably due to the use of harder binders. Preliminary results suggested that seal coats do not have an effect on the product of β1 and β3. The results of the analysis as obtained with linear regression are given in Table 6.10.

Table 6.10: Influence of the Treatment Type and Climate on the Product of β1 and β3

	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 to determine which of the experimental variables has a significant effect on the bias correction factors β1 and β3. 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 6.11.

Table 6.11: Parameter Estimates for β1 and β3

Parameter Estimates for β1			
	Coefficient	**t-stat**	**p-value**
Intercept	46.2	5.5	0.000
β3	-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 β3			
Intercept	0.550	1.9	0.089
β1	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 β3. Also, seal coats as a treatment type were found to have no impact on the values of the bias correction factors.

6.6 Preliminary Calibration Findings

The evaluation of how various rehabilitation and maintenance strategies influence the bias correction factors led the researchers to some interesting conclusions. 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. In summary, for calibrating the MEPDG to Texas's conditions (or any other mechanistic-based design procedure), one set of calibration factors have to be developed for new design and a different set for rehabilitated pavements. Until new data becomes available, the factors estimated as part of Research Project 0-5513 should be used for new pavements while the calibration factors for rehabilitated pavements should be determined as a function of the existing conditions and the maintenance/rehabilitation strategy. Some specific findings follow.

Firstly, it was observed that using a mix containing RAP is more likely to retard the rate of rutting development, which testing confirmed. 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 $\beta 1$. 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.

Chapter 7. Recommendations

7.1 Recommendations

This report and the corresponding database (http://pavements.ce.utexas.edu/) represent the end product of a 7-month effort for the continuation of the TFPD. It constitutes the platform for what could become one of the most important sources of data for the purposes of validating and calibrating data-intensive pavement design models—undoubtedly, the most important in the state of Texas.

Based on the results obtained from previous studies, it is strongly recommended that TxDOT continue to monitor the sections contained in the TFPD on an annual basis for at least the next 10 years. The performance monitoring should consist of collecting roughness (in IRI) automatically and rutting and cracking data manually as part of the visual surveys. Once accurate automatic equipment becomes available, rutting and cracking should be collected by means of automated systems to avoid human interpretation and subjectivity.

In addition, as new materials become popular in the state of Texas (such as WMA and the increased use of RAP), the original experimental design should be periodically reviewed and new sections should be incorporated accordingly.

One of the shortcomings of the current TFPD is the lack of a representative number of sections in North Texas (Panhandle region) and in East Texas. The continuation of the database effort should rectify this issue. As the database use increases, more extensive and intensive interaction and cooperation with the Districts is expected.

As suggested in the final report for 0-5513, the calibration methodologies that were developed should be carried forward and applied to new data as they become available. This will increase the confidence in the results and will produce more reliable calibration factors and, in general, more robust pavement performance models. Additionally, 0-6275 demonstrated that the transfer function for new design and rehabilitated pavements should be calibrated separately and independently.

At the same time, a few more enhancements should also be developed on the database side. Enhancements could include site-specific data on traffic and materials, an interface for users to run user-specified queries, a feature that allows users to download raw data, and other features that could facilitate the implementation of mechanistic-empirical design principles in Texas.

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