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16. Abstract

In recent years, a number of databases have been developed as part of TxDOT's Research Program for a diversity of goals and objectives. However, there is no one database that collects and stores lessons learned from long-term field application of different technologies or methods—in particular, after the original research project has concluded. Establishing a database that can hold information from research and construction projects provides TxDOT personnel, practitioners, and researchers with a tool to reference information from past studies to supplement data and avoid duplication of efforts. A database of this type is useful not only for tracking the performance of different types of materials or construction techniques, but also as a warehouse to archive case studies and observations about different projects that can be accessed by TxDOT engineers and others. This access is fundamental for ensuring that the experience and knowledge gained by the TxDOT personnel through the years by developing and implementing research technologies is preserved and the mistakes from the past are not repeated. Besides, the long-term monitoring and documenting of the performance of the various projects could provide valuable information in terms of the effectiveness and the efficiency of the various technologies and products developed during the original study.

Based on the analysis of the advantages and limitations of existing database systems developed previously under TxDOT's Research Program, this project created a wiki-type online database system to house all of the required information and meet the project's requirements and goals. The availability of wiki tools allowed for the development of an online database system that can store all of the required information and be easily accessed and modified by users to accommodate changing requirements and needs. A wiki consists of a collection of web pages designed to enable anyone with access to contribute or modify content, using a simplified markup language. The project's website address is http://pavements2.ce.utexas.edu:8080/txdot. This study was a "low-cost, high-payoff" project because most of the research and expensive testing had already been done but several valuable lessons were missing. This database is a useful tool that can help communicate the experiences and lessons learned not only from different projects, but from engineers themselves.

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Monitoring of Experimental Sections Using a Web-based Database

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Chapter 1. Introduction

Over the years, the Texas Department of Transportation (TxDOT) has developed several databases for very different purposes. However, there is no one database that collects and stores the important experiences gained from field application of different technologies or methods. Furthermore, TxDOT funds a significant number of research projects and often requests that these projects submit the information and data collected in database format at the conclusion of the project. Establishing a database that can hold information from various TxDOT research projects provides TxDOT personnel, practitioners, and researchers with a tool to reference information from past studies to supplement data and avoid duplication in their own projects. A database of this type is useful not only for tracking the performance of different types of materials or construction techniques, but also as a warehouse to archive case studies and observations about different projects that can be accessed by TxDOT engineers and others. This access is fundamental for ensuring that the experience and knowledge gained by the Department through the years in developing and implementing different technologies is preserved and the mistakes from the past are not repeated.

The recently completed TxDOT Project 0-5472, entitled *A Database for Successful Pavement Sections in Texas—Including Both Experimental and Non-Experimental Pavements,* developed a database system that stores information on sections that have been deemed "successful" according to criteria established by a panel of experts composed of engineers from several TxDOT Districts and the Texas Transportation Institute (TTI). The project defined a successful pavement section, which was a minimum of one mile, as *"a structure that has met performance expectations over its service life with normally expected levels of maintenance for its age, materials utilized, traffic loads, and local conditions."* In terms of performance data, the database contains scores only from the Pavement Management Information System (PMIS): pavement distress score, pavement condition score, and pavement ride score. Additionally, one Ground Penetrating Radar (GPR) survey, one Falling Weight Deflectometer (FWD) test, one Dynamic Cone Penetrometer (DCP) test, and some general laboratory results are included. This database has a significant limitation in the number of years for which it can store performance data.

The Texas Flexible Pavement Database (TFPD), recently developed by The University of Texas at Austin (http://pavements.ce.utexas.edu/), has no such limit and is actually well suited to store pavement performance data. The philosophy behind the TFPD has two distinctive characteristics: *"open-to-all"* and *"one-stop shopping."* However, the TFPD has an even greater advantage: it is linked to the PMIS, Design and Construction Information System (DCIS), and SiteManager databases, making it unique among its kind in the U.S. and worldwide. Additionally, the research team maintains a PC-based version of the database in MS Access, in cases that web access is a limitation.

Finally, based on the analysis of the limitations and advantages of existing database systems such as the one developed as part of TxDOT Project 0-5472 and the TFPD, this project created an online database system that can house all of the required information and meet the project's requirements and goals.

1.1 Background

1.1.1 TxDOT 0-5472

TxDOT Project 0-5472, *A Database for Successful Pavement Sections in Texas—Including Both Experimental and Non-Experimental Pavements,* resulted in the development of a database system that stores information on sections deemed successful according to criteria established by a panel of experts (Krugler et al., 2007). The one-mile-minimum pavement sections, identified by personnel from TxDOT Districts, were selected because the sections could each be defined as a *"structure that has met performance expectations over its service life with normally expected levels of maintenance for its age, materials utilized, traffic loads, and local conditions."*

More specifically, recommendations were sought for pavement sections with successful performance that are over 6 years old, meet a maintenance expenditure criteria (to rule out pavement sections that display good performance through time only as a result of constant expenditures in maintenance and rehabilitation), and meet at least four of the following six criteria: 1) minimum condition score average, 2) standard deviation of score averages, 3) minimum distress score average, 4) standard deviation of distress scores, 5) minimum ride score average, and 6) standard deviation of ride scores.

During the 2-year project duration, 25 successful pavement sections (meeting some of the criteria established previously) were entered into the database. Approximately 68 percent of the included pavement sections are subjected to medium traffic (average daily traffic [ADT] between 500 and 10,000), and only one section has traffic lower than 500 ADT. Additionally, 32 percent of the pavement sections are surface treatments, 24 percent are medium thickness asphalt concrete pavement pavements (2.5 in.–5.5 in.), and 40 percent are thick or composite pavements. However, only one thin asphalt concrete pavement section showed successful performance and was included in the database.

1.1.1.1 Database and Web Application Development

The 0-5472 database structure was divided into three components. The first component deals with general pavement section information, but also includes maintenance history. The second component stores the performance and material properties that were collected for the 25 successful pavement sections. The third element of the database is mostly intended for administrative users, and allows for managing and accessing information.

Component 1 contains location, weather, layer, and traffic data (acquired from the design and construction records). Component 2 consists of distress and roughness data (only characterized through PMIS distress scores: pavement distress score, pavement condition score, and pavement ride score). This information cannot be considered actual performance data, as it is the result of the weighted sum of actual distresses. Pavement performance data should consist of actual roughness, rutting, cracking, etc., in addition to the above-mentioned scores. Moreover, one GPR survey, one FWD test, one DCP test, and some general laboratory results are included. A significant limitation of the database design is that "*the database and website are designed to allow entry and storage of up to 10 years of PMIS performance score information*" (Krugler et al., 2007, pp. 100). This limitation means that if new data were to be imported, the older information would be overwritten and, thus, lost. There should be no limit to the amount or longevity of performance history allowed into the database, as a longer record allows for more robust pavement performance analysis.

The web application that serves as a user interface for the database was designed to allow for four levels of access: guest, read-access, read-write-approve-access, and administrator-access. Guest access is limited to viewing and exporting data stored in the database, while read-access provides the additional functionality of nominating new candidate successful pavement sections. Read-write-approve-access has the permissions of the previous access levels but can also edit information stored inside the database and change the status of nominated pavement sections to approved successful pavement sections. The administrator-access level has the additional capability of registering new users, and modifying the types of users.

1.1.1.2 Pavement Testing

The 0-5472 research team conducted testing with a scope similar to that of a forensic investigation. Forensic testing is performed on a pavement section to determine the cause of failure, but in this case the main purpose was to study those factors that were involved in the successful performance of the selected pavement sections. Each of the locations contains only one one-mile long section on a single lane in one travel direction (generally the outside lane was chosen).

As mentioned previously, a single measurement of the following non-destructive tests (NDT) was performed on the pavement sections: GPR, FWD, and DCP. GPR from the right wheel-path was used for the purpose of determining variation in layer thickness. It also allowed for analysis of sub-surface conditions. Although the raw data obtained from the GPR should be downloadable from the web application, only a low-resolution color map of the data collected by the GPR is observable on the web application while browsing through the different pavement sections. This functionality limits users who are trying to easily select pavement sections with certain properties, mainly because GPR color maps are difficult to read for people with little experience with the test output. A simple set of summary statistics such as average layer thicknesses and variation of layer thicknesses would be highly desirable.

The FWD test was conducted with the objective of characterizing the structural integrity of the pavement sections and backcalculating their moduli of elasticity. Thirty test points were evaluated along the outer wheel-path of the one-mile sections using a standardized 9,000 lb. load. The web application allows the user to observe an image of the deflection below the seven FWD sensors along the pavement, and also displays modulus backcalculation results for some of the pavement layers.

The DCP test was used for the purpose of measuring the stiffness of the layers of the successful pavement sections. The test was performed using an 8-kg sliding hammer at two locations along the one-mile pavement section. Summary statistics of elastic modulus, California Bearing Ratio (CBR), and number of blows are presented in the web application.

In addition to the non-destructive testing described earlier, researchers extracted core samples of the asphalt mixture and stabilized layer and loose samples of base and subgrade materials from two locations along the one-mile pavement sections (500 ft from each endpoint of the section). The samples extracted from asphalt concrete layers of sufficient thickness were used in specific gravity and air void determination, asphalt content through different extraction procedures (solvent extraction for non-modified binders and ignition method for the polymer-modified asphalt binders), mixture aggregate gradation, and penetration grading of the asphalt binder.

Cores from pavement sections that consisted of surface treatment or seal coats over the base layer were only used for thickness measurements. Cores obtained from stabilized layers were tested in a similar manner as that of the previously mentioned asphalt concrete layer cores. Loose materials obtained from the flexible base layers and subgrade were used to determine field moisture content, Atterberg limits, gradation, and mineral identification through visual examination, calcite content, and hardness.

In summary, clearly a significant effort was carried out during TxDOT Project 0-5472. However, the project objectives were not fully compatible with TxDOT's objectives for the new Project 0-6357, *Monitoring of Experimental Sections Using a Pavement Database,* thus requiring a different approach.

1.1.2 From the Old to the New Texas Flexible Pavement Databases

For more than 30 years, in an almost continuous effort that began in 1972, TTI maintained a TFPD. This database originally consisted of 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, and FM and RM roads). However, due to the importance of the interstate system, these facilities were sampled at a higher rate. The data collected and contained in this database was used for developing the performance equations and pavement condition predictions that were incorporated into various optimization routines, which eventually became part of the Texas Flexible Pavement System (FPS) software for flexible pavement design.

In addition to structural and basic condition information, deflection measurements were performed and complete condition surveys were carried out to determine the serviceability index of the various sections. Weather data were also taken from the records of weather stations in the counties where the sections were located. With the advent of mechanistically based pavement design approaches, popularization of the FWD, back-calculation techniques, and the increased need for designing overlays, data needs became more demanding and maintaining such a large database for design purposes became increasingly cumbersome. Thus, in 1988, TxDOT Project 0-187-6, *Preserving the Texas Pavement Database*, was initiated with the following objectives:

- Preserve, update and improve the TFPD;
- Store all condition and deflection data that were collected by TxDOT personnel on the pavement sections in the database; and
- Revise, using the new data, the pavement distress and performance equations for each type of pavement represented in the database.

Once Project 0-187-6 concluded, a period of time followed during which data were not collected and the database was not maintained until 2001, when another project modification was put in place to re-establish the TFPD and to facilitate its full implementation. The objective of this modification was to fill in the experimental cells that were lacking, primarily covering different pavement structures in different environmental regions. The full experimental design included the following variables: 1) type of pavement structure, 2) environmental conditions, 3) traffic loads, 4) layer thickness, and 5) material types. The experimental design necessary to take into account reasonable levels for all these variables was not economically feasible so the project focused efforts on a partial experiment that was more realistic. The implementation plan established that the database was "*to be used to validate and verify design data being generated by District Pavement Engineers.*" The database was to be applied for calibrating the performance curves used in FPS-19W and other design algorithms used by TxDOT.

The experimental design considered in this project consisted of almost 500 sections that included 1) six pavement types, 2) two subgrade types (weak and strong), 3) five traffic levels, and 4) five environmental regions (dry-cold, dry-warm, wet-cold, wet-warm, and mixed). Although logical, this goal turned out to be impractical due to the enormous effort that it implied.

In 2003, another project modification contemplated the incorporation of the data corresponding to the Long Term Pavement Performance (LTPP) studies contained in the Federal Highway Administration's (FHWA) DataPave database (www.ltpp-products.com; FHWA 2004). Sections from the General Pavement Studies (GPS) and Specific Pavement Studies (SPS) were incorporated into the scope of the project. These data were to be used to perform a sensitivity analysis of the design variables using the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG). TTI also carried out this effort. Unfortunately, the research objectives were not fully accomplished. Data have been collected over a period of more than 30 years but, unfortunately, most of the data has either been lost or is not accessible.

Finally, in 2006, TxDOT Research Project 0-5513, *Development of a Flexible Pavements Database*, was initiated. This database will be referred to as the "new" TFPD. The work was conducted at UT-Austin following a different approach. It was recognized that to perform reliable performance analysis, reliable and sustainable databases are essential. These databases should include material properties, pavement structural characteristics, highway traffic information, environmental conditions, and performance data, such as cracking, rutting, and roughness. Ideally, all information should be together and should be easily accessible to everyone so that at the end of the project, the information is not lost.

Clearly, databases that store important and valuable data have been developed in Texas over the years. Regrettably, a lot of the information has been lost or is not accessible in a useful way to pavement engineers. Furthermore, the databases do not interact with each other. A recent joint effort between TxDOT and UT-Austin (Smit and Cleveland, 2004) produced a very successful tool for linking some of the existing databases—DCIS and PMIS—and "mining" them to extract the desired data. This effort, however limited in scope, demonstrated the feasibility and benefits of the approach. Building on the success of this effort and the lessons learned, a similar approach was proposed for the development of the new TFPD.

Note that as design methods and materials change over time, so also do traffic characteristics, especially wheel loads, tire inflation pressures, tire types, axle configurations, and suspension types. An overly ambitious plan has the potential to derail or to be so cumbersome that the means becomes the goal. As discussed earlier, to some extent the same applies to local efforts with similar objectives. TTI's work on the development of the "old" TFPD has been ongoing for more than 30 years. The research objectives were logical; however, those objectives were too wide-ranging and became unachievable and unrealistic within limited budget and time constraints. The result was that after 30 years there was little to show.

In summary, as explained in the proceeding sections, clearly the objectives and deliverables of TxDOT Project 0-5513 are more compatible with the objectives of this new TxDOT Project 0-6357, *Monitoring of Experimental Sections Using a Pavement Database*. Furthermore, the availability of "wiki" tools allowed for the development of an online database system that can house all of the required information and can be easily accessed and modified by all users involved to accommodate changing requirements and needs.

The original developer of the wiki described it as *"the simplest online database that could possibly work."* A wiki consists of a collection of web pages designed to enable anyone with access to contribute or modify content, using a simplified markup language. The website for the current project can be found at http://pavements2.ce.utexas.edu:8080/txdot. Figure 1.1 shows an example of the database main interface and welcome screens.

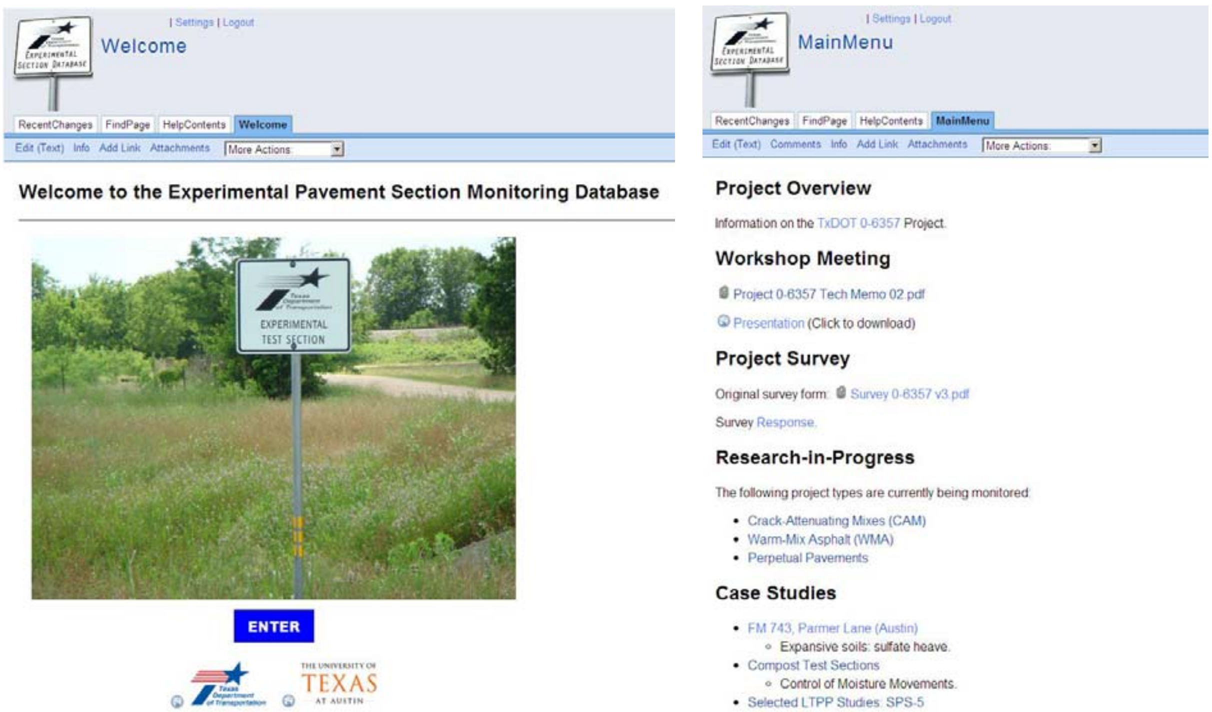


Figure 1.1: Welcome page and extract of the main menu for the Experimental Pavement Section Monitoring Database

1.2 Project Objectives

The goal of the current project was to develop a database of selected experimental flexible pavement sections constructed over the years by the different TxDOT districts. This project provided for collection of these sections' performance data in order to determine the types of experiments that have been successful and provided good performance. To achieve this goal, the objectives of TxDOT Project 0-6357, *Monitoring of Experimental Sections Using a Pavement Database*, are to 1) select experimental sections that have the potential for greater impact in Texas, 2) develop and agree upon a data collection plan, 3) perform data collection and data population, 4) recommend a process for analyzing the data, and 5) develop a long-term plan for the maintenance and management of the database by TxDOT. This database will also be a fully GIS-based system, and thus will be compatible with the latest developments currently being undertaken by the Department. Towards this goal, a web-based wiki was developed as a front end to serve content developed as part of the study.

The intent of the project was to deliver organized data and analysis from those experimental sections that have the potential to result in more reliable and economical pavement design. Once these findings are implemented, all TxDOT districts and divisions can use them and potentially realize significant savings by affecting design and construction methods and current specifications.

1.3 Scope, Methodology, and Report Outline

After the current Introduction Chapter, the report follows this structure: Chapter 2 introduces the effort involved in selecting the projects and case studies that are treated as part of the current project. The chapter summarizes all the projects that were initially considered, the selection criteria, and the final project selection. Chapter 3 introduces the recommended analysis tools or methodologies that can be used in quantifying the differences between different alternatives or technologies. These tools were used to compare and analyze the performance of the different technologies used in the projects and cases studies selected in Chapter 2.

Chapters 4 through 8 summarize project information, performance monitoring, analysis and findings associated with the different projects and case studies that were included in the project. Chapter 4 looks at the effect of recycled asphalt pavement (RAP) on the LTPP SPS-5 sections. Chapter 5 looks at the use of composts to reduce the deterioration of the pavement structure (mainly in the shoulder). Chapter 6 looks at the effect of different warm mix asphalt (WMA) technologies on pavement performance. Chapter 7 describes the performance of observed perpetual pavement structures. Chapter 8 analyzes the performance of crack attenuating mixes (CAM) mixes in Texas.

Finally, Chapter 9 summarizes the main findings and observations.

Chapter 2. Selection of Candidate Sections

2.1 Proposal of Candidate Sections

A kick-off meeting for this research project was held on September 22, 2008. During this meeting, participants agreed to include both "successful" and "unsuccessful" projects from which knowledge can be gained, provided that the projects still exist. Furthermore, a district's interest in the long-term performance and outcome of a project should drive the study. The participants decided to select the final database format after completing a review of candidate projects for possible inclusion in the database.

The candidate projects and sections were selected based on the recommendations of the research team, as well as on the result of a survey that was sent to the District Offices. The results of this survey are included in Appendix A.

This chapter outlines the survey and summarizes responses received from the districts towards identifying candidate projects. Additionally, this chapter expands on the topics identified in the survey, providing topic-specific background information to assist TxDOT in the selection of candidates and to identify a number of relevant projects in the different districts. The following sections outline the researchers' recommendations, based on specific criteria, for (a) candidate projects to track and (b) case studies to document as part of the study.

2.1.1 Survey and Literature Review

A pavement candidate survey was e-mailed to each of the TxDOT districts to gather information pertaining to past and ongoing projects for the selection of candidate projects to include in the study. Responses obtained from the survey are summarized in Table A.1, (Appendix A), which provides the location and description of projects identified by district personnel for potential inclusion in the study.

To further identify potential projects for monitoring, a literature review of past TxDOT research was conducted, based primarily on responses highlighted in the district survey. This information was synthesized and is summarized by grouping the feedback from the survey and literature review into specific topics and subjects as follows:

- Geogrids and Geosynthetics
- Rubblization
- High Performance and Heavy Duty Flexible Bases
- Perpetual Pavements
- Ultrathin Bonded Wearing Courses
- Warm Mix Asphalt
- Asphalt Rubber Pavements
- Full Depth Reclamation
- Sulfate Heave Problems
- Jointed Concrete Pavement Rehabilitation

- Long-Term Pavement Performance Studies
- Use of Compost

After a project progress meeting in spring 2009, participants decided to exclude all previously identified concrete-related and porous friction course (PFC) projects as these projects were being evaluated by other ongoing TxDOT studies. These particular projects are shaded in Table A.1. Note that Project #16 as listed in Table A.1 stands apart from the others, describing two weigh-in-motion installations constructed using portland cement concrete (PCC) and hot mix asphalt (HMA).

2.1.1.1 Geogrids [TxDOT Project 0-4829]

Geosynthetics can be defined as planar products manufactured from polymeric material, which are used with soil, rock or other geotechnical engineering related material as an integral part of a man-made project, structure or system (ASTM, 1995). Geosynthetics are widely used in many geotechnical and transportation applications.

Geosynthetics have numerous material properties. Many of the reported properties are important in the manufacture and quality control of geosynthetics; however, many others are also important in design. The material properties related to the manufacture and quality control of geosynthetics are generally referred to as index properties and those related to the design as design or performance properties. Considering their different properties, the several geosynthetic products can perform different functions and, consequently, they should be designed in order to satisfy minimum criteria to adequately perform these functions. The geosynthetic functions are: separation, reinforcement, filtration, drainage, infiltration barrier, and protection.

A research study sponsored by TxDOT at The University of Texas at Austin looked at both geogrids and geosynthetics in pavement applications. The focus of this TxDOT study was on the use of geogrids, as they have been the primary product used in projects involving pavements over subgrade soils sensitive to volumetric changes (i.e., heaving and shrinkage of soils). The primary concern addressed by geogrids is pavement cracking over high plasticity index (PI) clay and weak subgrades.

In Texas, the geogrid is placed at the interface of the cement-treated or lime-treated sub-base and a 3 to 4 inch flexible base. *Particularly* poor performance has been reported with the use of glass grids on some projects. In these cases this product has been shown to physically disintegrate over time [Personal communication with Dar-Hao Chen].

The project report identified the following districts as having geogrid as reinforcement in pavements.

<u>Fort Worth (3 Projects)</u>

The pavements in this area faced problems with cracking over high PI clays and weak subgrade. They used geogrids to span weak subgrade and reduce the severity of reflective cracking from the subgrade. The three projects executed in 2004–2005 were:

- FM 2331 (4.3 miles) between FM 4 and SH 171,
- FM 917 (1.4 miles) between US 67 and the Ellis county line, and,
- FM 157 (2.0 miles), in five separate section areas between SH 1774 and FM 2880.

The geogrid used was Tensar Type 2 geogrid. The pavement structure for each of the Johnson County sections was 2 inch ACP, seal coat, 8 inch cement stabilized base, geogrid, and clay-sandy subgrade.

Williamson (1 Project)

The area faced cracking problems with pavements over high PI clays and weak subgrade. The project was executed on **US 79** (2.063 miles), west of Taylor, where a layer of geogrid was introduced between the lime-treated subgrade and flexbase to reduce reflective cracking over high PI clays.

Hidalgo (2 Projects)

The pavements in this area had cracking problems when placed over high PI clay and weak subgrades. A mill and overlay test section was constructed in April 2001 on **FM 1926**, 0.66 miles long, consisting of six 500-ft sections using the following geogrids: GlasGrid 8501, HaTelit C40/17, Pave-Dry 381, StarGrid GPS, Bitutex Composite by Synteen, and Petro-Grid 4582. These sections were constructed to monitor the ability of a geosynthetic to prevent transverse and longitudinal cracks. Further, a new project was started in March 2005 at **FM 3462** that was 1.5 miles long. The geogrid was used to prevent or retard cracks along the shoulders, which were evident before reconstruction.

Jefferson (1 Project)

The Area Engineer experienced problems with pavement distress over weak subgrades. Two layers of geogrids were used as base reinforcement to span the weak subgrade. The project was executed on **SH 73** and completed in 2000, running from 0.5 miles west of Taylor Bayou to the Chambers County line. The total length of project was 17.686 miles and used two geogrid products, Tensar and Tenax.

Houston (1 Project)

Harris County experienced problems with pavement cracking over high PI clays and weak subgrade; geogrids were used to span the weak subgrade. Tensar geogrid was used in this project on **FM 357** (0.35 miles) at Wallace Creek on a temporary detour for a bridge replacement project in 2005. But the conditions were so poor that the subgrade had to be cement treated in addition to using geogrid.

Navarro (2 Projects)

Navarro County experienced problems with pavement cracking over high PI clays and weak subgrade. The geogrids were used in the first project in the **northbound lane of IH 45**, 1 mile south of US 287 in Corsicana for 0.2 miles, to span weak and wet subgrade over a 12-inch crushed concrete base. The project was completed in 2002. In the second project, the 3-mile section **of IH 45** from SH 14 to north of Richland Creek was reinforced with geogrid in 2004.

The grid was used to span soft subgrade covered with 6 to 10 inches of flexible base. The remaining section had a 4-inch bond breaker and 13.5 inches of concrete pavement.

Panola (4 Projects)

Panola County experienced problems with pavement cracking over high PI clays and weak subgrade. The geogrid was used for spot treatment to act as reinforcement, mitigate subgrade cracking, and span weak subgrade. Four projects were reported. The first project was executed on **US 59** in 1991–1994. It used a nonwoven geotextile, Phillips 66, 5 ft below the pavement surface. The three other projects were executed in 2004, using Tensar geogrid: **FM 123** from US 79 to FM 31, **FM 699** from US 59 to FM 2517, and **FM 2517** from FM 699 to FM 31.

Titus (1 Project)

Titus County experienced problems with pavement cracking over high PI clays and weak subgrade. The geogrids were used to mitigate subgrade cracking. A 0.52-mile section of **FM 1402** from IH 30 to US 67 was reinforced in August 2004 using Tenax Type 1 geogrid.

Bowie (1 Project)

The pavements in Bowie County were experiencing problems due to cracking when placed over high PI clay and weak subgrade. The geogrids were used as base reinforcement and to mitigate subgrade cracking. **US 259** was reinforced using Tensar BX 1100 geogrid with 0.59 miles of single placement and 0.19 miles of double placement in 2004.

Harrison (2 Projects)

In one project, executed in 1983, geotextile was used to span weak subgrade. In a 4.3-mile section of **FM 3251**, a layer of geotextile was placed directly below the flexible base. The geotextiles used were Typar 360, Mirafi 500X, and True-Tex MG 200. Subsequent base failures occurred throughout the project. *The reasons for failure were not clear*. In other projects, geogrids were used for spot treatment. A section of **FM 2199** from US 80 to IH 20 was reinforced using Type 1 geogrid to mitigate subgrade cracking in 2003.

Walker (5 Projects)

Walker County had problems with pavement cracking over high PI clays and weak subgrade. The main application of the geogrid was for base reinforcement purposes. Five projects were executed from 2001 to 2003 using Type 1 geogrid:

- **FM 1696** (1.549 miles) between SH 75 and IH 45 in 2001,
- **FM 39** (7.854 miles) between US 190 and County Line in 2002,
- **FM 1428** (3.376 miles) between SH 21 and FM 2158 in 2003,
- **IH 45** (6.017 miles) from the Leon County line to 0.2 miles north of MP 187, and,

- **FM 1375** (4.315 miles) from the IH 45 West frontage road to 4 miles west.

Grayson (3 Projects)

Grayson County had problems with pavement cracking over high PI clays and weak subgrade. The geogrids were used in areas of high stress and at signalized intersections. Approximately 3 miles of a section on **US 69** from Spur 503 to MLK at Denison were reinforced in 1999. A geotextile was used under hot mix asphalt concrete (HMAC) in **US 82** from Coke to Beaver Creek in 1998. Geogrids were also used for area widening in **SH 121** from the county line to SH 11 in Fannin. The project was completed in 2003.

Midland (1 Project)

Longitudinal cracking was observed on **US 67** extending from the Reagan County line to 3 miles west of it. The cracks were due to shrinkage occurring in the subgrade near the shoulder but later they continued across the roadway. Completed in 2003, the project placed the layer of Tensar geogrid between the base and subgrade.

Taylor (An example where geogrids were tried but failed)

Taylor County had problems with pavement sections cracking when placed over high PI clays. They tried using a geocomposite and glass grid. *In both cases they ended up removing the material from the project because it did not meet the specifications set by TxDOT.*

For the majority of the projects the geosynthetics were placed in the pavement during construction and no post-construction performance evaluation was conducted, making it difficult to quantify the benefits of using geosynthetics in pavements.

Other Case Studies

- A forensic investigation conducted at a newly constructed pavement on FM 542 (Leon County) was reported. Longitudinal cracks were observed in the geogrid reinforced pavement before it was open to traffic. When the site was excavated near the cracks, no geogrid was found below the pavement section. Further investigation revealed that the contractor had laid a 9.8-ft roll of geogrid beneath a 14-foot-long pavement, meaning that the remaining 4.2-ft section was unreinforced and hence cracked, indicating the benefits of geogrid.
- Field performance of two geogrid-reinforced pavement sections with high plasticity clay subgrade was reported. The pavement had two different types of geogrid. Whereas one section reinforced with Type 1 geogrid (polypropylene) was found to be performing well, the other section reinforced with Type 2 geogrid (polyester) showed longitudinal cracking. The review of the material properties led to the preliminary conclusion that poor performance in the Type 2 geogrid sections was due to inadequate junction efficiency but closer inspection indicated a higher tensile modulus of geogrid in this section. Because tensile modulus is an important property of geogrid, the need for better material characterization is stressed to predict the actual cause of differences observed in field performance.

- In the third pavement, three sections were constructed. The two geogrid reinforced sections, i.e., Sections 1 and 2, had base course thicknesses of 8 inches and 5 inches respectively; the Control Section (with no geogrid reinforcement) had an 8 inch (0.20 m) thick base course layer. FWD testing showed higher pavement modulus for the geogrid-reinforced section with an 8 inch thick base course layer over the Control Section; lower modulus values were predicted for the geogrid-reinforced section having a 5 inch thick base course layer. This modulus value would indicate better performance for Section 1 and poor performance of Section 2 when compared with the Control Section. Field visual assessment, however, showed cracking in the Control Section and that the two geogrid-reinforced sections were performing well. The geogrid-reinforced sections outperformed the unreinforced sections although the FWD testing indicated otherwise. This result shows the inadequacy in the present analysis technique of using nondestructive testing to quantify the benefit of geogrids in pavements.

<u>Bryan Experiment (32 Sections)</u>

A total of 32 test sections (4 reinforcement types x 2 stabilization approaches x 4 replicates) were placed as part of a road reconstruction project on **FM2** in the Bryan District (see Figure 2.1). Instrumentation was implemented in order to characterize the patterns of moisture migration under the pavement. Construction was completed in January 2006 and performance evaluation of the newly reconstructed road was conducted with special focus on the 32 test sections. Field monitoring was conducted before reconstruction and immediately after reconstruction. However, post-construction field evaluation requires continued monitoring for at least two complete seasons.

Figure 2.1 shows the 32 sections on FM 2 constructed on the eastbound (E) and westbound (W) lanes, K1 and K6, respectively. The blue-shaded cells indicate sections with lime treatment and the numbers in each cell indicate the type of reinforcement applied, as shown on the legend to the right of the figure. The experiment is replicated as indicated in each cell as *a* and *b*.

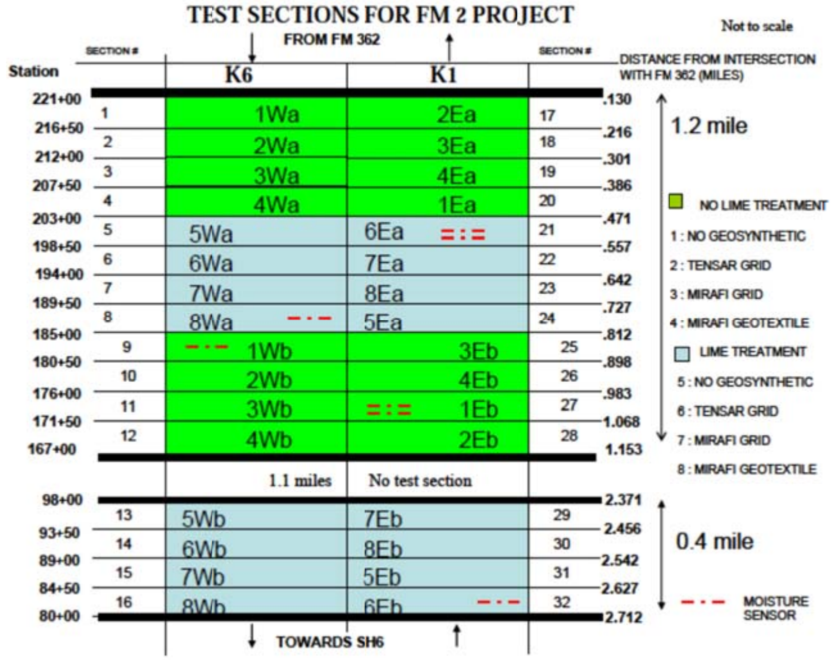


Figure 2.1: Section layout of geotextile/geogrid experiment

The report recommended the following field monitoring program:

- Continued FWD testing to be conducted at least four times a year to assess the effect of seasonal moisture variations in the pavement performance,
- Continued field monitoring of moisture sensor profiles that have already been installed to monitor the horizontal and vertical changes in moisture within the pavement layers,
- Continued condition surveying to document and quantify the field performance of the sections,
- Continued gathering and evaluation of relevant weather data, and
- Quantification and assessment of cracks and deterioration that may develop in the 32 monitored sections. This process should also include trenching at locations of identified failures.

<u>Newer sections identified in the survey</u>

- Survey information provided by Darlene Goehl: Geogrid was used on SH 6 in lieu of chemical stabilization. Due to concerns with sulfates and organics on this project, the working platform was designed with geogrid and a lift of flexible base.
- Geogrid was used with RAP to rehabilitate FM 734 (Parmer Lane).
- The FHWA has six sections in Texas with geotextile reinforcement. These are shown in Table 2.1. The geotextile is a non-woven polypropylene fabric meeting the requirements of DMS-6220. Item 356 (Fabric Underseal) was modified for

construction equipment and laydown operations. Data being collected on these sections include:

    - GPR test.
    - Automated distress survey.
    - Rutting measurements.
    - Ride Quality measurements.
    - FWD test.
    - Video (roadway and right-of-way).

- On US 259 in Rusk County, Type 1 fiberglass reinforcing grid was used to counter reflective cracking of HMA over a cracked concrete pavement.

Please note that not all geogrid/geosynthetic applications have been successful, and as such, it is important to record TxDOT's experience in this regard so that valuable knowledge on application of geogrids is transmitted, and mistakes are not repeated.

Table 2.1: FHWA geotextile sections

Controlling CSJ	Project Description	Length (ft)	County	ADT	Location	Typical Section	Fabric Location	Tack Coat	Seal Coat Binder	Aggregate Source
0055-02-021	November 2003 Letting		Hamilton							
Project CSJ										
0049-01-080	SH 6 From FM 1860 in Riesel To .5 mi West	2640	McLennan	2700	East of IH 35	Divided Highway with Two 12-ft lanes with inside and outside shoulders	Northbound Lanes only	PG 64-22	AC20 - 5TR	Martin Marrietta
1835-02-048	FM 1741 From FM 93 To .5mi. North	2640	Bell	31000	East of IH 35	Four 12-ft lanes with 16-ft continuous left turn lane (C&G section) (67-ft total width)	Entire Roadway Width	PG 64-22	AC20 - 5TR	Capitol Aggregates
0232-02-017	SH 53 From Bell/Fall CL To .5 mi East	2640	Falls	1300	East of IH 35	Two 12-ft lanes with 6-ft shoulders	Both Travel Lanes	PG 64-22	AC20 - 5TR	Martin Marrietta
0413-04-031	SH 164 From 200' east of Navasota River Bridge To 3/4 mi. East	3960	Limestone	2200	East of IH 35	Two 12-ft lanes with 10-ft shoulders	Both Travel Lanes	PG 64-22	AC20 - 5TR	TXI Lightweight

Controlling CSJ	Project Description	Length (ft)	County	ADT	Location	Typical Section	Fabric Location	Tack Coat	Seal Coat Binder	Aggregate Source
1187-01-024	December 2003 Letting		Coryell							
Project CSJ										
0386-04-012	FM 1991 From SH 22 South 3/4 mile	3960	Bosque	330	West of IH 35	Two 10-ft lanes NO shoulders	Both Travel Lanes	CRS-2P	CRS-2P	Capitol Aggregates
1187-01-024	FM 929 From SH 36 To 0.5 mi. East	2640	Coryell	2100	West of IH 35	Two 12-ft lanes with 10-ft shoulders (C&G roadway)	Mainlanes and Shoulders	PG 64-22	CRS-2P	Capitol Aggregates

2.1.1.2 Rubblization [TxDOT Project 0-4687]

Rubblization is a unique means of rehabilitating concrete pavements by in-place conversion of the old concrete pavement into a useable base. Rubblization employs machinery that will break apart the concrete in place and leave pieces small enough so that reflective cracking problems are significantly reduced or ideally eliminated. *Rehabilitation of concrete pavements is a major issue within TxDOT. The department has many miles of old jointed and continuously reinforced concrete pavement approaching the end of their service life. Black topping and white topping can be used to gain additional life, but these treatments are often impacted by reflection cracking. In many instances the existing concrete pavement is structurally deteriorated to the point that simple overlays will not provide adequate performance.* TxDOT does not have a lot of experience with rubblization. The TxDOT Project 0-4687 report discusses the following three constructed projects.

<u>Bryan</u>

On the **US 79** project, the Bryan District reported problems with rubblizing. On approximately 20 percent of the project, the slabs would not break and had to be replaced with full depth hot mix, leading to an expensive field change. The District placed 7 inches of new flexible base and 3.5 inches of HMA on top of the rubblized concrete. The rubblized section is now part of a major intersection. Most of the rubblized pavement is in the central turn lane. Performance to date has been good; the District reported one longitudinal crack in the area where they widened the existing slab with full depth hot mix. The section was constructed around 1998.

<u>Atlanta</u>

The Atlanta District section was constructed on **US 67** by the Mount Pleasant Area Office. The process was not considered a success. It was reported by the District Lab Engineer that

> the process was not effective over joint; big unbroken pieces remained which had to be replaced with full depth Asphalt Concrete Pavement (ACP). The rubblizer rutted the processed section and displaced the concrete. Upon coring it was found that water was present in the rubblized concrete beneath the ACP.

The average rubblized concrete moduli value was less than 50 ksi, which is less than would be anticipated for a Class 1 flexible base. This low value may be attributed to the fact that water may be trapped in the base. As with the Bryan District project, no edge drains were installed.

<u>Childress</u>

The Childress District rubblized a section of **US 83** in 2003. TTI evaluated a continuous section 0.9 miles long from FM 3256 northward. This section is still performing excellently—monitoring of the section reveals good performance from the pavement, with the rubblized layer retaining a relatively uniform and low moisture level. Results show the average modulus of the rubblized layer continues to increase with time. Originally 114 ksi within 1 year of rubblization, this value was 200 ksi after 2 years. Currently the average rubblized base modulus is 323 ksi.

The latest survey at this site took place in April 2007 with GPR and FWD. Visually the section still appears to be in excellent condition. The rubblized layer modulus, originally approximately 4 percent of the prefractured modulus, shows a trend of increasing modulus with time. This increase is likely due to self-cementing properties of the rubblized layer.

In the summer of 2006, the Childress District rubblized a portion of the westbound travel direction of **US 70** in Foard County starting at FM 267. A pre-construction site investigation conducted in the fall of 2005 concluded that although the section was suitable for rubblization, the collected data were spot specific, so a chance existed for areas between test locations where problems could occur. The pre-construction investigation estimated that 10 to 20 percent of the project would require removal and replacement. Unfortunately, stability problems were discovered during the paving operation. Although only approximately 5 percent of the rubblized section had problems, most of the rubblized concrete was removed and rubblization abandoned due to the unfortunate timing of the problem's discovery. Approximately 3,000 feet of rubblized concrete remained in place. Problems arose on the project because the unstable areas were not detected prior to paving. This project highlighted the importance of re-evaluating the project screening phase, the rubblization operation, the enforcement of specifications, and the crucial role of proof rolling. In the rubblized section that TxDOT retained, the average rubblized layer modulus was 138 ksi approximately 8 months after construction. A review of the project following the occurrence of the failures revealed the following contributing factors:

- Wet/weak subgrade was not detected in the project pre-screening evaluation. This project highlights the weakness of the spot test nature of the pre-screening tests. Additionally, the GPR survey *did not* indicate wet subgrade areas. The thickness of the HMA cover when the GPR was performed could have contributed to this lack of detection.
- Large, out-of-spec particle sizes did not trigger action during inspection of the rubblization process. Because particles cannot be broken down as small as required by rubblization when poor subgrade support exists, failing to meet particle size specification indicates a weak spot may exist. Stricter enforcement of the particle size specification could have helped avoid the failures encountered on this project.
- The construction process itself may pump water. The rubblization process and compaction of HMA produce vibrations that could pump water to the bottom of the concrete, contributing to even poorer subgrade conditions.
- Proof rolling *did not* detect weak spots. Although TxDOT reports proof rolling was conducted, the wheel loads apparently were not high enough or the process was not monitored sufficiently.

These are important lessons that may be beneficial to other TxDOT engineers considering rubblization.

<u>Rubblization project identified in the survey</u>

In the survey response, Darlene Goehl describes a project on FM 1155/FM912. This rubblization section has multiple sections and a flexible base overlay. This project included two new special specifications: one for rubblization and one for drainable base. The information is documented in TxDOT Project 0-4687.

2.1.1.3 High Performance Flexible Bases and Heavy Duty Flexible Pavements [TxDOT Project 0-4358]

Traditional Texas flexible bases specified under Item 247 perform well as long as they are kept dry. However, rapid and sudden failures can occur if water enters these bases. In Project 0-4358, draft specifications (proposed Item 245) were developed for high-performance flexible base materials. These specifications tighten all existing specifications by placing an upper limit of 10 percent on the amount of material passing the No. 200 sieve, and introduce new procedures to ensure that the base is not moisture susceptible.

In Project 0-4358, two TxDOT pavements containing bases that met the proposed high-performance base specifications were constructed. No handling or segregation problems were encountered. The main concern found by the contractors was the use of nuclear density gauges for measuring density. Alternative methods were investigated. The initial field moduli were measured to be 60 ksi. **The long-term benefits of these low-fines bases could not be demonstrated in this short project**, because all the applicable sections are new and performing well. However, performance problems were encountered on a third section constructed on SH 43. In that case the design caused a “bathtub” effect and water became trapped in the low fines base. Based on this experience a “day-lighting” requirement was placed in the Item 245 specification. High-performance bases will cost more than traditional bases, and they are not needed in many areas of west Texas where rainfall is low. However, these bases will be economically viable in many areas of northeast Texas, especially with the escalating prices of traditional road building materials.

<u>SH 31 Tyler District</u>

In April 2005 the Tyler District constructed a short section on SH 31 using the Granite Mountain flexible base. Granite Mountain base is from Arkansas and is not commonly used by TxDOT although it is used extensively for oilfield gravel road construction in east Texas. The reported advantage is that it performs much better as an unsurfaced gravel road than traditional Texas limestone materials. Granite Mountain material is hauled by train from Arkansas and stockpiled in Longview.

Densities were checked with a standard nuclear device. One comment made about all of these bases is that the nuclear test could be problematic with these granular low-fines bases, particularly as they do not retain moisture. When driving the rod for the nuclear gauge, cracks appear in the base, and when removing the rod some disturbance (uplift) of the base is sometimes observed. This result was not a problem on the SH 31 base, but it was a large concern with the section constructed on US 287 in Bryan.

<u>US 287 Bryan District</u>

This section was constructed in the summer of 2005, and it is approximately 2 miles long, stretching from near the intersection with FM 488 to the Trinity River Bridge. The subgrade in the area is very wet; this entire area is next to a large dam and is largely wetlands. The pavement is built up on select fill embankment. The pavement structure initially called for 8 inches of lime-treated subgrade, 10 inches of Grade 1 limestone base, an underseal, and 4 inches of HMA surfacing. However, because of transportation problems the limestone bases (from central Texas) were not available for this project. At the last minute the contractor proposed a change to a Mill Creek granite base from Oklahoma. This new material was supplied at the same cost as the original Grade 1 limestone.

The GPR and FWD data from the base looked reasonable, but concerns were raised about the quality of the HMA surfacing and the permanency of the lime-stabilized subgrade. The current visual condition is very good. This short project did not perform long-term monitoring of these sections, and both sections were around 1 year old at the conclusion of the project. As a minimum it was recommended that visual condition, GPR, and FWD surveys be completed when the sections were 3 years old (in the summer of 2008).

<u>SH 43 Atlanta District</u>

In late 2004 a test section of low fines base was included as part of a rehabilitation project on SH 43 in the Atlanta District. The four-lane section had an existing asphalt surface. All four lanes were overlaid with a minimum of 8 inches of granular base. The final surface of the project was a surface treatment. In the northbound lanes the base used was a traditional crushed limestone from the Beckmann Pit in San Antonio. In the southbound lanes, a low fines base from the Jones Mill pit in Arkansas was placed.

A surface treatment was placed and the initial performance of both sections was good. However, problems arose when the hot weather arrived in late spring of 2005. In both cases shelling of the surface seal occurred and the top of the flexible base was exposed. Substantial rainfall then occurred and additional stability problems (rutting) were noted in the southbound lanes containing the Jones Mill base.

The TTI researchers considered that at least the following factors contributed to the stability problems in the Jones Mill base:

- The initial loss of the surface seal was not base related as it occurred on both bases.
- The cause of the stability problems in the Jones Mill bases was attributed to the bases permeability (as compared to the limestone) and more importantly that the base was placed directly on an existing asphalt surface and the base was not "day-lighted" at the edges. As is normal practice in Texas the existing material was bladed up against edge of the base. The existing shoulder material was clay. This approach in fact caused a bathtub effect, trapping water where moisture entered the base through the surface.
- Typically, Texas limestone bases are practically impermeable, particularly with slush rolling, which causes the fines to rise to the surface.
- The clay contamination of the Jones Mill base certainly contributed to the eventual failure.

Bases designed according to the Item 245 specification, which was developed as a deliverable of this study, will be more permeable than traditional high fines limestone bases. Therefore, it is important to ensure that they are day-lighted at the edge, hopefully minimizing the risk of a bathtub situation. On SH 43 the Jones Mill base was eventually treated with 3 percent cement and it is now performing well.

Additional sections reported in Project Report 0-4358-1 are shown in Table 2.2. Three experimental road sections constructed in Texas between 1997 and 1999 under different projects are included in this study. The base courses of these pavements were constructed to carry high traffic loads and are therefore good candidates for a field study on heavy-duty flexible bases. One of the important features of these sections is that they have been subdivided into smaller test sections with the objective to investigate the performance of different granular bases.

Table 2.2: Additional experimental sections reported in Project 0-4358

Project	District and County	Year Completed	Base Types
SHRP/FHWA SPS-1 Project 4801 US-281	Pharr District, Hidalgo County	1997	Crushed Limestone (CSAB) Dense Graded Aggregate (DGAB) Lime Rock Asphalt (LRA) Caliche (CAL) Crushed Concrete (CCAB)
TxDOT Project 1869 US-77	Corpus Christi District, Nueces County	1997	Yucatan Limestone Lime Rock Asphalt Caliche + lime Caliche + cement
TxDOT Project 7-3931 FM-1810	Fort Worth District, Wise County	1999	Regular Grade 1 Large Stone Gradation

US 281 Pharr District (10 Sections)

These sections were originally part of the LTPP program developed by the Strategic Highway Research Program (SHRP) and the FHWA. The experimental section is located in the southbound lanes of US 281 in Hidalgo County north of McAllen in the Rio Grande Valley. The project site falls in the dry no-freeze climatic zone. The rainfall in this area is very low, averaging less than 12 inches per year. The natural soil is sand; therefore, with the low rainfall and free-draining subgrade soils, moisture intrusion from below is not a concern. The project section is a four-lane divided highway with 12-foot wide lanes and 10-foot wide outside shoulders. The final report on the SPS-1 project indicates that the estimated traffic at that time included 32.8 percent heavy trucks with annual average daily traffic in two directions of 10,180 vehicles. The designs were based on a total of 10 million 18-kip equivalent single axles over a structural design period of 20 years. The road was opened to traffic in April 1997.

The experimental sections evaluated are shown in Table 2.3. FWD testing was done on these sections over time. The change in base modulus over time is shown in Figure 2.2.

Table 2.3: Experimental section on US 281

Section (Cell ID)	Sequence (South)	Pavement Structure	Begin Station (foot × 100)	End Station (foot × 100)
480113	1	4" AC & ACB 8" Dense Graded Aggregate Base (DGAB) 12" Subgrade stabilized with 2% lime	1558 + 00	1551 + 00
480114	5	7" AC & ACB 12" DGAB 12" Subgrade stabilized with 2% lime	1522 + 00	1515 + 00
480160	9	5" AC & ACB 10.5 " Lime Rock Asphalt (LRA) 12" Subgrade stabilized with 2% lime	1460 + 00	1453 + 00
480161	8	5" AC & ACB 8.5" LRA 12" Subgrade stabilized with 2% lime	1468 + 00	1461 + 00
480162	7	5" AC & ACB 8.5" Crushed Limestone Aggregate Base (CSAB) 12" Subgrade stabilized with 2% lime	1482 + 00	1475 + 00
480163	6	5" AC & ACB 10.5" CSAB 12" Subgrade stabilized with 2% lime	1490 + 00	1483 + 00
480164	3	5" AC & ACB 10.5" Crushed Concrete Aggregate Base (CCAB) 12" Subgrade stabilized with 2% lime	1542 + 00	1535 + 00
480165	2	5" AC & ACB 10.5" CCAB 12" Subgrade stabilized with 2% lime	1550 + 00	1543 + 00
480166	10	5" AC & ACB 14" Caliche + ½% lime (CAL) 12" Subgrade stabilized with 2% lime	1452 + 00	1445 + 00
480167	4	5" AC & ACB 14" CAL 12" Subgrade stabilized with 2% lime	1530 + 00	1523 + 00

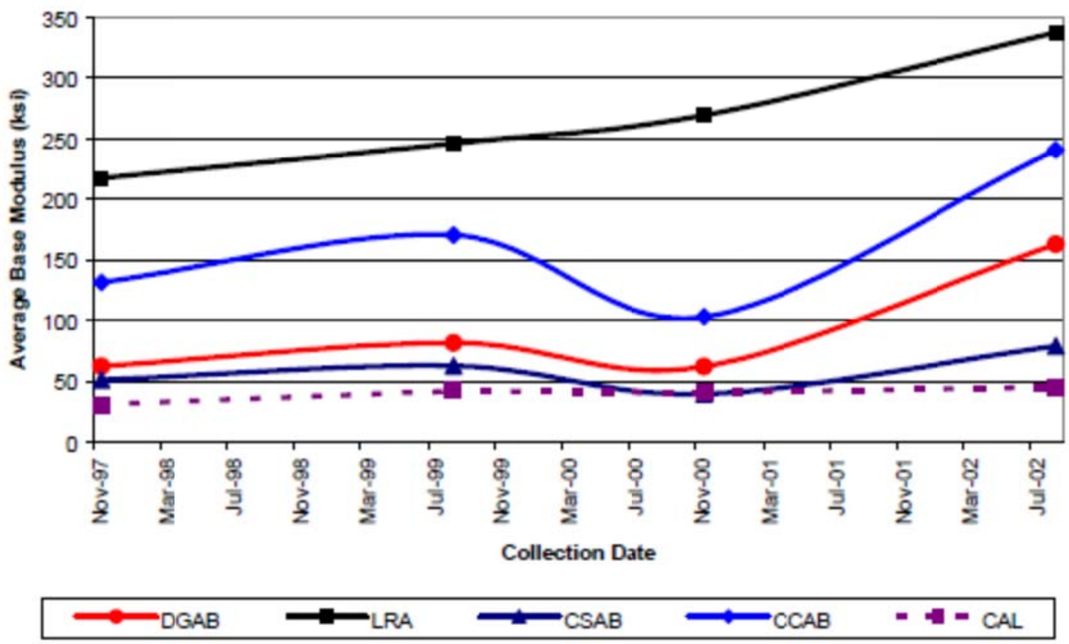


Figure 2.2: Base layer moduli for US 281 sections over time

The following conclusions were reported for the US 281 sections:

- The caliche appears to be the worst performing base; it has the lowest in-place modulus and highest dielectric value. In many areas of the state, designers use 70 to 80 ksi for Class 1 bases when placed on stabilized layers; however, the average value for the caliche is in the 40 to 50 ksi range.
- Both the limestone rock asphalt (LRA) and crushed concrete provide moduli values well above those typically used for flexible base material. The maximum value ever found for normal flexible bases in Texas is in the 100 to 125 ksi range. This supports the conclusion that these bases are "setting up" themselves.
- All of these bases were classified as Class 1 materials but the in situ performances are substantially different. The most probable explanation for this is the self-cement action of two bases and the very low in situ moisture contents of all the bases especially the dense-graded asphalt base (DGAB). These results are clearly impacted by the following factors: (a) the favorable sub-base support (very thick stiff layer), (b) climate (little or no rainfall), and (c) free-draining subgrade (pure sand).

US 77 Corpus Christi District (4 Sections)

These base sections are shown in Table 2.4. Base moduli backcalculated from FWD testing are shown in Figure 2.3 and Figure 2.4. The Corpus Christi District initiated the construction of experimental test sections on US 77 to establish design parameters for pavement structures historically built in this district and to investigate the cost effectiveness of alternative structures with different base types. These test sections would also represent heavy-duty flexible pavements capable of accommodating loads associated with interstate highways. A control section and three treatment sections were constructed and have been monitored since 1997. The four sections are located on US 77 in Nueces County near Robstown. Although this experimental site is within the dry no-freeze climatic zone, it is located near the Nueces and Corpus Christi Bays. Therefore, the influence of moisture may be more pronounced on a micro-climatic scale in comparison with the US 281 site. Each section is approximately 1,000 feet in length and includes the northbound and southbound lanes of the four-lane divided highway. The experimental sections were designed to carry an estimated 15 million equivalent standard axles (ESALs) over a design life of 20 years.

Table 2.4: US 77 sections

Section (No.)	Sequence (North)	Pavement Structure	Begin Station (feet × 100)	End Station (feet × 100)
4	1	8" AC Surface 18" Caliche stabilized with 2% lime 12" Subgrade stabilized with 4% lime	160 + 00	170 + 00
1	2	8" AC Surface 18" Caliche stabilized with 4% cement 12" Subgrade stabilized with 4% lime	170 + 00	180 + 00
2	3	8" AC Surface 18 " Yucatan Limestone 12" Subgrade stabilized with 4% lime	180 + 00	190 + 00
3	4	8" AC Surface 18" Lime Rock Asphalt 12" Subgrade stabilized with 4% lime	150 + 00	160 + 00

Notes:
- All sections across northbound and southbound lanes.
- Section numbers are the original numbers as shown on the plans.
- Section 4 was originally the control section.

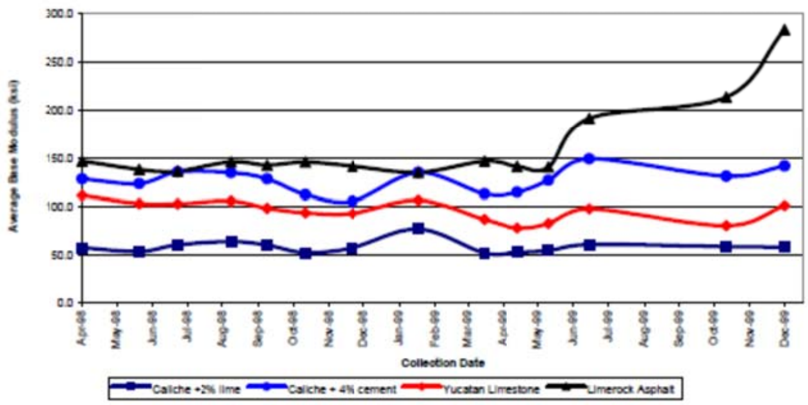


Figure 2.3: Base layer moduli on US 77 from 1998 to 1999

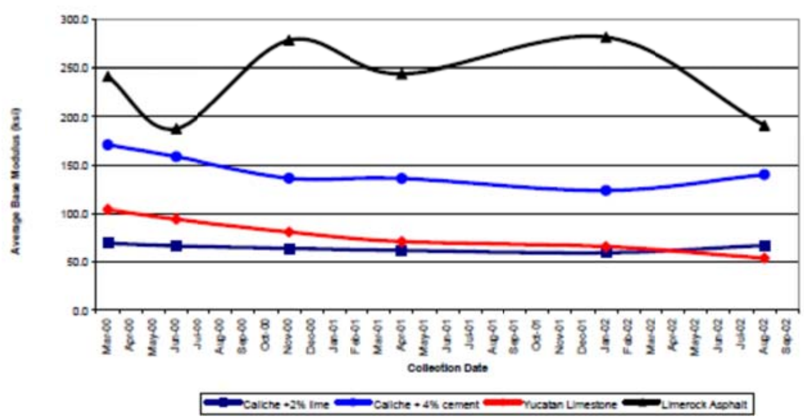


Figure 2.4: Base layer moduli on US 77 from 2000 to 2002

The main conclusions from the nondestructive data presented above are as follows:

- The caliche materials stabilized with lime have the lowest base moduli of all of the materials used.
- The LRA and caliche stabilized with cement have moduli values that are consistently higher than those typically found for flexible base materials in Texas.

The reason for the reduction in field moduli for the Yucatan limestone is not known at this time. **This reduction should be the subject of further study**. The section has developed some longitudinal cracks that are not thought to be load related but may be associated with edge drying. However, these cracks may permit moisture to enter the base layers. The GPR dielectric values of all of the bases have increased over time, but the values are still thought to be reasonable. Clearly, it would be beneficial to take base samples from these sections.

FM 1810 Fort Worth District

Two base sections in Fort Worth District were constructed as shown in Table 2.5. Base moduli measured on these sections over time are shown in Figure 2.5. Research Project 0-3931 was conducted in-house by TxDOT in cooperation with the FHWA from September 1998 to December 2000. Several roadway failures reported in the Fort Worth District were thought to have originated in the flexible base course. According to Research Report 0-3931, this district had not adopted the triaxial classification as part of the specifications of base course materials, and in recent years it was recognized that their standard flexible base requirement did not provide adequate supporting structures for overlying courses. Although the district used the gradation specification of the 1993 Texas Standard Specifications, this situation revealed the detrimental effect that a high fines content can have on the strength of the aggregate mass if it is not indirectly controlled by another means, such as a triaxial test parameter. The current specification allows a variation of up to 35 percent of the fines, which includes the material passing the No. 200 sieve (0.075 mm). The main objective of this research was to investigate the influence of fines on strength and to propose a new gradation envelope. A proposed large stone gradation and a regular gradation were used in the base courses of two experimental sections constructed in August 1999. These sections of FM 1810 are located in Wise County, northwest of Decatur, near Chico. Within the broad climatic regions defined in Chapter 2, this project can be categorized under intermediate freeze-thaw.

The experimental sections of FM 1810 carry a large amount of heavy trucks, as FM 1810 serves as an access route to the Pioneer Quarry. The design was based on an average annual daily traffic of 5,280 vehicles in 2000 and 8,480 vehicles in 2020, with 29.3 percent trucks (which is a low estimate). The 20-year design ESAL was 6.38 million. The road was opened to traffic in September 1999, and researchers estimate that the total ESALs carried to date is approximately 500,000.

Table 2.5: FM 1810 sections

Section (North)	Pavement Structure	Begin Station	End Station
1	7.5" ACP Surface 12" Large aggregate crushed stone 12" Cement stabilized subgrade	1 + 1000	1 + 900
2	7.5" ACP Surface 12" Regular graded crushed stone 12" Cement stabilized subgrade	4 + 000	4 + 800
Note: All sections are located across northbound and southbound lanes.			

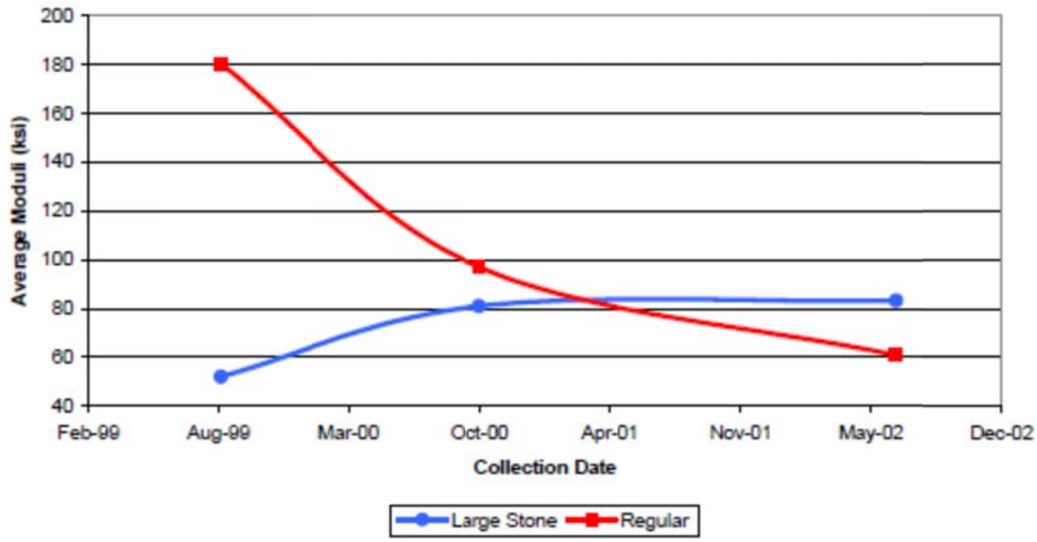


Figure 2.5: Base layer moduli on FM 1810 from 1999 to 2002.

The trends shown in Figure 2.5 are very interesting. The regular base with high minus 200 content (>20 percent) had a very high initial pavement stiffness. Researchers theorize that this trend may be related to the practice of "slush rolling," whereby the process of watering and rolling with a steel wheel roller causes the excess fines to migrate to the upper base. When they dry, they create a dense stiff layer, which was measured with the FWD to have a backcalculated modulus of 180 ksi. However, over time this modulus drops. After almost 3 years in service the average backcalculated modulus has dropped to around 60 ksi. The large stone base shows a different trend. The initial modulus is low, on the order of 50 ksi, but over time this increased gradually to a value of over 80 ksi after almost 3 years. This could indicate that these bases need trafficking to consolidate, and with time they will provide a dense stiff support layer.

2.1.1.4 Perpetual Pavements [TxDOT Project 0-4822]

So-called *perpetual pavements* are designed to limit the horizontal strain levels (~ 100$\mu\varepsilon$) beneath the asphalt layer and the vertical compressive strains (~ 200$\mu\varepsilon$) on top of the subgrade layer to threshold values at which fatigue and rutting failures of these layers are negated. These structures are designed to last 50+ years with a renewable surface that can be milled and replaced intermittently to provide the required functional performance. An asphalt cement (AC) rich bottom layer (RBL) is placed beneath the asphalt structure to mitigate the initiation of bottom-up cracking. A typical structural section as used in Texas is shown in Figure 2.6. The total thickness of HMA used in these pavement structures may vary from 15–20 inches!

PFC(SS3231)	**1.0″-1.5″ Porous Friction Course**	**Sacrificial Layer**
HDSMA(SS3248)	**2.0″-3.0″ Heavy Duty SMA ½″ Aggregate with PG76-XX**	**Impermeable Load Carrying**
SFHMAC(SS3249)	**2.0″-3.0″ Stone-Filled HMAC ¾″ Aggregate with PG76-XX**	**Transitional Layer**
SFHMAC(SS3248)	**8.0″-'variable' Stone Filled HMAC or CMHB, 1.0″-1.5″ Aggregate with PG76-XX**	**Load Carrying Layer**
Superpave (SS3248)	**2.0″-3.0″ Superpave or 3146 ½″ Aggregate with PG64-XX Target lab molded density 98%**	**Stress Relieving Impermeable Layer**
Stabilized foundation	**6.0″-8.0″stiff base or stabilized subgrade. Primarily to serve as construction working table or compaction platform for succeeding layers**	

Figure 2.6: Typical Texas full depth structural sections

TxDOT Project 0-4822 was initiated to perform a structural assessment of perpetual pavements constructed in Texas, to identify strengths and weaknesses in the existing structures, and to provide guidance for future designs. **Problems identified are segregation of stone-filled asphalt layers and debonding between the AC layers.** On a positive note, the stone-filled mixes are considerably stiffer than conventional dense graded AC. The pavements evaluated as part of Project 0-4822 include the first four shown in Table 2.6, which lists the existing perpetual pavements in Texas as of December 2005. Construction details of the first four pavements are shown in Table 2.7. To date all the mixes used for perpetual pavements were designed using the Superpave volumetric design system with 100 gyrations to achieve the 4 percent air voids; the mixes also were required to pass the Hamburg Wheel tracking test.

Table 2.6: Existing perpetual pavements in Texas (as of December 2005)

District	CSJ	Highway	Description
Laredo Zumwalt 1	0017-08-067	IH 35	North of Cotulla TRM 69 +0.44 miles to 74.0, Northbound lanes only Zumwalt Construction Completed 2004
Laredo Gilbert	0018-01-063	IH 35	South of Cotulla; TRM 58 to 65 + 0.36 miles Gilbert Construction Completed early 2003
Laredo Zumwalt 2	0018-02-049	IH 35	Near Artesia Wells TRM 49 + 0.43 miles to 53 + 0.43 miles Zumwalt Construction Completed Summer 2005
Laredo	0018-05-062	IH 35	Loop 20 to Uniroyal Road (approximately 6 miles) Price Construction Project just underway; scheduled for completion in 2007
San Antonio	0016-04-091 0016-04-094	IH 35	In New Braunfels 0.5 mile south of SH 46 to 0.35 m N of FM 306 Hunter Industries Project Underway; scheduled for completion in late 2006
Waco 1	0015-01-164	IH 35	McLennan County from Myers Loop to US 77 (2.2 miles) Young Brothers Completed 2003
Waco 2	0048-09-023	IH 35	North of Hillsboro at "Y" Young Brothers Construction Project Underway scheduled for completion in late 2006/early 2007
Fort Worth	0353-01-026	SH 114	Wise County, TRM 580 + 0.8 miles to the Denton County line, approximately 5 miles Duininck Brothers To be completed early 2006

Table 2.7: Structural section, layer thickness, and PG grade of binder

District	Laredo			San Antonio	Waco		Fort Worth
CSJ Contractor	0017-08-067 Zumwalt 1	0018-01-063 Gilbert	0018-02-049 Zumwalt 2	0016-04-091 Hunter Ind.	0015-01-164 Young Br.	0048-09-023 Young Br.	353-01-026 Duininck
PFC	-	-	-	1.5 (72-22)	**1.5 (76-22)**	1.5 (76-22)	-
SMA	**3 (76-22)**	**3 (76-22)**	**3 (76-22)**	2 (76-22)	**2 (76-22)**	2 (76-22)	2 (76-22)
¾″ SF	**3 (76-22)**	**3 (76-22)**	**3 (76-22)**	2 (64-22)	**3 (70-22)**	3 (70-22)	**2.5 (76-22)**
1″ SF	**8 (70-22)**	**8 (70-22)**	**8 (70-22)**	**12 (64-22)**	**10 (70-22)**	12 (70-22)	**13 (70-22)**
RBL*	**4 (64-22)**	**2 (64-22)**	**3 (70-22)**	**4 (64-22)**	**4 (64-22)**	4 (64-22)	**3 (64-22)**
Foundation	**3% Lime (8 in)**	**3% Lime (8 in)**	**2% Cement + Precrack (8 in)**	**3% Lime (6 in)**	**6 in. Flex Base + Emulsion Over 6% Lime (8 in)**	6 in. Flex Base + 6% Lime (12 ins)	**6% Lime (8 in)**

Bold text indicates structure at time of testing. * where RBL is Rich Bottom Layer

2.1.1.5 Ultra-Thin Bonded Hot Mix Wearing Course (UTBHMWC)—Novachip

UTBHMWC is a surface course composed of a warm spray-applied polymer modified emulsion membrane followed immediately with a hot plant mixed gap-graded paving mixture. This mixture provides a wearing course with a minimum thickness of 1/2 in. for Type A, 5/8 in. for Type B, and 3/4 in. for Type C. It is currently being applied through Special Specifications SS3127 and SS3142.

This mix is placed in a thin lift (< 1 inch) so it is ideal for curb and gutter type installations. It can be constructed relatively quickly requiring only one roller pass. Applications include improving friction and reducing noise, splash, and spray. It has been used on a number of projects in Texas as an alternative to PFC. Some of these projects are shown in Table 2.8.

Table 2.8: UTBHMWC projects in Texas

CSJ	DISTRICT	COUNTY	ROUTE
1685-06-027	Houston	Harris	SH6
0156-03-044	Wichita Falls	Wichita	SH240
0044-02-072	Wichita Falls	Clay	US287
0224-01-054	Wichita Falls	Clay	US287
0441-05-038	Odessa	Reeves	IH10
0140-03-042	Odessa	Pecos	IH10
1068-04-083	Dallas	Dallas	IH 30
0142-02-025	San Antonio	Kerr	IH10
0142-06-026	San Antonio	Kendall	SH 27
0017-14-014	San Antonio	Medina	SH 132
2104-02-028	San Antonio	Bexar	FM 1957
1433-01-026	San Antonio	Bexar	FM 2252

2.1.1.6 Warm Mix Asphalt (WMA) [TxDOT Project 0-5597—Initial Investigation]

The use of WMA has several advantages over conventional HMA. Because its production releases fewer emissions, it is better for workers and can be used in areas where air quality is a concern. WMA doesn't require new technologies, because it is laid like traditional hot-mix asphalt; in fact, it's easier to work with because the mix is less stiff at lower temperatures. Because outdoor temperature isn't as big a concern, the paving season could be extended. One drawback is the additional cost, which may run as much as $5 a ton for the WMA additives.

TxDOT Project 0-5597 provided a synthesis of WMA addressing:

- current state of the art/practice of WMA,
- benefits and costs of WMA technology,
- plant modifications to accommodate certain WMA processes,
- mixture design and analysis,
- pavement structural design,
- durability and performance,
- performance-related testing,
- quality control,
- specifications, and
- construction guidelines.

A summary of findings and recommendations is provided. Also included in this synthesis is a complete documentation of the first warm-mix asphalt field trial conducted by TxDOT. This trial comprised the use of Evotherm on Loop 368 in Bexar, San Antonio in 2006. No reports on the performance of this section are available as yet.

Table 2.9 shows a listing of WMA projects in Texas, indicating where these mixes are paved, the type of HMA, lift thickness, and tonnage as well as the WMA additive or process applied.

Table 2.9: WMA projects in Texas

<table>
<tr><th>District</th><th>Roadway</th><th>Lift Thickness</th><th>Mix Type</th><th>Approximate Tonnage</th><th>WMA Additive or Process</th></tr>
<tr><td rowspan="3">Atlanta</td><td>SH 8</td><td>2 inches</td><td>Type D</td><td>5,000</td><td rowspan="3">Astec D.B. Green</td></tr>
<tr><td>FM 3129</td><td>3 inches</td><td>Ty C&D</td><td>2,700</td></tr>
<tr><td>IH 30</td><td>3 inches</td><td>Type C</td><td>15,000</td></tr>
<tr><td rowspan="3">Austin</td><td rowspan="2">SH 71</td><td rowspan="2">2 inches</td><td rowspan="2">Type C</td><td>7,000</td><td>Evotherm</td></tr>
<tr><td>-</td><td>Control</td></tr>
<tr><td>IH 35</td><td>2 inches</td><td>SMA C</td><td>16,500</td><td>Evotherm</td></tr>
<tr><td rowspan="2">Beaumont</td><td>IH 10</td><td>1.5 inches</td><td>SMA C</td><td>8,000</td><td>Redi-Set WMX</td></tr>
<tr><td>US 190</td><td>2 inches</td><td>Type C</td><td>40,000</td><td>Redi-Set WMX</td></tr>
<tr><td>El Paso</td><td>SH 118</td><td>2 inches</td><td>CMHB-F</td><td>20,000</td><td>Terex</td></tr>
<tr><td rowspan="7">Fort Worth</td><td rowspan="2">BU 287</td><td>3 inches</td><td>Type B</td><td rowspan="2">52,600</td><td rowspan="2">Evotherm</td></tr>
<tr><td>2 inches</td><td>Type D</td></tr>
<tr><td>FM 1220</td><td></td><td>Type D</td><td>20,000</td><td></td></tr>
<tr><td rowspan="2">FM 1938</td><td>2 inches</td><td>Type D</td><td>20,000</td><td rowspan="2">Evotherm</td></tr>
<tr><td>8 inches</td><td>Type B</td><td>22,000</td></tr>
<tr><td rowspan="2">FM 156</td><td>2 inches</td><td>Type D</td><td>11,000</td><td rowspan="2">Evotherm</td></tr>
<tr><td>10 inches</td><td>Type B</td><td>3,600</td></tr>
<tr><td>Lufkin</td><td>FM 324</td><td>1 inch</td><td>Type D</td><td>3,800</td><td>Advera
Akzo Nobel Rediset
Evotherm
Sasobit
Control</td></tr>
<tr><td>San Angelo</td><td>US 83</td><td>2 inches</td><td>CMHB-C</td><td></td><td></td></tr>
<tr><td rowspan="3">San Antonio</td><td rowspan="2">Loop 368</td><td rowspan="3">2 inches</td><td rowspan="3">Type C</td><td>1,200</td><td>Evotherm</td></tr>
<tr><td>-</td><td>Control</td></tr>
<tr><td>IH 37</td><td>50,000</td><td>Evotherm
Sasobit</td></tr>
</table>

District	Roadway	Lift Thickness	Mix Type	Approximate Tonnage	WMA Additive or Process
Waco	SH 6	4 inches	Type B		Astec D.B. Green
	US 290		SMA-F		
Wichita Falls	US 380	2 inches	Type D	75,000	Astec D.B. Green
	US 82	1.5 inches	Type D	8,000	Astec D.B. Green
	US 183	2 inches	Type D	35,000	Astec D.B. Green

2.1.1.7 Asphalt Rubber Projects

Texas has a long history of utilizing asphalt rubber in construction and rehabilitation of pavements. The first reported use of asphalt rubber in Texas was in 1976 by the Bryan and El Paso Districts. In an extensive research project conducted by Texas A&M University in 1982 (Schuler), the researchers evaluated performance of nearly 800 miles of seal coat and underseal constructed throughout Texas from 1976 and 1981 (Project 287-2). The researchers concluded,

> asphalt rubber binders can be effectively used in seal coat construction to reduce alligator cracks and raveling when compared to conventional seal coat performance. However, shrinkage cracking and flushing performance is respectively equal and less desirable than conventional seal coat performance.

The research conducted at Texas A&M University also identified some of the shortcomings in design and construction practices which may have led to less than desirable performance with respect to flushing.

Asphalt rubber is the produced through the blending process known as the "wet process." This process involves on-site blending of at least 15 percent crumb rubber with asphalt cement. In Texas, asphalt rubber has been used in four different types of applications:

- Chip Seal Coat: In this application, asphalt rubber is used as the binder for the seal coat, which is the finished pavement layer. This application is also known as SAM (Stress Absorbing Membrane).
- Underseal: In this application, asphalt rubber is used as the binder for chip seal application. After construction of a chip seal layer, an asphalt overlay is applied on the chip seal layer. The function of this underseal is to waterproof the existing pavement and retard reflective cracking. This application is known as SAMI (Stress Absorbing Membrane Interlayer).
- Hot Mix: Asphalt rubber is used as the binder for hot mix.
- Porous Friction Course: Asphalt rubber is used as the binder for open graded PFC.

TxDOT has constructed several asphalt rubber projects in each of the previously mentioned categories. Some of these are shown in Table 2.10 through Table 2.12.

Table 2.10: Asphalt rubber seal coat projects in Texas

Date	District	County	Highway	ADT	Project	CSJ	Location
92	Bryan	Madison	FM 247	1,250	CPM-475-3-44,etc	0475-03-044,etc	From Midway on FM 247 about 9.5 miles south to Bedios creek. One section on IH 45, about 2 miles south and 2 miles North of SH 21. This section may be SAMI (CSJ 0645-05-034)
92	Bryan	Madison	OSR, etc	470	CPM-475-3-44,etc	0475-03-044,etc	There are 9 different locations. Main locations on OSR From IH 45 to Midway about 14 miles. One section on IH 45, about 2 miles south and 2 miles North of SH 21. This section may be SAMI (CSJ 0645-05-034)
96	Childress	Donely	US 287	7,200	NH 96(11)R,etc	0042-07-045, etc	US 287 South Bound, From Clarendon to Hedly
89	El Paso	Presidio	US 90	770			Between Alpine and Marfa
95	El Paso	Culberson	IH 10	9,000			Between Van Horn and IH 10/ IH 20 split
96	El Paso	El Paso	SH 20,etc	3,300	CPM2-3-16 etc	002-03-016, etc	From Fabens S for about 12 miles
96	El Paso	El Paso	FM 76	17,800		674-01-xxx	From Loop 375 to FM 1109
2000	El Paso	Culberson	SH 54	210			From Van Horn North to Guadalupe Mountain
95	Odessa	Crane	US 385	1,800			Between Crane and McCamey
87	San Antonio	Medina	SH 173	1,350		0421-02-xxx	Devine, From IH 35 S for 5 miles
88	San Antonio	Bexar	FM 1604	15,800			From SH 151 to US 90, AR chip seal is in turn lane, Info from S. Cox
91	San Antonio	Bexar	SP 536	12,300	CPM 17-1-31	0017-11-031	North of Bank Street
96	Wichita Falls	Montague	US 287	13,200	CSR 224-3-49	0224-03-49	US 287 from FM1125 just S. of Bowie North to Montague/ Clay Co. line
93	Amarillo	Potter	FM 1061	1,600	SMERP 4800-00-011	SMERP 4800-00-011	0.75 mile E of FM 2381 to 2 mile E
93	Odessa	Ector	FM 181	1,650	SMERP 4800-00-012	SMERP 4800-00-012	Andrews Co. Line to 5.5 Miles N of SH 158
93	Odessa	Martin	SH 349	1,800	SMERP 4800-00-013	SMERP 4800-00-013	Near FM 87 to Dawson Co. Line, Ref Marker 300 to 302, 48F06H
93	Brownwood	Brown	US 67	3,900	SMERP 4800-00-013	SMERP 4800-00-013	Blanket Creek Bridge to 1.0 Mile N.
93	Brownwood	McCullouch	US 377	2,100	SMERP 4800-00-013	SMERP 4800-00-013	1.0 Mile N. of FM 2996 S. to FM 2996

Table 2.11: Asphalt rubber HMA projects in Texas

Date	District	County	Highway	ADT	Project	CSJ	Location
92	Lubbock	Parmer	US 84	3,300	CPM 52-1-28	52-1-28	From Lariat to Farewell, has been seal coated due to ravelling
92	Wichita Falls	Wichita	FM 369	14,900	CRP 91(42)M	802-02	Southwest Parkway, From Ray Road to the Stadium
93	Abilene	Callahan	SH 36	5,400	STP 93(115)RM	0181-02-021	From Taylor Co line 6.8 Miles south east near FM 603
93	Lufkin	Angelina	SH 63	3,000	STP 93(142)R	0244-01-040	From FM 2743 to the Jasper County Line
94	Lufkin	Nacagdoches	US 259	17,000		138-6-XX	From US 59 North
94	Beaumont	Hardin	US 96	30,000		65-05-117	North side of Lumberton from West Chance Cutoff to Village Creek
95	Houston	Fort Bend	FM 1994	1,300	AR 1965-1-5	1965-001-005	From SH 36 North to near intersection of FM 762
94	Odessa	Ector	IH 20	13,200	IM20-1(122)111	0004-07-087	From Midland County line to West of FM 1936
98	Odessa	Crane	SH 385	4,700	STP 97(291)R	0600-03-016	Starts S of Crane city limits, goes N to Crane Co. Line
99	Abilene	Nolan	IH 20	15,060	CPM 6-2-91		From Sweetwater to West of Roscoe, 9.3 Miles East Bound Lanes only

Table 2.12: Asphalt rubber PFC projects in Texas

Date	District	County	Highway	ADT	Project	CSJ	Location
94	Odessa	Midland	IH 20	11,200	IM 20-1(124)154	0005-03-054	IH-20 loop around Stanton. From Intersection of BI-20 West of town to 1 mile East of intersection of BI-20 East of town.
95	Odessa	Martin	IH 20	11,100	IM 20-1(129) 158	0005-04-055	From end of CSJ 005-04-0054 to Howard County line
95	Lufkin	Polk	SH 146	8,300	STP 95(85)HEC,etc	0388-01-035,etc	South of Livingston. From Intersection of US 190 South to FM 1988
2000	Odessa	Ward	IH 20	5,900		0004-04-073	From E of Monahans W to Pyote. First half
2000	Odessa	Ward	IH 20	5,700		0004-04-075	From E of Monahans W to Pyote. second half

2.1.1.8 Full-Depth Reclamation/Recycling Stabilization (FDR) [TxDOT Project 0-4182]

Rehabilitating an old pavement by pulverizing and stabilizing the existing pavement is a process referred to as *full depth recycling*. The stabilized layer becomes either the base or sub-base of the new pavement structure. Types of stabilizers and the frequency of TxDOT district use are shown in Figure 2.7.

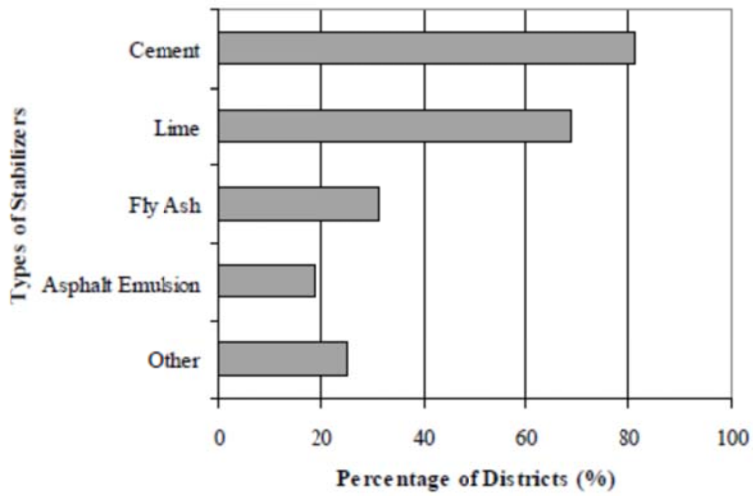


Figure 2.7: Stabilizers used by TxDOT

TxDOT Project 0-4182 evaluated the performance of various pavements throughout Texas that used stabilizers to strengthen the base/sub-base materials. Pavements were identified in the following districts:

- **Bryan (39 sections)** has primarily used either cement or lime to upgrade its Farm-to-Market (FM) network. The subgrades in this District are varied, but it has large areas of highly plastic clays. The existing FM road network is very thin (typically 6 inches of flexible base) and has multiple surface treatments. The District has high rainfall, high humidity, and relatively warm/wet winters.
- **Lubbock (10 sections)** has made widespread use of fly ash to treat its low-volume FM roadways. This District has relatively good subgrades, low humidity, and low rainfall.
- **Amarillo (8 sections)** has used a variety of stabilizers with mixed success. Although it found problems with calcium-based stabilizers, the District has several sections constructed with asphalt emulsions that are reported to be performing well. The District has relatively good subgrades and low humidity, but it does experience cold winter weather with several freeze/thaw cycles each year.
- **Childress (17 Sections)** largely uses fly ash to treat its roadways. This District uses FDR techniques for both high-volume and low-volume roadways. The subgrades in this District are good due to low humidity.
- **Yoakum (FM 237 Emulsion treated)** largely uses lime to rehabilitate its FM network. The researchers selected this District because it constructed an experimental section in which the traditional lime treatment was compared with an asphalt emulsion section. The District has typically fair subgrades and high humidity.
- **Waco (4 Sections)** has not made widespread use of FDR techniques, but it did construct an experiment on a FM roadway in which four different treatments were compared.

2.1.1.9 Case Study: Sulphate Heave

Gypsum is a major source of sulfate that produces sulfate-induced heave in lime-treated soils beneath roads and other paved structures. This deformation of pavement subgrade is known to result from the growth of the basic hydrous calcium aluminum sulfate mineral, ettringite, or a silica-bearing analog, thaumasite. These materials are highly expansive when wetted.

The pavement on FM 734 (Parmer Lane) from Samsung Road to Harris Branch in the Austin District experienced severe distresses (shown in Figure 2.8). Frequent swells and dips were evident, due to sulfate heave and seasonal expansive soil swelling. In addition, the pavement was experiencing longitudinal shrinkage cracking with cracks as wide as 1 inch and deeper than 5 feet caused by shrinkage of expansive soil during the drought in 2006.

Figure 2.8: Shrinkage cracking distress on FM 734

The existing pavement consisted of 10 in. HMA, 12 in. flexible base, and 8 in. lime-treated subgrade. The natural subgrade, underlying the pavement, is a highly plastic soil (PI >35), and has high amounts of organics and sulfate contents. A test pit indicated shrinkage cracking extending into the subgrade as shown in Figure 2.9.

Based on the results of trial sections, the use of a geogrid and a RAP sub-base was recommended to address the heaving problem. The final pavement structure used for rehabilitation is shown in Figure 2.10. It was found that the RAP sub-base promotes better interlock and performance of the geogrid placed between RAP sub-base and cement-treated base.

Microcracking was used to address potential shrinkage cracking of the cemented base layer.

Figure 2.9: Reflective shrinkage cracking seen in test pit

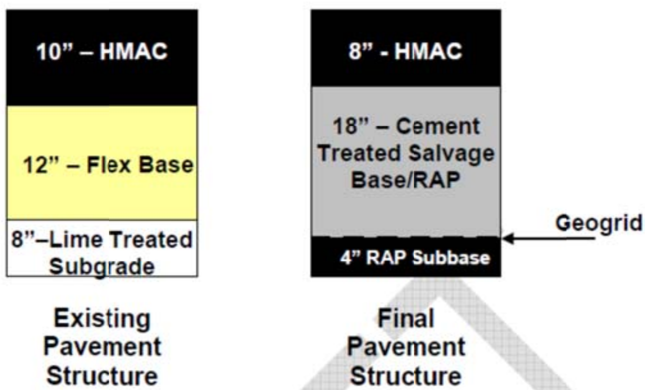


Figure 2.10: FM 734 pavement redesign

Micro-cracking is a construction process used to reduce the potential for reflective cracks in pavements that have cement-treated bases (CTB). Any cement-treated material will shrink slightly as it cures and gains strength. Thin cracks in a cement-treated base occur naturally every 20–40 feet as the result of this shrinkage. The objective of micro-cracking is to induce hundreds of tiny cracks to accommodate the shrinkage, and hopefully prevent formation of individual cracks that have the potential to reflect up into the surface layer.

Micro-cracking is accomplished by loading the CTB with a vibrating roller approximately 2 days after construction. About four passes of the roller will complete the process. This action does not permanently damage the cement-treated base, because it is still “green” and will regain the strength lost due to the micro-cracking procedure.

For the FM 734 section, once the test section had been compacted to specification requirements and wet cured for 48 hours, additional rolling was applied by 12-ton vibratory rollers. The roller traveled at a speed of 2 mph, vibrating at maximum amplitude. After each pass and 100 percent coverage was achieved, the moduli were tested to determine the overall reduction in moduli from its 48-hour wet cure state. It was determined that two passes achieve the 40 percent reduction in moduli, as demonstrated in Figure 2.11. As shown in Figure 2.12, the stiffness of the base increases over time, with a substantial increase in modulus 14 days following the micro-cracking.

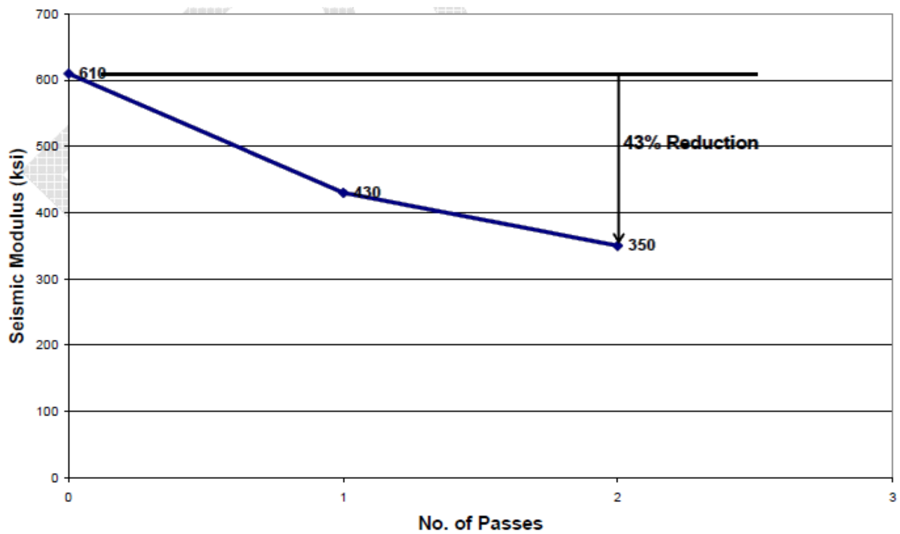


Figure 2.11: Reduction of base modulus with micro-cracking

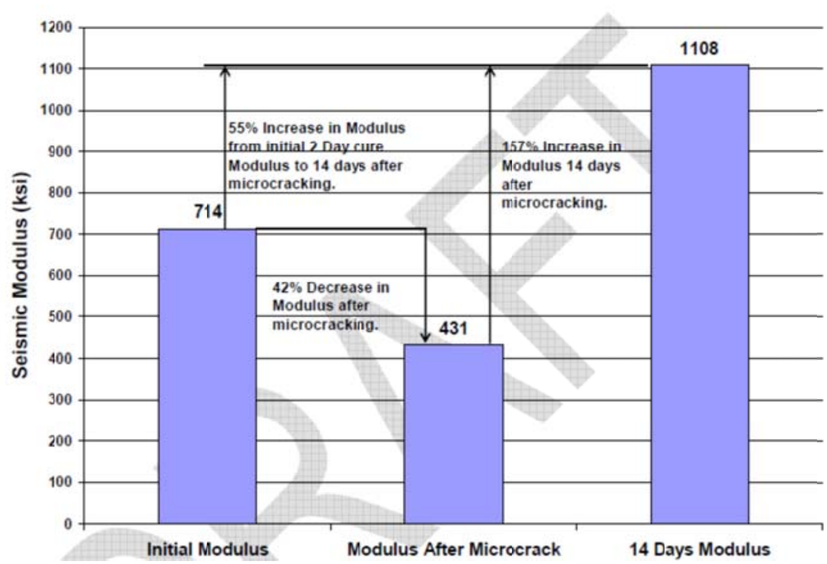


Figure 2.12: Overview of micro-cracking effectiveness

TxDOT has explored alternatives to the traditional use of lime or cement to stabilize soils with high sulfate concentrations. One project is using a soil mix design that is a combination of 4 percent Ground Granulated Blast Furnace Slag (GGBFS) and 1 percent hydrated lime. The GGBF slag/lime mixture is added into the existing subgrade soil in an effort to provide soil stabilization and mitigate the potential for sulfate induced heave.

The performance of the FM 734 section is being monitored by the Austin Area Office to evaluate the effectiveness of the rehabilitation applied. Some localized cracking of the section has been reported.

2.1.1.10 Case Study: Jointed Concrete Pavement Rehabilitation

Reflective cracking through Jointed Concrete Pavement (JCP) overlays has been a persistent problem. Texas has used several different rehabilitation strategies. The Design Division has evaluated the performance of strategies used for rehabilitating JCP in the past 10 years. A number of sections were evaluated using products and materials, including:

- Strata®, Petromat Fabric Underseal, and Control Sections on SH 3
- Crack Retarding Grid on IH 45
- Seven Different Strategies on US 59—(a) full depth repair, (b) break-and-seat, (c) crushed stone base interlayer, (d) open-graded AC interlayer (Arkansas mix), (e) SBS Modified interlayer plus a 75mm overlay, (f) dense graded overlay, and (g) thin dense graded overlay.
- Crack Retarding Grid vs. Crumb Rubber Modified Asphalt on US 96
- Crack Retarding Grid vs. Control on US 59
- Strata® vs. Crack Retarding Grid on US 175
- Crack Retarding Grid vs. Petromat Fabric Underseal on US 377

The following lessons learned are cited:

- The performance of the crack retarding grid has been disappointing. It has caused several premature failures through debonding. The small openings in the crack retarding grid and the lack of an effective bond may be the causes of the debonding.
- The flexible base and the Arkansas (large stone) mix have performed well. The flexible base was able to absolve the joint movement and retard the reflective cracking. Recommend Texas Triaxial class 1 material or better with classification of “Good” in suction/dielectric tests. Also, it is advisable to have the moisture content of flexible base 1–2 percent below optimum.
- Strata® has been performing well over 2 years of monitoring. In the control section of that project, all cracks reflected through in the first year. Petromat fabric underseal has been performing adequately to retard reflective cracking. The cost of Strata® is 10–20 times higher than the Petromat fabric underseal.
- The break-and-seat section was the worst performing on US 59, due to weak subgrade support. The subgrade was unable to support cracked concrete, which led to a rocking action under traffic loads. For future projects, this method should not be applied on subgrade with DCP penetration rate exceeding 0.98 in. (25 mm) /blow.

2.1.1.11 Case Study: Long Term Pavement Performance (LTPP) SPS-5 Section in Texas

Texas has a number of LTPP sections. One of these sections, SPS-5 on US 175, is particularly interesting because it provides evidence of the benefits of specific rehabilitation strategies that may be applied to reduce reflection cracking of HMA pavements over lime stabilized layers. The purpose of the SPS-5 experiment is to compare the effectiveness of rehabilitation treatments for thin and thick overlays, constructed with virgin material and material incorporating RAP, on milled and non-milled surfaces.

The 20-year design traffic (2007–2027) for US 175 was 12.7 million ESALs with an annual average daily traffic (AADT) of about 34,000 vehicles. The asphalt overlay surface varies in thickness (4 in. and 7 in.) and is paved over 12 inches of lime stabilized base and 18 inches of lime stabilized subgrade. Eight test sections representing different combinations of the three design factors are placed adjacent to each other for comparison. These sections were constructed in 1991.

Lime stabilization of the base and sub-base layers provides a sound pavement foundation but can be prone to cracking. After 18 years of service, the RAP sections on SPS-5 have performed well. Dr. Dar-Hao Chen (TxDOT) did an analysis of these sections from which he concluded that (1) milling of existing pavement does not reduce reflective cracking if existing cracking is not completely removed, (2) a thicker overlay does contribute to transverse cracking resistance, and (3) on average, the pavement life for RAP sections are approximately 0.4 times as long as virgin AC sections. The results from the SPS-5 study also suggest that a flexible mix will be able to bridge over the cracked surface. An interesting finding from the study was that even after 17 years of service, overlay crack testing of the AC overlay with the virgin asphalt mix indicated that this mix still maintains excellent resistance to reflective cracking, as the specimens exceed 2,000 cycles to failure. This is considerably more than 750 cycles in the overlay crack tester typically used to indicate crack resistant mixtures. This was attributed to three factors: (1) the mix has a high AC content, (2) the mix was compacted to high density (96–99 percent), hence low air voids to reduce oxidative aging of the binder in the mix, and (3) the mix incorporates aggregates with low absorption. The aggregates used on the SPS-5 sections all have water absorption less than 0.75 percent. Following are other conclusions from the study:

- The virgin sections performed better than the RAP sections, although both were acceptable.
- Thicker overlay sections perform better in terms of reducing roughness and reflective cracking.
- The current Hamburg requirements are probably conservative.
- No significant difference was found between the milled and non-milled sections.

2.1.1.12 Case Study: Compost

Compost has been in use by 22 of the 25 TxDOT Districts for promoting vegetation growth and to control erosion of embankments with excellent results. Composted material may refer to animal manures, municipal wastes (solid waste and wastewater sludges), and other waste materials, as well as application of the composted materials mixed with different soils (composted manufactured topsoil) (TxDOT Project 0-4403).

More recently, Texas has also used compost materials as a means to reduce shoulder cracking as Figure 2.13 shows. The reader should refer to Kirchoff et al (2002) for details on the process.

(a) Control Plot (b) Dairy Manure as compost

Figure 2.13: Use of compost to reduce shoulder cracking

2.2 Selected Candidate Sections and Case Studies

The review of past TxDOT projects and survey responses indicated a wide variety of different sections and case studies that could potentially be included in the study database. Selection of specific projects that could be documented and evaluated was necessary. The following criteria were used to recommend specific projects for inclusion in the study:

- Statewide impact
- Potential economic impact
- Promising technologies
- Proven technologies (including successes and failures)
- Data for long-term performance studies
- Sections not being tracked by other studies
- Important lessons

It was recommended that the study consider two applications: (1) collection of project specific and performance data for experimental sections and (2) documenting case studies of successful/failed projects. The first application will provide a database to track and query specific experimental sections and the case studies will outline lessons learned and provide guidance that will serve to inform TxDOT personnel in the correct application of particular technologies.

Within the time and budget constraints of the current study, the research team recommends tracking the following experimental sections:

- Crack Attenuating Mixes.
    - 4 sections + 2 control sections
- Warm Mix Asphalt.
    - 4 sections + 2 control sections
- Perpetual Pavements.
    - 4 sections + 2 control sections

The specific sections to evaluate were decided in conjunction with TxDOT. It was further recommended to document the following case studies:

- NCAT Section.
- Compost Sections.
- Selected LTPP Studies: SPS-5.
- Thin and thick overlays; with and without RAP; milled and non-milled surfaces.

Chapter 3. Data Analysis

A very important component of this research involves the analysis and comparison of different project alternatives or research products that are used in different pavement-related projects. This chapter introduces recommended methods or tools that can be used in this regard.

Several tools are used to compare the performance of different technologies and methods applied to the design and construction of pavement sections. The comparison could focus on evaluating the performance of the pavement structures after a given amount of time (e.g., during the design period). This approach will always result in selecting the pavement technologies that provide the highest performance. However, this approach (looking only at pavement performance) has no regard for the costs involved in constructing and maintaining the different alternatives (Prozzi and Hong, 2006). Therefore, it might be the case that the best performing pavement technologies are inefficient due to high or excessive costs, rendering that approach invalid.

Thus, the research team decided that the best approach to comparing different alternatives should involve economic considerations, not only of design and initial construction activities, but also maintenance and rehabilitation activities as well. With this in mind, Life Cycle Cost Analysis (LCCA) was selected as the recommended economic tool for comparing various pavement technologies.

3.1 Life Cycle Cost Analysis

LCCA is generally based on economic indicators such as the Benefit/Cost (B/C) ratio, the Internal Rate of Return (IRR), the Net Present Value (NPV), or the Equivalent Uniform Annual Cost (EUAC). The B/C ratio corresponds to the discounted benefit divided by the discounted cost from a project. A B/C ratio greater than one indicates that the benefits exceed the costs. However, there is generally difficulty in differentiating the benefits from some costs in applications such as pavement analysis. Therefore, B/C analysis is generally not recommended for LCCA.

The IRR corresponds to the discount rate that will allow the benefits to be equivalent to the costs. This indicator is generally more applicable to industry applications where the effect of discount rate is evaluated.

Finally, the NPV is the discounted value of the expected costs or benefits of a given project. It is computed by assigning monetary value to the different costs and benefits that will be incurred during the life of a pavement project and can be estimated by subtracting the sum of discounted costs from the sum of discounted benefits (Walls and Smith, 1998) as follows:

$$NPV = PV_{Benefits} - PV_{Costs} \tag{3.1}$$

In this approach it is expected that the benefits provided by the different pavement alternatives are the same. Therefore, the benefits component of Equation 3.1 can be removed from the equation and the NPV can then be estimated as follows:

$$NPV = \text{Initial Costs} + \sum_{k=1}^{N} \text{Rehabilitation Cost}_k \left[\frac{1}{(1+i)^{n_k}} \right] + \text{SalvageValue} \left[\frac{1}{(1+i)^{n_s}} \right] \tag{3.2}$$

where "i" corresponds to the discount rate and "n" corresponds to the year in which the expenditure was made. The superscript n_k represents year k while n_s represents the end of the analysis period or the year in which the salvage value is calculated. The concept of NPV is summarized in Figure 3.1.

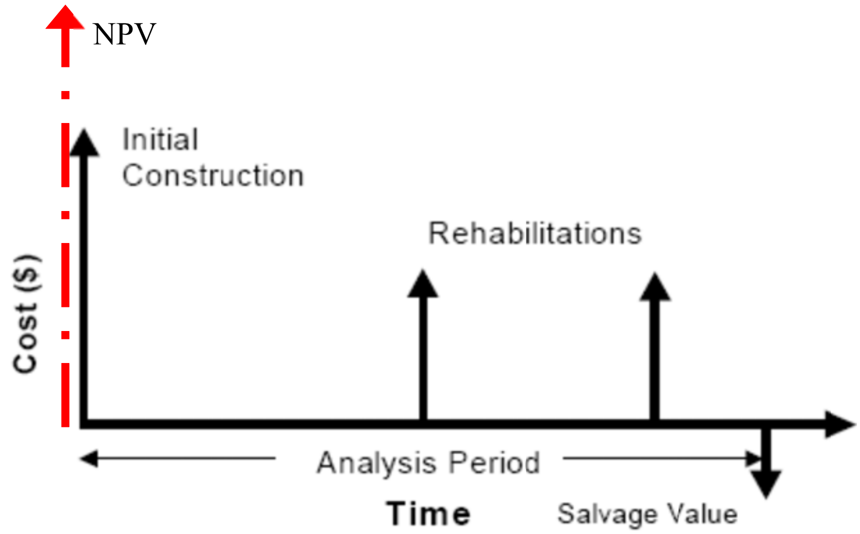


Figure 3.1: Distribution of costs for a given project.

The NPV can also be expressed by means of the EUAC which corresponds to all the costs and benefits associated to the NPV as if it occurred uniformly throughout the service life of the pavement structure. Therefore, the EUAC can be estimated from the NPV as follows:

$$EUAC = NPV\left[\frac{(1+i)^n}{(1+i)^n - 1}\right] \quad (3.3)$$

3.1.1 Estimated Costs and Discount Rates

The estimate of costs and benefits can be obtained based on constant (real) dollars or based on nominal dollars. Constant dollars represent dollars that have the same amount of purchasing power over time; therefore, using constant dollars and activity that costs $1 today will still cost $1 in 1 or 10 years from now. On the other hand, nominal dollars account for dollar values that fluctuate with time. Using nominal dollars, the cost of an activity in the future will be a function of the cost of the activity today and the year in which the activity will be performed.

LCCA can be performed using both constant and nominal dollars. However, it is important to note that the analysis has to be based on only one of them, and constant and nominal dollar values cannot be mixed.

The same applies to the discount rates. Constant discount rates reflect the actual time value of money with no inflation and should be used with non-inflated values for the future costs. Nominal discount rates account for inflation and should be used with inflated values for the future costs.

Discount rates can vary significantly and have an important effect on the estimates of the LCCA. Therefore, reasonable values should be selected based on observation of extended historical trends. The FHWA (Walls and Smith, 1998) recommends using a discount rate between 3.0 to 5.0 percent. However, their analysis is based on data observed between March 1991 and August 1996.

The research team has collected economic data for the period ranging from 1990 to 2010. Figure 3.2 displays the historical trends on 10-year treasury notes. On the figure, the upper line (10-year Treasury Note) represents the nominal rate of return while the lower line (Real Treasury Interest Rate) corresponds to the return rate adjusted due to inflation. Note that the FHWA recommendation is valid for the period prior to 1996. However, for recent years a value in the order of 3.0 to 3.5 percent is more appropriate.

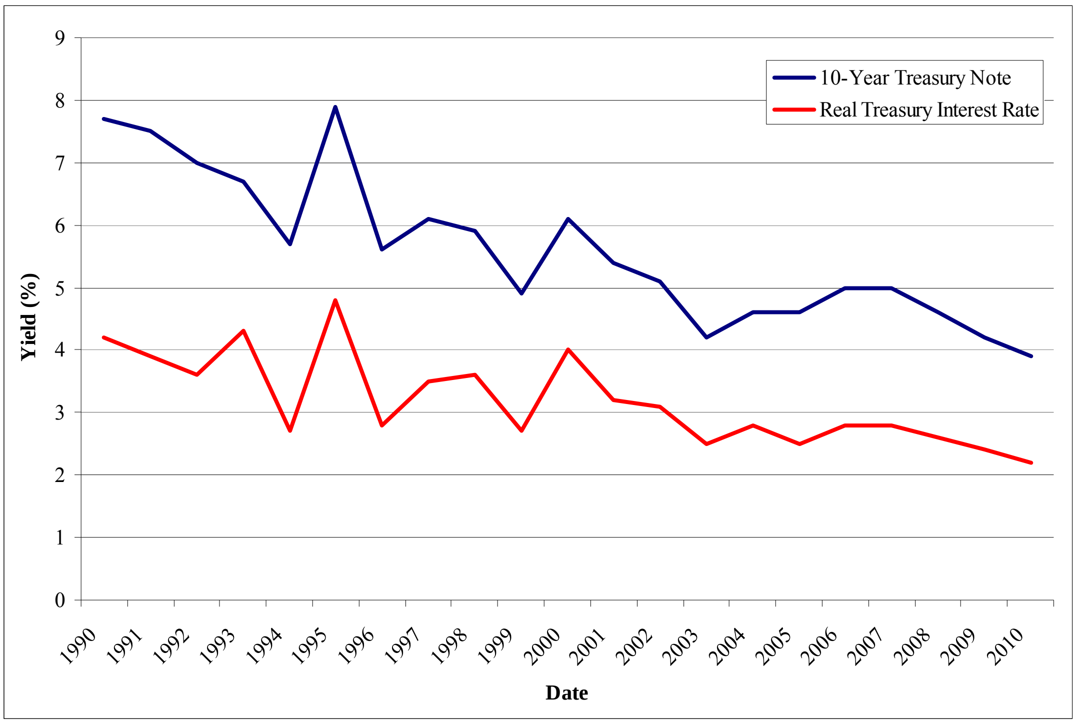


Figure 3.2: Historical trends on 10-year treasury notes

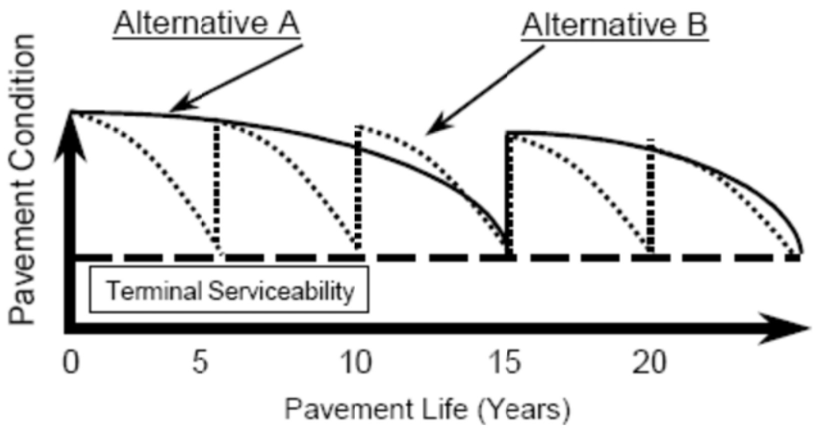
3.1.2 Analysis Procedure

Although the concept behind any LCCA is the same, the models used in estimating costs can differ. For this purpose, the research team has decided to follow the FHWA approach to LCCA. The FHWA approach is consistent with current pavement practices and is flexible enough to allow project specific input parameters. Additionally, the FHWA has developed the RealCost software package that can be used to estimate the LCC of different alternatives.

The FHWA's LCCA procedure can be summarized in the following steps (Walls and Smith, 1998):

- Develop alternatives to accomplish the performance goal of the project.
- Define schedule of initial and future activities involved in implementing each alternative (Figure 3.3).

- Estimate costs of the activities (direct costs + user costs).
- Form the projected life-cycle cost stream for each design alternative.
- Convert the costs to present dollars and sum for each alternative (discounting).
- Determine which alternative is more cost effective.



Source: Walls and Smith, 1998

Figure 3.3: Programming of activities for the different alternatives to be analyzed

As can be inferred from the previous steps, LCCA is a tool used that can be used by the pavement engineer to compare the relative costs of new construction, maintenance, and rehabilitation strategies of different alternatives. The purpose of the LCCA analysis is to identify the alternative with the lowest relative cost, while accounting for all agency-related costs and user-related costs (which can amount to very large amounts, sometimes exceeding the actual agency costs). Finally, as was previously highlighted, the LCCA analysis assumes that the benefits provided by the different alternatives are the same; therefore, the analysis focuses in comparing the differential costs between the different alternatives to be analyzed.

It is also important to consider that the LCCA might be very sensitive to parameters such as the discount rate, or many of the required input parameters that may not be fixed values but that may follow given distributions (e.g., traffic). Because of this reason, LCCA might be performed in two distinct ways: deterministic and probabilistic. In the deterministic approach, the LCCA input variables are considered as fixed and discrete values. Consequently, a single life-cycle cost is determined for each alternative. On the other hand, in the probabilistic analysis, the LCCA input variables can be assigned probability distributions. In this case, the life-cycle costs distributions for each alternative can be determined.

3.1.3 RealCost Software

RealCost allows for LCCA analysis following the FHWA models. The easy-to-use software is implemented in Excel by means of macros and includes a user-friendly interface. RealCost allows the pavement engineer to compare two alternatives at a time; however, because the input files for any alternative can be saved, unlimited number of alternatives can then be evaluated.

The implementation of the LCCA analysis is performed as per Table 3.1.

Table 3.1: Implementation of LCCA in RealCost

LCCA Step	RealCost
1. Establish Design Alternatives	• Project Details (general description) • Analysis Options (units, analysis period, discount rate, include specific costs) • Traffic Data (AADT, % trucks, % growth, traffic flow properties) • Traffic Hourly Distribution • Value of User Time • Added Vehicle Time and Cost
2. Determine Activity Timing	• Alternative 1 and 2
3. Estimate Agency and User Costs	• Alternative 1 and 2
4. Compute Life-Cycle Costs	• Deterministic Results • Simulation
5. Analysis Results	• Deterministic Results • Probabilistic Results

The main graphical interface for RealCost is shown in Figure 3.4. Note that the program can account for user costs. The user costs are based on the FHWA's work zone user cost calculation method, which evaluates traffic demand to roadway capacity on an hourly basis, indicating expected traffic conditions.

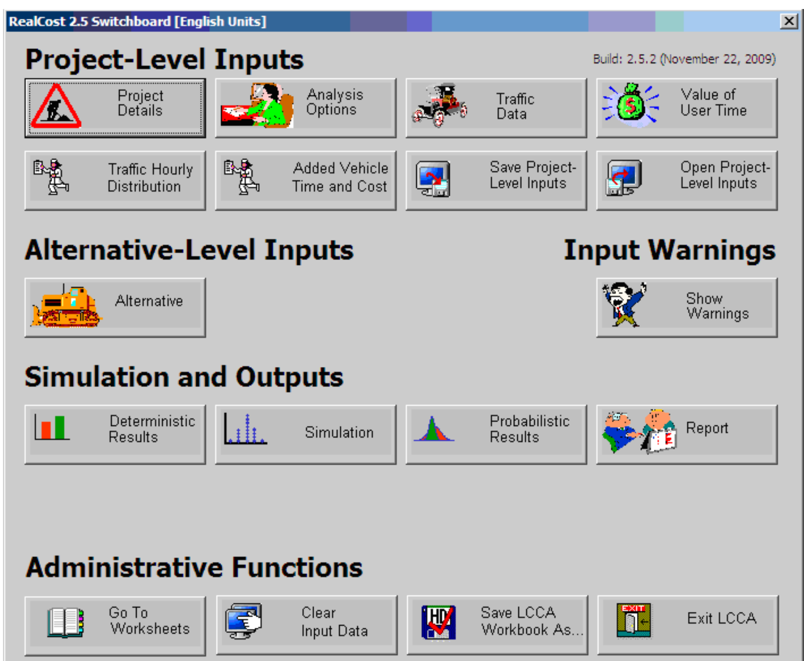


Figure 3.4: RealCost main user interface

3.1.4 Value of User Time

As compared to the engineering and construction costs associated with new construction or rehabilitation, the value of user time or delay costs is generally more contentious. However, the user's costs can have a considerable impact on the LCC of the project. Towards this purpose, the FHWA (Walls and Smith, 1998) analyzed several studies that have previously recommended possible values of user time, including studies carried out by the U.S. Department of Transportation in 1997 and results from the Highways Economic Requirement System (HERS) Model in 1997. Based on the analyzed studies, the FHWA made recommendations for adequate values of user time. Unfortunately, their recommendations correspond to 1996 dollars.

Consequently, the research team has updated the values initially recommended by the FHWA to reflect the current dollar value (2010 dollars), shown in Table 3.2.

Table 3.2: Value of user time in 2010 dollars

Vehicle Class	$ Value Per Vehicle Hour		
	Mean	Minimum	Maximum
Passenger Vehicles	17.72	15.30	19.89
Single-Unit Trucks	28.37	26.01	30.60
Combination Trucks	34.13	32.13	36.72

3.2 Regression Analysis

If the time history data in the performance of different types of pavement mixes, pavement construction methods or maintenance, and rehabilitation strategies are insufficient, then it may be infeasible to perform the LCCA analysis. For this reason, the research team would also like to highlight a valuable tool in characterizing and comparing the performance of different alternatives: regression analysis and analysis of variance. Hence, a brief description follows.

Multiple regression analysis allows for estimating the effect of changing one independent variable ($X_{1,i}$) on a response variable (Y_i), while holding all other factors constant ($X_{2,i}$, $X_{3,i}$, $X_{4,i}$, and so on). In linear regression analysis, the relationship between the independent variables ($X_{j,i}$) and the dependent variable (Y_i) can be defined by a linear expectation function as follows:

$$E(Y_i \mid X_{1,i} = x_1, X_{2,i} = x_2, \ldots, X_{k,i} = x_k) = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \ldots + \beta_k x_k \quad (3.4)$$

where "k" corresponds to the number of regressors or independent variables and $E(^{\cdot})$ corresponds to the conditional expected value function. The coefficient β_0 corresponds to the model intercept and very generally represents the expected value of Y_i when all the independent variables ($X_{j,i}$) are zero. The remaining coefficients β_1, β_2, …, β_k are slope coefficients for the independent variables $X_{1,1}$, $X_{2,i}$, …, $X_{k,i}$. The interpretation of the β_j coefficient corresponds to the effect on Y_i of a unit change in $X_{j,i}$ while keeping the remaining dependent variables constant. In other words,

$$\beta_j = \frac{\Delta Y}{\Delta X_j}, \text{ while } \Delta X_{h \neq j} = 0 \quad (3.5)$$

The most common method of estimating the β_j parameters is by means of Ordinary Least Squares (OLS). Assume $b_0, b_1, b_2, \ldots, b_k$ are estimators of $\beta_0, \beta_1, \beta_2, \ldots, \beta_k$. Then the predicted values of Y_i will be given by,

$$\overline{Y}_i = b_0 + b_1 X_{1,i} + b_2 X_{2,i} + \ldots + b_k X_{k,i} \tag{3.6}$$

However, as with any estimator, there is some error in the predicting $\overline{Y}_i$ that is given by $Y_i - \overline{Y}_i = Y_i - (b_0 + b_1 X_{1,i} + b_2 X_{2,i} + \ldots + b_k X_{k,i}) = Y_i - b_0 - b_1 X_{1,i} - b_2 X_{2,i} - \ldots - b_k X_{k,i}$.

Then, define the sum of the squared errors as,

$$\sum_{i=1}^{N} u_i^{\,2} = \sum_{i=1}^{N} \left(Y_i - b_0 - b_1 X_{1,i} - b_2 X_{2,i} - \ldots - b_k X_{k,i}\right)^2 \tag{3.7}$$

Then, the set of b_j coefficients that minimize Equation (3.7) are defined as the ordinary least squares (OLS) estimators of $\beta_0, \beta_1, \beta_2, \ldots, \beta_k$. Although it is possible to compute the OLS estimators of $\beta_0, \beta_1, \beta_2, \ldots, \beta_k$ by means of trial and error, it is recommended to derive the estimators using calculus. Furthermore, OLS estimation routines are incorporated in most statistical packages and spreadsheet software such as MS Excel.

If the following conditions are also met—i) the relationship between Y_i and $X_{1,i}, X_{2,i}, \ldots, X_{k,i}$ is linear as per Equation 3.4, ii) the independent variables are not perfectly correlated (multicollinearity), iii) the expected value of the error term for all observations is zero, iv) the variance of the error term is constant for all the observations, and v) the errors from different observations are uncorrelated and independent—then the OLS estimates are the best linear unbiased estimators (BLUE) for this class of estimators.

In actual application of OLS, the interest is not only in estimating and interpreting the β_js, but also in determining how important or significant a given independent variable's effect on Y_i is. Determining the significance of each $X_{j,i}$ for Y_i can be done in several ways. However, the most common approach consists of estimating a test statistic that indicates how important each variable is.

Most test statistics require estimation of the standard error associated with each of the parameters (and the covariance between parameters as well). Under the previous set of assumptions, it can be proven that the variance matrix associated with the parameters is given by the following:

$$\mathrm{Var}\begin{pmatrix} b_0 \\ b_1 \\ \vdots \\ b_k \end{pmatrix} = \sigma^2 \left(\begin{bmatrix} 1 & X_{1,1} & \cdots & X_{k,1} \\ 1 & X_{1,2} & \cdots & X_{k,2} \\ \vdots & \vdots & \ddots & \vdots \\ 1 & X_{1,N} & \cdots & X_{k,N} \end{bmatrix}^{T} \begin{bmatrix} 1 & X_{1,1} & \cdots & X_{k,1} \\ 1 & X_{1,2} & \cdots & X_{k,2} \\ \vdots & \vdots & \ddots & \vdots \\ 1 & X_{1,N} & \cdots & X_{k,N} \end{bmatrix} \right) \tag{3.8}$$

where an estimate of σ^2 is given by s^2 as follows:

$$s^2 = \frac{\sum_{i=1}^{N} u_i^{\,2}}{N-k} \tag{3.9}$$

Then under the assumption that the error is normally distributed, t statistics can be applied because,

$$\frac{b_j - \beta_j}{s_{b_j}} \sim t_{N-k} \tag{3.10}$$

for j = 1, 2, …, k.

Then if we are interested in evaluating the assumption that a given independent variable ($X_{j,i}$) is significant, we can test the hypothesis that $\beta_j = 0$. If the t statistic has a low value based on the observed data, then it means that we cannot reject the hypothesis that $\beta_j = 0$ and therefore the effect of $X_{j,i}$ on Y_i is insignificant.

Chapter 4. Effect of Reclaimed Asphalt Pavement (SPS-5 Sections)

The use of RAP by TxDOT is steadily increasing due to, among other factors, the continued increase in the cost of quality aggregates and asphalt binder, a decline in the availability of aggregates meeting specification and project requirements, or due to an overall increase in awareness of the environmental impact of not properly storing and disposing of material generated when milling or full depth-removing asphalt pavements. TxDOT allows up to 30 percent RAP on their base mixtures and up to 20 percent RAP is allowed on the surface mixtures (TxDOT, 2004).

Traditionally, because RAP contains a given percentage of aged asphalt binder, the amount of new asphalt that needs to be added to a mixture containing RAP may be reduced. For example, the Asphalt Institute suggests that using 20 percent RAP (with 5 percent asphalt content) can be approximately translated to a savings of 1 percent of new asphalt binder.

However, the use of RAP introduces an additional variable to the performance and deterioration of the pavement structure. Characterization and better understanding of the interactions between the aged binder and the new or virgin asphalt is required. Depending on the percentage of RAP to be included in the mixture, it is expected that to achieve a target performance grade (PG) for the binder in the mixture with a percentage of RAP over a given threshold, the PG grade of the new asphalt binder needs to be "dumped" (reduced) one grade to offset the effect of the aged binder in the RAP (McDaniel and Anderson, 2001; Newcomb et al., 2007).

Furthermore, in addition to the initial economic benefits of using RAP (reduction in the requirement of new aggregate and asphalt binder), an improvement in the performance of the pavement structure can be expected during the initial service years of the pavement structure. This is due to an increased stiffness of the HMA mix resulting from the inclusion of RAP. The long-term effects in the field, however, have not yet been quantified.

An additional argument for the use of RAP is the initial gain in resistance to permanent deformation of the HMA mix. However, it is still unclear how HMA mixes with high percentages of RAP will perform in terms of cracking. It has been identified that the initial rate of aging in mixes containing RAP is higher than that of virgin mixes. Therefore, it is important to determine what happens in the long term to the binder in the prior type of mix. If the rate of aging of the binder remains relatively constant, the HMA mix might become very stiff, resulting in loss of flexibility and resistance to fatigue and low temperature cracking.

4.1 Texas SPS-5 Sections

The purpose of the present chapter is to evaluate the performance benefits or limitations associated with the use of RAP in an HMA mix. In order to do so, it is imperative to have enough field performance history so that the deterioration trends of the mixes with RAP can be fully captured. A short observation period will likely result only in concluding that the rutting behavior of the mix with RAP was considerably improved because of the increased stiffness associated with using a given percentage of RAP in the mix. However, within this time frame it is impossible to assess how other types of deterioration, such as cracking, will develop in the long run. The use of Accelerated Pavement Testing (APT) will also not help due to its inability to capture the long-term effects of the environment and aging of the binder and the mix.

Therefore, it was necessary to identify pavement sections with monitoring history of over 10 years in the state of Texas. It was found that the LTPP SPS-5 sections have a recorded monitoring history of over 15 years since their original rehabilitation and were, therefore, selected as the candidate sections to analyze as part of the current study.

The SPS-5 experiment was designed to examine the effects of climatic region, condition of existing pavement, and traffic rate on pavement sections incorporating different methods of rehabilitation with HMA overlays. These rehabilitation methods consisted of routine preventive maintenance or intensive preparation (cold milling). The experimental design also involved two types of asphalt overlay (virgin and recycled), and two overlay thicknesses (2 in. and 5 in.). Finally, the experimental design resulting from the combination of study factors resulted in eight different rehabilitation options to be constructed at each test site.

The following are the SPS-5 sections (Figure 4.1), which were built in Kaufman County of the Dallas District along US 175:

- A501 (Control, no treatment),
- A502 (Thin overlay—recycled HMA mix),
- A503 (Thick overlay—recycled HMA mix),
- A504 (Thick overlay, virgin mix),
- A505 (Thin overlay, virgin mix),
- A506 (Thin overlay, virgin mix, with milling),
- A507 (Thick overlay, virgin mix, with milling),
- A508 (Thick overlay, recycled mix, with milling), and
- A509 (Thin overlay, recycled mix, with milling).

Figure 4.1: Texas SPS-5 pavement structures

4.1.1 Performance of SPS-5 Sections

The field performance of the Texas SPS-5 sections in terms of rutting and cracking is shown in Figures 4.2 through 4.6. As expected, three of the sections with RAP (thick overlay with and without milling, and thin with milling) outperform the sections with no RAP in terms of rutting resistance. However, note that the average difference in rutting between the sections with RAP and the sections with no RAP is only in the order of 0.07 in. All sections, in general, performed well. Nonetheless, regardless of the difference in rutting between the sections with and without RAP, a clear distinction exists between the performances of the two types of mix.

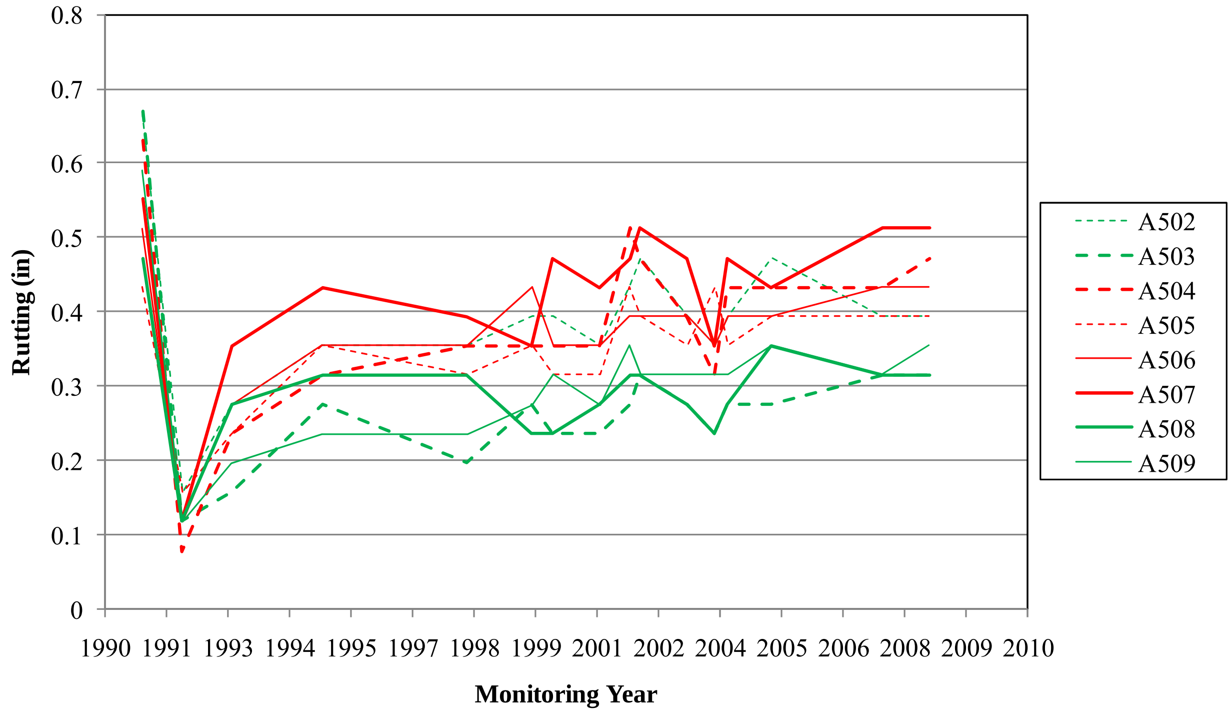


Figure 4.2: Rutting of Texas SPS-5 pavement structures

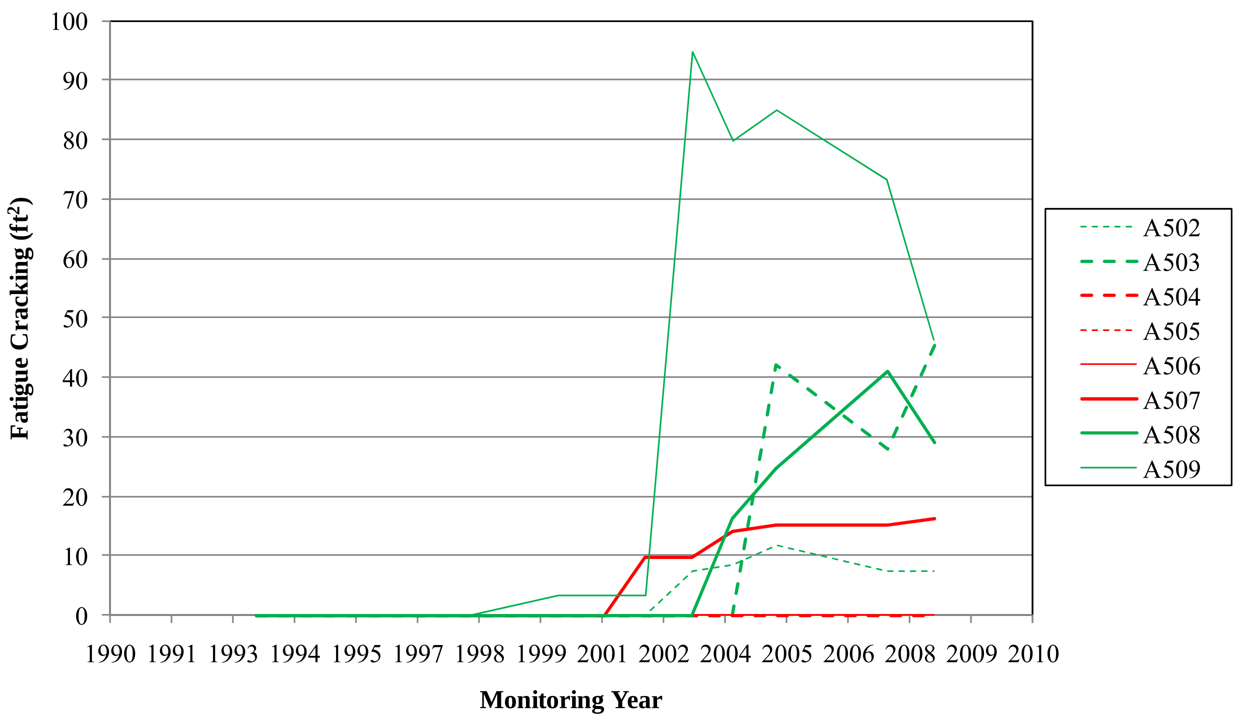


Figure 4.3: Fatigue cracking of Texas SPS-5 pavement structures

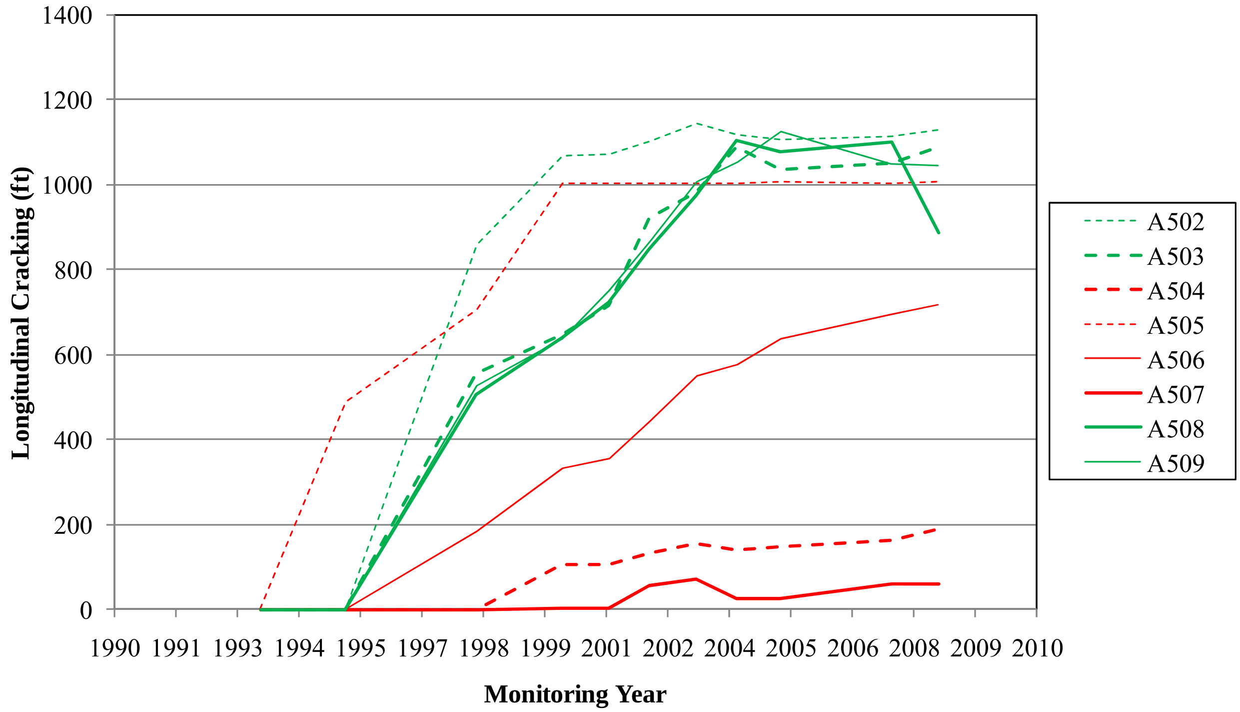


Figure 4.4: Longitudinal cracking of Texas SPS-5 pavement structures

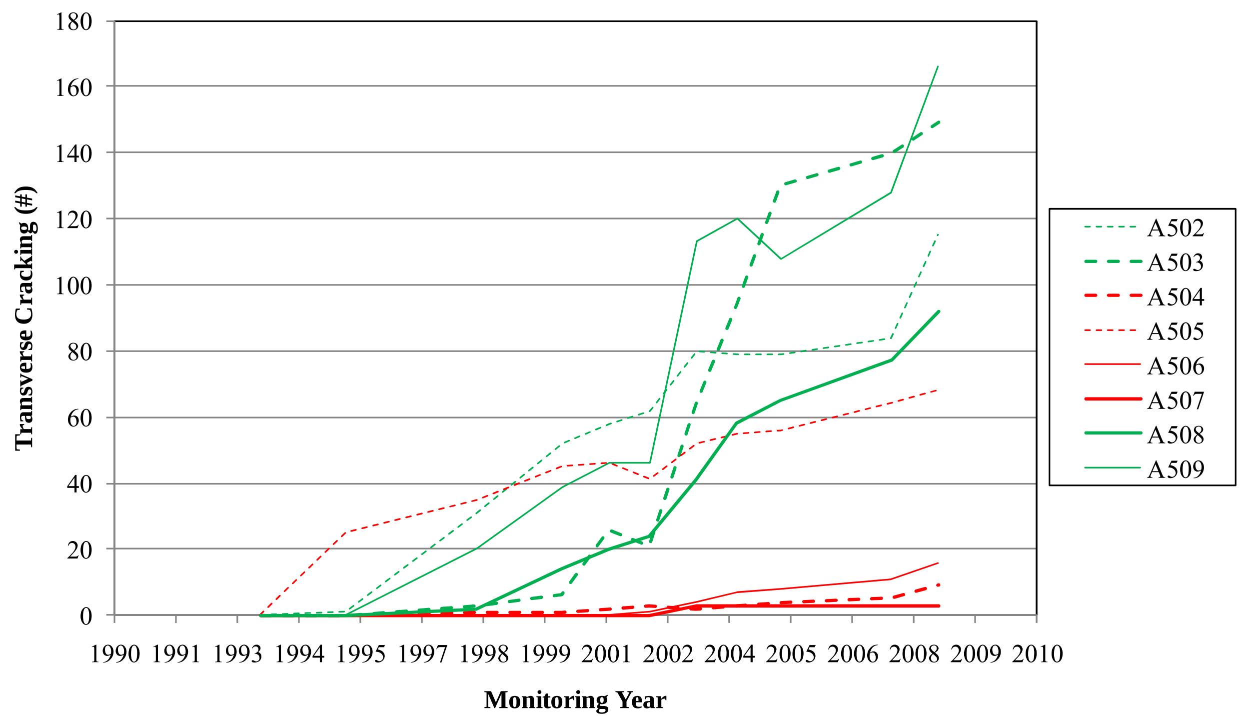


Figure 4.5: Transverse cracking of Texas SPS-5 pavement structures

Contrary to the case of rutting, Figures 4.3 through 4.5 illustrate that the pavement sections with RAP consistently exhibit earlier cracking initiation (alligator, longitudinal, and transverse cracking) and a higher rate of cracking progression, as compared to pavement sections where RAP was not included. In the case of fatigue cracking, two of the pavement sections with no RAP (thin overlay with and without milling) did not develop alligator cracking during the entire observation period. Similarly, in the case of transverse cracking, three of the pavement sections with no RAP (thick overlay with and without milling, and thin with milling) have developed very few transverse cracks at a slower rate than the remaining pavement sections that include RAP.

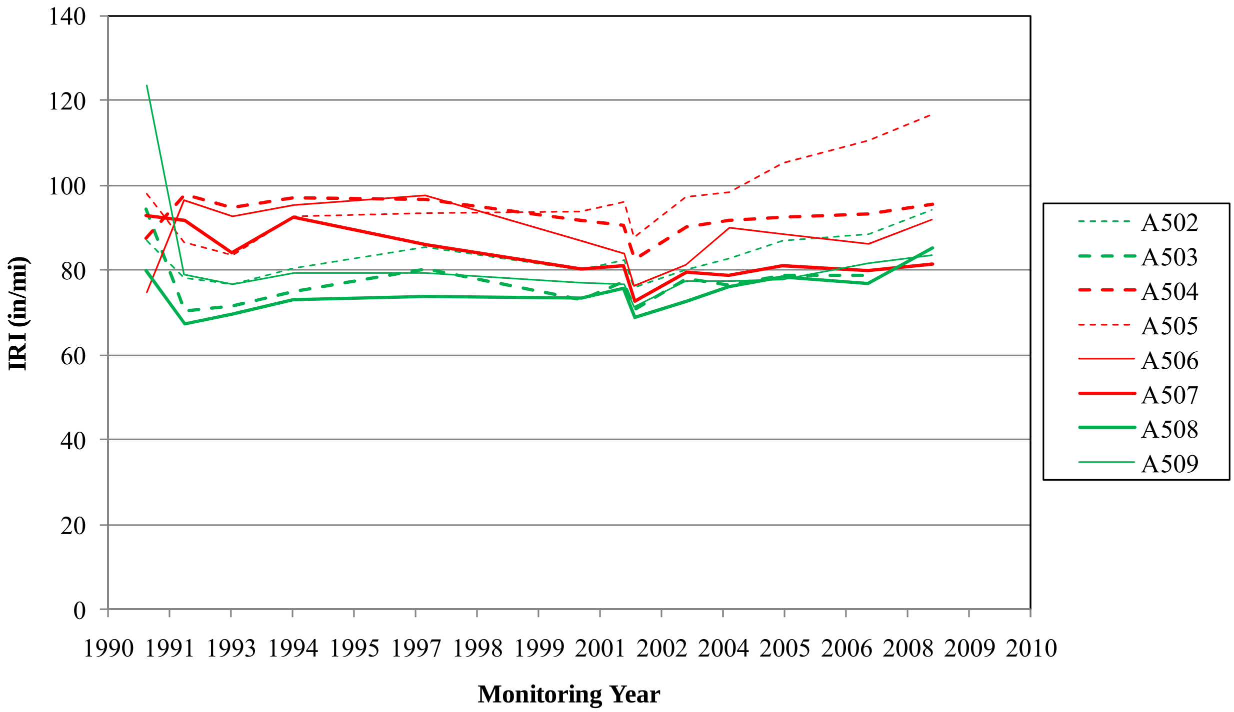


Figure 4.6: Roughness of Texas SPS-5 pavement structures

Figure 4.6 shows roughness in terms of International Roughness Index (IRI) for the analyzed pavement sections. As shown, most of the sections have exhibited very constant roughness during the analysis period. Therefore, the pavement sections with RAP have on average an IRI value of approximately 10 in./mi lower than that of the pavement sections with no RAP. The difference is due mainly to the pavement sections with no RAP where no milling was performed prior to rehabilitation. Nonetheless, the higher IRI rating of the pavement sections with no RAP is expected because of the significant effect that rutting has on the roughness of the pavement structure, therefore increasing the IRI values of the sections with no RAP.

4.2 Modeling the Performance of SPS-5 Sections

When performing economic comparisons and evaluations of HMA overlays with or without RAP, as well as other properties, it is important to develop models that can accurately predict the performance of said pavement structures. For this purpose, two models will be developed—a rutting model and a transverse cracking model—to account for failures due to both types of deterioration.

The rutting model proposed in this study is similar to that developed by Hong et al. (2010). The model corresponds to a rutting progression model intended to capture the incremental rutting after the initial observation (rutting at time 0). The functional form of the model is as follows:

$$Rut - Rut_0 = (\beta_1 + \beta_2 Mill + \beta_2 TH)t^{(\beta_4 + \beta_5 RAP)} \tag{4.1}$$

where:

- Rut: rutting at time t = T (in)
- Rut_0: rutting at time t = 0 (in)
- Mill: 1 if milling was performed, 0 otherwise
- TH: 1 if the overlay is thick, 0 otherwise
- RAP: 1 if RAP was used in the HMA mix, 0 otherwise
- T: time (years)
- β_1-β_5: model coefficients

The model parameters were estimated by means of non-linear least squares (NLS) using Matlab based on the SPS-5 dataset. Table 4.1 shows the model estimates.

Table 4.1: Rutting model parameter estimates

Parameter	Mean	Standard Error	t - Statistic
β_1	0.1247	0.0129	9.66
β_2	0.0096	0.0060	1.60
β_3	0.0176	0.0061	2.87
β_4	0.3095	0.0419	7.39
β_5	-0.1785	0.0199	-8.99

From the model estimates, the researchers concluded that the effect of milling (β_2) is not significant at the 95 percent level of confidence. On the other hand, the thickness of the overlay is significant based on the previous level of confidence. However, based on the model estimates, the use of RAP has the highest effect on the rutting progression of the pavement structure. Figure 4.7 shows a comparison of the performance predicted by the model for a pavement section with and without RAP.

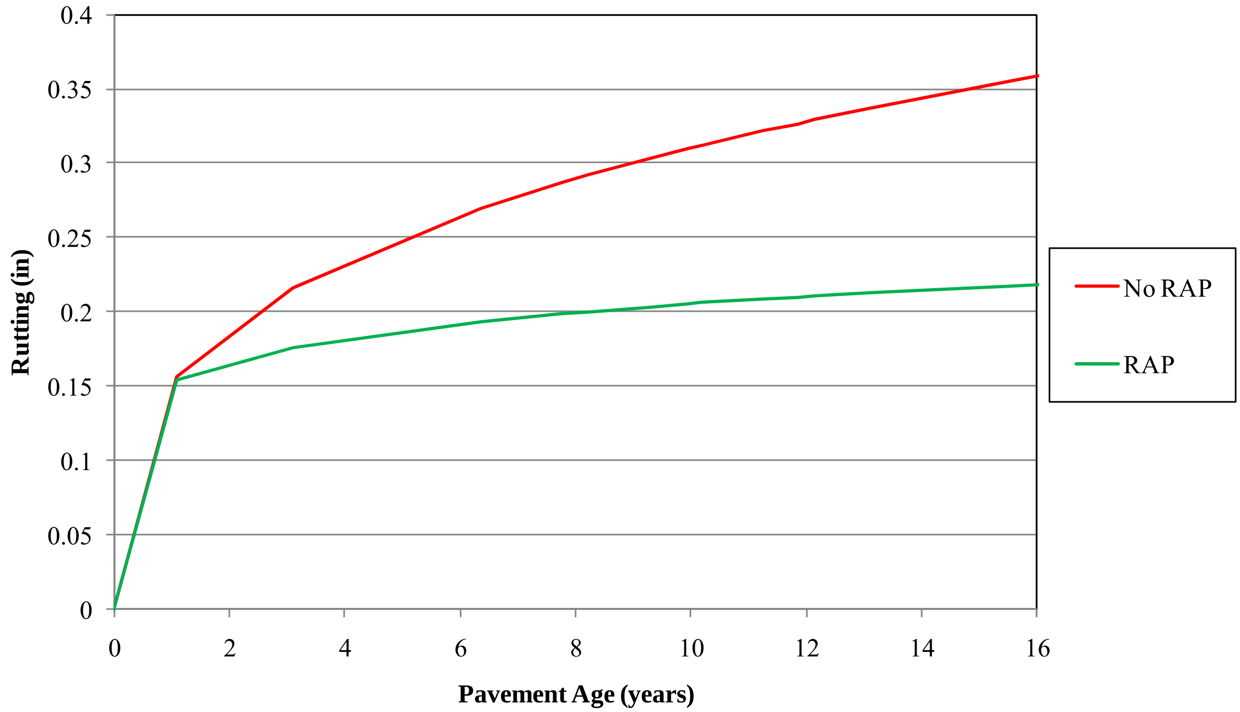


Figure 4.7: Rutting progression on a pavement with a thick overlay (with milling prior to overlaying)

The transverse cracking model proposed for the present analysis is a Heckman-type model that consists of two separate models or components: a crack initiation model and a crack progression model. The functional form of the model is the following:

$$\Pr ob(Crack? \mid Mill, TH, RAP, t) = \Phi(\alpha_1 + \alpha_2 Mill + \alpha_3 TH + \alpha_4 RAP + \alpha_5 t) \tag{4.2}$$

$$Crack = \gamma_1 + \gamma_2 Mill + \gamma_3 TH + \gamma_4 RAP + \gamma_5 t \tag{4.3}$$

where:

Crack?:	1 if transverse cracking has occurred, 0 otherwise
$\Phi(\cdot)$:	normal cumulative density function.
Crack:	number of transverse cracks
Mill:	1 if milling was performed, 0 otherwise
TH:	1 if the overlay is thick, 0 otherwise
RAP:	1 is RAP was used in the HMA mix, 0 otherwise
T:	time (years)
α_1-α_5, γ_1-γ_5 :	model coefficients

In the previous model, Equation (4.3) the amount of cracking is conditional on whether cracking has initiated or not, which is defined by Equation (4.2). Equation (4.2) represents the probability that a given pavement structure has cracked, and for the modeling specification it has been assumed to follow a normal distribution. The previous type of joint model is usually referred to as a Heckit model, and is estimated in a two-step procedure which involves initially estimating the crack initiation model (Equation (4.2)), and then using some information from the initial estimation ($\lambda(\alpha_i X_i) = \phi(\alpha_i X_i)/\Phi(\alpha_i X_i)$, where $(\alpha_i X_i) = \alpha_1 + \alpha_2 Mill + \alpha_3 TH + \alpha_4 RAP + \alpha_5 t$ and $\phi(\cdot)$ is the normal density function) in the estimation of the crack progression model. For more details on the Heckit model, please refer to Heckman (1976).

The estimation procedure was coded in Matlab and was used to estimate the model parameters shown in Table 4.2.

Table 4.2: Transverse Cracking Model Parameter Estimates

Crack Initiation				Crack Progression			
Parameter	**Mean**	**Standard Error**	**t - Statistic**	**Parameter**	**Mean**	**Standard Error**	**t - Statistic**
α_1	–45.8	12.2	–3.7	γ_1	–2.68	1.09	–2.4
α_2	–21.3	6.37	–3.3	γ_2	–3.90	1.47	–2.6
α_3	–27.9	6.04	–4.6	γ_3	–1.23	0.772	–1.6
α_4	56.4	6.21	9.0	γ_4	2.75	1.07	2.6
α_5	8.09	1.06	7.6	γ_5	0.901	0.289	3.1

The model estimates can lead to the conclusion that, for the case of crack initiation, both milling prior to overlaying and the thickness of the overlay have a significant positive effect (both of these factors decrease the probability that the pavement will crack at time t). Inversely, the use of RAP has the highest impact on the crack initiation of the pavement structure (using RAP highly increases the probability of cracking).

Then, based on the estimates from the cracking progression model, it can be concluded that milling prior to overlaying results in a lower number of transverse cracks, while adding RAP to the HMA mix increases the number of transverse cracks. Figure 4.8 shows a comparison of the performance predicted by the model for a pavement section with and without RAP.

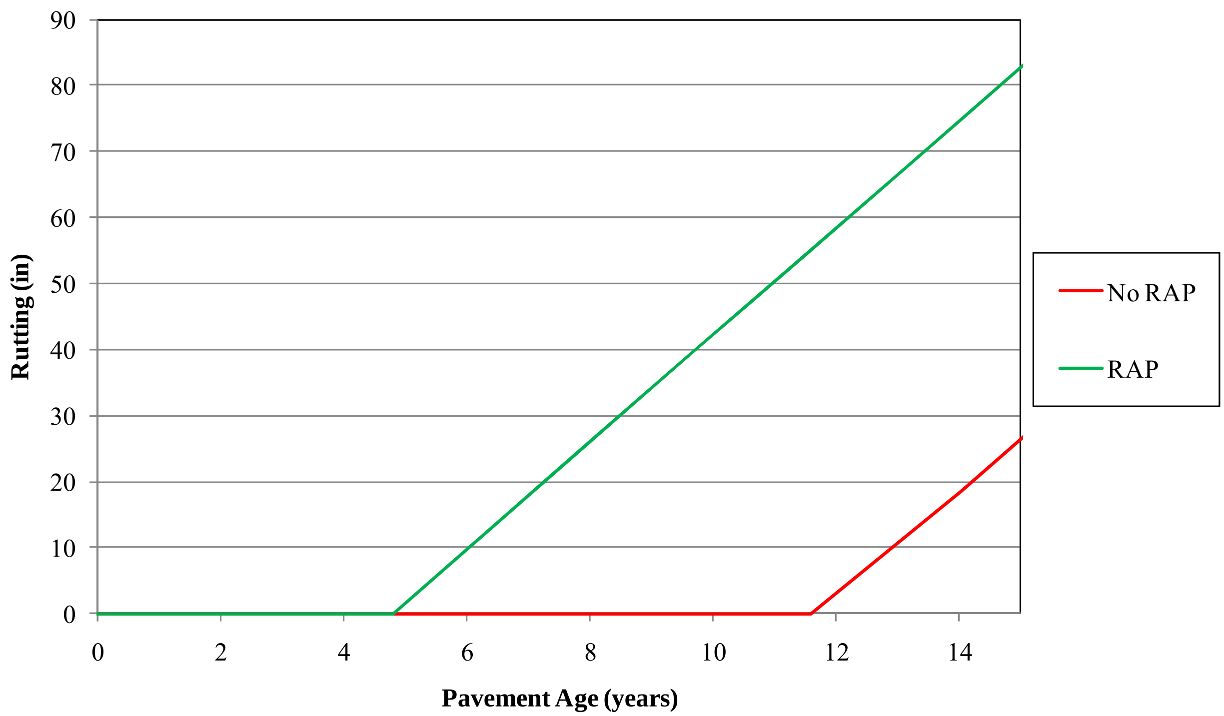


Figure 4.8: Transverse cracking progression on a pavement with a thick overlay (with milling prior to overlaying)

4.2.1 Predicting Pavement Life

In order to use the previous models to estimate pavement life, it is necessary to define what degree of the given deterioration types can be considered as failure. This is a fundamental requirement in estimating pavement life, which is very sensitive to changes in the failure threshold.

In the case of rutting, the MEPDG recommendation of 0.20 in. of rutting will be adopted as the allowable progression of rut depth before failure can be defined (ERES, 2004). However, the analysis could be repeated for any other alternative criterion that is desired. Based on this criteria, any pavement structure where rutting has increased by 0.20 in. since initial monitoring of the pavement section will be declared as failed. With regard to transverse cracking, the criteria that will be selected to describe failure is 5 cracks per 100 ft station. This can be translated to 25 cracks in a normal LTPP section that measures 500 ft, assuming the cracking is uniform throughout the section.

Based on the previously defined deterioration models and failure criteria, the estimated life of the pavement sections of interest can be summarized in Figure 4.9. Note that milling prior to overlaying increases the life expectancy of the pavement structure when no RAP is used in the mix. In the case where RAP is used, the effect of milling is reversed. Furthermore, the expected increasing trend of pavement life as a function of overlay thickness can be verified for the case of pavement where RAP was used. The reverse trend is observed in pavements with no RAP (however, this trend can be verified by observing the actual field cracking and rutting on the SPS-5 sections in Figures 4.2 through 4.5).

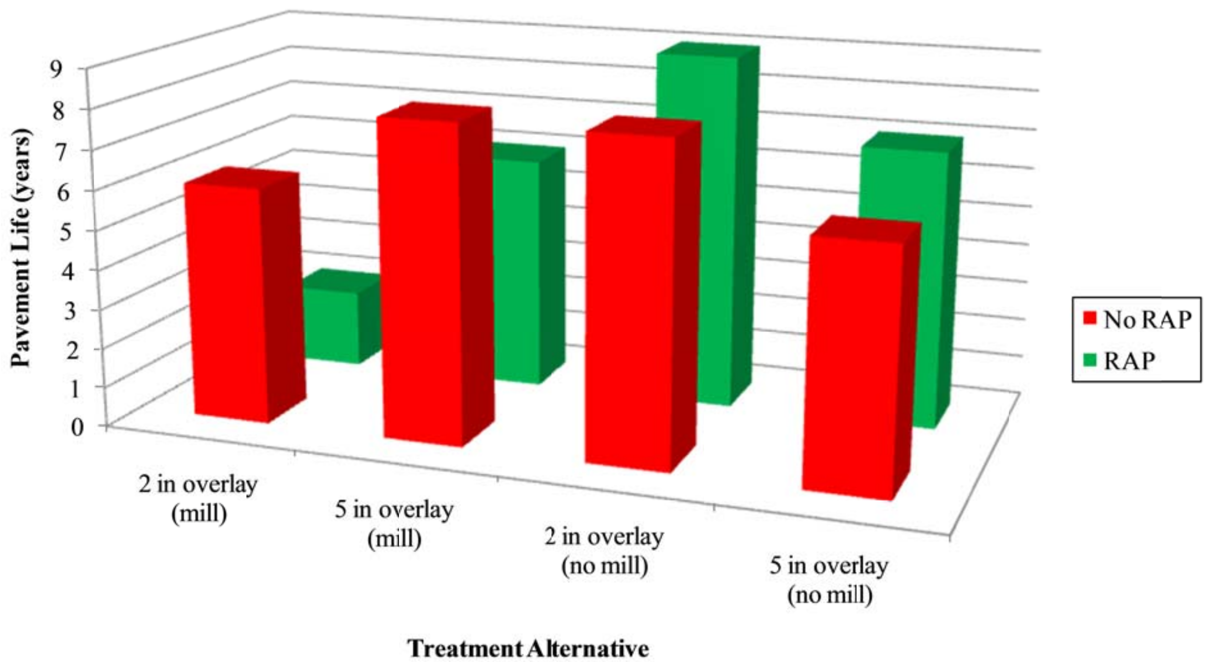


Figure 4.9: Estimated pavement life of analysis pavement sections

4.3 LCCA Analysis

In order to perform an economic evaluation of the effect of using RAP in an HMA, a LCCA based on FHWA's RealCosts (Walls and Smith, 1998) was performed. The agency costs are directly related to the costs incurred by the agency over the service life of the pavement structure, ranging from the initial engineering design costs to the actual costs of constructing and maintaining the pavement structure. Because all the alternatives are considered mutually exclusive, only the costs that are different between alternatives are considered. The agency can incur additional costs at the end of the service life of the pavement structure, such as the salvage value of the pavement structure. This negative cost will be neglected in the current analysis.

Under the assumption that the underlying structure of the pavements to be analyzed is the same, the only difference in material costs between the different alternatives will be due to the use of RAP in the overlay. Based on TxDOT average low bid unit prices, a Type C mix with RAP has been assumed to have a cost of approximately $49.60/ton (PG76-22 binder with RAP), while a Type C mix with no RAP has been assumed to have a cost of approximately $57.42/ton (PG64-22 virgin binder) in 2010 dollars.

The user costs are costs that are directly incurred by and affect the user, such as vehicle operating costs (change in speed, number of stops, additional miles, and hours of idling), user delay costs, and crash costs (Walls and Smith, 1998). The differential user costs are estimated by multiplying the different user costs incurred due to a construction or maintenance/rehabilitation strategy by the unit cost of each of these components. The values of time have been assumed as per Table 4.3.

Additionally, in order to estimate the remaining costs on the user, the following assumptions of free flow traffic and traffic under the work zone were used based on the FHWA recommendations (Walls and Smith, 1998).

Table 4.3: Traffic inputs required for the estimation of user costs

Traffic Parameter	Value
AADT Construction Year (two-direction)	9,380
Single Unit Trucks (%)	5.23
Combination Trucks (%)	12,22
Annual Growth Rate of Traffic (%)	3
Speed Limit (mph)	65
Lanes in each direction	2
Free Flow Capacity (vphpl)	2,017 (*)
Queue Dissipation Capacity (vphpl)	1,818 (*)

(*) Based on Highway Capacity Manual recommendations.

The comparison between the pavement sections to be analyzed has been performed for a 20-year design period under the following assumptions: i) the initial cost of construction of the overlay is equal to the cost of the rehabilitation for a given pavement section, and ii) the deterioration rates of the subsequent rehabilitation strategies are equal to that of the initial overlay.

Finally, the NPV of the different alternatives has been estimated by means of the FHWA's RealCost software. The results are summarized in Figure 4.10. In the case of thick overlays, the long-term costs of using RAP in the mix appear to be similar to the costs when no RAP is included in the mix. However, in the case of the thin pavement structure with no milling, there is a clear economic benefit to *not* using RAP in the long run—primarily because the increased cracking expected in thin RAP sections may be mitigated by milling of the sections prior to overlay.

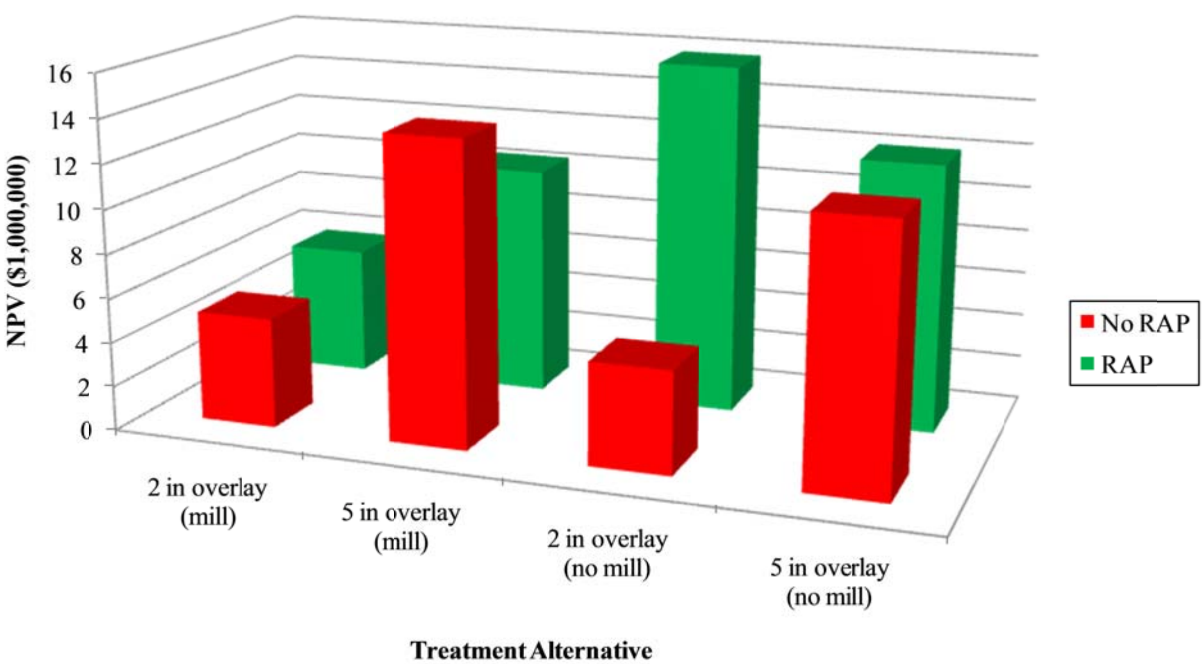


Figure 4.10: NPV of total costs for the different alternatives

The LCCA analysis presented in this chapter may be used to evaluate and compare the benefits of alternative pavement engineering strategies. The importance of long-term reliable and accurate performance data is emphasized. Through the analysis of the SPS-5 sections in Texas many valuable lessons were learned. There are no doubts that use of RAP is an appropriate sustainable technology and should be promoted because RAP is an effective material for delaying and reducing rutting. However, RAP is neither effective nor efficient against cracking (low temperature cracking and fatigue cracking). Please note that RAP is not always the solution; its use depends on project-specific conditions and location. More data and research are needed to assess long-term implications, in particular with fractionated RAP and with recycled asphalt shingles (RAS).

Chapter 5. Use of Composts

Composts have been used in 22 of the 25 TxDOT districts to promote vegetation growth and control erosion of embankments with excellent results. *Compost* may refer to animal manures, municipal wastes (solid waste and wastewater sludges), and other waste materials, as well as application of the composted materials mixed with different soils (composted manufactured topsoil) (Kirchhoff et al., 2002).

More recently, compost materials have been used in Texas as a means to reduce pavement shoulder cracking. Although many types of compost materials are available, not all work properly with particular types of soil. Only two composts, Dairy Manure Compost (DMC) and Biosolids Compost (BSC), meet TxDOT and the United States Environmental Protection Agency (USEPA) specifications for use as potential soil additives (Kirchhoff et al., 2002).

Contrary to Kirchoff findings, subsequent research demonstrated that other regionally available products such as cotton burr, grease and tree trimmings are also effective as soil treatments to retard pavement cracking. BSC results from treatment of solid waste or wastewater sludges from treatment plants and DMC through the activity of aerobic microorganisms on dairy manure. Previous TxDOT Research compared control soil (CS) to the following material mixes: 1) 75 percent DMC and 25 percent CS, 2) 100 percent DMC, 3) 20 percent BSC and 80 percent CS, and 30 percent BSC and 70 percent CS (Puppala et al., 2004). Laboratory testing revealed that the composts reduced linear shrinkage strains, increased the shear strength, increased the swell strain potential, and slightly reduced the permeability of the natural soil.

In order to validate the results, several field test plots were constructed to evaluate the effect of the composts on different soil types throughout Texas. The first and most comprehensive test section was constructed on SH 108 near Stephenville in Erath County, Texas. Sixteen test plots of different widths and thicknesses were constructed and studied. Two widths (5 ft and 10 ft) and two thicknesses (2 in. and 4 in.) were studied. One Control Plot (CP) with no compost material was established for comparison. Moisture and temperature sensors recorded real time volumetric moisture content and temperature data (Puppala et al., 2004).

Mean moisture content variations were not statistically different from those recorded on the control plot. However, approximately half of all 16 plots had lower variations than the CP and the moisture variations in all plots varied from 11.6 to 22.2 percent. With respect to temperature variations, few were statistically different from the CP; however, the temperature variations of about two-thirds of the test plots were lower than that of the CP, with temperatures variations ranging from 15.18 ^{o}F to 30.29 ^{o}F temperature variations (Puppala et al., 2004).

On average, the BSC plots exhibited less shrinkage than the CP, while the DMC exhibited higher shrinkage than that of the CP. The study found new cracks on the CP and the 5 ft x 2 in. plots, while no cracks appeared on the wider and thicker plots using both compost materials (10 ft by 4 in.) (Puppala et al., 2004).

In order to extend and verify the effectiveness of compost materials in different soil types and climatic regions, an implementation study was conducted at three distinct test sites located in Lubbock, Bryan, and Corpus Christi regions, representing the Panhandle Plains, Prairies and Lakes, and Gulf Coast regions of the state, respectively (Puppala and Intharasombat, 2006).

The Lubbock site was located on US Highway 82, west of Crosbyton (Crosby County), Texas. Two composts, Feedlot Compost and Cotton Burr Compost, were acquired from local sources and mixed with the control soil at 20 percent dry weight to form two types of Compost

Manufactured Topsoils (CMTs). The Bryan site was located on FM 2818 about 2 miles north of SH 60 on the west side of Texas A&M University. Two compost sources were used for the field studies: BSC from the City of Bryan, Texas, and wood compost from Conroe, Texas. Shortly after installation, the District added a seal coat to the existing roadway surface, negating observance of the existing cracks. This section was no longer used in the Implementation Project. The Corpus Christi site was located on FM 188 east of Sinton (San Patricio County), Texas. Cow Manure Compost and Biosolids Compost were combined with the soil.

The trends observed in all the sites were similar to those of the Stephenville site with regard to the mean moisture variation: no significant differences appeared with respect to a control plot. However, most of the compost material treated plots had significantly lower temperature variation, as compared to a control plot on each section. Additionally, most of the plots exhibited less percentage cracking than the respective control plot, with the exception of the Cow Manure plot in Corpus Christi. Nonetheless, none of the plots in Corpus Christi exhibited new cracks, while most of the plots in the other locations did.

5.1 Performance of Compost Test Plots

As part of this research project, several of the compost sections were visited and evaluated to assess the long-term performance. Field performance data such as visual cracking assessment and automated roughness measurements by means of a TxDOT profiler van were collected. The collected data can be readily viewed on the wiki site developed for the current project at http://pavements2.ce.utexas.edu:8080/txdot/Compost.

As an example of the data that was collected, the performance data for one of the time observations for the Stephenville compost test plots are displayed in Figures 5.1–5.3. In order to easily identify each section's treatment mix, please refer to Table 5.1.

Table 5.1: Different plots in compost test section in Stephenville, TX

Section	Soil	Shoulder Width (ft)	Thickness (in)
1	CBS-30	10	4
2	CBS-20	10	4
3	DM-100	10	4
4	DM-75	10	4
5	CBS-30	10	2
6	CBS-20	10	2
7	DM-100	10	2
8	DM-75	10	2
9	CBS-30	5	2
10	CBS-20	5	2
11	DM-100	5	2
12	DM-75	5	2
13	CBS-30	5	4
14	CBS-20	5	4
15	DM-100	5	4
16	DM-75	5	4
17	CS	10	4

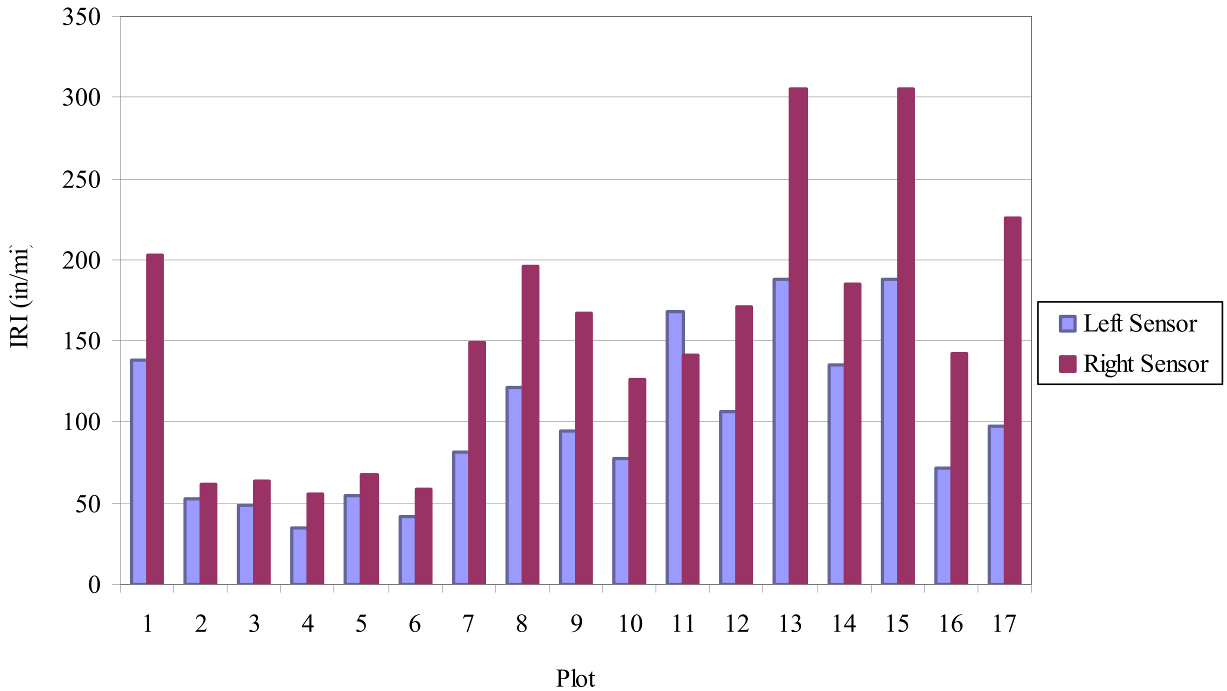


Figure 5.1: Roughness measured along shoulder for each of the test plots (01/10)

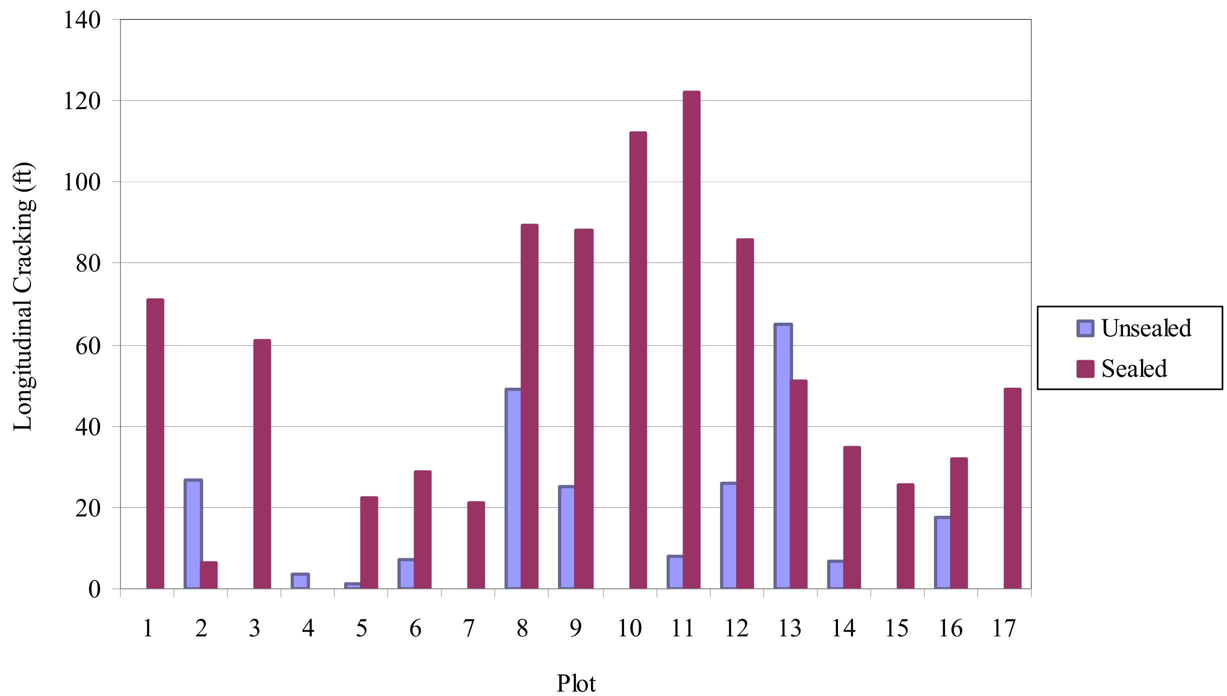


Figure 5.2: Longitudinal cracking measured along shoulder for each of the test plots (01/10)

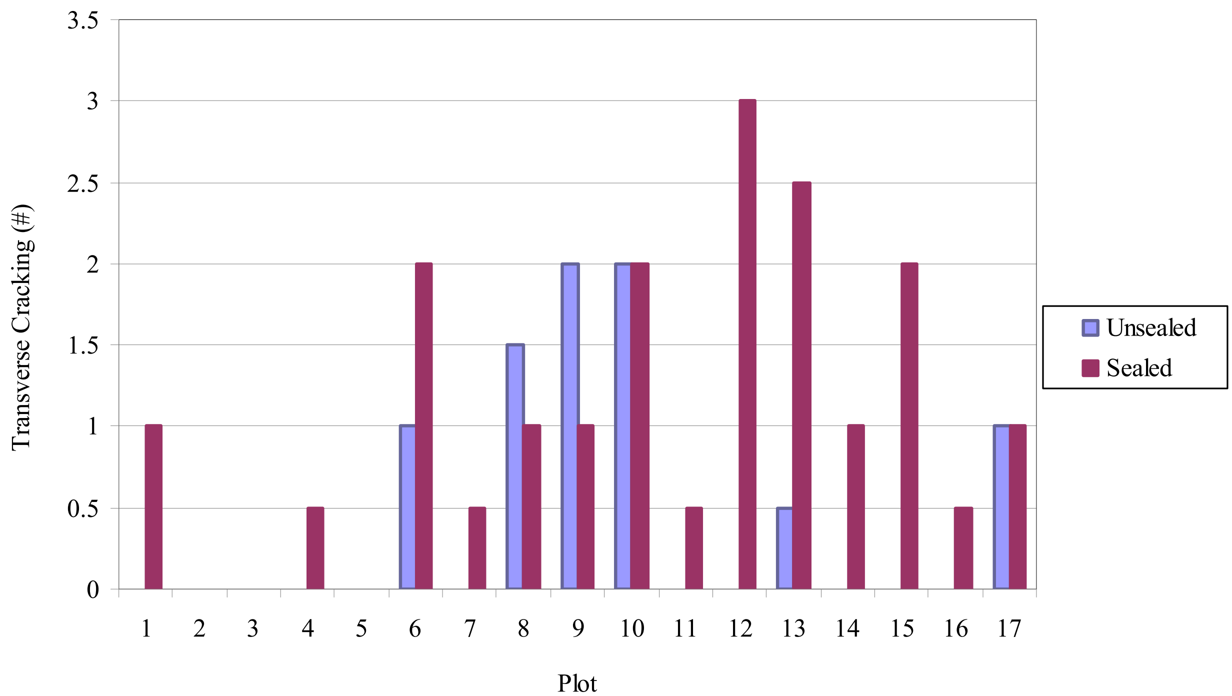


Figure 5.3: Transverse cracking measured along shoulder for each of the test plots (01/10)

Overall, the figures demonstrate that after almost 7 years of field performance (the plots were constructed in March 2003), the plots with higher section width (10 ft as opposed to 5 ft) seem to exhibit better overall performance. However, in order to better quantify the effect of the different factors involved, regression analysis (OLS) was performed.

Ideally the research team would have liked to perform LCCA analysis. Unfortunately, 2 years of data collection is insufficient to observe performance trends in the pavement structure that can be properly used to model the field performance of similar structures. For this reason, the research team strongly recommends continued monitoring of the sections so that the effect of compost on the pavement structure performance, mainly on the shoulder, can be better described by means of times series data. The same recommendation applies to other sections constructed as a result of research projects. The monitoring of these sections is typically terminated with the project and many valuable lessons can still be learned by quantifying the long-term performance.

5.1.1 Effect of Compost on Pavement Performance (Shoulder)

The researchers performed a regression analysis to characterize the effect of the different factors evaluated in the test plots (type and amount of compost material, and depth and width of treatment) and better quantify their impact on the performance of the shoulder. Tables 5.2 through 5.4 show the results of the regression analysis and the variance analysis.

Table 5.2 confirms that both compost materials aid in reduction of roughness along the shoulder. However, the effect of the different types of compost material is statistically significant for approximately an 80 percent level of confidence. Additionally, the amount of compost and the depth of the treatment are not statistically significant in improving resistance to roughness (for a 95 percent level of confidence). What is significant is the width of the treatment: a 1-ft increase in the width of the treated soil can result in a 13.4 in./mi decrease in roughness.

Table 5.2: Effect of different factors on shoulder roughness of the compost test plot sections

Parameters	Coefficients	Standard Error	t Statistic
Intercept	243.5	65.7	3.7
CBS	-56.0	49.2	-1.1
DM	-75.4	96.9	-0.8
Compost Content	0.3	1.0	0.3
Treatment width	-13.4	3.8	-3.5
Treatment depth	4.9	9.5	0.5

Table 5.3: Effect of different factors on shoulder longitudinal cracking of the compost test plot sections

Parameters	Coefficients	Standard Error	t Statistic
Intercept	213.9	49.0	4.4
CBS	-12.5	36.7	-0.3
DM	-22.5	72.4	-0.3
Compost Content	0.0	0.7	-0.02
Treatment width	-9.8	2.8	-3.5
Treatment depth	-16.8	7.1	-2.4

Table 5.3 shows that the type of compost material is not significant in the resistance to longitudinal cracking. Similarly, the amount of compost is not statistically significant in improving resistance to longitudinal cracking. However, Table 5.3 indicates that the depth and the width of the treatment are highly significant: a 1-ft increase in width of treated soil can be associated with a 9.8-ft decrease in longitudinal cracking and a 1-inch increase in treated soil depth can be associated with a 16.8-ft decrease in longitudinal cracking.

Table 5.4: Effect of different factors on shoulder transverse cracking of the compost test plot sections

Parameters	Coefficients	Standard Error	t Statistic
Intercept	6.8	1.5	4.5
CBS	-0.3	1.1	-0.3
DM	1.2	2.2	0.6
Compost Content	-0.04	0.02	-1.7
Treatment width	-0.2	0.1	-2.8
Treatment depth	-0.6	0.2	-2.6

Table 5.4 shows the effects of the different factors on resistance to transverse cracking. Similarly to the case of resistance to longitudinal cracking, the type of compost used has no

significant effect on the resistance to transverse cracking. However, the effect of the content of compost material is significant: a 10 percent increase in compost material can be related to a decrease of approximately half a transverse crack. Furthermore, the table indicates that the depth and the width of the treatment are very significant: it was found that a 1-foot increase in width of treated soil can be associated with a 0.2 decrease in the number of transverse cracks and a 1-inch increase in depth of treated soil can be associated with a 0.6 decrease in the number of transverse cracking.

5.2 Summary and Recommendations

The short-term benefits of using composts have been demonstrated by previous TxDOT-sponsored research projects (Puppala et al., 2004; Puppala and Intharasombat, 2006). The researchers previously showed that that cracking is reduced along the shoulder when composts are used as additives to improve the behavior of the soils in the surrounding area of the shoulder. However, the researchers had originally expected the effect to be only short term.

The current analysis has demonstrated that even after 7 years following construction, some of the pavement sections where compost was used still exhibit less cracking (both transverse and longitudinal) along the shoulder and less roughness, as compared to the control sections. Thus, the benefit is even higher than initially anticipated.

The data collected indicated that the type of compost additive that was used is not significant in the long run. What is important is the width of soil that is treated (as measured by the distance from the shoulder outwards). The study found that plots where the treatment width was only 5 ft did not differ significantly in performance from the control sections; however, sections where the compost additive was applied to widths of 10 ft (regardless of the depth of treatment) do exhibit improved resistance to cracking and lower roughness.

The previous results apply mainly to the shoulder. On the driving lanes of the visited sections, the effect of compost was not clear. However, it is very important to highlight the buffering effect that compost material can potentially have on pavement sections with little to no shoulder. In these cases, performance benefits would be expected from the soil treatment.

Furthermore, the types of composts should not be limited only to the ones used in the preceding projects, but can be extended to other material such as wood shavings that are typically obtained as a byproduct from roadside maintenance. Such byproduct materials can be incorporated at little to no cost, and can also absorb higher degrees of moisture, further improving pavement performance.

Chapter 6. Warm Mix Asphalt

WMA utilizes lower mixing and placement temperatures compared to its more traditional counterpart, HMA. This difference in temperature implies several environmental advantages that include fewer emissions, odors, and smoke, less energy consumption, and improved working conditions at the paving site. Because its production releases fewer emissions, WMA is better for workers and can be used in areas where air quality is a concern. WMA energy consumption represents a reduction of about 30 percent when compared to HMA. WMA does not require new construction technologies, because it is laid as traditional HMA; in fact, it is easier to work with because the mix is less stiff at lower temperatures. WMA often requires less compaction to achieve the same level of density. And because outdoor temperature is not as big a concern, the paving season can be extended. This mix is workable at lower temperatures, allowing for a longer period to haul it and compact it. One potential drawback is the additional cost, which may run as much as $5 a ton for the additives.

Various proprietary technologies have been developed in different parts of the world for producing WMA, such as Aspha-Min, WAM-Foam, and Low-Energy Asphalt (all European), Sasobit (from South Africa), and Evotherm (developed in the U.S.), all of which reduce the viscosity or expand the volume of the asphalt binder at a given temperature.

TxDOT Project 0-5597 performed a synthesis of WMA. A summary of findings and recommendations is presented in TTI Report 0-5597-1 (Button et al., 2007). Also included in this synthesis is a complete documentation of the first warm-mix asphalt field trial conducted by TxDOT. This trial consists of the use of Evotherm on Loop 368 in Bexar County, San Antonio, in 2006.

6.1 WMA Sections

This project monitored two groups of sections: the San Antonio sections on Loop 368, and the Lufkin sections on FM 324.

6.1.1 San Antonio, Loop 368

This project was the first warm-mix asphalt trial placed by TxDOT. The section is a four-lane roadway divided by a median with curb and gutter and many businesses along each side. The existing pavement (prior to placement of the warm mix and control sections) consisted of a cold-milled asphalt surface that had been seal coated with AC-15P and a Grade 4 precoated aggregate. The seal coat had been under traffic for about a month prior to the overlay. All of the paving for this project was conducted at night; Figure 6.1 illustrates the paving sequence of the Evotherm section.

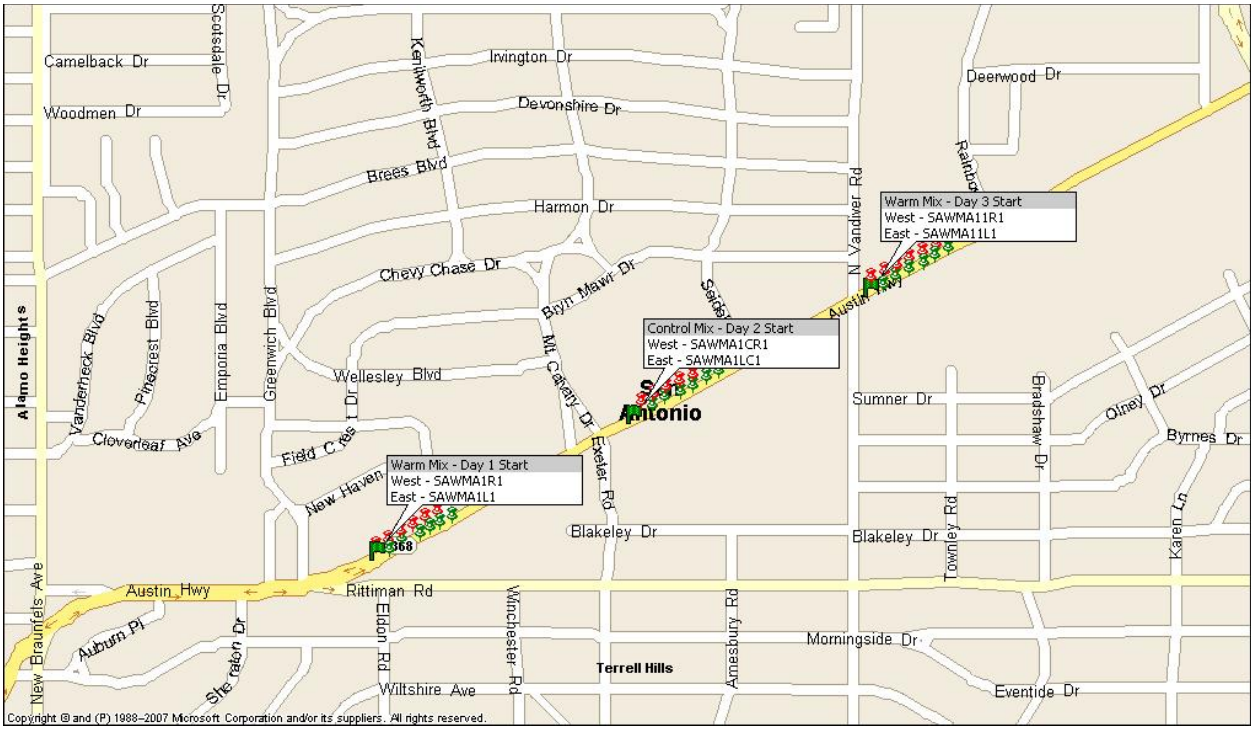


Figure 6.1: Location of WMA sections on Loop 368, San Antonio

The warm and control mixes were produced by Vulcan Materials of San Antonio and placed by Dean Word Company of New Braunfels. The field trials are placed within the limits of a much larger HMAC paving project (CSJ 0016-08-027).

The control and warm mixtures met the gradation requirements of a TxDOT Item 341, Type C, dense-graded HMAC. The mixture designs were performed by Vulcan Materials laboratory. The asphalt used for the control HMAC was Valero PG 76-22. The base asphalt for the warm mix started as a Valero PG 64-22 prior to modification. Ergon modified and emulsified the asphalt using chemistry supplied by MeadWestvaco. Once modified, the warm-mix binder met the specifications of PG 76-22. The modified asphalt was then emulsified and provided to Vulcan Materials laboratory to perform the mixture design. Two aggregate sources were used for the mixtures: Vulcan's Helotes Pit limestone and the Harris Pit field sand. The same aggregate sources and gradations were used for both the warm mix and the control. Both warm and control mixtures were designed using a Texas gyratory compactor with a target density of 96.5 percent.

Both the warm mix and control were produced at the Vulcan Materials plant in San Antonio. The plant was a parallel flow Astec with external coater. The emulsion was pumped from the tanker trucks into the end of the drum through the regular plant metering system. Production rate for the warm mix was about 190 tons per hour (conventional hot mix for this plant is around 250 tons per hour). The production rate was less than expected due to high moisture content in the aggregate stockpiles (primarily the field sand) from rain during the day prior to the first night of WMA production. The combined stockpile moisture content ranged from 4.8 to 5.2 percent. (The normal stockpile moisture content for these aggregates was between 3 and 4 percent.) The limit on the production rate was due to problems with the external coater motor. It would have been desirable to have a mix discharge temperature of 200 °F, but at temperatures below 220 °F, the plant started having trouble with the coater (asphalt too viscous), causing the motor to trip. Amperages were about 20 percent higher with the warm mix. The drag chain, which transfers the mix up into the silos, was operating normally. On the average, the fuel consumed was the same for the warm mix as for the hot mix. No reduction in fuel use was observed for the warm mix because of the high moisture content in the aggregates. The baghouse had no moisture problems—lots of steam was observed, but bags were not plugged up or caked over. For the three nights that the test mixes were produced, the plant started producing mix around 7:00 p.m. and began shipping mix out at around 9:00 p.m. About 180 tons of mix were stored during this first 2 hours of production but after the first 180 tons of mix, there was no need to silo mix. Temperature of the warm mix at the time of loading into the haul trucks was 220 °F. Warm-mix plant samples were compacted in the laboratory under three different curing conditions: no cure, curing for 2 hours at 200 °F, and curing for 2 hours at 240 °F.

6.1.2 Lufkin, FM 324

The project was developed as an experimental test section to compare the performance of several types of WMA technologies. Among the evaluated products are Advera, Akzo Nobel Rediset, Evotherm DAT, and Sasobit. Additionally, a control section where HMA was used was also constructed. Construction ended in March 2008. The mix consisted of a Type D dense graded mix with PG64-22 binder. Figure 6.2 indicates placement of the four WMA products.

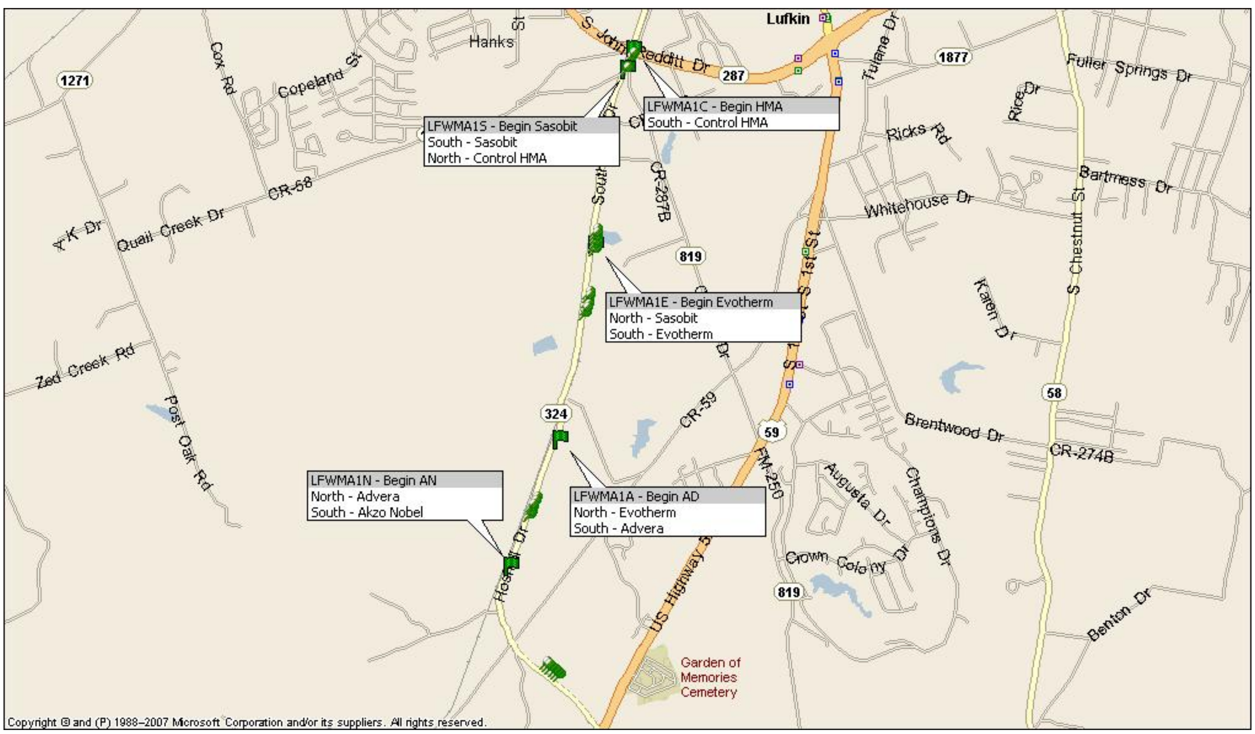


Figure 6.2: Location of WMA sections on FM 324, Lufkin

Based on TTI's assessment, the HMA material appeared to have greater temperature differentials when compared to the WMA (Estakhri et al., 2010). This result is to be expected because the WMA compaction temperatures are closer to ambient temperature, and consequently the temperature difference should be less.

6.2 Performance of WMA Sections

As part of the current project, the previously mentioned WMA sections in Texas were visited and evaluated. Field performance data such as surface rutting (with 6 foot straight edge), visual cracking assessment, FWD, and automated roughness measurements by means of TxDOT profiler vans were collected. The collected data can be readily viewed on the wiki site developed for the current project at http://pavements2.ce.utexas.edu:8080/txdot/WMA.

As an example of the data that were collected, the performance data for one WMA section are shown in Figures 6.3–6.7. Please note that Figure 6.7 shows no transverse cracking. The data correspond to the WMA with Advera placed on the southbound lane of Loop 368 (LFWMA1AS).

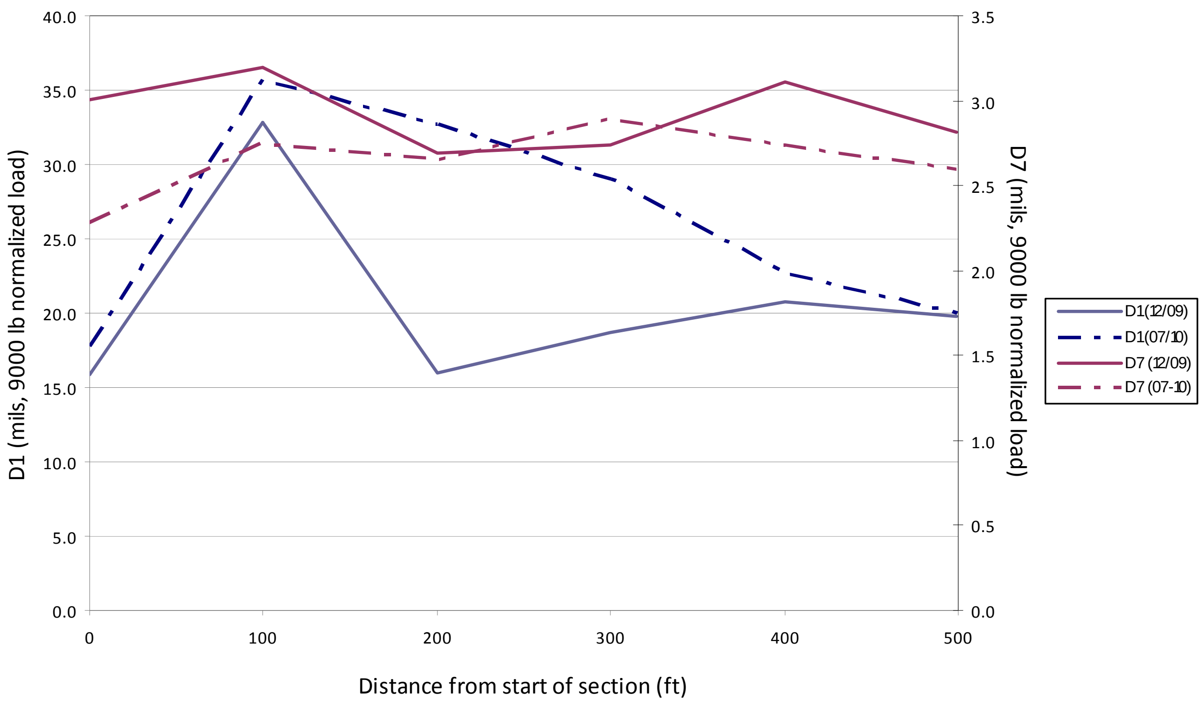


Figure 6.3: Deflections along WMA (Advera) in FM 324, Lufkin (LFWMA1AS)

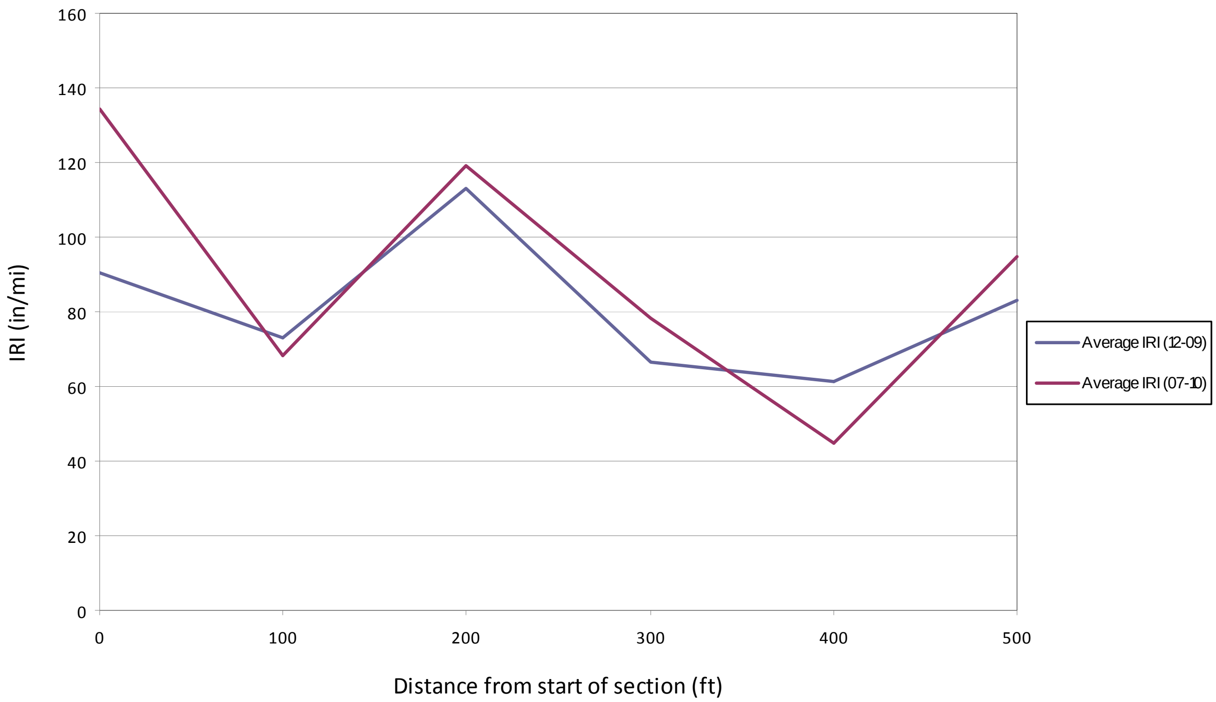


Figure 6.4: Roughness along WMA (Advera) in FM 324, Lufkin (LFWMA1AS)

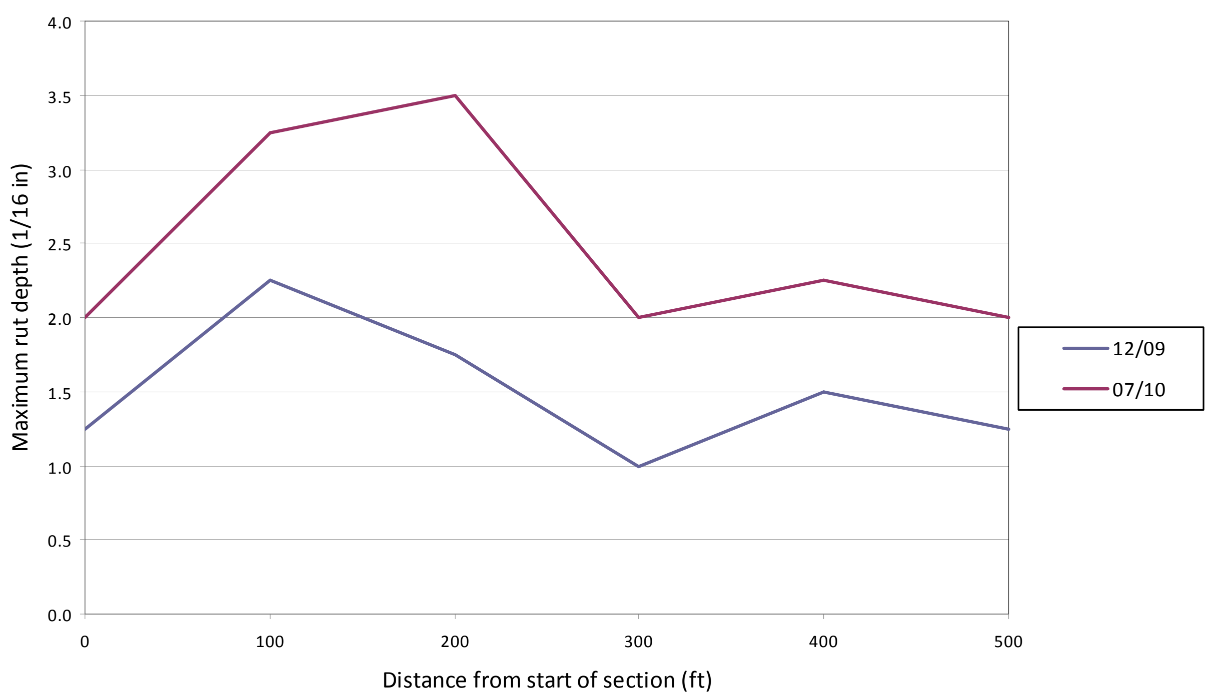


Figure 6.5: Rutting along WMA (Advera) in FM 324, Lufkin (LFWMA1AS)

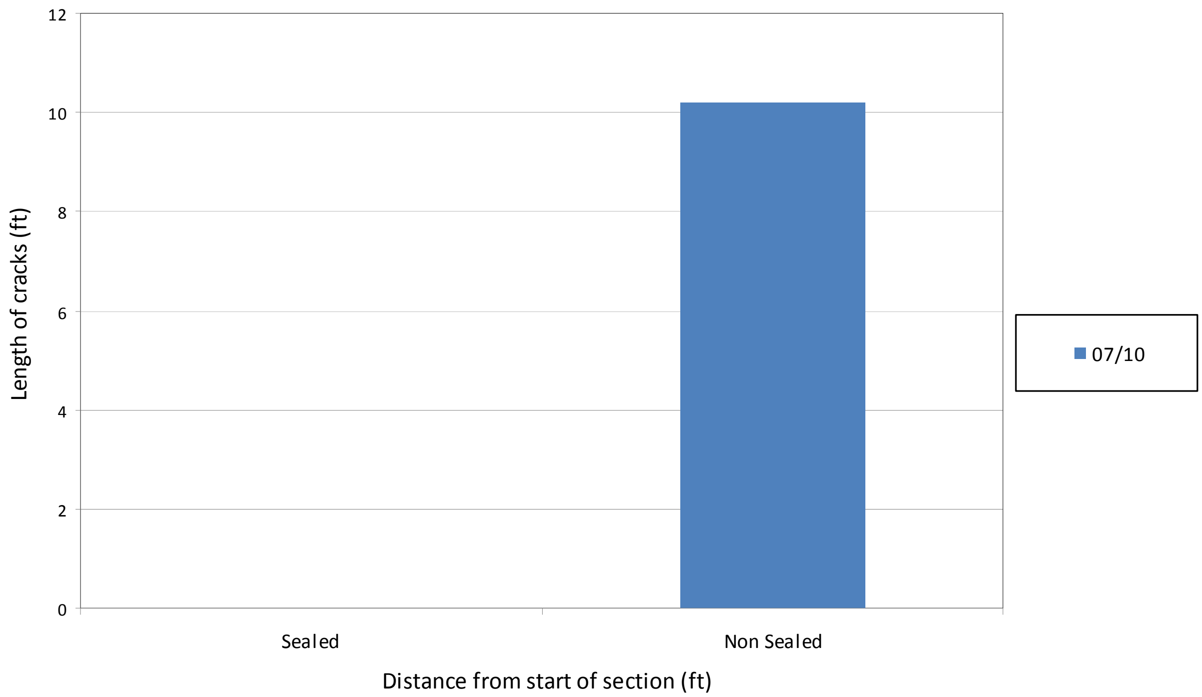


Figure 6.6: Longitudinal cracking along WMA (Advera) in FM 324, Lufkin (LFWMA1AS)

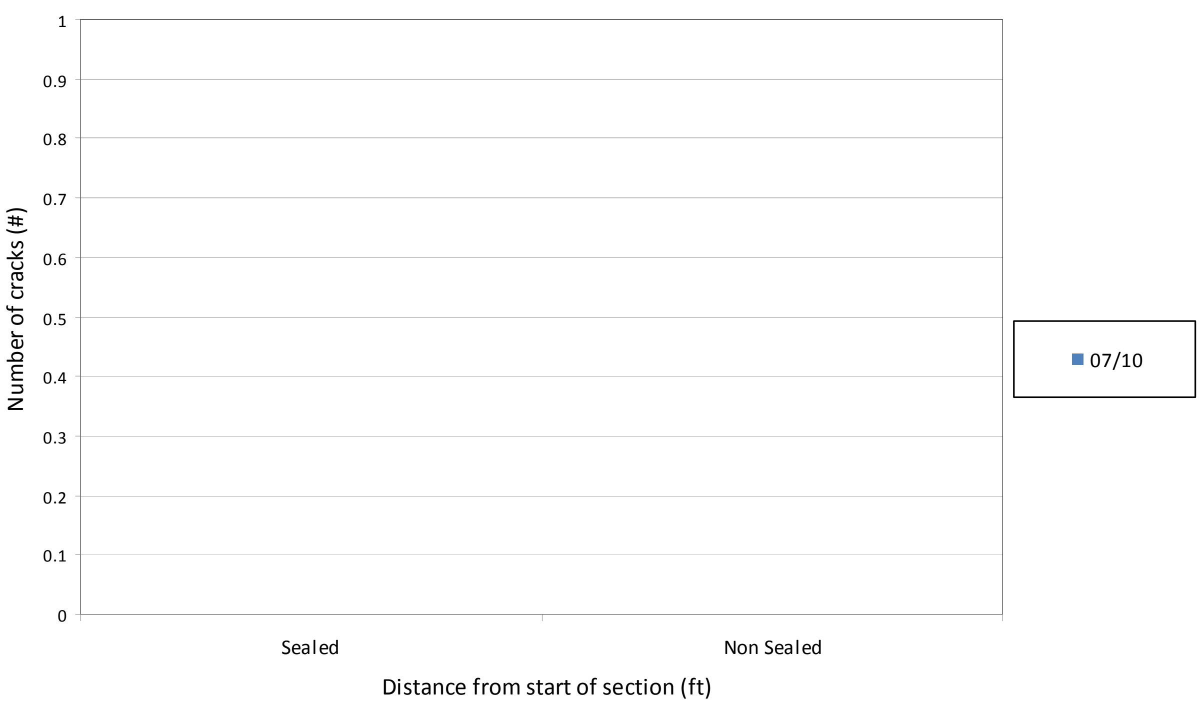


Figure 6.7: Transverse cracking along WMA (Advera) in FM 324, Lufkin (LFWMA1AS)

The results indicate that the performance has slightly decreased based on the observation periods collected as part of the current study. However, overall it appears that the section is still performing well. Nevertheless, in order to better quantify the effect of the different factors involved, regression analysis was performed.

Ideally the research team would have liked to perform LCCA analysis because the experiment designed in Lufkin involves several factors that can be evaluated. Unfortunately, data collection over only 2 years is insufficient to observe performance trends in the pavement structure that can be properly used to model the field performance of other similar structures. Additionally, the pavement sections are still relatively new (slightly over 2 years after construction). For this reason, the research team strongly recommends continued monitoring of the sections so that the effect of WMA and WMA additives on the pavement structure's performance can be better described by means of more sophisticated modeling techniques.

6.2.1 Effect of WMA on Pavement Performance

A regression analysis was performed to characterize the effect of the different factors evaluated in the Lufkin WMA sections (mainly the type of WMA additive that was used) and better quantify their impact on the performance of the pavement structure. The results of the regression analysis are shown in Tables 6.1 through 6.3.

Table 6.1: Effect of different factors on deflection of the Lufkin WMA sections

Factor	D1			D7		
	Coefficients	Standard Error	t Statistic	Coefficients	Standard Error	t Statistic
Intercept	23.95	2.12	11.31	2.52	0.19	13.09
Advera	-0.47	2.87	-0.16	0.27	0.26	1.03
Akzo Nobel	-4.35	2.87	-1.52	-1.33	0.26	-5.10
Evotherm	5.10	2.87	1.78	-0.54	0.26	-2.08
Sasobit	3.55	2.87	1.24	-0.61	0.26	-2.36

In Table 6.1, D1 and D7 correspond to the FWD deflections measured under the loading plate and farther from the loading plate, respectively. Table 6.1 indicates that WMA modified with Advera does not perform significantly different from the control HMA section when observing FWD data (95 percent level of confidence). In the case of WMA modified with Sasobit and Akzo Nobel, the additives have a significant effect on the deflection of the lower layers; however, the effect on the WMA layer is not significant. As for the WMA modified with Evotherm, the additive has a significant effect on the deflection of both the upper and lower layers of the pavement structure.

Note that while all of the additives that significantly affect the deflections of the lower layers of the pavement structure (as measured by D7) produce a reduction in deflections as compared to the control HMA pavement structure, a similar conclusion cannot be generalized for the deflection in the upper layers (as measured by D1). While the sections with the Akzo Nobel additive can be associated with a reduction in deflection, the Evotherm additive resulted in increased deflections. These data imply the degree of surface stiffness afforded by the different WMA additives, i.e., the Akzo Nobel produces a significantly stiffer mixture compared to the other additives.

Table 6.2: Effect of different factors on roughness of the Lufkin WMA sections

Factor	Coefficients	Standard Error	t Statistic
Intercept	87.19	12.92	6.75
Advera	-1.61	17.49	-0.09
Akzo Nobel	53.61	17.49	3.06
Evotherm	-33.19	17.49	-1.90
Sasobit	-38.59	17.49	-2.21

As was the case with deflections, Table 6.2 shows that the Advera additive has no significant effect on the roughness of the WMA. The use of the three remaining WMA additives does have a significant effect on the roughness of the pavement structure. The table shows that while WMA with Evotherm and Sasobit exhibit a reduction in roughness, the WMA with Akzo Nobel is significantly rougher than the control HMA test section.

Table 6.3: Effect of different factors on rutting of the Lufkin WMA sections

Factor	Coefficients	Standard Error	t Statistic
Intercept	1.50	0.20	7.59
Advera	0.50	0.27	1.87
Akzo Nobel	0.35	0.27	1.32
Evotherm	0.42	0.27	1.56
Sasobit	-0.21	0.27	-0.78

The case of rutting shows a different trend from what was observed in the case of roughness and deflection of the WMA. Table 6.3 indicates that only WMA with Advera has a significant effect on the rutting resistance of the pavement structure. However, the effect is negative in the sense that WMA is expected to have higher rutting as compared to the control HMA mix. This was also the case with the all of the other WMA additives (with the exception of Sasobit, which showed rutting levels on par with the control HMA mix). To observe the previous observation, please refer to http://pavements2.ce.utexas.edu:8080/txdot/WMA#Performance.

6.3 Summary and Recommendations

The use of WMA in Texas is still relatively new; the first section, corresponding to Loop 368 in San Antonio, was finished in September 2006. Since its initial applications, TxDOT project 0-5597 monitored several sections and observed mixed results. However, properly characterizing WMA's long-term performance requires continued monitoring and the research team highly recommends it.

Based on the limited analysis performed in this study (only 2 years of observations), WMA might be prone to rutting but that cannot be concluded statistically. Rutting is an important issue in Texas and proper mix design needs to be ensured as well as the continued monitoring of the sections. Performance-related tests such as the Hamburg Wheel Tracking Test (HWTD) need to be performed to increase the probability that rutting will not be a problem.

Similarly, steps must be taken to ensure that the mixture is resistant to cracking. For example, in the case of FM 324 in Lufkin, while WMA with Evotherm and Sasobit has displayed little to no cracking in the 500 ft analysis sections selected as part of the current project, the WMA sections with Advera and Akzo Nobel have recorded 3 and 40 times more cracking than the control HMA section, respectively. This result is very critical because the sections are only approximately 2.5 years old. Note, however, that the conditions of the underlying pavement structure before overlaying are not known and may be significant contributors to this observed performance.

This project's analysis shows that WMA can improve roughness and reduce deflections in the lower layers of the pavement structure (indicating an improvement in the overall pavement strength). For this reason it is important that comprehensive projects such as the one on FM 324 (Lufkin) continue to be monitored for years to come to establish proper characterization of the expected performance of the WMA pavements.

The conclusions in this chapter should be taken with caution because they are preliminary and based on very limited performance data: only 2 years. Furthermore, the conditions of the pavement structure and the subgrade before the overlay were not known at the time of the analysis. Due to the significant tonnage of WMA being placed in Texas, the researchers strongly recommend that the monitoring of these and other WMA section continue. Many unknowns remain with WMA technology that only time and data will reveal.

Chapter 7. Perpetual Pavements

A perpetual pavement is defined as an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal to repair superficial distresses. The concept of perpetual pavements, or long-lasting HMA pavements, is not new. Full-depth and deep-strength HMA pavement structures have been constructed since the 1960s, and those that were adequately designed and constructed have been very successful in providing long service lives under heavy traffic.

The concept of a perpetual pavement is that a very thick layer of HMA is placed over a strong foundation design, such that it will not sustain structural damage under heavy traffic loads. If surface-initiated cracking and rutting can be detected and corrected by minimal surface repairs before they impact the structural integrity of the pavement, the pavement design life could greatly increase. The basic structural concept uses a thick HMA pavement over a strong foundation design with three layers, each one tailored to resist specific stresses, namely, from bottom to top:

- HMA base layer. This is the asphalt bottom layer designed specifically to resist fatigue cracking. Two approaches are used to resist fatigue cracking in the base layer. First, make the total pavement thickness great enough such that the tensile strain at the bottom of the base layer is insignificant. Alternatively, use an extra-flexible HMA for the HMA base layer, most easily accomplished by increasing the asphalt content or using modified binders. Combinations of these two approaches can also work.
- Intermediate layer. This is the middle layer designed specifically to carry most of the traffic load. Therefore, it must be stable (able to resist rutting) as well as durable. Stability is best achieved by using stone-on-stone contact in the coarse aggregate and using a binder with the appropriate high-temperature grading.
- Wearing surface. This top layer is designed specifically to resist surface-initiated distresses such as top-down cracking and rutting. This layer is considered a renewable surface that can be milled and replaced intermittently to provide the required functional performance, in relation to the appearance of distresses on the surface.

When scheduled surface restoration is performed, perpetual pavements can be maintained easily and cost-effectively without removing the road structure for reconstruction, saving time and money, and causing little inconvenience to the users.

7.1 Evaluation of Perpetual Pavement Performance

This project evaluated perpetual pavements along IH 35 in Laredo and San Antonio, and on SH 114 in Fort Worth.

7.1.1 SH 114 in Fort Worth

This perpetual pavement is located on SH 114 in Wise County, only in the eastbound direction (let in 2003). The section is approximately 0.2 miles east of the intersection with US Highway 81/287. It is approximately 2.2 miles in length, consisting of two 12-ft lanes and 10-ft shoulders, ending at the Denton County line. SH 114 is a heavily trafficked highway with an ADT of approximately 18,000 (Walubita and Scullion, 2007). As of 2003, truck composition was about 27.3 percent, with a designated maximum speed limit of 70 mph. In 1997, the Texas Hot Mix Asphalt Paving Association (now known as the Texas Asphalt Pavement Association, or TxAPA) approached the District about constructing a full-depth ACP perpetual pavement adjacent to a continuously reinforced concrete pavement in the eastbound direction of SH 114. The District agreed; the sections were constructed and opened to traffic in 2005.

To monitor the performance of the perpetual pavement structure, this project defined a 500 ft long section and collected performance data on both the outside and inside lanes (eastbound direction). For a better description of the location, please refer to Figure 7.1.

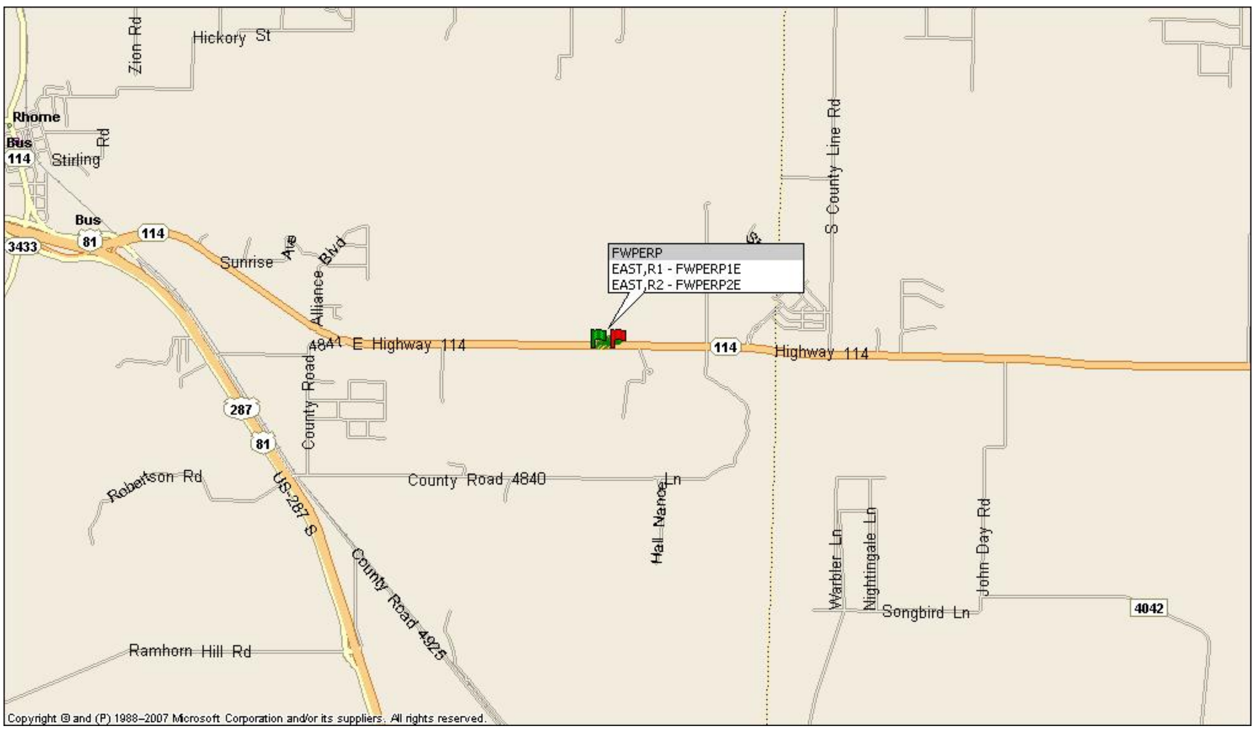


Figure 7.1: Location of perpetual pavement sections on SH 114, Fort Worth

The District also developed pavement alternates (full-depth AC perpetual pavement using FPS-19W and CRCP using the AASHTO procedure) and included them in the plans for the westbound direction of SH 114. The contractor chose the full-depth AC option in the westbound direction. The perpetual pavement designs used Superpave specifications. During construction of the perpetual pavement in the eastbound direction, District personnel discovered that the Superpave layers were permeable. As a result, the District executed a Change Order with the contractor to install under-drains and to place a seal coat on the full-depth ACP pavement before placement of the stone matrix asphalt (SMA) surface. This step prevented further moisture intrusion and resulting damage according to District personnel. However, the District realized that the under-drains will need to be maintained and the seal coat replaced when the SMA is removed in the future. As a result of the experience with the permeability of the full-depth AC structure, the District did change the design in the westbound direction to CRCP (again through executing another change order with the contractor). Because of the construction and performance issues with the Superpave mixes, the Area Engineer decided to incorporate a section of conventional full-depth asphalt into the SH 114 project. The final SH 114 full-depth asphalt pavement (FDAP) project consisted of two structural sections:

- Superpave section (about 1.7 miles), designated herein as FW 01, designed with a Superpave mix
- Conventional section (about 0.25 miles), designated herein as FW 02, designed with the conventional TxDOT mix.

In terms of construction, the Superpave section is at least 1 year older than the Conventional section. The structural design was mechanistic-empirically based using the FPS 19W for a 30-year design period, at a 95 percent reliability level. The total pavement thickness on both sections is 30 inches, i.e., 22 in. of four HMA layers plus 8 in. of 6 percent lime-treated subgrade as the base. The mix design is based on the Superpave volumetric design system with 100 gyrations to achieve 4 percent air voids (AV) (i.e., 96 percent density) as the design criterion, except for the RBL at 97 percent. Additionally, the HMA mixtures also had to pass the Hamburg wheel tracking test at a maximum rut depth of 0.5 in. The rut-resistant layer is 13 in. thick and consists of a 1-in. SFHMAC mixture (4 percent PG 70-22, plus limestone) on the Superpave section and TxDOT Type B (4.5 percent PG 64-22, plus limestone) on the Conventional section. The fatigue-resistant layer (the RBL) is 4 in. thick and consists of a 0.75-in. SFHMAC mixture (4.2 percent PG 64-22, plus limestone) on the Superpave section and TxDOT Type B (5.3 percent PG 64-22 plus limestone) on the Conventional section.

The eastbound perpetual pavement sections, which are of most interest to this chapter, are performing properly as Figures 7.2–7.4 show. The data corresponds to the outside lane; however, data are also available for the inside lane. To access the information, please refer to:
http://pavements2.ce.utexas.edu:8080/txdot/Perpetual%20Performance.

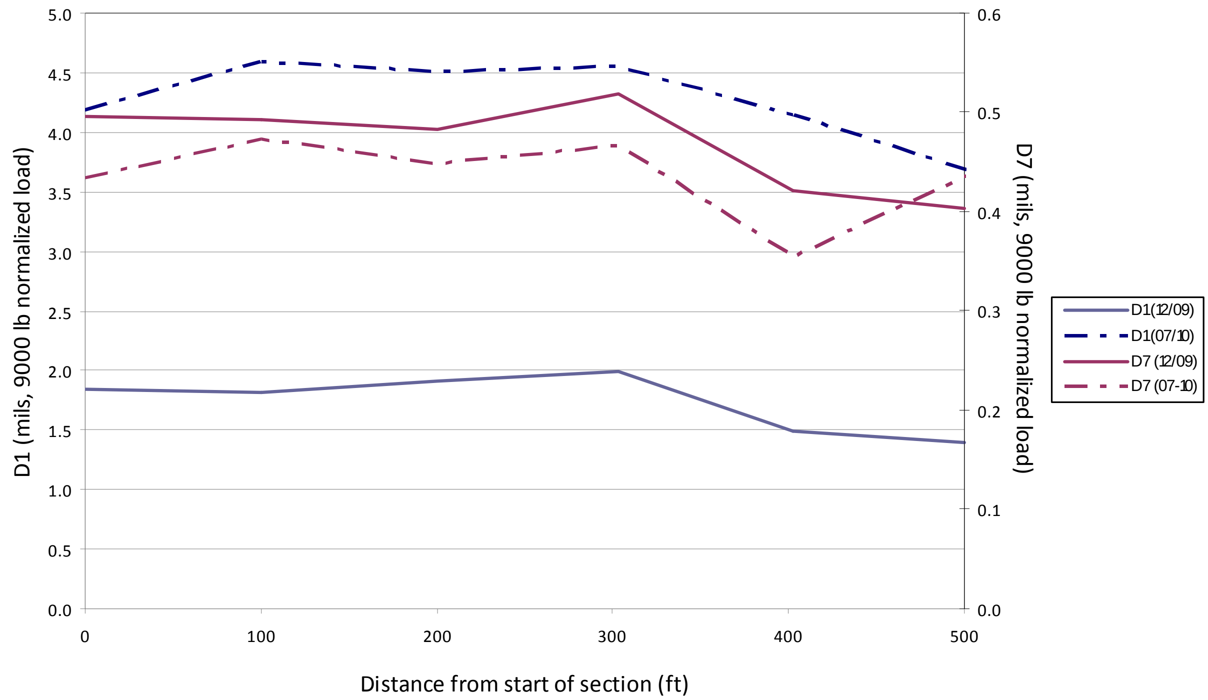


Figure 7.2: Deflections along perpetual pavement structure, R1, Fort Worth (FWPERP1E)

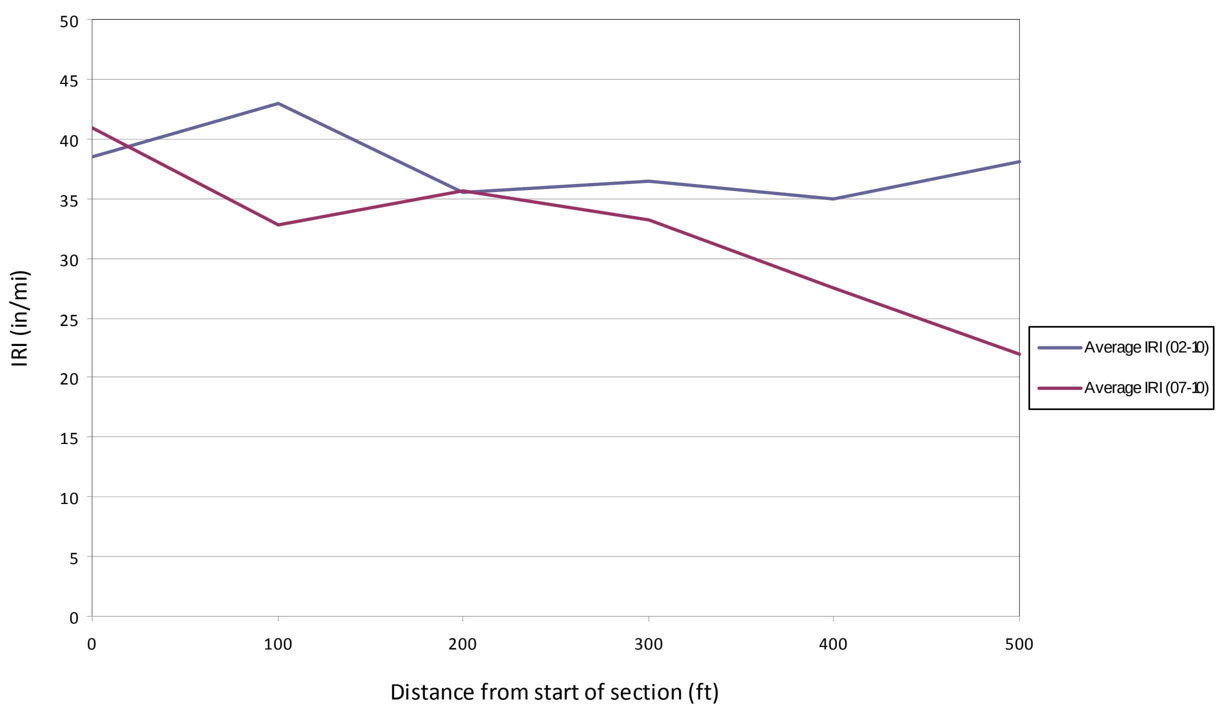


Figure 7.3: Roughness along perpetual pavement structure, R1, Fort Worth (FWPERP1E)

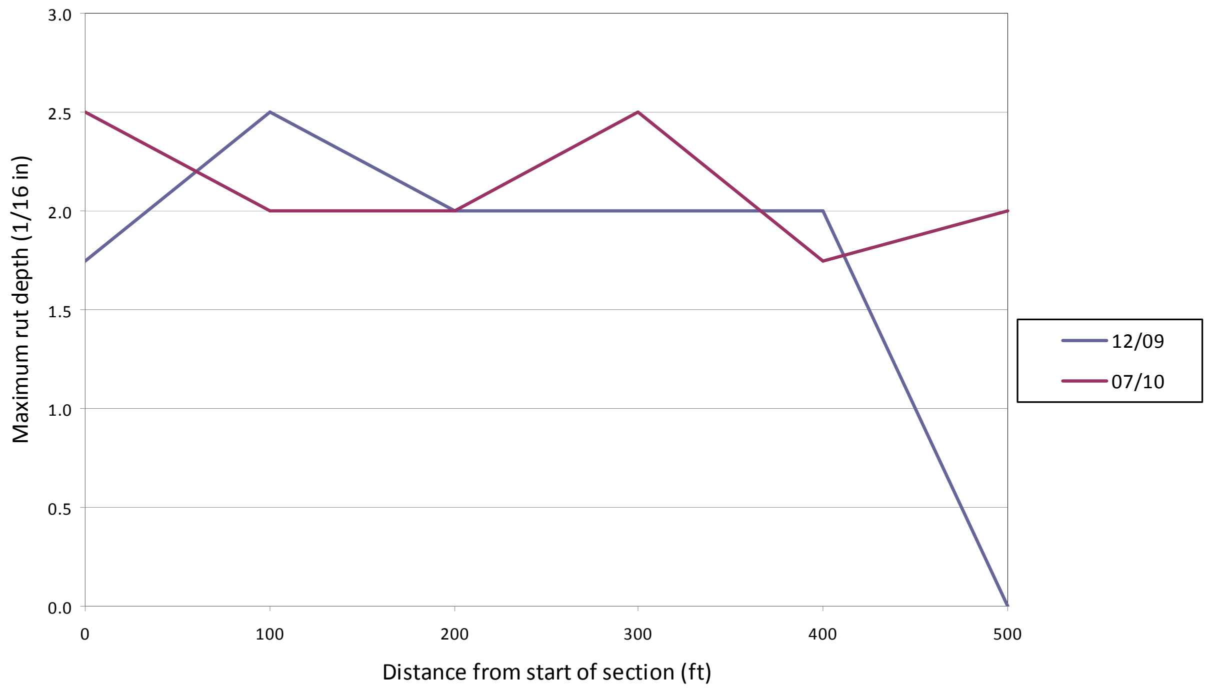


Figure 7.4: Rutting along perpetual pavement structure, R1, Fort Worth (FWPERP1E)

Overall the perpetual pavement section has displayed proper performance after 5 years of service in the field. The structure shows more than adequate strength, as evidenced by the small deflections under the normalized FWD load. Additionally, the IRI values are low, also indicating proper riding quality. Furthermore, the sections display very little rutting (averaging under 2/16 in.) and no cracking.

Unfortunately, for a perpetual pavement section, the structure is relatively new. Also, in order to properly characterize its performance, 20 or 30 years of observation are required to evaluate if the perpetual pavement structure holds to its name. For this reason, the research team strongly suggests that monitoring of the section continue in the future. Continued monitoring will provide very valuable information that can be used to properly compare a perpetual pavement structure with alternatives and to proper quantify the economic benefits of this type of pavements.

7.1.2 IH 35 in San Antonio

This perpetual pavement structure is located along IH 35 in New Braunfels, San Antonio (0.5 mi south of SH 46 to 0.35 mi north of FM 306). The structure of the pavement consists of a 1.5-in. PFC (PG72-22), 2 in. SMA (PG76-22), 2 in. Superpave Type F with a NMAS of 0.75 in. (PG64-22), 12 in. Superpave Type F with a NMAS of 1 in. (PG64-22), 4 in. RBL (PG64-22), and 6 in. of subgrade treated with 3 percent lime. The section was completed late in 2006.

For this project's purposes, two 500-ft pavement sections were selected to characterize the performance of the perpetual pavement structure. The location of the pavement sections can be observed in Figure 7.5. FWD, IRI, rutting with a 6-ft straight edge, and visual cracking assessments were performed.

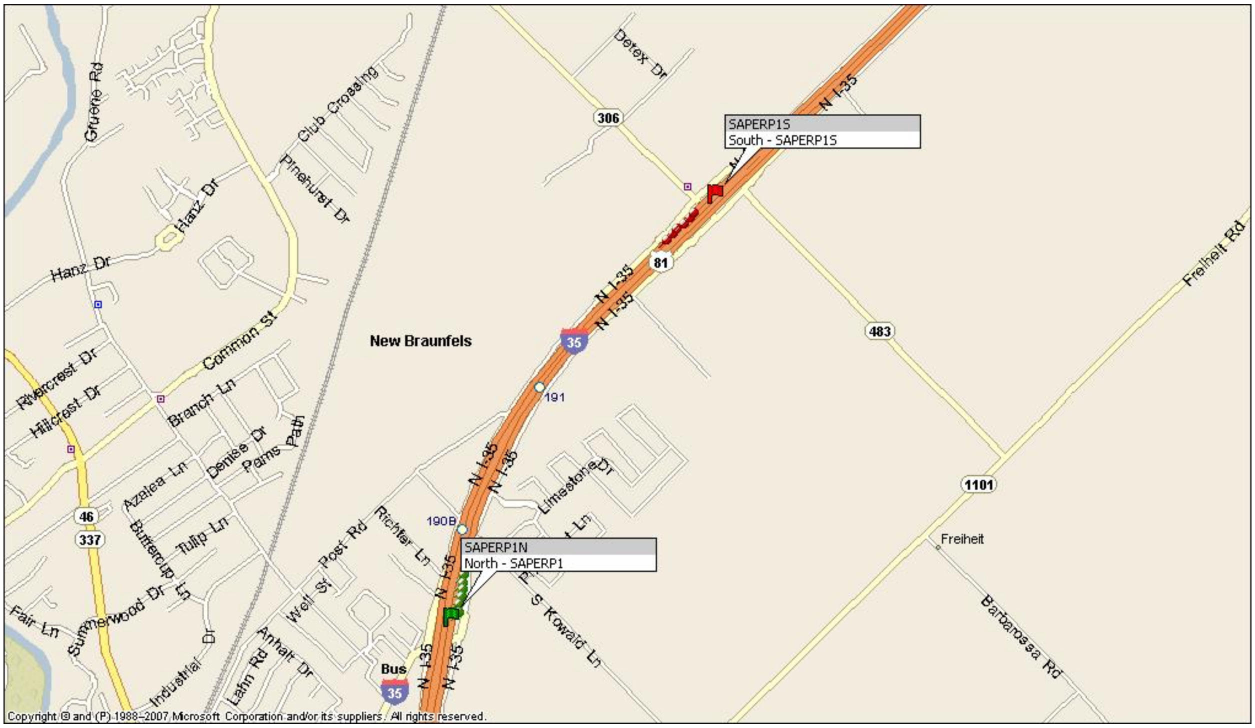


Figure 7.5: Location of perpetual pavement sections on IH 35, San Antonio

As an example of the collected data, Figures 7.6–7.8 show the performance of the northbound perpetual pavement section. The data corresponds to the outside lane (R1). Data is also available for the southbound lane (L1). To access the information, please refer to http://pavements2.ce.utexas.edu:8080/txdot/Perpetual%20Performance.

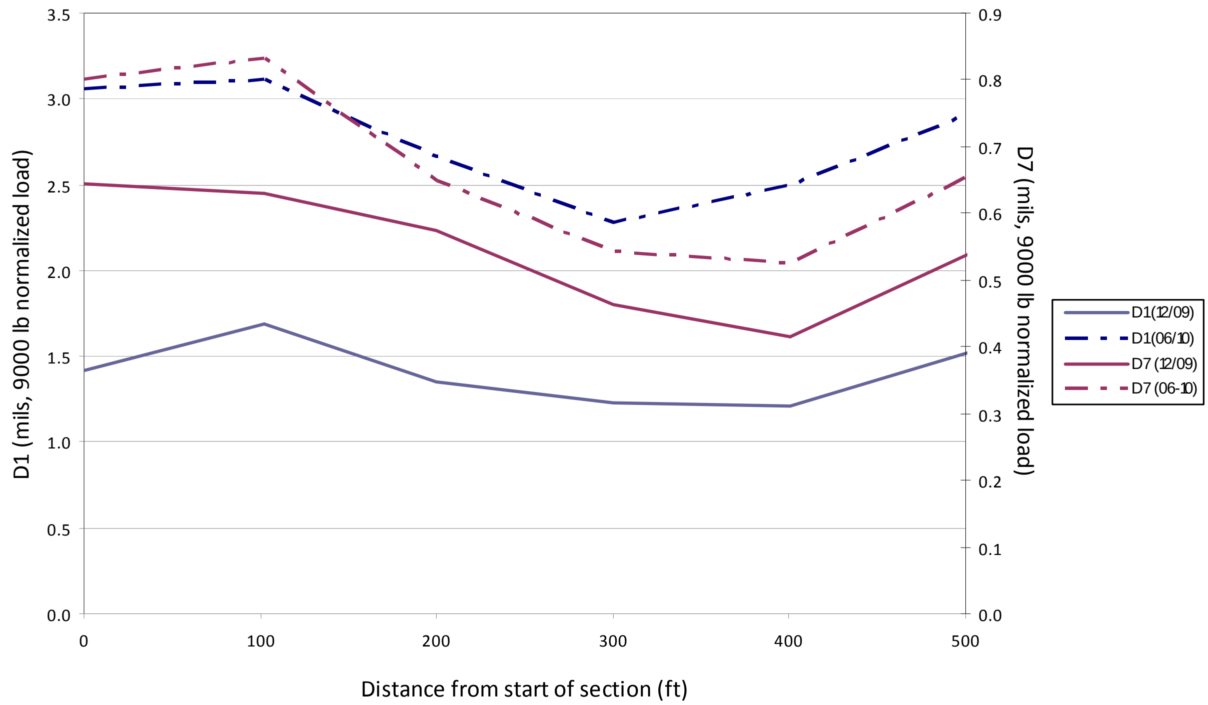


Figure 7.6: Deflections along perpetual pavement structure, R1, IH 35 (SAPERP1N)

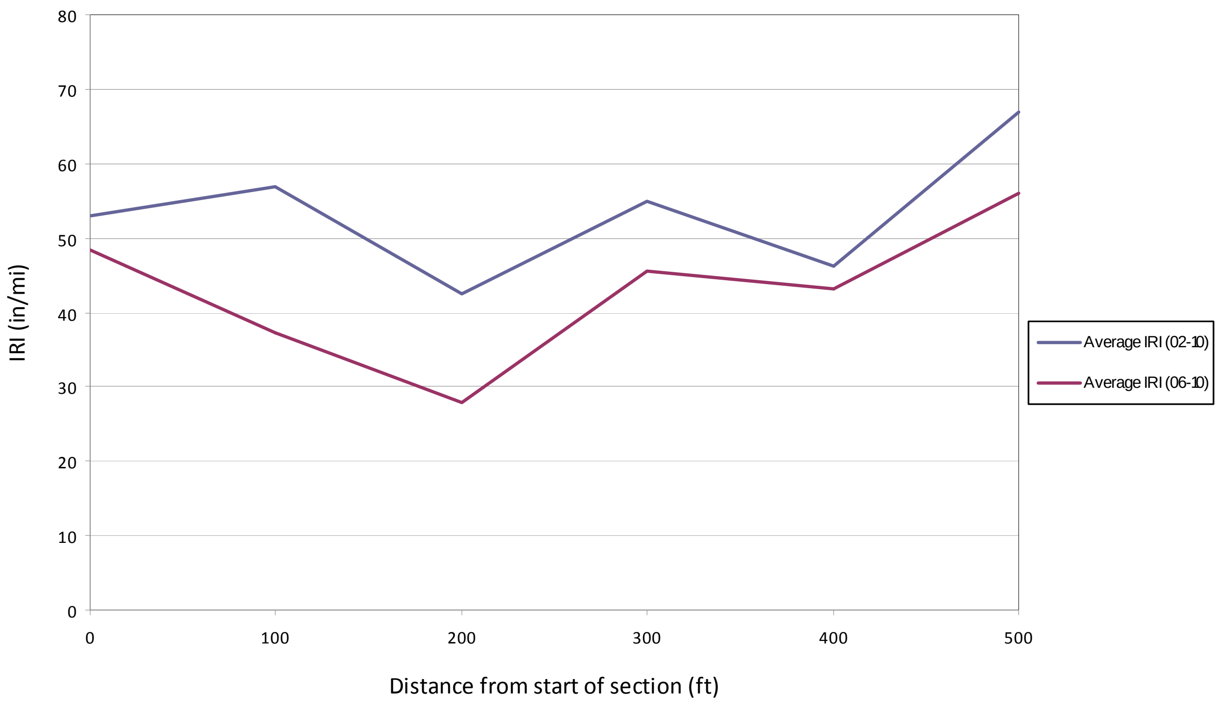


Figure 7.7: Roughness along perpetual pavement structure, R1, IH 35 (SAPERP1N)

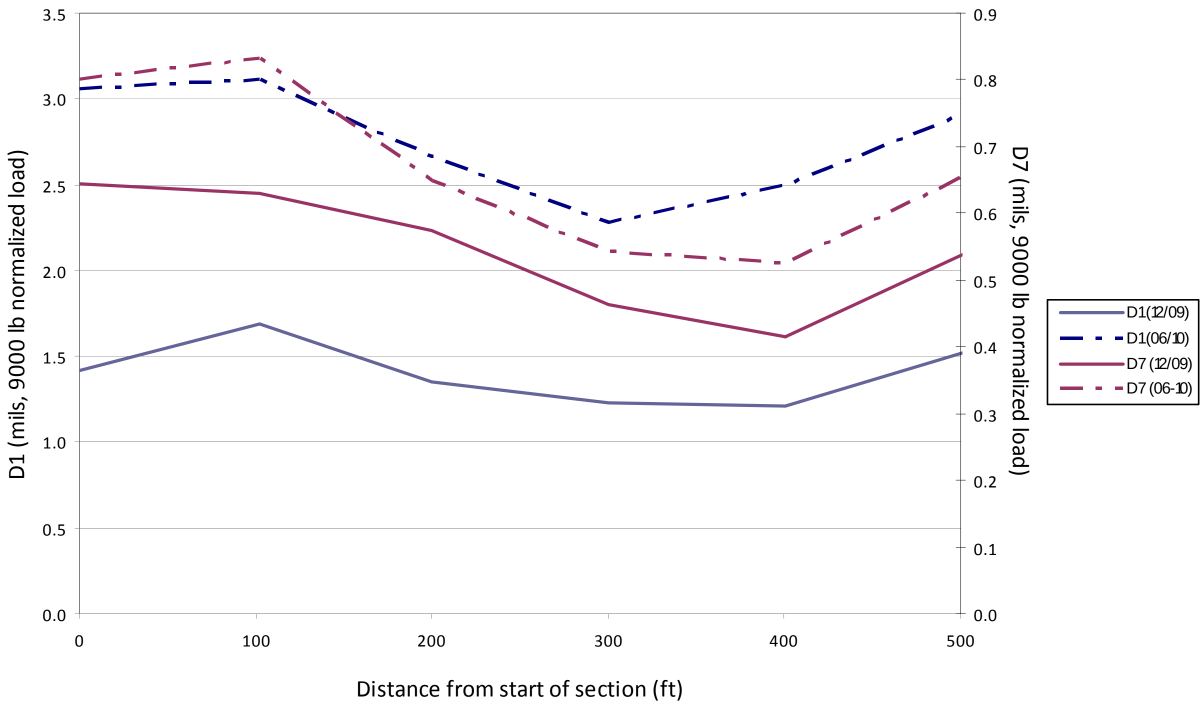


Figure 7.8: Rutting along perpetual pavement structure, R1, IH 35 (SAPERP1N)

In general, the perpetual pavement section has displayed acceptable performance after 4 years of service in the field. The structure exhibits high structural capacity, as evidenced by the small deflections under the normalized FWD load. The IRI values are slightly higher than the perpetual pavement section in SH 114 in Fort Worth. However, the traffic in these sections is much higher and the IRI is still relatively low. Furthermore, the sections display very little rutting (averaging under 3/16 in) and no cracking can be observed.

Ideally, properly characterizing the performance of the perpetual pavement structures requires many more years of data. For this reason the research team recommends continued collection of performance data of these pavement sections. However, in an effort to compare the performance of the these pavement structures to conventional HMA pavement structures, PMIS data was identified for the sections and compared to the performance of conventional HMA pavement sections (along the same lanes) 10 miles north along IH 35.

The researchers found that for the southbound lanes, in 2007 and 2008 the performance of the perpetual pavement was excellent, as described by a Distress Score (DS) and a Condition Score (CS) of 100. On the other hand, the conventional HMA pavement section exhibited a 10 point loss in both DS and CS from 2007 to 2008, so it is deteriorating at a much faster rate (the conventional HMA pavement section was completed 1 year prior to the perpetual pavement section). The perpetual pavement also showed higher Ride Score (RS) and lower IRI values (32 percent lower). Note that the perpetual pavement section is performing better even though it is subjected to higher AADT: in 2008 45,750 versus 30,978 (and kESALs: 31,163 vs. 25,252). Also important to note is that the annual maintenance costs for the perpetual pavement section have been almost 2.5 times lower than those of the conventional HMA structure.

For the northbound lanes, in 2007 and 2008 the performance of the perpetual pavement was also excellent, as described by a DS and a CS of 100. On the other hand, the conventional

HMA pavement section exhibited a 5 point loss in both DS and CS from 2007 to 2008, so it also deteriorated at a higher rate. The perpetual pavement also showed higher RS and lower IRI values (51 percent lower). Similar to the southbound lanes, the perpetual pavement section is performing better even under higher AADT: in 2008 43,750 versus 30,978 (and kESALs: 31,163 vs. 25,252). The annual maintenance costs for the perpetual pavement section were also 2.5 times lower than those of the conventional HMA structure.

7.1.3 IH 35 in Laredo

These perpetual pavement structures are located along IH 35 in Laredo: 1) northbound lanes between TRM 69+0.44 and TRM 74; 2) between TRM 58 and TRM 65+0.36; and 3) between TRM 49+0.43 and TRM 53+0.43). The structure of the pavements consists of a 3 in. SMA (PG76-22), 3 in. Superpave Type F with a NMAS of 0.75 in. (PG76-22), 8 in. Superpave Type F with a NMAS of 1 in. (PG70-22), a RBL (PG64-22), and 8 in. of subgrade treated with either 3 percent lime or 2 percent cement. For details of the thicknesses of the RBL and details of the treated subgrade please refer to Table 2.7. The sections were completed between 2003 and 2005.

For research purposes, the project selected five 500-ft pavement sections to characterize the performance of the perpetual pavement structures. Figure 7.9 shows the location of the pavement sections.

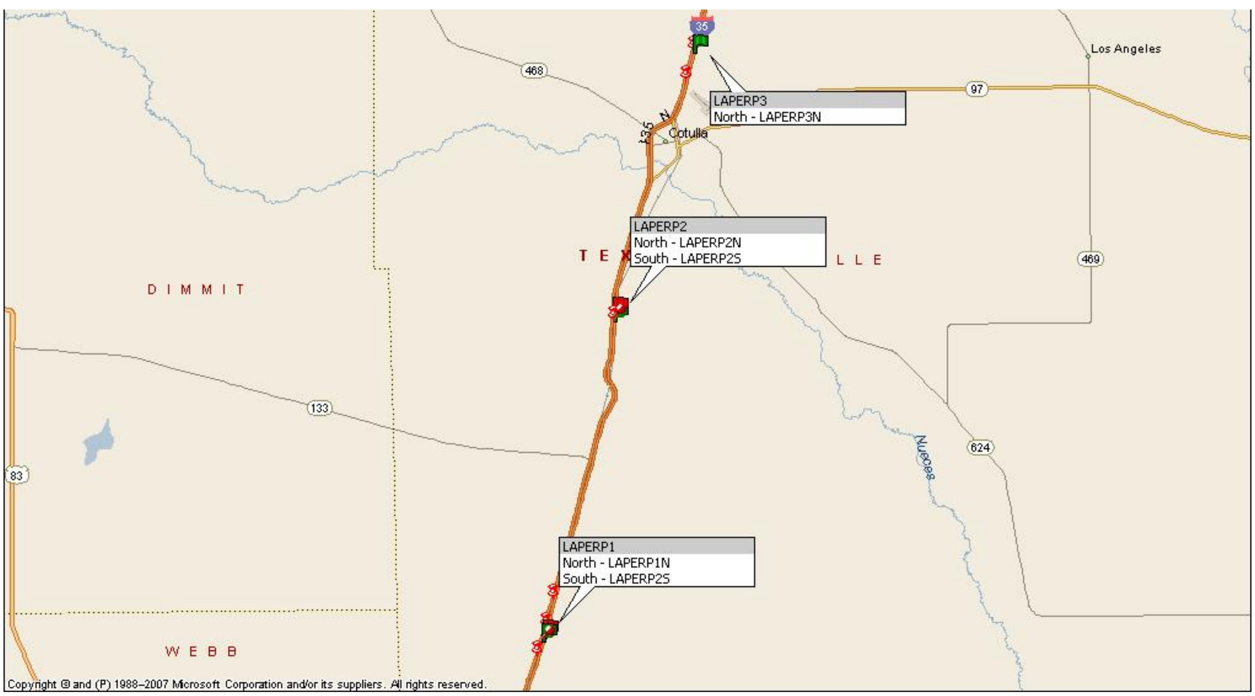


Figure 7.9: Location of perpetual pavement sections on IH 35, Laredo

Figures 7.10–7.12 provide an example of the collected data: the performance of one of the northbound perpetual pavement sections (LAPERP1N). The data corresponds to the outside lane (R1). Notice that Figure 7.12 displays the average rut depth for the left and right wheel path, 2/16 and 1/16 in., respectively. Data are also available for the remaining four sections. To access the information, please refer to:

http://pavements2.ce.utexas.edu:8080/txdot/Perpetual%20Performance.

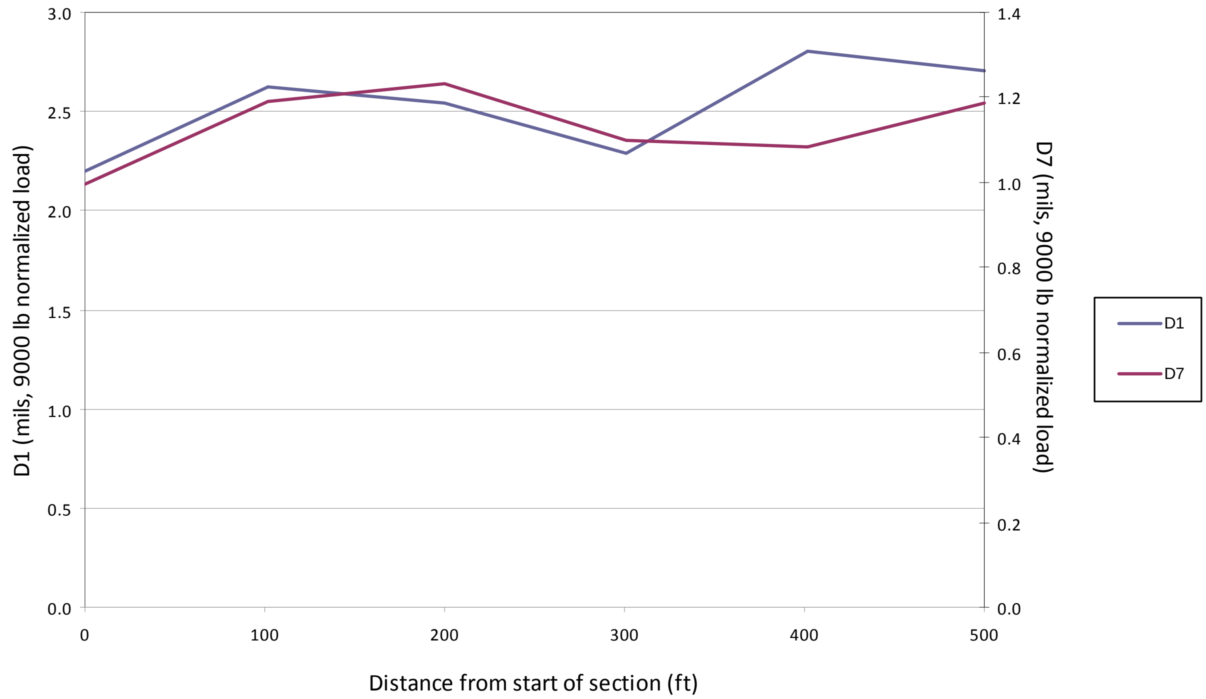


Figure 7.10: Deflections along perpetual pavement structure, R1, IH 35 (LAPERP1N)

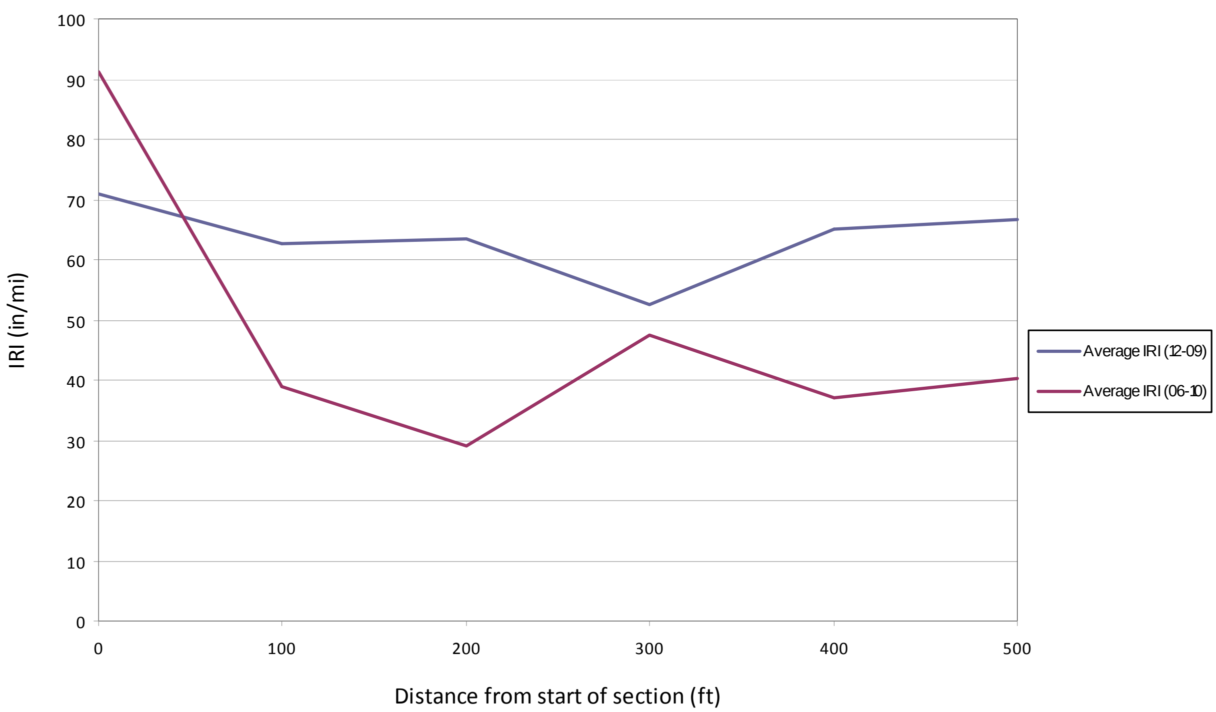


Figure 7.11: Roughness along perpetual pavement structure, R1, IH 35 (LAPERP1N)

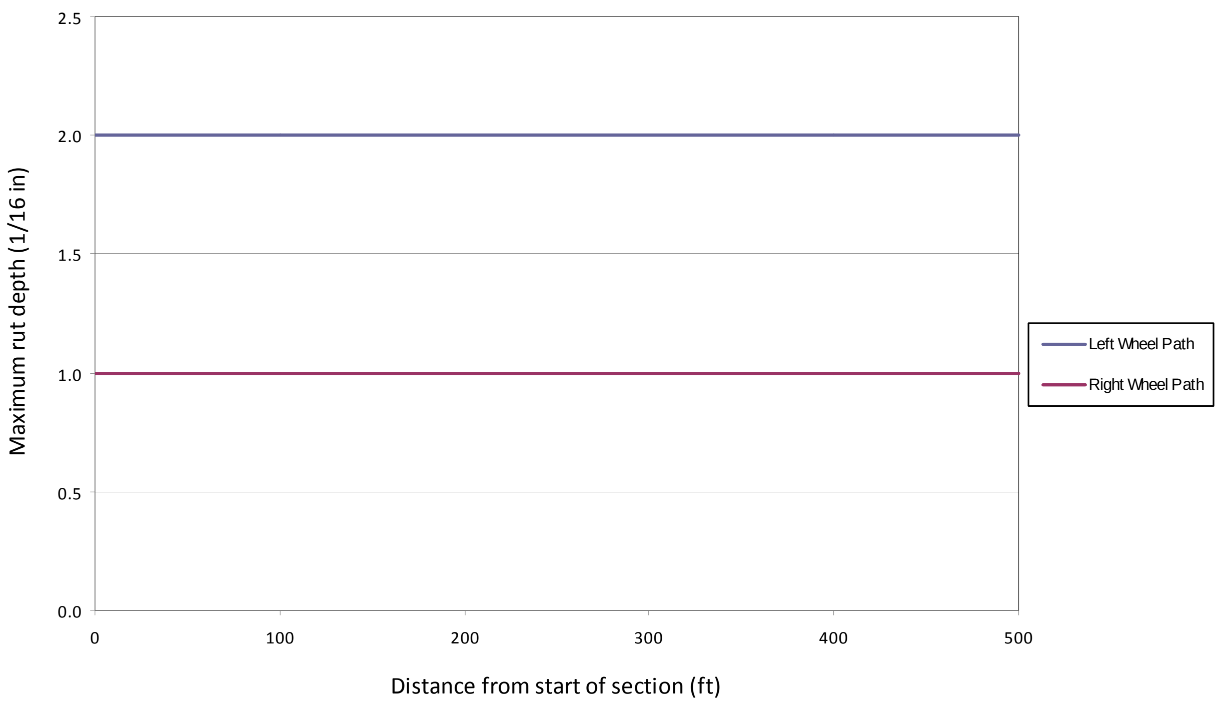


Figure 7.12: Rutting along perpetual pavement structure, R1, IH 35 (LAPERP1N)

The Laredo perpetual pavement structures display adequate performance after 5 to 7 years of service in the field. These structures also show high strength, as the small deflections under the normalized FWD load evidence. However, out of all the evaluated perpetual pavements, the IRI and rutting are on average slightly higher. Even the observed values of IRI and rutting are still low and can be considered appropriate. Furthermore, these sections have been in the field longer than the perpetual pavements in San Antonio and Fort Worth.

The researchers performed an analysis similar to the one performed for the San Antonio perpetual pavement based on available PMIS data. PMIS data was identified for the sections and compared to the performance of a conventional HMA pavement sections (along the same lanes) immediately north of the perpetual pavement, along IH 35.

For the southbound lanes, in 2007 and 2008 the performance of the perpetual pavement was excellent, as described by a DS and a CS of 100. In the case of the conventional HMA pavement section, considerable maintenance work was performed in 2006. Regardless, it exhibited a 13 point loss in CS and a 3 point loss in DS, indicating a much higher deterioration rate. The perpetual pavement also showed higher RS and lower IRI values (127 percent lower). The traffic in both the perpetual pavement section and the conventional HMA pavement section was nearly identical. However, due to higher deterioration rates in the conventional HMA structure, higher yearly maintenance investment was required: 1.4 times in 2007 and 8.4 times in 2008, as compared to the perpetual pavement structure.

For the northbound lanes, in 2007 and 2008 the performance of the perpetual pavement was also excellent, as described by a DS and a CS of 100. The conventional HMA pavement section exhibited a 2 point loss in both DS and CS from 2007 to 2008, so it is performing much better than the conventional HMA pavement structure in the southbound direction; however, the performance of the perpetual pavement structure is still superior. The perpetual pavement also showed higher RS and lower IRI values (28 percent lower). Similar to the southbound lanes, the traffic is almost equivalent for both the perpetual pavement structure and the conventional HMA structure. Regardless, higher yearly maintenance investment was required for the conventional HMA pavement structure: 1.1 times in 2007 and 7.5 times in 2008, as compared to the perpetual pavement structure.

7.2 Summary and Recommendations

As based on the initial findings, the use of perpetual pavements appears to be promising. Although initial construction costs might be higher than those of a conventional pavement, the deterioration rates of such pavements are considerably lower than that of conventional HMA pavement structures. Considering the costs of maintenance and associated user costs, the life-cycle cost of a perpetual pavement may be lower than that of conventional HMA pavement structures.

Proper LCCA would require long-term monitoring of the perpetual pavement structures to identify how they perform over time. For this reason, the research team recommends continued monitoring of the perpetual pavement structures, especially because most of the perpetual pavement structures evaluated are less than 8 years old, which is relatively short considering the 25 or more years of service life for which these pavement types are designed. These sections should be monitored for at least another 10 years for the conclusions to be meaningful. However, the application of perpetual pavement technology in Texas has proven to be very effective.

As was confirmed by the previous analysis, perpetual pavement structures show high strength (as indicated by very small deflections under the FWD load). Additionally, most of the evaluated sections exhibited very little rutting and little to no cracking. On the other hand, some of the sections do show an increase in roughness. However, surface renewals can easily address the roughness to improve ride quality, because additional structural capacity is not required.

Chapter 8. Crack Attenuating Mixes

A CAM is a HMA mixture designed to reduce reflective cracking in hot mix overlays, but also exhibit high rut resistance. It is typically placed as an interlayer between an existing pavement and a surface layer of hot mix. The current Texas statewide special specification (SS) for CAM is SS 3165. However, on many occasions CAM mixes have been placed as a surface course under the right conditions. A few CAM mixes have been placed as the final riding surface and are being monitored to quantify their performance. It should be kept in mind that this mix, being fine-graded, lacks macro-texture, and vehicles may become more susceptible to loss of friction. While this may not be an issue on residential streets or roadways with lower posted speed limits (< 45mph), friction and surface texture are needed at highway speeds.

The requirement of both Hamburg Wheel and Overlay Tests makes the CAM, in principle, a good choice to address rutting, fatigue, and reflective cracking distresses. RBL mixtures also provide improved resistance to cracking. In some instances CAM and RBL have been used interchangeably; however, they are very different mixes with different intended applications. Both mixes are fine graded and provide an integral contribution to the overall pavement design; however, they are designed to perform different functions to address different pavement conditions. A CAM may be used in lieu of a RBL; however, using a RBL in lieu of a CAM is not recommended.

CAM is designed to a laboratory density of 98 percent at Ndes = 50 gyrations using the Superpave Gyratory Compactor (SGC). The gradation used to design this mix is sometimes referred to as a *screenings mix*. The minimum binder content is 7.0 percent (7.0 to 8.0 percent is typical). Although not specified in the plans, a PG 76-22 binder is usually required to meet both the Hamburg Wheel Test and Overlay Test requirements outlined in the specification. Several mixes have been designed and successfully produced in the past with PG 70-22 and have performed well. Class A aggregate is not required unless used as a surface course and desired by the District; however, most contractors have found that Class A aggregates are needed to meet the Hamburg and Overlay requirements.

TxDOT Project 0-5598 evaluated various thin HMA overlays in Texas. Report 0-5598-1 presented some preliminary results (Walubita and Scullion, 2008). This study designed and evaluated mixes based on the balanced mix-design concept and the CAM special specification 3109 for their potential application as very thin overlay mixes. The research methodology incorporated extensive laboratory testing and field experiments including the Hamburg Wheel, Overlay Tester, and GPR. Due to some conflicting design and performance information about CAM, more research is recommended with these mixes. With the HMA mixes, promising laboratory results have been obtained with fine-graded (3/8-inch nominal maximum aggregate size) mixes; predominantly composed of Type F rock and screenings and an asphalt-binder content of over 7 percent. Based on the TxDOT CAM SS 3109 specification, high quality clean Class A aggregates, such as granite, exhibited superior laboratory performance and are recommended. However, acceptable laboratory designs were also obtained with good quality sandstone and limestone materials. The initial field performance of these mixes has been acceptable; however, some problems have been found when these mixtures have been used at intersections.

8.1 CAM Performance Evaluation

This project evaluated six CAM projects in Texas, located in the Paris, Bryan, and Laredo Districts. Table 8.1 summarizes the list of the evaluated projects.

Table 8.1: Location of observed CAMs

District	County	Route	Layer	PG Grade	SAC
Laredo	Webb	LP 20	Surface	76	A
Paris	Grayson	US 69	Surface	70	A
Paris	Hopkins	FM 499	Surface	70	A
Bryan	Brazos	FM 60/2347	Surface	70	A
Bryan	Brazos	FM 2154	Surface	70	A
Bryan	Brazos	FM 1179	Surface	70	A

Each of these previous projects selected a 500 ft long section for evaluation. In all of the sections a cracking visual survey, surface rutting measurements, and roughness testing (with TxDOT profiler) were performed. For detailed location of the sections and their performance data, please refer to http://pavements2.ce.utexas.edu:8080/txdot/CAM.

Some additional information on the quality control/quality assurance (QC/QA) testing performed during the construction of three of the CAM sections was identified in SiteManager as shown in Table 8.2 (AASTHO 2002). This information includes asphalt contents, gradations, and design air voids.

Table 8.2: Material properties for some of the monitored CAM sections

Route	HWTD (cycles)	Asphalt Content (%)	Air Voids (%)	Percent Passing (%)			
				3/8"	No.8	No.30	No.200
FM 69	-	7.20	2.04	99.0	36.9	20.6	6.1
FM 499	17,500	7.73	2.03	99.9	43.1	23.4	5.7
FM 1179	-	7.33	2.02	98.2	54.9	22.4	4.8

From the previous table it is clear that the asphalt binder contents used are high, but this is in part due to the higher fine content associated with CAMs and to ensure cracking resistance. The air void content is low, and although only one of the projects contained HWTD (indicating that the mix passed the test), these mixtures may be susceptible to rutting as was observed from rutting measurements and visual assessments of the sections.

As an example, the following figures display the performance data for the Laredo section (Loop 20). Figure 8.1 shows the location of the surface CAM pavement section, and Figure 8.2 shows rutting along the section.

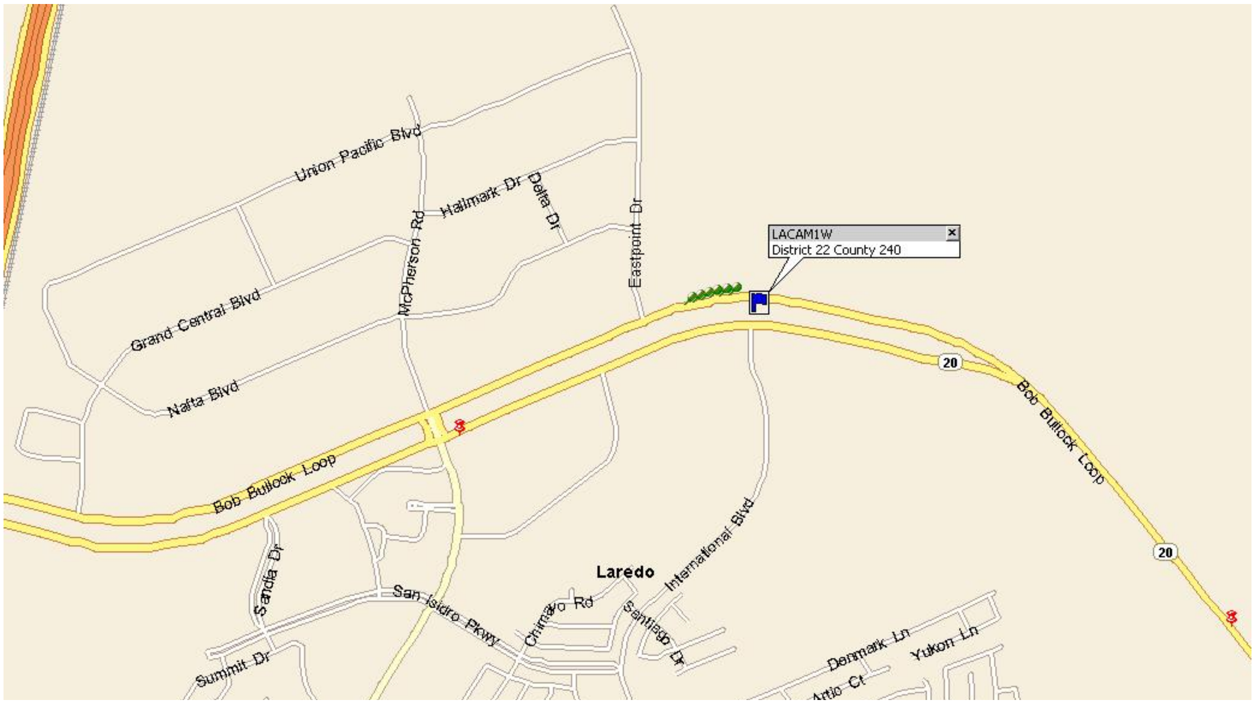


Figure 8.1: Location of CAM section on LP 20, Laredo

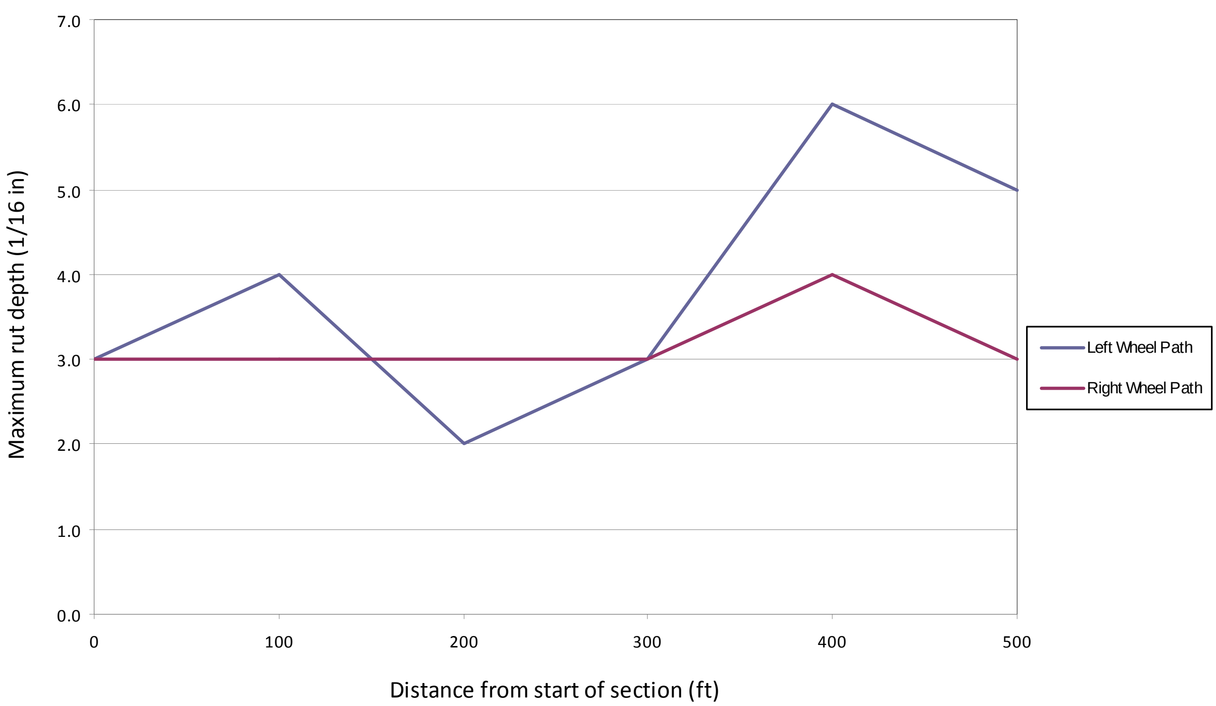


Figure 8.2: Rutting along CAM section on LP 20, Laredo (LACAM1W)

Figure 8.2 shows that this particular section does exhibit rutting. On the other hand, while the section does show some signs of cracking along the shoulder (Figure 8.3), there is no cracking in the driving lanes. A similar trend was observed in the Bryan sections, which exhibited rutting but no cracking. This scenario can result from the higher temperatures in the southern and southeastern regions of Texas. On the other hand, the sections in Paris exhibit considerably less rutting (around half of that in the previous sections) but do have a considerable amount of transverse and longitudinal cracking.

The poor performance of the CAM mixtures both in terms of rutting and cracking as observed from the sections monitored in the study is reason for concern. The CAM sections are still relatively new constructions but are showing early signs of distress, suggesting the improper application of these mixtures. In other sections of Texas CAM mixes have been used successfully.

Figure 8.3: Cracking along pavement shoulder in CAM section on Loop 20, Laredo

8.2 NCAT Test Section

In 2006, TxDOT sponsored a test section (S12) constructed at the National Center for Asphalt Technology (NCAT) test track to evaluate the effect of thin RBL on resistance to cracking. For this purpose, the section was designed as a 3 inch Type D mix (Class A granite aggregates, 5.4 percent AC content, and PG76-22S binder) over a 1-inch RBL (sandstone, 7.1 percent AC content, and PG70-22S binder).

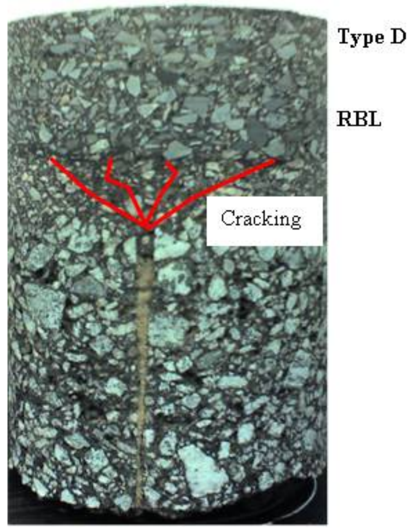
After the NCAT experiment ended (10 million ESALs according to NCAT's own estimation) the pavement section exhibited considerable rutting (1.02 in. [26 mm] of surface rutting). The rutting was attributed to poor construction that resulted in binder contents in the Type D mix in excess of 1.3 percent from the design AC value. The rutting allowed for water ponding, as Figure 8.4 shows.

Source: TTI Tech Memo

Figure 8.4: Rutting and water ponding in NCAT S12 section

The Tech Memo prepared by TTI indicated that little permanent deformation was identified in the RBL layer, although the extracted samples also indicated higher asphalt contents than those the mix had been designed to.

Roughness in the section was measured to be 150 in./mi. More importantly, no surface cracks were identified after the experiment ended. NCAT reported that no cracking propagated through the RBL (Figure 8.5). This finding is important because the underlying layers had been pre-cracked to observe the attenuating properties of the RBL layer.



Source: TTI Tech Memo

Figure 8.5: Cracking in underlying structure that did not propagate through RBL

8.3 Summary and Recommendations

The performance results of CAM in the field have been mixed and inconsistent. Some sections have performed very well, in particular when CAM was used as a RBL. In some other cases, the CAM performed below acceptable levels, in particular when used as a surface layer. For the projects evaluated in this chapter and proposed by District personnel, the use of CAM mixes as surface layers can result in considerable rutting (due to the high content of asphalt binder and fine gradations that are used) and lack of surface texture. However, the NCAT study showed that using the CAM mixes as a RBL results in a flexible mix that can withstand cracking better than conventional HMA mix, therefore reducing crack initiation at the bottom of the asphalt layers, and also minimizing crack propagation from stiff treated bases.

The main objective of the CAM is to retard and minimize cracking. Some of the sections evaluated have not achieved this objective (CAMs in Paris District already exhibited both longitudinal and transverse cracking). However, most of the sections are relatively new and distresses associated with cracking tend to occur after several years of service life. For this purpose, monitoring of the current sections, and sections where the CAM mix is used as a RBL, should continue.

Chapter 9. Conclusions

Over the years, the research program sponsored by TxDOT and managed by the Research and Technology Implementation (RTI) Office has developed many significant technologies and generated numerous valuable products. The benefits and impacts of these technologies and products go well beyond Texas' borders and extend outside the United States. However, the research projects are, in general, only two years long and many longer term lessons are not captured, except by a few individuals who continue to be involved with the research projects after completion under their own initiative. Many valuable lessons out there can still be captured and documented at a very low cost and, consequently, enhancing the efficiency of the TxDOT research program beyond initial estimated benefits.

This research study highlighted the importance of the availability of a database system that can archive information gathered from different TxDOT research and construction projects. This study was a "low-cost, high-payoff" project because most the research and expensive testing had already been done but several valuable lessons were missing. This database is a useful tool that can help communicate the experiences and lessons learned not only from different projects, but from engineers themselves. Researchers need to continue to assess the effectiveness and efficiency of the research studies beyond their initial termination point. To this effect the researchers recommended the implementation of a plan for gathering, analyzing, updating, and tracking performance of selected experimental sections (see Appendix B).

The database system itself was developed using the "wiki" approach, which consists of a website that allows the easy creation and editing of any number of interlinked web pages via a web browser using a simplified markup language. This system is useful and flexible because anyone that wishes to share their experiences or lessons learned from any given project can quickly and easily create a new wiki page within the website (or simply edit an existing one) and enter the information, images, and documents to share. More importantly, a user-friendly interface resembling that of a text editor can carry out most of these tasks.

Because the website is open to all, TxDOT personnel or the website administrator also has the option to decide what type of access should be granted to the different users. The access can range from read-only access, to editing of existing pages, to editing and creating new pages, to full administrator rights (ability to delete pages, check for changes, etc.).

Access to the database is simple; users just enter this URL into any web browser: http://pavements2.ce.utexas.edu:8080/txdot. As was highlighted throughout the document, the Main Menu of the website contains links to the different types of projects and case studies that were monitored as part of the current research project. Additionally, it has links to the different products that have resulted from the project and to several other websites of interest.

The projects and case studies that were monitored include CAM projects, WMA projects, Perpetual Pavement projects, compost use evaluation projects, and the LTPP SPS-5 sections addressing the use of RAP.

9.1 Summary of Specific Findings

This research project's evaluation and analysis of the different projects and case studies resulted in the following summarized findings. For more details on each of the different projects or case studies, please refer to each individual chapter.

1) Recycled Asphalt Pavement (RAP)

- Analysis of the data on the LTPP SPS-5 sections in Texas indicated that, in the case of rutting, the effect of milling did not significantly affect performance. On the other hand, the thickness of the overlay significantly improved rutting resistance. However, it was also found that the use of RAP has the most significant effect on reducing the rutting progression of the pavement structure.
- Additionally, based on the analysis of cracking in the LTPP SPS-5 sections, researchers identified that both milling prior to overlaying and the thickness of the overlay significantly reduce the time for cracking to initiate. However, based on the same data, it was also observed that the use of RAP has the most significant effect on accelerating the time of crack initiation in the pavement structure.
- Based on the previous sections, the researchers also identified that, in sections where the cracking process has started, milling prior to overlaying results in a lower number of transverse cracks, while adding RAP to the HMA mix increases the number of transverse cracks.
- The use of RAP is clearly an appropriate sustainable technology and should be promoted in Texas. RAP is an effective material for delaying and reducing rutting; however, it is neither effective nor efficient in preventing cracking. Therefore, RAP is not always the most economical option. Assessing the long-term implications requires more data and research.

2) Long Term Performance of Compost

- In the case of pavement sections where compost material was used to treat the soils along the shoulder, the type of compost additive used did not have a significantly different effect in the long run. In the short term, however, the type of compost does make a difference as identified by previous research.
- The width of soil that is treated (as measured by the distance from the shoulder outwards) has a significant effect on the performance of the shoulder and, possibly, of the pavement structure. Researchers also found that while plots where the treatment was only 5 ft did not differ significantly in performance from the control sections, the sections where the compost additive was applied to widths of 10 ft did show (regardless of the depth of treatment) improved resistance to cracking and lower roughness. On average, a 1-ft increase in the width of the treatment resulted in a 13.4 in./mi reduction in roughness and 9.8-ft reduction in longitudinal cracking.
- Additionally, the depth of the treatment was also found to be significant in minimizing deterioration of the shoulder. The study found that on average, a 1-ft increase in the depth of treated soil resulted in 16.8-ft decrease in longitudinal cracking.
- The use of compost to minimize the effect of moisture variation on the performance of the adjacent pavement has been an effective and successful experiment.

3) Warm-Mix Asphalt (WMA) Technologies

- In projects involving WMA, the researchers found that WMA modified with Advera does not significantly perform differently from the control HMA section when observing FWD data. In WMA modified with Sasobit and Akzo Nobel, the additives have a significant effect on the deflection of the lower layers; however, the effect on the overall deflection is not significant. As for the WMA modified with Evotherm, the additive has a significant effect on the deflection of both the upper and lower layers of the pavement structure. Also, while the Akzo Nobel additive can be associated with a reduction in the surface deflections, the Evotherm additive resulted in increased deflections.
- As was the case with deflections, the Advera additive demonstrated no significant effect on the roughness performance of the WMA section. The use of the three remaining WMA additives did have a statistically significant effect on the roughness of the pavement structure. Noted that while WMA with Evotherm and Sasobit exhibit a reduction in roughness, the WMA section with Akzo Nobel is significantly rougher than the control HMA test section.
- For the case of rutting in WMA pavements, researchers observed a different trend: only WMA with Advera exhibited a significant effect on rutting resistance of the pavement structure. However, the effect is negative in the sense that the WMA is expected to have higher rutting as compared to the control HMA mix.
- This report emphasizes that the observations described are based on only 2 years of performance, which is limited data. Besides, the performance of the sections built in the Lufkin District may be affected by the variability of the thickness of the overlay and the different conditions of the pavement before the overlay. Validation of the early findings requires further monitoring.

4) Perpetual Pavements

- Examining the perpetual pavement structures revealed that, when comparing the perpetual pavement structure to a conventional HMA structure (along the same road and with the same traffic), the deterioration rates for the perpetual pavement structure are significantly lower than those of the conventional HMA structure.
- In general, the perpetual pavement structures showed low deflections under the FWD, indicating high stiffness and structural capacity. Additionally, little distress (cracking and rutting) was observed in the evaluated sections. Furthermore, the perpetual pavements showed lower IRI values. These results were all confirmed using PMIS data, which also showed that the perpetual pavement structures showed better Distress, Condition, and Ride Scores as per TxDOT’s evaluation manual as compared to the conventional HMA structures.
- Additionally, results indicated that typically, the yearly maintenance investment on the perpetual pavement sections was considerably lower than that of the conventional counterpart. Therefore, the application of perpetual pavement technology in Texas has been another very successful story. Perpetual pavements have proven to be effective. The final chapter of this story will be written when the

long-term performance of these sections is determined through continued monitoring of these sections. At that point, detailed life-cycle cost analysis should be carried out to determine the efficiency of the technology.

5) *Field Performance of Crack Attenuating Mixtures (CAM)*

- The CAM section evaluation indicated that the use of CAM mixes as surface layers can result in considerable rutting. However, the main use of CAMs should not be for surface layers, but actually as RBLs to help reduce cracking originating from the bottom of the HMA layer. This use was confirmed with the TxDOT NCAT section that used CAM mixes as a RBL. The NCAT experiment revealed that the RBL ably reduced crack initiation at the bottom of the asphalt layers, and also minimized crack propagation from stiffed treated bases.
- It should be emphasized that CAMs were, in principle, developed as a RBL mixture; however, the necessity in Texas for a crack-resistant mixture for relatively thin overlays over severely cracked pavements has created the need for a CAM-type mix for surfaces.

6) *General Issues*

- For all of the previously evaluated projects and case studies (with the exception of the LTPP SPS-5 sections), the monitoring of the performance was conducted over a relatively short period of time. To properly and accurately characterize many of the evaluated technologies, longer monitoring of the pavement sections is required. For this purpose the research team highlights the importance of continued monitoring of most of these pavement sections (and potentially others) to further populate the database, enable use of more detailed analysis techniques, and develop more sophisticated performance models to better characterize the different technologies. The ultimate objective should be to perform a full life-cycle cost analysis of the sections to quantify the efficiency of the various technologies.
- The research team recommends that the enhancement, maintenance, and updating of this database be a continued effort. Many valuable lessons remain out there that can be captured at a low cost and consequently, any investment in gathering these lessons will have a high benefit/cost ratio.

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Appendix A. Section Selection Survey Results

Table A.1 summarizes the results of the survey that was sent to the different Districts.

Table A.1: Survey Responses

#	Location	Brief Description	Proposed by
1	FM 2	Multiple **geogrid** test sections. It contains several sections including control sections set up on the project and documented in Research Project 4829.	Darlene Goehl, P.E. Bryan District Lab
2	FM1155 FM912	This is a **rubblization** section with multiple sections and a flexible base overlay. This project included 2 new special specifications, one for rubblization and one for drainable base. The information is documented in Research Project 4687.	Darlene Goehl, P.E. Bryan District Lab
3	FM 6	**Geogrid** used in lieu of chemical stabilization. Due to concerns with sulfates and organics on this project, the working platform was designed with geogrid and a lift of flexible base.	Darlene Goehl, P.E. Bryan District Lab
4	IH 45	**CAM** mix as surface on several ramps. This project was the first to use the CAM mix SS.	Darlene Goehl, P.E. Bryan District Lab
5	US 59 Cass County	Concrete Pavement Contraction Design (CPCD) w/ siliceous gravel, 4” asphalt treated drainable base w/ edge drain & storm drain, and lime treated subgrade	Miles Garrison Atlanta District
6	US 59 Panola	**Item 344 CMHB-F mixes**, no field sand, 76-22 and 70-22 binders, 5.5 to 6.0 AC content, 1200 cycles on overlay tester, 4 to 7 mm rut depth Hamburg, no fiber, lower permeability when designed correctly, excellent ride, no anti-strip additive required.	Miles Garrison Atlanta District
7	Statewide Compost Sections	Research and Implementation Projects 0-4573 and 5-4573 looked at adding **compost** to the soil adjacent to the pavement structure. The purpose was to mitigate the shrink-swell characteristics of the adjoining soil to maintain the moisture under the pavement structure and greatly minimize longitudinal cracks. There are 5 sections still in service that need to have data collected for continued evaluation and determination of the service life of the compost before reapplication. The sites are in the Fort Worth, Lubbock, Corpus Christi, Tyler, and Yoakum Districts.	Richard S. Williammee, Jr., P.E. Fort Worth District

#	Location	Brief Description	Proposed by
8	SH 360 2 miles south of IH 20 Tarrant County	The Fort Worth District experimented with *a quarry fines select fill and a RAP select fill* on a four-lane divided highway constructed 2 years ago under CRCP pavement. The intent was to replace high swelling clay with more inert, somewhat waste products in a recycling effort. Horizontal inclinometers and pressure cells were placed to measure small movements that might indicate future unacceptable swelling. Only 1 year's worth of data has been secured. Additional data is required in an attempt to determine the usefulness of these select fill alternatives.	Richard S. Williammee, Jr., P.E. Fort Worth District
9	SH 251 0134-01-021	This was one of the first **Warm Mix Asphalt (WMA)** projects constructed in Texas.	Jim Black Wichita Falls District
10	US 183 0125-03-036	This was one of the first **WMA** projects constructed in Texas using RAP.	Jim Black Wichita Falls District
11	SH 194	**Warm Mix Asphalt (WMA)** Project	Mike Craig Lubbock District
12	Marsha Sharp Freeway	**Open Graded Friction Course** (OGFC) Project	Bryan Wilson Lubbock District
13	Various PFCs Project 0-5185	Several **Porous Friction Course** Projects throughout Texas	Amy Epps Martin Texas A&M / TTI
14	US 180	The section in downtown Albany presented a unique problem in that there is concrete pavement buried below the asphalt pavement surface. The section constructed consists of 2" **CAM mix** directly on top of the old concrete pavement covered by 2.5" SMAR mix. This section is currently in its second winter. No reflective cracking has occurred to date. The section prior to construction contained severe reflective cracking from the concrete pavement. The purpose of the newly constructed section is to delay reflective cracking resulting in an acceptable life cycle and is potentially a cost effective method in dealing with buried concrete pavement.	Paul Hoelscher Abilene District
15	FM 1761 1120-01-014	1.5 mile project through town. Consists of planing existing ACP (2–3") and placing 2" of a **Warm-Mix Asphalt** on a one-course surface treatment. This project could be useful because it is the district's first warm mix asphalt project. Lessons learned could be acquired from both the contractor and TXDOT.	Ciro Baeza Odessa District

#	Location	Brief Description	Proposed by
16*	IH 20 0005-13-043 IH 10 0441-09-036	These two projects are weigh-in-motion projects. They collect traffic data specific to our area for use in pavement designs. IH 20 WIM Site installed under/during project, CSJ 0005-13-043 and IH 10 WIM Site installed after project, CSJ 0441-09-036. These projects could be useful because one was installed in a *concrete* section (IH 20) and the other was installed in a *hot-mix* section (IH 10) to show pros and cons or any differences in implementation and/or durability.	Ciro Baeza Odessa District
17	IH 10 0140-01-074	9-mile project. ¾" **Ultra Thin Bonded Hot Mix Wear Course** placed over thin ACP (less than 2") consisting mostly of surface treatments. This project could be useful to show the effectiveness an UTBHMWC over a thin ACP on interstate.	Ciro Baeza Odessa District
18	BU 59-G	Crack Attenuating Mix (**CAM**)	Richard Boles-Gracia Lufkin District
19	FM 324	Warm Mix Asphalt (**WMA**) Project	Richard Boles-Gracia Lufkin District
20	FM 770	FM 770 would be a good experimental roadway for the captioned survey. We could compare two locations with different pavement designs (one without a **CAM** and one with a **CAM**). Number 1: FM 770 from Daisetta to FM 160 involved full depth repair of concrete pavement, cleaning and sealing joints of widened 9-6-9 concrete pavement. Placing one inch of Type D hot mix and finally placing two inches of Type C hot mix. Number 2: FM 770 from SH 105 to Batiste Creek (March 2009 Letting) will involve full depth repair of concrete pavement, cleaning and sealing joints of widened 9-6-9 concrete pavement. Placing one inch CAM and finally placing two inches of Type C hot mix.	John E. Sudela, PE Beaumont District
21	SH 87 (16th St) 305-07-058 BU90 (Simmons) 0028-14-099	The project consisted of resurfacing old jointed concrete pavement that had been previously covered with ACP. The old ACP was planed off; joints in the concrete pavement were cleaned and sealed. Broken slab corners were repaired before a riding surface of 1 1/2" of **Thin Bonded Porous Friction Course** (TBPFC) was placed. The TBPFC resulted in a reduction of traffic noise and a reduction of water spray from vehicles.	Clark Slacum Beaumont District

#	Location	Brief Description	Proposed by
22	US 69, Cherokee Co. “7-mile Hill” 0199-01-071	The section of this project that may be useful was constructed with 1.25” of **PFC** (PG76-22 TR) on 4” HDSMA. The roadway geometry consists of a moderate vertical grade in conjunction with horizontal curves, which was contributing to **higher than average crash rates**. The PFC was chosen to combat these issues as well as serve as a test bed for this type of material in the Tyler District. The pavement has held up well to date and no unforeseen issues have arisen.	Bill Willeford, P.E. Cal Hays, P.E. Tyler District
23	US 259, Rusk Co. 0138-03-137	This project consisted of rehabilitation of the southbound lanes of a 4-lane divided highway with heavy truck traffic. The southbound roadbed is the original US 259 and has the old 9”-7”-9” concrete pavement still in place, which was causing severe reflective cracking to any overlays previously placed. Our solution was to use 1.5” of TY D HMA level-up (PG70-22), followed by **fiberglass reinforcing grid** (TY 1), and 2” of TY C HMA surface (PG70-22). If more specific information is needed about the reinforcing grid, we can provide it later.	Bill Willeford, P.E. Cal Hays, P.E. Tyler District
24	FM 734 (Parmer Lane)	**Sulphate heave and swelling** addressed through the use of **geogrids, RAP, and microcracking**.	Mike Arellano Austin District
25	Spur 16 in Maverick County	The use of **ground granulated blast furnace slag** for stabilizing subgrade soil with potential for sulphate induced heave.	Caroline Herrera Austin
26	SH 16	**Full-depth Reclamation** (FDR) using Asphalt Emulsion	Caroline Herrera Austin
27	7 Projects Statewide	**Jointed Concrete Pavements Rehabilitation** Strategies in Texas	Dar-Hao Chen Austin
28	5 Projects Statewide	**Full-Depth Reclamation** (FDR) using Asphalt Emulsion (TTI/Semmaterials Fortress Study)	Caroline Herrera Austin
29	NCAT Test Sections	The NCAT Texas Section (S12), **Perpetual Pavement Sections** (N8, N9). Other states sections (Florida, Georgia, Missouri, North Carolina, South Carolina).	Fujie Zhou Lubinda Walubita Tom Scullion

#	Location	Brief Description	Proposed by
30	US 180 Stephens County	This test section is 3,200’ long, built in 1997 as part of a 6 mile long project, CSJ 0011-09-059. The project consisted of **scarifying and compacting** top 8” of existing pavement, adding 8” of new flexible base, and a two course surface treatment (2CST). For the 3,200’ section, we did not scarify and compact the existing pavement. We just placed 8” of new flexible base and a 2CST. It is faster and cheaper if the existing pavement is not scarified and compacted prior to placing base and surface. Also, a forensic investigation on FM 2818 in the Bryan District showed the portion of the roadway that performed the best was the section where the existing pavement was not scarified and compacted prior to placing the new base and surface.	Elias Rmeili Brownwood District
31	FM 1176 Brown County	This experiment consisted of five 2,500’ long test sections to study the effect of the **different type of stabilizations** on pavement performance. They were built in 1997, part of a 11.5 mile long project, CSJ 1365-05-010, etc. The project consisted of **scarifying** and stabilizing top 8” with lime, adding 4” flexible base, and 2CST. The 5 test sections consisted of: Section 1. Scarify and reshape top 8”, add 4” flexible base treated with liquid stabilizer EN1, and 2CST. Section 2. Scarify and treat top 8” with cement and EN1, add 4” flexible base, and 2CST. Section 3. Scarify top 8”, cement treat outer 8 feet, compact, add 4” flexible base, and 2CST. Section 4. Scarify and treat top 8” with cement, add 4” flexible base, and 2CST. Section 5. Scarify and reshape top 8”, add 7” flexible base, and 2CST. So far, the section without any stabilization performed the best.	Elias Rmeili Brownwood District
32	IH 35 McLennan County From Myers Lane to BU 77	The experimental **perpetual pavement design** used on this project was the first built in the Waco District. Information being gathered in Research Project 0-4822, along with data gathered during the construction of this project, makes this project one of the most studied in the district. **Application of the lessons learned here is changing the way in which perpetual pavements are being designed in Texas**. The continued study of this roadway should prove to verify this design and future designs of this type.	Billy S. Pigg Waco District

#	Location	Brief Description	Proposed by
33	IH 35 Hill County From FM 310 to FM 286	This is the first full scale **Post Tensioned Concrete Pavement** to be built in the district (actually being built at this time). Most likely the first (**cast-in-place**) in the state, from what we know. This project (as Implementation Research Project 5-4035) is being built as a continuation of Research Project 0-4035. The site is to be instrumented to farther evaluate the performance of this type pavement. To track the performance of this section would prove to provide justification of this type pavement as a suitable substitution for our CRCP pavements.	Billy S. Pigg Waco District
34	SH 6 NB Falls County From Riesel to Marlin	This project, although not possessing any experimental designs, holds the particular advantage of having the ability to **monitor the rehabilitation strategies** resulting from forensic studies performed on failures (both ride quality and structural) during a post construction (but prior to release of the contractor) period of time. The difficulties occurred only in the northbound lane covering most, if not all, of the limits of the project. Based upon the results of the CSTM&P forensics study, it would provide invaluable information to monitor the strategies employed to access their effectiveness. Multiple strategies were employed at various locations throughout this project to achieve an acceptable project.	Billy S. Pigg Waco District
35	IH 35 Bell County From US 190 (in Belton) to South LP 363 (in Temple)	This overlay project consisted of a seal coat , a 1" **CAM** layer, overlaid with a 1-1/2" **PFC** surface course. The first CAM mix used in the Waco District. This project should be beneficial in evaluating the performance of CAM mixtures and Also PFC mixtures.	Billy S. Pigg Waco District
36	US 287 Armstrong County From Carson County Line East 8.6 miles	Existing section is approximately 6 inches of ACP over 20 inches of sand and gravel base. Proposed section will remove from 0 to 2 inches of ACP to establish a 2 percent cross slope on the main lanes, a one-course surface treatment of A-R Binder with Grade 4 Aggregate, a 1 inch **RBL** using PG70-28 asphalt, a 3 inch SMA with PG76-28. This project is scheduled for an April 2009 letting. CSJ 0042-03-038	Kenneth R. Petr, P.E Amarillo District

#	Location	Brief Description	Proposed by
37	US 287 Armstrong County From East City limits of Claude East 5.7 miles	Section is constructed of 6 inches of **Lime Stabilized Subgrade**, 10.5 inches of Sand and Gravel Base, a One Course Surface Treatment using AC-5 or CRS-1P Asphalt, a 6 inch layer of TY-B ACP using PG70-28 Asphalt, a 3 inch surface layer of TY-D ACP using PG70-28. This project was completed in April 2003. CSJ 0042-04-034	Kenneth R. Petr, P.E Amarillo District
38	US 287 Armstrong County From Donley County Line West 6.0 miles	Section is constructed of 10 inches of **Fly Ash Treated Emulsion Stabilized Base**, a One Course Surface Treatment using AC-5 Asphalt, a 7 inch layer of TY-B ACP using PG64-22 Asphalt, a 2.5 inch surface layer of TY-C ACP using PG76-28. This project was completed in July 2008. CSJ 0042-05-029	Kenneth R. Petr, P.E Amarillo District
39	Various Statewide	**Hot In-place Recycling** (HIR)	Maghsoud Tahmoressi

Appendix B. Recommendations for Data Collection

During this research study, the research team has identified the need for continuing assessment of the effectiveness and efficiency of the research studies sponsored by TxDOT's Research and Implementation Office (RTI) beyond their initial termination point. To this effect, the researchers recommend the implementation of a plan for gathering, analyzing, updating and tracking performance of selected experimental sections. It is also recognized that this activity may consume a significant amount of resources, therefore, the researchers have developed the following recommendations for minimum data collection keeping in mind current economic constraints but at the same time ensuring that data collection will capture the benefits of the research projects. In addition, the recommendations are presented at three levels so the Research Engineer may select the appropriate level for the resources available and the potential benefits of the specific project:

1) **Level 1:** Minimum Data Requirements (Essential). This level of effort represents the minimum data elements that are necessary in order to perform analyses to qualify and quantify the benefits of a particular research study. This level is proposed in order to optimize the benefits with a minimum cost.
2) **Level 2:** Important Data Requirements. This level of effort contains data elements that are not collected on a routine basis, but depending on project specific conditions, may be essential part of the economic and performance analyses. In general, the cost will be higher than that of Level 1 because this level will require traffic control and the operation or installation of specific pieces of equipment and technology.
3) **Level 3**: Specific Data Requirements (Desirable). Level 3 is recommended when the necessary resources are available and the research study has the potential for a high pay-off (or realize significant savings) in the short time.

In the next paragraphs, we present the types of data elements that should be collected at each of the above-mentioned levels as well as preliminary recommendations in terms of temporal (how often) and spatial (how dense) frequencies for data collection. It should be highlighted that these are general recommendations and the data elements and frequency, in all cases, should be determined by the research team in conjunction with the Implementation Director (ID) and the District personnel affected by the project. It is also important to note that these three levels have been conceived as incremental; i.e., Level 2 includes Level 1 data collection and Level 3 includes Level 2.

1) **Level 1: Minimum and Essential Data Requirements**
    a. Location: the importance of accurate project location should not be underestimated, in particular for short experimental sections. Location information is inexpensive so redundant information is desirable. This information should include: facility type and facility number, distance from origin (DFO), Texas Reference Marker (TRM) and displacement, GPS coordinates (including altitude) and descriptive information that may facilitate the location of the experiment in the future.

b. Photographic Records. Pictures of the section are useful to locate the sections and to represent and objectively record the condition of the section at a given time. Digital video should also be considered.

c. Ground Penetrating Radar (GPR). In some cases where structural information is not available, GPR is a good alternative to obtain estimates of the thickness of the surface layer and, sometimes, of the base layer. For long experimental sections, this information is also useful to determine homogenous sections.

d. Environmental Information. At Level 1, this information should include maximum and minimum monthly temperatures, annual precipitation (monthly if available) and subgrade type (AASHTO or Unified Soil Classification). Most of this information is readily available online.

e. Existing Research Data: All research reports and products should be gathered as well as any additional unpublished information that could aid in the analysis of the performance data. This information could consist of project memoranda, test results and any spreadsheet or database developed during the project. Very often this information is lost at the end of the project.

f. Traffic Estimates. Although traffic estimates are available in the Pavement Management Information System (PMIS) and from TxDOT's Transportation Planning and Programing Division (TP&P), it is often the case that Districts or Area Offices have additional updated traffic data which can significant improve available estimates.

g. Visual Distress Survey. Visual distress surveys should be conducted as per current TxDOT's procedures (from a moving vehicle) but the Research Engineer should ensure that the specific lane where the project is located is the one surveyed. At Level 1, estimates of rutting and cracking should be obtained from the visual assessment. This is until the time TxDOT implements automatic rutting and cracking data collection.

h. Riding Quality (i.e., Roughness). Longitudinal profiling and roughness determination is to date the most consistent and reliable piece of performance information and can be safely collected at highway speed.

i. Temporal Frequency: at least once a year.

j. Spatial Frequency: at the very minimum 3 to 10 measurements, depending on the length of the section and the characteristic being measured.

2) Level 2: Important Data Requirements

a. Visual Distress Survey. At Level 2, the visual distress survey should be conducted by walking through the experimental section. This should include the assessment of the drainage system. The procedure recommended by the Long-Term Pavement Performance (LTPP) program is recommended.

b. Rutting. Surface rutting should be collected manually using TxDOT's mechanical profiler or a 6-ft straight edge.

c. Cracking. Surface cracking should be collected manually as recommended by the LTPP program. In addition, photographic records of the sections should be collected using TxDOT's 3-D camera. If not available, a digital camera can be used.

d. Falling Weight Deflectometer (FWD). It is recommended that two annual FWD surveys be conducted to determine the effect of the environmental conditions on the structural capacity of the section. Depending on the specific characteristics of the sections, these surveys should be conducted during the dry and wet seasons or during the warm and cold seasons. It is recommended that at least three different load levels are utilized and that the full time-history of the deflections is collected and stored.

e. Rolling Weight Deflectometer (RDD). While the FWD is the most popular deflection measuring equipment, the RDD may offer significant advantages for project and research level investigations because it can capture a quasi-continuous deflection profile of the entire section. Thus, allowing the identification of weak spots. The decision whether to use the FWD or the RDD should be left to the Research Engineer and the Implementation Director (ID).

f. Traffic Counts. Accurate traffic data are paramount for pavement performance estimation. Traffic characterization is the most challenging and variable input into the pavement design and analysis system. Traffic varies annually, seasonally, and hourly and all types of variability are important. At the minimum traffic counts should be conducted four times a year for at least three days each time (preferably one entire week).

g. Temporal Frequency: twice a year

h. Spatial Frequency: a minimum of 10 to 30 measurements, depending on the length of the section and the characteristic being measured.

3) Level 3: Desirable and Specific data Requirements

a. Coring and Trenching. Coring and trenching of the experimental sections are expensive activities, however, in some instances are the only means for obtaining the information necessary for establishing meaningful comparisons.

b. Laboratory Testing. Depending on the specific project objectives, field cores should be collected and subjected to testing. Specific testing should include the determination of the volumetric properties and performance testing (e.g., Hamburg Wheel Tracking Device, Asphalt Pavement Analyzer, Flow Time and Flow Number, Indirect Tensile Strength, Repeated Direct Tension, Semi-Circular Bending Beam, Overlay Tester, Third-Point bending Beam, etc.).

c. Environmental Information. When specific environmental information is vital for achieving the project objectives, the research team should consider the installation of a weather station at the site of the project.

d. Detailed Traffic Data: for Level 3, in addition to site specific traffic counts, automatic vehicle classification is recommended. For projects where site-specific axle load distribution is essential, the installation of weigh-in-motion (WIM) equipment should be considered.

e. Temporal Frequency: two to twelve times a year, depending on the project and data element.

f. Spatial Frequency: a minimum of 30 measurements, depending on the length of the section and the characteristic being measured.