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16. Abstract

The work contained in this interim report provides a case study describing the design, construction, initial structural evaluation, and performance predictions of the full-depth perpetual pavement constructed on SH 114 in the Fort Worth District. Based on the research findings, recommended improvements to design and construction practices are proposed. The research methodology and scope of work included data collection, laboratory and field testing, computational simulations, and performance predictions. Laboratory testing for characterizing the asphalt mixture properties included the Hamburg, the Overlay Tester, Dynamic Modulus, and Repeated Load Permanent Deformation tests. Asphalt-binder testing was accomplished with the Troxler Ignition Oven and the Dynamic Shear Rheometer, respectively. Field testing involved visual surveys, coring, infra-red quality control tests, forensic investigations, and non-destructive performance evaluations using Ground Penetrating Radar (GPR) and Falling Weight Deflectometer (FWD) measurements. Computational analyses included the FPS, PerRoad, VESYS, and MEPDG software.

The SH 114 design utilized the perpetual pavement concepts, with a thick 1-inch stone-filled layer as the main structural component. This layer was found to be considerably stiffer than the traditional TxDOT mixes. However, this material was also found to be highly permeable and subject to vertical segregation. During construction, the Fort Worth District found it necessary to retrofit edge drains to minimize problems with water trapped in the asphalt layers. For comparison purposes, the Fort Worth District also included a similar section constructed with traditional dense graded mixes. Laboratory and field results from both sections are included in this report.

Recommendations are also provided for the structural design of future perpetual pavements in Texas. From the results presented in this project, the current designs are very conservative. The results generated support the transition to higher design moduli for these full-depth pavements.

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PERPETUAL PAVEMENTS IN TEXAS: THE FORT WORTH SH 114 PROJECT IN WISE COUNTY

by

Lubinda F. Walubita
Assistant Transportation Researcher
Texas Transportation Institute

and

Tom Scullion
Senior Research Engineer
Texas Transportation Institute

Report 0-4822-2
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TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

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TABLE OF CONTENTS

TABLE OF CONTENTS (CONTINUED)

TABLE OF CONTENTS (CONTINUED)

LIST OF FIGURES

Figure **Page**

LIST OF FIGURES (CONTINUED)

LIST OF FIGURES (CONTINUED)

LIST OF TABLES

Table **Page**

LIST OF NOTATIONS AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
ADT	Average daily traffic
DM	Dynamic modulus
ESAL	Equivalent single axle load
FDAP	Full-depth asphalt pavement
FPS	Flexible pavement system
FWD	Falling weight deflectometer
GPR	Ground penetrating radar
HDSMA	Heavy-duty stone mastic asphalt
HMA	Hot-mix asphalt
HMAC	Hot-mix asphalt concrete
HWTT	Hamburg wheel tracking test
IRI	International roughness index
MEPDG	Mechanistic empirical design guide
MTD	Material transfer device
OT	Overlay tester
PFC	Porous friction course
PG	Performance grade
PI	Plasticity Index
PP	Perpetual pavement
PSI	Pavement serviceability index
RBL	Rich-bottom layer
RLPD	Repeated load permanent deformation test
RRL	Rut-resistant layer
SF	Stone-fill or Stone filled
SMA	Stone mastic asphalt
SS	Special specification
ε_t	Horizontal tensile strain measured in microns ($\mu\varepsilon$)
ε_v	Vertical compressive strain measured in microns ($\mu\varepsilon$)

CHAPTER 1

INTRODUCTION

In March 2001, the Texas Department of Transportation (TxDOT) issued a memorandum recommending the use of full-depth asphalt pavements (FDAPs) on heavy truck trafficked highways where the 20-year estimate of 18 – kip equivalent single axle loads (ESALs) is in excess of 30 million (TxDOT, 2001). This recommendation is necessary to cope with the ever increasing traffic and to minimize the cyclic and costly structural rehabilitation and/or reconstruction processes. To date, there are eight Texas full-depth asphalt pavement projects constructed since 2001. The proposed Texas FDAP structural section was based on the perpetual pavement (PP) concept developed by the Asphalt Institute (Newcomb et al., 2001). Figure 1-1 shows the proposed FDAP structure for Texas Perpetual Pavements (TxDOT, 2001).

	Layer Designation, Materials, and Functions				Thickness (inches)
Layer 1	PFC (SS3231)	Porous Friction Course		Sacrificial layer	1.0 – 1.5
Layer 2	HDSMA (SS3248)	Heavy-Duty SMA	1/2″ Aggregate + PG 76-XX	Impermeable load carrying layer	2.0 – 3.0
Layer 3	SFHMAC (SS3249)	Stone-Filled HMAC	3/4″ Aggregate + PG 76-XX	Transitional layer	2.0 – 3.0
Layer 4	SFHMAC (SS3248)	Stone-Filled HMAC	1.0-1.5″ Aggregate + PG 76-XX	Stiff load carrying layer	8.0 - Variable
Layer 5	Superpave (SS3248)	Superpave (RBL)	1/2″ Aggregate + PG 64-XX (Target lab density=98%)	Stress relieving impermeable layer	2.0 – 4.0
Layer 6	Stiff base or stabilized subgrade		Construction working table or compaction platform for succeeding layers		6.0-8.0
Subgrade					∞

Figure 1-1. Typical Texas FDAP Structural Section.

In Figure 1-1, SMA stands for stone matrix (or mastic) asphalt, HMAC for hot-mix asphalt concrete, RBL for rich bottom layer, and PG for performance grade. SF, HD, SS, and PFC stand for stone-filled, heavy-duty, special specification, and porous friction course, respectively. In this report, the acronym FDAP is used synonymously with the acronym PP.

In Figure 1-1, Layer 4 represents the stiff rut-resistant layer with a minimum thickness of 8″ to ensure adequate structural capacity in terms of load spreading capability. Layer 5 represents the flexible and typically high binder content fatigue-resistant layer. Because of its characteristically high binder content, Layer 5 is generally referred to as the rich-bottom layer. Layer 3 is a transitional load-carrying layer. Layers 1 (PFC) and 2 (SMA) are intended to improve the resistance to weathering, thermal cracking, rutting resistance, water proofing/drainage, safety, and durability characteristics of the pavement. In particular, SMAs provide very good stone-on-stone contact with generally high stiffness values (i.e., modulus greater than 500 ksi at 77 °F). The PFC is optional in the current FDAP design, but these layers generally reduce traffic noise and improve drainage characteristics. Layer 6 and the subgrade provide the working platform and pavement foundation, respectively. The structure for the perpetual pavements proposed by the Asphalt Institute (APA, 2002) is shown in Figure 1-2.

		Thickness (inches)
Layer 1	High quality asphalt layer (durability & wear resistant)	1.5 – 3.0
Layer 2	Stiff rut-resistant layer	4.0 – 7.0
Layer 3	Flexible fatigue-resistant layer	3.0 – 4.0
Layer 4	Strong pavement foundation	(variable to infinite)

Figure 1-2. Typical Perpetual Pavement Structural Section (APA, 2002).

The Texas FDAP structure (Figure 1-1) is substantially more conservative. The FDAP structure shown in Figure 1-1 encompasses more layers and has a greater total pavement thickness than the typical PP structure shown in Figure 1-2. In Figure 1-2, the impermeable load carrying and transitional layers (Layers 2 and 3 in Figure 1-1) are nonexistent. Also, the Texas-recommended minimum thickness for the rut-resistant layer (Layer 4) is twice (8″ versus 4″) that of the typical PP structure shown in Figure 1-1. Theoretically, the Texas FDAP would be expected to have better structural capacity; however, ultimate field performance is a function of many variables including materials, mix-designs, and construction practices.

The current structural (thickness) design and analysis of Texas FDAP is mechanistic-empirically based using the Flexible Pavement System (FPS) 19W software (Scullion and Liu, 2001). However, the thickness design is also often checked with the PerRoad software (Timm, 2004; Timm and Newcomb, 2006).

One critical issue for the FPS 19 design is that this mechanistic-empirical procedure requires the designer to input layer moduli for each layer in the structure. These moduli are obtained from nondestructive testing of existing structures using the Falling Weight Deflectometer (FWD). However, for the Texas FDAP, many of the layers such as Layer 4, the 1″ Stone Filled Superpave (1″ SFHMAC) layer, have never been placed in Texas before. Therefore, many of the initial designs were completed assuming values based on results for traditional Texas mixes. One major goal of this project is to determine design properties for the materials used in the existing full-depth pavements, nondestructively in the field and on recovered samples in the laboratory, to provide guidelines for future thickness designs.

OBJECTIVES AND SCOPE OF WORK

Because of limited experience with perpetual pavements, TxDOT through the Texas Transportation Institute (TTI) initiated a research project (Project 0-4822) in 2004 to monitor the performance of existing FDAP sections and those to be constructed (TxDOT, 2001; Scullion, 2006). The project objectives include material testing both in the field and laboratory, and identifying lessons learned from these initial projects to improve future Texas FDAP designs and construction practices. The primary focus is the thick rut-resistant 1″ SFHMAC layers (Layer 4 in Figure 1-1). Overall, the following are the project research goals (Scullion, 2006):

- Validate the FDAP design concept by relating field and laboratory results to pavement performance monitored after construction.
- Create a database of design moduli for the current FPS design system and the National Cooperative Highway Research Program (NCHRP) 1-37A mechanistic empirical pavement design guide (MEPDG) and the Asphalt Alliance design methodology (PerRoad).
- Use the data collected to verify and enhance TxDOT's design, materials, and construction specifications.

Within this framework, the specific objective of this interim report provides a case study for the design, construction, initial structural evaluation, and performance predictions of the FDAP project on State Highway (SH) 114 in the Fort Worth District in Wise County (Texas, USA). The work will focus on the 13″ thick rut-resistant layers in the asphalt pavement. The second objective is to develop recommendations to improve the current Texas FDAP design and construction practices. The research methodology and scope of work includes:

- monitoring and recording the construction process;
- data collection (design, construction, and performance) for the Texas FDAP database;
- visual surveys and pavement surface performance profiling;
- coring and forensic evaluation of field-extracted cores;
- Binder Dynamic Shear Rheometer testing (for characterizing the binder rheological properties) and binder extraction tests with the Troxler Ignition Oven;
- laboratory testing of asphalt mixtures/cores with the Hamburg wheel tracking (HWTT) device (for rutting resistance characterization), the Overlay Tester (OT) (for cracking resistance characterization), the Dynamic Modulus test (for characterizing the mixture visco-elastic properties), and the Repeated Load Permanent Deformation test (for characterizing the permanent deformation characteristics of the mixtures);
- non-destructive field testing and performance evaluations with ground penetration radar (GPR) and falling weight deflectometer measurements;
- computational simulations and numerical performance predictions with the FPS, PerRoad, VESYS5, and MEPDG software;
- comparative analyses of laboratory, computational modeling, and field data; and
- documentation of research findings with conclusions and recommendations.

The work plan basically involved comparative evaluation and performance predictions of the SH 114 structures, assessing the adequacy and quality of the construction methods, and relating design (in particular mix-design) to constructability aspects such as workability and compactability. The first report in this Project 4822-1 (Scullion, 2006) noted that constructability is one of the critical issues for Texas perpetual pavements, in particular for the 1″ SFHMAC mixtures. This will be further explored in evaluating the SH 114 sections.

The work plan also includes investigating the applicability of the numerical software MEPDG Version 0.910 and VESYS5 for modeling, analyzing, and performance prediction of Texas perpetual pavements. This interim report provides an insight into assembling and processing the input data required for the FPS, PerRoad, VESYS5, and MEPDG software. These input data include traffic, material properties, environment and climatic conditions, the pavement structure, etc. In contrast to TxDOT's FPS system, the material properties for the PerRoad, VESYS5, and MEPDG must be generated by laboratory testing. Note that in this project, both the laboratory and field tests were utilized to generate input data for the software analyses. For instance, all these software require asphalt mixture modulus values as input data. Also, the MEPDG (Level 1) require binder rheological properties as input data. Finally, the correlation among laboratory testing, computational simulations, and field data were investigated. This aspect of the research enabled establishing how these approaches complement each other and how they could effectively and simultaneously be engaged in performance evaluation studies of this nature.

DESCRIPTION OF CONTENTS

This interim report is comprised of seven chapters, including this chapter (Chapter 1) that provides the introduction, research objectives, research methodology, the work plan, and scope of work. Chapter 2 provides a description of the SH 114 FDAP project including the location, pavement structures, design (structural and mix), and construction details. Chapter 3 presents the laboratory testing and associated results and includes the Dynamic Shear Rheometer, the Hamburg wheel tracking device, the Overlay tester, the Dynamic Modulus test, and the Repeated Load Permanent Deformation test. Field testing and performance evaluations including visual monitoring surveys, radar measurements, falling weight deflectometer tests, and forensic investigations are discussed in Chapter 4. Computational simulations and numerical analyses with the FPS, PerRoad, VESYS5, and MEPDG software are presented in Chapter 5, which include processing of the input data, structural analyses, and performance predictions. Chapter 6 is a comparative analysis, discussion, and synthesis of the results and research findings. The report concludes in Chapter 7 with a summary of findings and recommendations.

SUMMARY

In this introductory chapter, the background and the research objectives were discussed. The research methodology, scope of work, and work plan were described followed by a description of the report contents.

Note also that in this interim report, the symbol ″ is used to represent "inches," interchangeably with "mm," as a dimensional unit, i.e., 1″ = 1 inch ≅ 25 mm. Also the acronym FDAP is used to represent "full-depth asphalt pavement" synonymously with the acronym PP, which stands for "perpetual pavement," respectively. Additionally, as some of the laboratory tests such as the Hamburg and Dynamic Shear Rheometer use standard metric (SI) units, some of the test results have consequently been reported in metric units, e.g., use of "mm" for the Hamburg test results.

Through this report, all the Superpave and Stone Fill HMA mixtures including the SMA are specified by their nominal maximum aggregate size (NMAS), e.g., 1″ SFHMAC, ¾″ SFHMAC, and ½″ HDSMA; where 1″, ¾″, and ½″ stands for 1″, ¾″ , and ½″ NMAS, respectively. The NMAS is defined as one sieve size larger than the first sieve to retain more than 10 percent of the aggregate material.

CHAPTER 2
THE FORT WORTH SH 114 PROJECT, WISE COUNTY

This chapter discusses the SH 114 FDAP project. This discussion includes location details, the pavement structures, the design (both structural and mix-design) aspects, and construction details. A summary is then provided to conclude the chapter.

LOCATION AND GEOGRAPHICAL DETAILS

The SH 114 FDAP project is located in the Fort Worth District, Texas (USA), on SH 114 in Wise County, 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 eastbound lanes and 10 ft shoulders ending at the Denton County line. SH 114 is a heavily trafficked highway with an average daily traffic (ADT) of approximately 18,000. As of 2003, truck composition was about 27.3 percent, with a designated maximum speed limit of 70 mph (Wimsatt, 2003).

THE SH 114 PAVEMENT STRUCTURES

The SH 114 FDAP project was designed according to TxDOT guidelines in the "Flexible Pavement Design Task Force Recommendations" dated April 23, 2001 (TxDOT, 2001). All of the structural layers were designed using the Superpave criteria, with the main rut-resistant layer being a 1″ stone-filled layer, designed to have a gradation that went below the "restricted zone." This design resulted in a coarse mix, which was subsequently found difficult to construct (Scullion, 2006). 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. As can be seen in Figure 2-1, the final SH 114 FDAP project consisted of two structural sections:

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

FW 01: Superpave

Layer	Material	Binder + Aggregate	Thickness (inches)
Layer 1	½" HDSMA	6.8% PG 70-28 + Igneous/Granite	2
Layer 2	¾" SFHMAC	4.2% PG 76-22 + Limestone	3
Layer 3	1" SFHMAC	4.0% PG 70-22 + Limestone	13
Layer 4	¾" SFHMAC (RBL)	4.2% PG 64-22 + Limestone	4
Layer 5	Stabilized Subgrade	6% Lime Treated	8
Subgrade			∞

FW 02: Conventional

Layer	Material	Binder + Aggregate	Thickness (inches)
Layer 1	½" HDSMA	6.8% PG 70-28 + Igneous/Granite	2
Layer 2	TxDOT Type C	4.4% PG 70-22 + Limestone	3
Layer 3	TxDOT Type B	4.5% PG 64-22 + Limestone	13
Layer 4	TxDOT Type C (RBL)	5.3% PG 64-22 + Limestone	4
Layer 5	Stabilized Subgrade	6% Lime Treated	8
Subgrade			∞

Figure 2-1. SH 114 FDAP Structural Sections.

Based on district preference, the Fort Worth District decided not to use a PFC surface. In Figure 2-1, there is also a variation in the binder types for both sections. FW 01 used PG 76-22 and PG 70-22 for Layers 2 and 3, while FW 02 used PG 70-22 and PG 64-22, respectively. However, Layers 1 and 4 are similar for both sections, albeit that the contractor switched to using PG 70-28 instead of the PG 76-22 original design for the surfacing SMA layer.

Neither of the two sections used the recommended stiff PG 76-22 binder for the rut-resistant Layer 3; FW 01 used PG 70-22, while FW 02 used PG 64-22. Variation in the binder contents is also evident; both sections used 6.8 percent PG 70-28, by weight of total mix, for the top ½″ HDSMA (Heavy Duty SMA) layer (Layer 1). Also, both sections used a 6 percent lime (by total dry weight) treated subgrade as the base. All the layer thicknesses are consistent with Figure 1-1, and all used limestone aggregates (Figure 2-1) except for Layer 1 (igneous/granite).

STRUCTURAL DESIGN

The SH 114 FDAP structural thickness design was mechanistic-empirically accomplished with the FPS 19W software (Version 1.136978) for an initial design period of 30 years at 95 percent reliability level (Wimsatt, 2003). The estimated design traffic ESALs was 63,307,000, with 27.3 percent truck composition. For a 20-year design period, the estimated design traffic ESALs was 37,242,000, also with 27.3 percent trucks (Wimsatt, 2003).

For the material properties, the initial design modulus values that were used for structural analysis are listed in Table 2-1.

Table 2-1. Initial Design Modulus Values and Poisson's Ratio (Wimsatt, 2003).

Layer/Material	Elastic Modulus (ksi)	Poisson's Ratio
Asphalt surfacing layers	500	0.35
Rut-resistant layer (1″ SFHMAC)	750	0.35
Bottom flexible fatigue-resistant layer	17	0.35
Lime-treated subgrade	17	0.45
Subgrade	9.1	0.45

Based on previous experience, a modulus value of 500 ksi was used for all the upper surface HMA layers, and a slightly higher value of 750 ksi was assigned to the rut-resistant Superpave layer (Wimsatt, 2003). However, no substantial stiffness was given to the RBL layer; a conservative assumption was that this layer would be assigned the same modulus as the lime-treated subgrade layer (17 ksi). The final thickness design and material characteristics are shown in Figure 2-1, with the top 8″ of the subgrade treated with 6 percent lime to ensure a strong and stable foundation. In the design process, the thickness of Layers 1, 2, and 4 were fixed, as recommended in Figure 1-1, as 2, 3, and 4″, respectively. The FPS 19W program was then used to design the required thickness of the rut-resistant layer. Based on the design assumptions, a 13″ thick layer was required. An initial structural design check with the Asphalt Institute Mechanistic Empirical fatigue and rutting models by Wimsatt (2003) had actually indicated that 20.5″ of asphalt layer thickness was structurally sufficient to perform satisfactory for at least 50 million ESALs.

MIX DESIGN

The current Texas FDAP mixture designs are based on the Superpave volumetric design system (Scullion, 2006). With regard to aggregate gradation, it was recommended that all the load-bearing layers should be designed with an aggregate gradation that passes below the restricted zone. The intention was to promote "stone-on-stone" contact and improve rut resistance. As was subsequently found in this research, this decision did result in mixes that were stiff and rut resistant, but it also had major bearings on mix constructability and permeability.

The mixture design criterion was based on 100 gyrations to achieve 4 percent air voids (AV) (i.e., 96 percent density), except for the RBL, which is designed at 97 percent density. Additionally, the mixtures are also required to pass the Hamburg wheel tracking test at a maximum rut depth of 0.5″ (Scullion, 2006). The design binder contents (by weight of total mix) are shown in Figure 2-1 and indicate the highest and lowest binder contents for the ½″ HDSMA and 1″ SFHMAC layers, respectively. In general, the 1″ SFHMAC layers are designed with relatively lower binder content to contribute to their rut-resistant characteristics. Typically, the RBL should be designed with a softer binder at slightly higher binder content to contribute to its fatigue-resistant characteristics. SMA mixtures are generally designed with higher binder contents, as evident in Figure 2-1, to contribute to their fatigue resistance and durability characteristics. In TxDOT's 2004 specifications book, the minimum binder content for the SMA is 6 percent; the recommended content for the SH 114 project was 6.8 percent (TxDOT, 2004).

Notice also that the Conventional asphalt layers (FW 02) generally have higher binder contents and used lower binder PG grades than the Superpave asphalt layers. For instance, FW 02 RBL has 5.3 percent of PG 64-22, while FW 01 has 4.2 percent. FW 01 Layer 3 (1″ SFHMAC) used 4 percent of PG 70-22, while FW 02 Layer 3 (TxDOT Type B) used 4.5 percent of PG 64-22. These differences in mix-design characteristics are likely to impact these sections' performance differently, as will be discussed in Chapters 3 through 7. Other mix-design characteristics including the binder contents (design and extracted), void in the mineral aggregate (VMA), and specific gravity (Rice) are discussed in the subsequent text.

Field Binder Contents

Table 2-2 summarizes the design and extracted binder contents. The extracted binder contents represent the binder extracted from the field cores using the Troxler Ignition Oven.

Table 2-2. Binder Contents.

Layer	Superpave Section		Conventional Section	
	Design	Extracted	Design	Extracted
Layer 1 (½″ HDSMA)	6.8%	6.67%	6.8%	6.67%
Layer 2	4.2%	5.23% (*¾″ SFHMAC*)	4.4%	4.51% (TxDOT Type C)
Layer 3 (Rut-resistant)	4.0%	3.35% (*1″ SFHMAC*)	4.5%	4.48% (TxDOT Type B)
Layer 4 (RBL: fatigue-resistant)	4.2%	4.22%	5.3%	5.21%
All binder content is by weight of total mix				

Comparing the extracted versus design, the binder contents on the Conventional section are reasonably more consistent than on the Superpave section; in fact, the deviation is less than ±5 percent for all the layers. The field binder contents for the Superpave mixes are a cause for concern. The 1″ FHMAC is approximately 16 percent below the target design 3.35 percent versus design 4 percent, while the ¾″ SFHMAC (Layer 2) is about 24 percent above the target design binder content. Note that the extraction binder results for Layers 2 (¾″ SFHMAC) and 3 (1″ SFHMAC) represent an average of over three replicate measurements of different cores, six in fact for the 1″ SFHMAC layer. The researchers do not have any explanation as to why there was such a wide variation in binder contents on the Superpave section. This variation could perhaps be a sampling error as all of the cores used in this project were taken from one location in the middle of the section.

Rice and Voids in the Mineral Aggregates

Table 2-3 is a list of the Rice and VMA data. Clearly, there is a considerable difference in the Rice and VMA characteristics for the asphalt mixtures used on these two sections. The Superpave exhibits relatively higher Rice and lower VMA values, respectively, and vice versa for the Conventional section.

Table 2-3. Rice and VMA.

Layer	Rice (G_t)		VMA	
	Superpave	Conventional	Superpave	Conventional
Layer 1	2.449	2.449	19.6	19.6
Layer 2	2.488	2.481	14.0	14.3
Layer 3	2.499	2.475	13.3	14.3
Layer 4	2.494	2.455	14.0	14.9

Aggregate Gradations

Figure 2-2 shows the aggregate gradations for the rut-resistant layers (1″ SFHMAC and TxDOT Type B) and shows both the design and the extracted aggregate gradations. The extracted aggregate gradations represent the aggregates that were extracted from the field cores after conducting the Ignition Oven Test. Complete gradation curves (both design and extracted) for all the other asphalt layers are included in Appendix A.

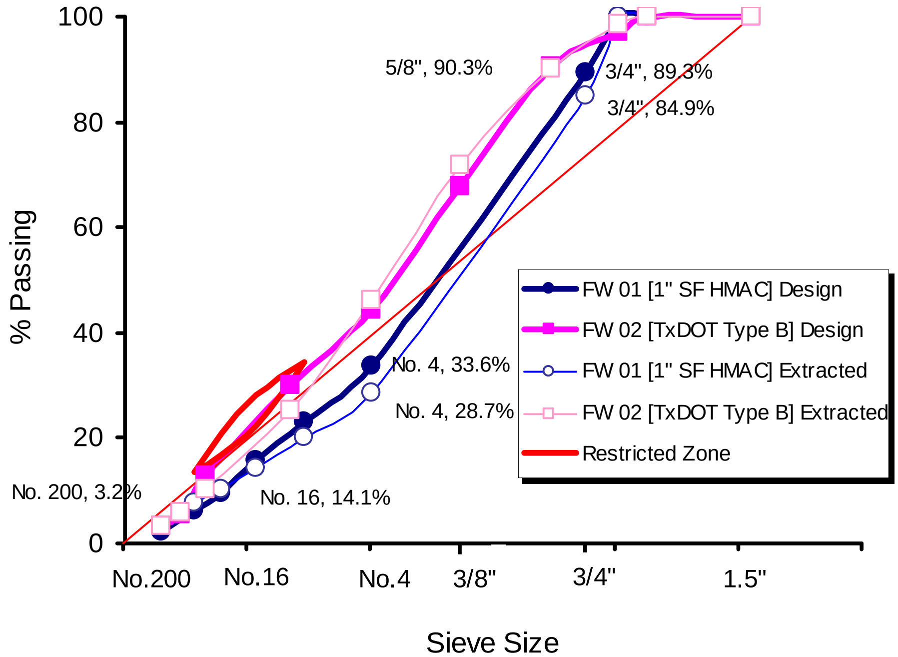


Figure 2-2. Aggregate Gradations.

As shown in Figure 2-2 and in comparison with Appendix A, the rut-resistant layers (Layer 3), in particular the 1″ stone-filled (1″ SFHMAC), are typically designed with a coarse aggregate gradation and are required to pass below the Superpave restricted zone (Scullion, 2006). With a good aggregate interlock and stone-on-stone contact, this coarse aggregate gradation contributes to this layer's rut-resistant characteristics.

Note also from Figure 2-2 that the 1″ SFHMAC is coarser than the TxDOT Type B mixture. For the extracted gradations, the 1″ SFHMAC used more of the coarser rock (in fact about 15.1 percent cumulative retained on the ¾″ sieve instead of the design 10.7 percent), whereas the TxDOT Type B used more of the medium-fine rock (i.e., 74.59 percent cumulative retained on the No. 10 sieve versus the design 69.80 percent). Based on these coarse gradations, traditional wisdom would suggest that the 1″ SFHMAC would be expected to be more rut-resistant than the TxDOT Type B, which in practice is not always the case, as aggregate interlock also plays an equally significant role. A tabulation of the design aggregate gradations together with the specification is also shown in Table 2-4 to supplement Figure 2.2.

Table 2-4. Aggregate Gradations for the Rut-Resistant Layers.

Sieve Size		% Passing (Design)		Specification	
	(mm)	1-inch SFHMAC	TXDOT Type B	1-inch SFHMAC	TxDOT Type B
1½″	37.5	100	100	100	
1″	25	100	100	90-100	98-100
⅞″	22.0	100	96.9		95-100
¾″	19.0	89.3	-		
⅝″	16.0	-	90.3		75-95
½″	12.5	-	-		
⅜″	9.375	-	67.7		60-80
No. 4	4.75	33.6	44.5		40-60
No. 8	2.36	23.2	-	19-45	
No. 10	2.0	-	30.2		27-40
No. 16	1.18	15.6	-		
No. 30	0.6	9.7	-		
No. 40	0.425	-	12.9		10.0-25
No. 50	0.3	6.2	-		
No. 80	0.18	-	5.6		3.0-13
No. 100	0.15	-	-		
No. 200	0.075	2.3	3.2	1.0-7	1.0-6
Pan		0.0	0.0		

Aggregate Blend Characteristics

Table 2-5 is a summary of the aggregate blend characteristics. Table 2-5 shows that the FW 01 Layer 3 (1″ SFHMAC) had more proportions of the coarser rocks in the blend than the FW 02 Layer 3 (TxDOT Type B). For instance, the 1″ SFHMAC (FW 01 Layer 3) has about 30 percent of the 1″ Class A rock against 22 percent for the TxDOT Type B (FW 02 Layer 3). Apparently, the Conventional section used more fines (sand) than the Superpave (Table 2-1), which is essential for workability in terms of compaction (among other functions). Notice also the high proportion of the ⅝″ rock (60 percent) compared to the other blend proportions in Layer 1 and presence of mineral filler (11 percent) and cellulose fiber (0.3 percent), respectively. This is a typical characteristic of SMA mixtures. Also, the ½″ HDSMA layer used igneous/granite aggregate while all the other asphalt layers used limestone aggregates.

Table 2-5. Aggregate Blend Proportions.

Degree of Coarseness	Rock Type	Blend Proportions					
		Layer 2		Layer 3		Layer 4	
		FW 01 (¾″ SF)	**FW 02** (Type C)	**FW 01** (1″ SF)	**FW 02** (Type B)	**FW 01** (¾″ SF)	**FW 02** (Type C)
Coarser (biggest)↑	1″ Class A	33%	-	30%	22%	32%	-
	1″ Belt Run	-	-	30%	-	-	-
	⅝″ -10	-	26%	-	-	-	-
	C-Rock	20%	10%	-	18%	20%	28%
	D-Rock	22%	24%	15%	28%	20%	32%
	Blend sand	10%	-	10%	-	15%	-
Finer (smallest)↓	Manufactured Sand	15%	40%	15%	32%	13%	40%
Total		100%	100%	100%	100%	100%	100%

Layer 1 (HDSMA): 60% (⅝″ - 10), 22.7% (½″ - 10), 6% (Manufactured sand), 11% (Mineral filler), & 0.3% (Fiber) = 100%; **FW 01** = Superpave section, **FW 02** = Conventional section

CONSTRUCTION DETAILS

This section provides an overview of the construction aspects of the SH 114 pavement structures and includes the subgrade treatment and placement of the asphalt layers. Non-destructive infrared temperature and in-place air void measurements are also discussed. A cost comparison of the pavement structures is also provided. The contractor for this project was Duininck Brothers, Inc.

Subgrade Treatment

Figure 2-3 shows the treatment process of the subgrade material in 2003/2004/2005. To ensure a durable and stable foundation support and because of the potential for high plasticity clay soils in the area, the top subgrade material was stabilized and treated with 6 percent lime, by dry weight. As shown in Figure 2-3, the lime treatment was applied as slurry in a liquid form, up to a total treatment depth of 8″. This 6 percent lime-stabilized top subgrade material formed the 8″ base of the pavement structures.

Figure 2-3. Subgrade Treatment with Lime Slurry.

The Fort Worth District has for many years routinely used 6 percent lime and an 8″ treatment depth in all clay soils where the plasticity index (PI) is less than 39. The soils from this project were tested at TTI to evaluate this level of treatment. The results of this evaluation are reported elsewhere (Scullion and Hilbrich, 2004). However, an excerpt summary of the major findings is listed as follows:

- The project soils are classified as clayey sand with a PI of 33. However, in many areas the bedrock comes very close to the surface so the clay soil is often interspersed with rock fragments.
- The sulfate content of the soil is low at 694 ppm.
- The organic content of the soil was found via the Ignition Oven to be very high at 5.4 percent. This content is well above the limit found in the literature for the use of lime, typically less than 1 percent. However, as will be discussed later, no problems were found with the lime-stabilized layer on this project.
- Based on TxMethod 121 E Part 1 (50 psi after 10 days capillary rise), the optimal lime content was found to be 3.5 percent.
- Based on TxMethod 121 E part 2 (PI and gradation), the optimal lime content was found to be 3.5 percent.

- Based on TxMethod 121 E Part 3 (Eades and Grim), the optimal lime content was found to be 3 percent.
- Based on 80 percent retained strength (wet/dry), the optimal lime content was found to be 3.5 percent.

In summary, the lime contents recommended from the laboratory testing indicated that 3.5 percent lime would be adequate for this highway. It is anticipated that the 6 percent used routinely by the district would provide a very good foundation layer for this pavement.

Placement of the Asphalt Layers

Apart from the top ½″ HDSMA layer (Layer 1), which was constructed in July 2006 on both sections, the underlying asphalt layers of each section were constructed on different dates in successive layers and lifts. The Superpave section (Layers 2, 3, and 4) was constructed in 2003/2004, while the Conventional section was completed in June 2006. So, the Superpave is basically over 1 year older than the Conventional section. However, while the Conventional section was placed within the same time frame, i.e., summer of 2006, there was generally an extended period of time between successive layers and lifts for the Superpave section.

With respect to construction and placement, the critical layer was the thick (13″) rut-resistant layer (Layer 3:- 1″ SFHMAC and TxDOT Type B, respectively), which had to be compacted in several lifts. For the Superpave section, the compaction lift thickness was 4″ + 4.5″ + 4.5″ and for the Conventional section 5″ + 5″ + 3″. The currently recommended maximum lift thickness for these layers is 5″. To attain the target density of 96 percent, the compaction rolling sequence was as follows:

- 1″ SFHMAC (Superpave section): two vibratory passes for the breakdown roller, three pneumatic passes for the intermediate roller, and one vibratory pass plus one static pass for the finishing roller.
- TxDOT Type B (Conventional section): five vibratory and two static passes for the breakdown roller, eight pneumatic passes for the intermediate roller, and one vibratory pass plus one static pass for the finishing roller.

Figure 2-4 shows an example of the compaction process and the 5″ lift thickness of the TxDOT Type B mixture on the Conventional section. It is clear from Figure 2-4 that with this type of a compaction lift thickness and coarse aggregate gradation, substantial compactive effort is necessary to attain the 96 percent target density. In general, there was an improvement in the compaction rolling pattern on the Conventional section.

5″ compaction lift thickness

Figure 2-4. Compaction Process of the TxDOT Type B Mix (Conventional Section).

All other layers followed typical compaction and rolling practices. However, the target compaction density for the RBL was 97 percent. This relatively high density contributes to this layer's fatigue resistance and impermeability characteristics, i.e., protecting the base/subgrade against moisture damage from top-down water infiltration as well as preventing upward water filtration from the base/subgrade. All other asphalt layers were compacted to 96 percent density.

Material Transfer Device (MTD)

The Roadtec material transfer device (MTD) was used to place all lifts of the Superpave section. As will be discussed in the next section, testing with TTI's infra-red (IR) segregation detection system had detected substantial temperature differentials in the initial placement of the RBL mix, prompting the contractor to switch from a Windrow pick-up system to the Roadtec MTD system. Additionally, since the Superpave layers were placed throughout the year, in both hot and cold seasons (the specifications permitted hot-mix asphalt [HMA] placement when the air temperature was 40 °F degrees and rising), it was decided to use the Roadtec on all layers.

However, as the section with conventional dense-graded mixes was constructed in the summer of 2006, it was decided to use only the Windrow pick-up for this section. Figure 2-5 shows an example of the Windrow pick-up MTD setup used on the Conventional section during placement of the TxDOT Type C mix (RBL). However, the Roadtec was used to place the final surfacing SMA on both sections in July 2006.

Figure 2-5. Windrow Pick-Up Setup for the RBL on the Conventional Section (May 2006).

Essentially, the Windrow pick-up MTD setup was such that the belly-dump trucks dump the hot-mix asphalt on the pavement surface and the Windrow picks it up with the help of conveyor belts directly into the paver. Under this setup, the Windrow and paver form one continuous operating system with the Windrow pick-up in front and the paver directly behind it. The driving mechanism and controls for the whole system are all on the paver. Compactors follow immediately behind the Windrow-Paver setup. The big difference between the Windrow and Roadtec MTD systems is the amount of remixing performed in the Roadtec.

Compaction Mat Temperature

To evaluate the placement of the first HMA layers, TTI used its infra-red monitoring system to measure mat temperatures. The latest version of this system is shown in Figure 2-6. This system is described in detail elsewhere (Sebesta and Scullion, 2002); it essentially consists of 10 infra-red sensors installed in a bar, which is attached to the foot plate of a paver. Custom built software displays the mat temperatures in real time.

Figure 2-6. TTI's Infra-Red System and Mat Temperature Measurements.

The first application of the IR system was to document the placement of the RBL layers on the Superpave section. This IR application was performed in December of 2003 while the air temperatures were in the range of 40 to 45 °F. The placement operation at that time is shown in Figure 2-7, and the measured surface temperature is shown in Figure 2-8.

Figure 2-7. Roadtec Lay-Down Operation for RBL on Superpave Section (Dec 2003).

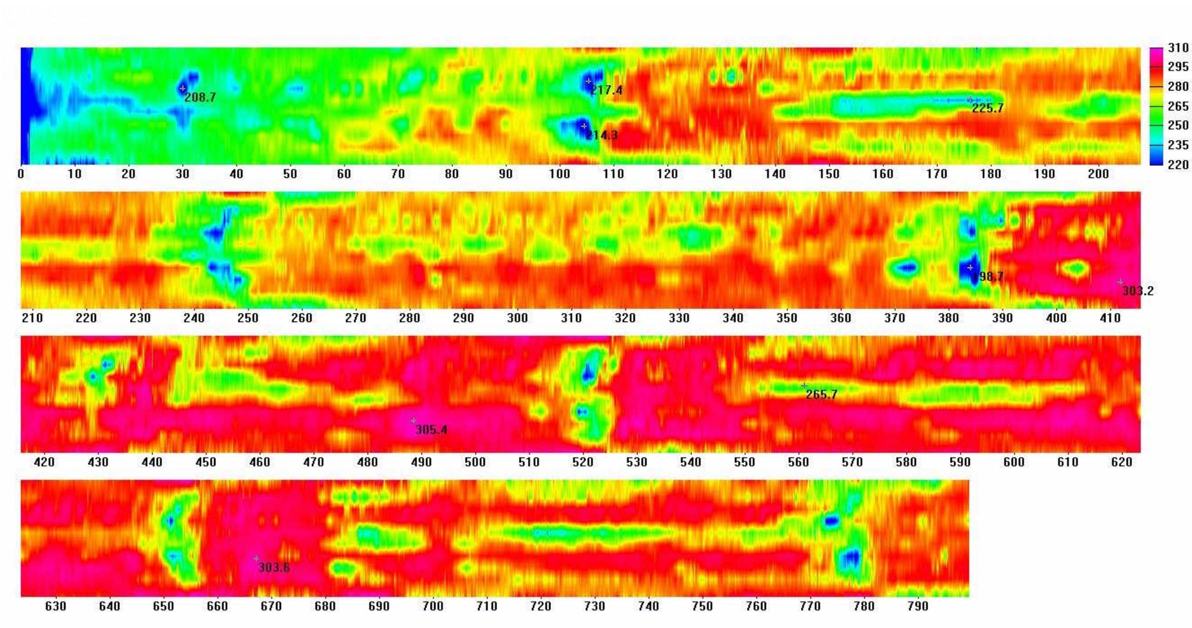


Figure 2-8. Temperature Profile for RBL Layer on Superpave Section (periodic thermal segregation- blue spots).

Figure 2-8 shows the temperature profile for the full lane width for 800 ft of new mat. The distance scale is under each plot. The key for the different color is shown in the upper right of the figure. Red colors represent temperatures around 300 °F, whereas blues are temperatures of around 220 °F. The green colors represent temperatures between 235 and 270 °F. The numbers on the plot are the actual temperatures at that location.

It is clear that there are intermittent cold spots in the mat at approximately 140 ft intervals. These cold spots coincided with the end of every truck load of hot-mix. TxDOT and the contractor reviewed these data and concluded that this placement process was unsatisfactory. For example at 410 ft, the mat temperature measured was 303 °F, whereas at 392 ft, it was less than 200 °F. It was decided to replace the Windrow elevator with the Roadtec device, shown in Figure 2-9. TTI again re-measured the temperature profile of the surface, and the resulting surface temperature profile is shown in Figure 2-10.

Figure 2-9. Revised Lay-Down Operation for All Subsequent Layers on Superpave Section.

The temperature profile in Figure 2-10 was a great improvement over that shown in Figure 2-8. It was decided to use the Roadtec for all HMA layers on the Superpave section. However, for the conventional mixes, the contractor again used the Windrow elevator system shown in Figure 2-6. Its use was primarily because the mixes were to be placed in the hot part of the year (summer), and the traditional dense-graded mixes are easier to place and compact than the coarser Superpave mixes.

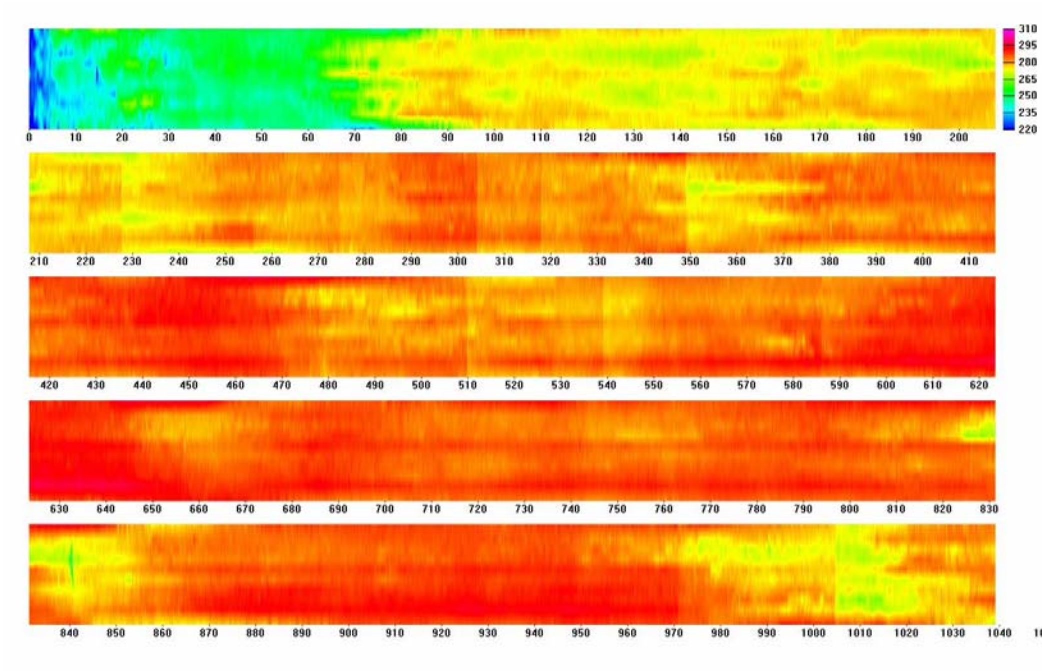


Figure 2-10. Temperature Profile with Roadtec MTD.

On May 23, 2006, TTI personnel tested the RBL layer placed on the Conventional section. This testing included infrared imaging and ground-penetrating radar surveys. The total section length placed and tested was approximately 1270 ft. The paving operation consisted of belly-dump trucks, a Windrow elevator, and a CAT AP 1000B paver. The hot-mix was placed directly on top of a prepared lime-treated subgrade. The temperature profiles from this test are shown in Figure 2-11. The observed pattern is similar to that shown earlier in Figure 2-8. In general, the Windrow pick-up system does not produce uniform surface temperature profiles. The main defects are:

- periodic cold spots in the mat (cold spots associated with the end of every truck load); and
- cooler strips down the middle of the mat where the original material was dumped on the pavement (See Figure 2-9 from 550 to 600 ft and Figure 2-12 from 360 to 390 ft).

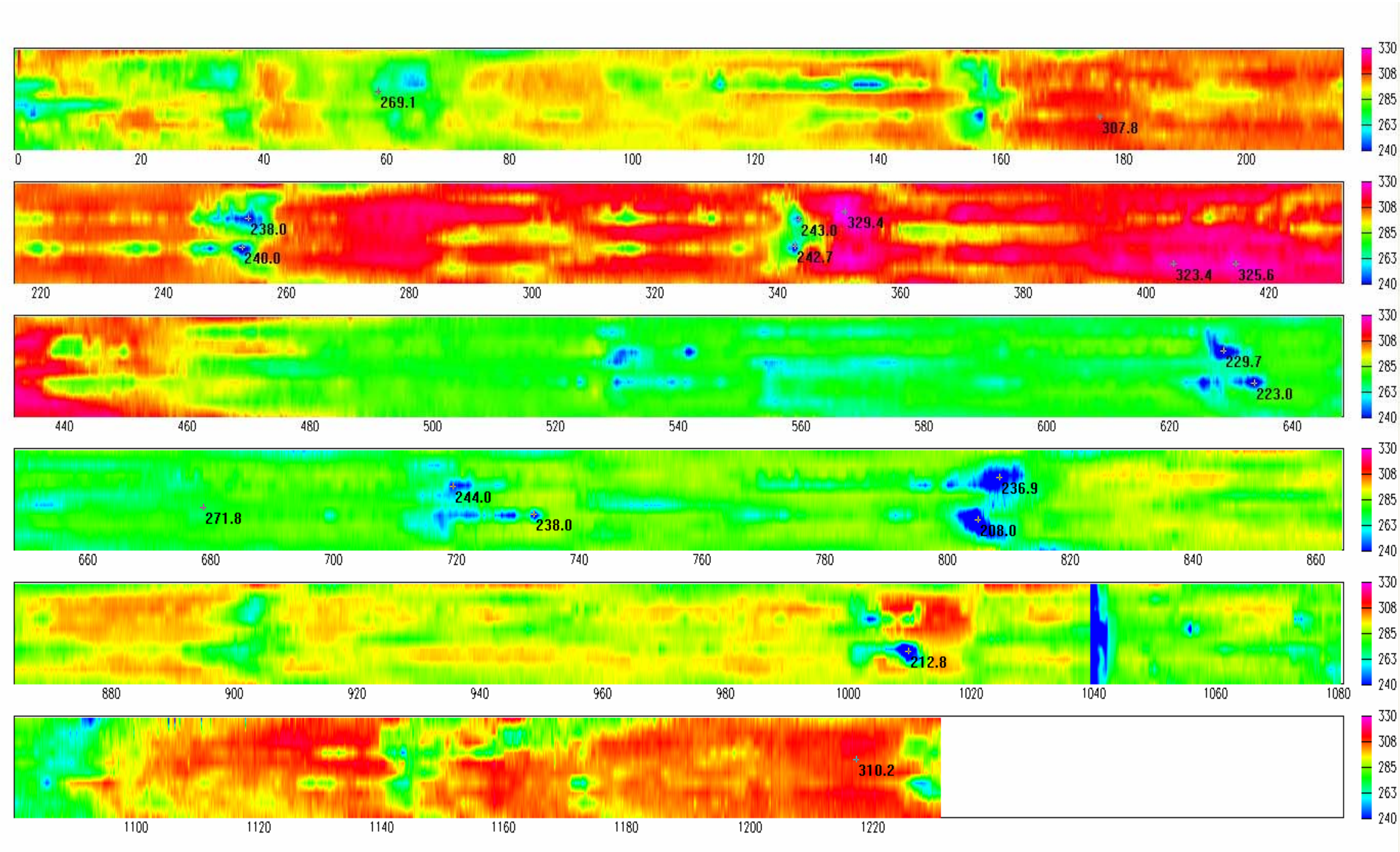


Figure 2-11. Mat Temperature Profiles (FW 02, TxDOT Type C [RBL] Mix).

GPR Determination of In-Place Air Voids

The IR data are collected before the compaction of the mat. To determine if the temperature differentials resulted in changes in the final air void distribution, a survey with TTI's Ground Penetration Radar was conducted. The techniques of using GPR to monitor in-place air void distribution are described elsewhere (Sebesta and Scullion, 2002).

Following the thermal data collection, an extensive GPR survey was conducted by performing five passes, each at different transverse offsets, over the center paving pass. Data were collected every foot to provide extensive coverage of the section. Based on these data, cores collected served to calibrate the HMA dielectric value (measured by GPR) to the air void content of the mix. Table 2-6 shows the calibration data, and Figure 2-12 shows the relationship determined.

Table 2-6. Core Data for RBL to Calibrate GPR with Density.

SH 114 RBL Core Densities		
Core	Density (pcf)	% Voids
1	139.2	9.2
2	137.6	10.4
3	143.1	6.7
4	140.9	8.1

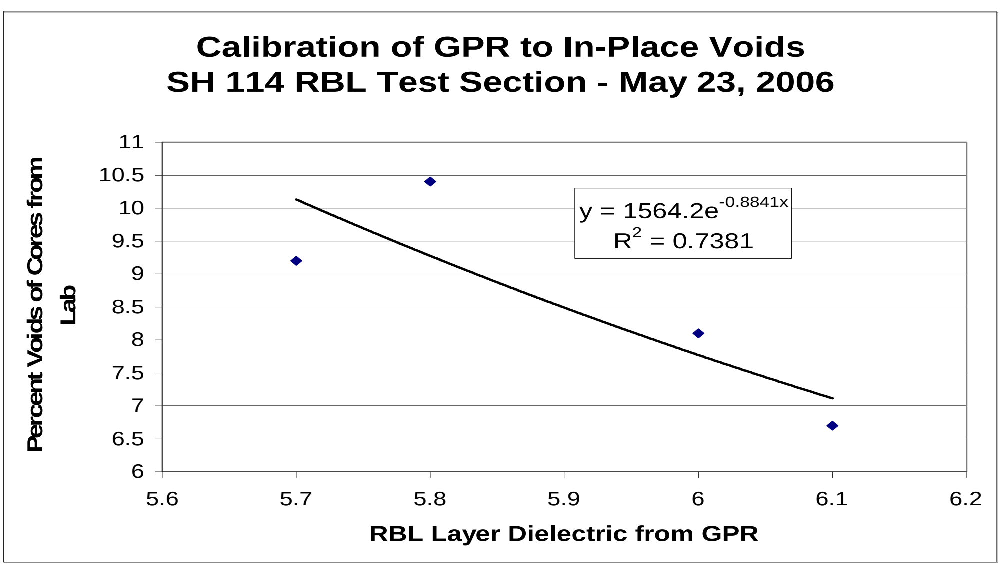


Figure 2-12. Calibration of GPR to In-Place Density on RBL.

Using the calibration relationship shown in Figure 2-12, a prediction of air void content at each of the GPR data points was made. This prediction equates to more than 6000 data points, which can be used to estimate the distribution of in-place voids over the project test site. Figure 2-13 shows this distribution. TxDOT desired 97 percent compaction; the data show the operation did not achieve that level of compaction. Ninety percent of the in-place void contents are expected to fall between 6 and 10 percent voids.

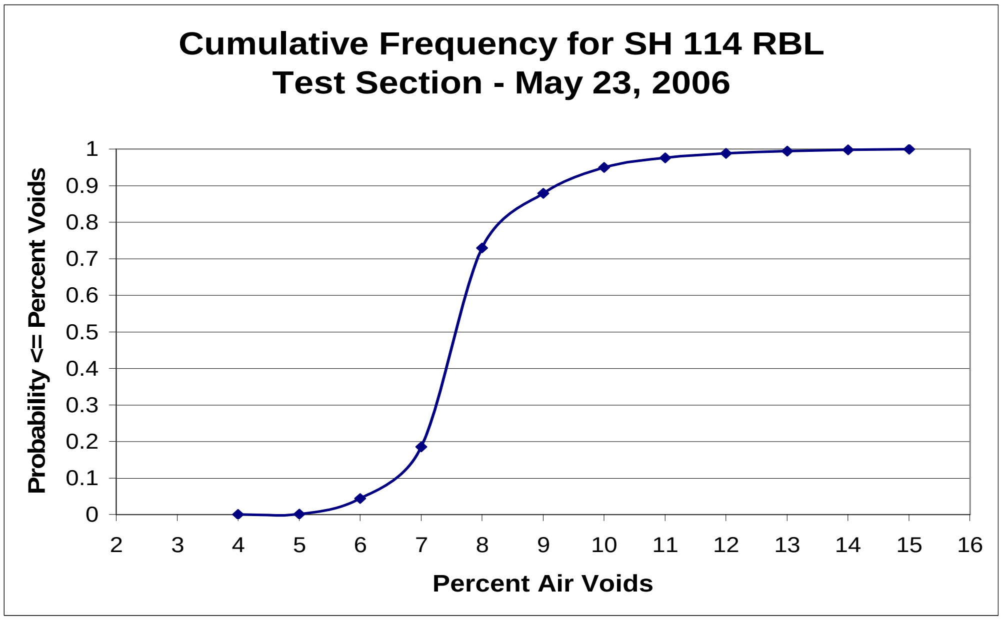


Figure 2.13. Expected Distribution of In-Place Voids on RBL Test Section.

Since the GPR data were collected at known transverse and longitudinal distances on the test section, the data also can generate a surface plot of the project showing where the locations of high void contents exist. Figure 2-14 shows this plot for the RBL section tested on the Conventional section. In Figure 2-14, the green, yellow, and red colors represent AV content less than 6 percent, AV content between 6 and 8 percent, and AV content greater than 8.5 percent, respectively. The lowest density locations primarily occurred at the unconfined edge and centerline area. The low air voids in the middle of the mat correlate with the observation that this lay-down operation tends to leave cold strips down the center of the mat. More research is needed in this area.

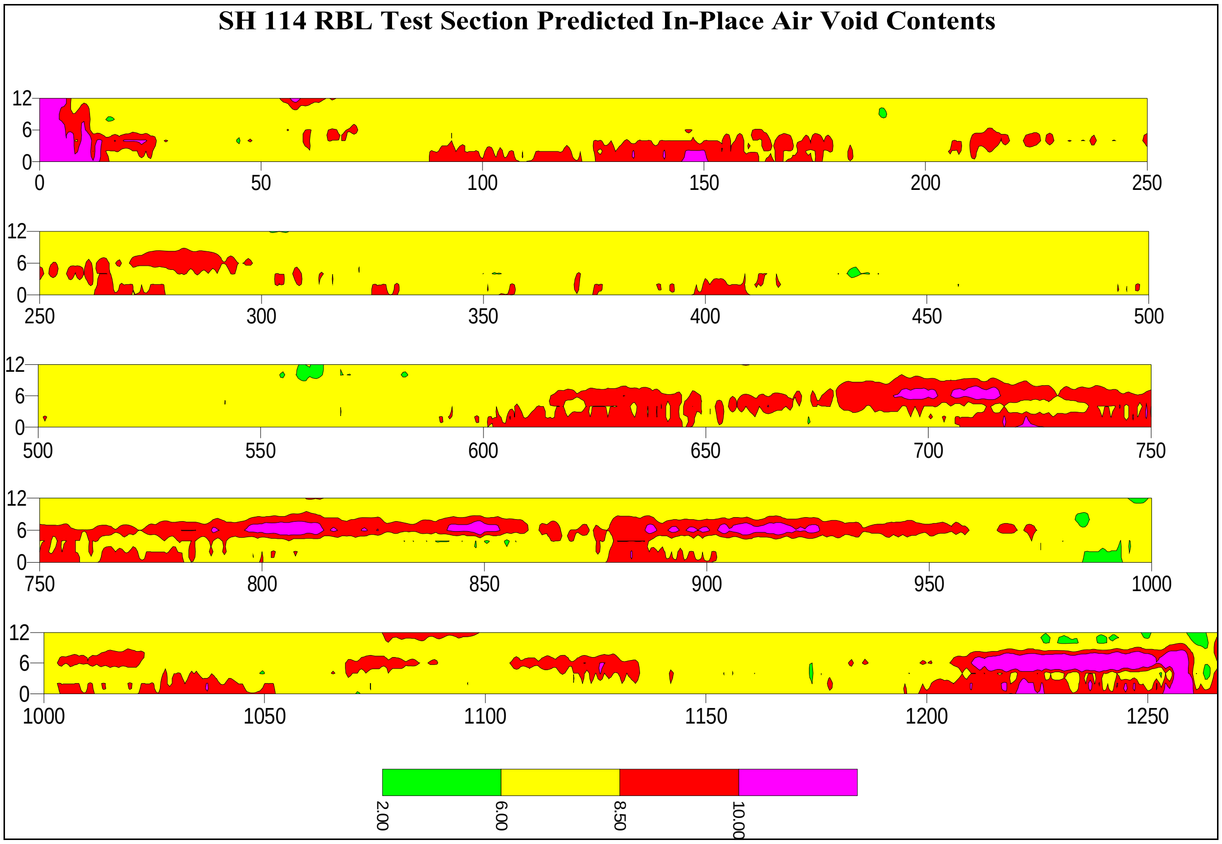


Figure 2-14. Surface Plot of Expected In-Place Voids on the RBL (Conventional Section).

Water and Permeability Problems on the Superpave Section

The underlying Superpave HMA mixes were found to be very porous after construction (Scullion, 2006). It was commented by the contractor (and other contractors working with this mix) that the material was difficult to compact and extremely permeable. The coarse nature of the mix is shown in Figure 2-15. In fact, it was found that the RBL and the clay shoulders had formed a "bathtub" with water retained in the 1″ SFHMAC. During construction of the Superpave section, the contractor noted that water was entering the 1″ SFHMAC layer. The contractor subsequently cut relief trenches in the shoulder material and, as shown in Figure 2-16, water flowed from the HMA layer. Consequently, edge drains were installed to drain the existing HMA layer, and a chip seal was applied to minimize future surface water ingress. Figure 2-17 shows the edge drain installation process. More details of the permeability problems and edge drain installation on SH 114 are discussed in the 0-4822-1 project report (Scullion 2006).

Figure 2-15. Coarse Nature of 1″ SFHMAC Superpave Mix Used on SH 114.

Figure 2-16. Water Draining from 1″ SFHMAC into Relief Trench.

Figure 2-17. Edge Drain Installation on the Superpave Section.

Construction Cost Comparisons

Table 2-5 is a comparative summary of the cost estimates and shows that the ½″ HDSMA with PG 70-28 binder was the most expensive in terms of unit cost (TxDOT, 2006a). Note that the original design binder type for the ½″ HDSMA (Layer 1) was PG 76-22, but the contractor switched to using PG 70-28 on site for cracking resistance issues. According to the contractor and area engineer, the "-28" is technically supposed to have more polymer added to resist the low temperature cracking. They believe that the aggregate structure will resist the rutting while the added polymer will take care of the cracking.

Table 2-7. Comparative Summary of Cost Estimates.

Layer	**Superpave Section**		**Conventional Section**	
	Material	**Cost**	**Material**	**Cost**
Layer 1	½″ HDSMA	\$55.00/mgr	½″ HDSMA	\$55.00/mgr
Layer 2	¾″ SFHMAC	\$32.80/mgr	TxDOT Type C	-
Layer 3	1″ SFHMAC	\$31.20/mgr	TxDOT Type B	\$28.80/mgr
Layer 4	¾″ SFHMAC (RBL)	\$30.10/mgr	TxDOT Type C (RBL)	\$40.30/mgr
Layer 5	Lime-treated subgrade			\$2.21/m^2
mgr = mega gram; m^2 = square meter (or meter squared)				

SUMMARY

The SH 114 FDAP project is summarized as follows:

- The SH 114 project consists of two structural sections: the Superpave mix-design and the conventional TxDOT mix-design. In terms of construction, the Superpave section is at least over 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 95 percent reliability level. The total pavement thickness on both sections is 30″, 22″ of four HMA layers plus 8″ 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 AV (i.e., 96 percent density) as the design criterion, except for the RBL at 97 percent. Additionally, the HMA mixtures are also required to pass the Hamburg wheel tracking test at a maximum rut depth of 0.5″.
- The rut-resistant layer is 13″ thick and consists of a 1″ 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 (RBL) is 4″ thick and consists of a ¾″ 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.
- In general, the Superpave section used coarser aggregate gradations, lower binder contents, and stiffer high PG binder grades than the Conventional section.
- The Superpave mixes were more difficult to construct than the conventional mixes. Observed permeability problems necessitated the addition of edge drains and a surface seal prior to placing the final SMA surface.
- The Windrow elevator HMA material transfer system was less effective than the Roadtec in eliminating thermal segregation.

CHAPTER 3
LABORATORY TESTING, RESULTS, AND ANALYSES

The laboratory tests and associated results are presented and analyzed in this chapter. These tests include the Dynamic Shear Rheometer, the Hamburg wheel tracking device, the Overlay Tester, the Dynamic Modulus, and Repeated Load Permanent Deformation. In this chapter, the binder tests are presented first, followed by a discussion of the asphalt mixture test specimens. Thereafter, laboratory tests and results for the asphalt mixtures are presented and discussed. A summary of the findings is then presented at the end of the chapter.

BINDER DYNAMIC SHEAR RHEOMETER TESTING

Characterization of the binder rheological properties in terms of the dynamic shear complex modulus (G*) and phase angle (δ) was accomplished with the Dynamic Shear Rheometer (DSR) (AASHTO, 1998). As well as providing a comparative visco-elastic analysis of the asphalt binders for the high temperature properties, the G* and δ constitute input data for the MEPDG Level 1 and 2 analyses (see Chapter 5). Figure 3-1 shows a schematic illustration of the DSR loading configuration.

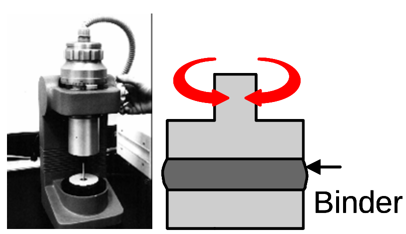


Figure 3-1. DSR Setup and Loading Configuration.

The DSR loading configuration consists of applying a sinusoidal shear stress at an oscillating angular frequency of 10 rad/s and various temperatures. In this project, the test temperatures ranged from 122 °F to 180 °F. An automated water bath is used to control and maintain the test temperatures. As per standard procedure, all the binder samples were rolling thin film-oven (RTFO) aged (short-term) prior to DSR testing (AASHTO, 1994).

A minimum of two samples were tested per binder type. During DSR testing, the measurable test data include the shear stress, temperature, angular frequency, G* values, and the phase angle. Figures 3-2 and 3-3 show plots of the DSR test results. A detailed tabulation of these results (G*, δ, and G*/Sin δ) is also included in Appendix B.

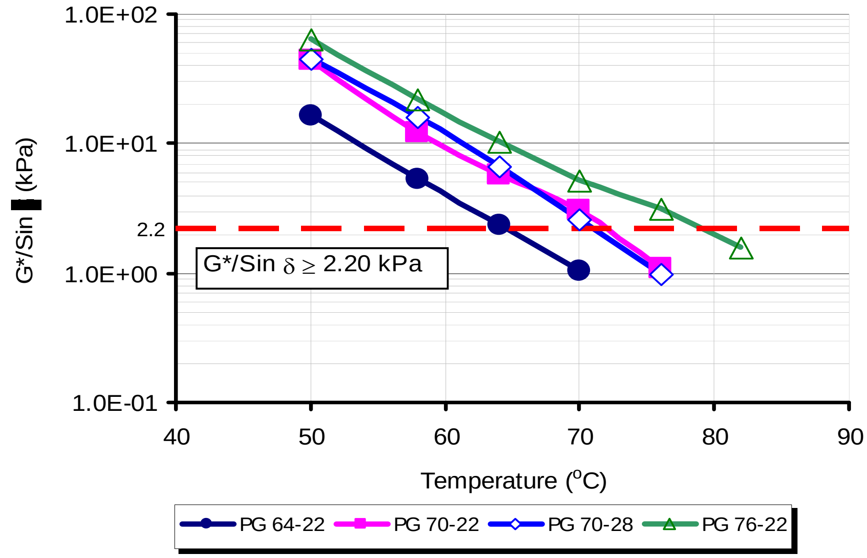


Figure 3-2. Binder G*/Sin δ (delta) versus Temperature.

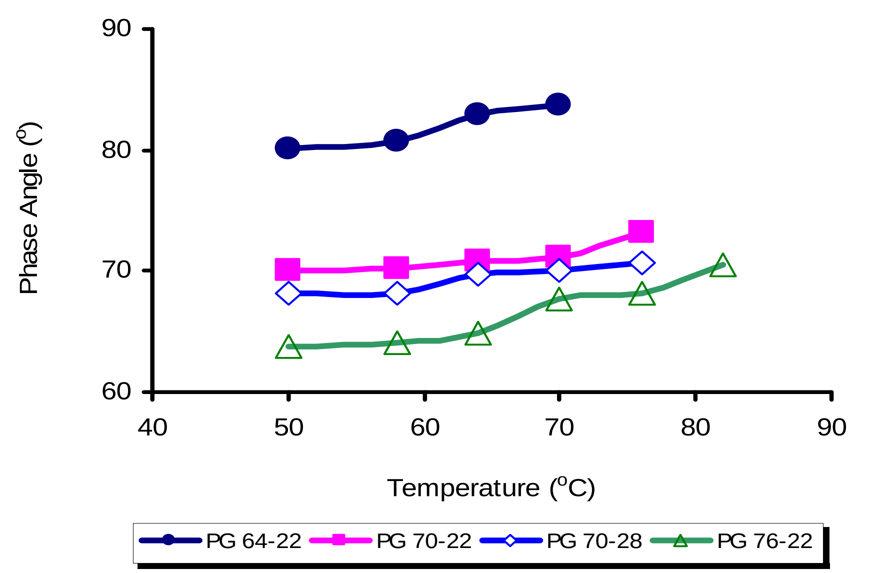


Figure 3-3. Phase Angle versus Temperature.

As would be theoretically expected, Figures 3-2 and 3-3 show that the softer PG 64-22 binder has the lowest G* values, but it has the highest phase angles and vice versa for the stiffer PG 76-22 binder. For the two PG 70-XX binders, which are in the intermediate stiffness range, these high temperature properties indicate that both the G* and δ do not differ significantly, albeit that the PG 70-22 seems to be more viscous with relatively higher phase angles than the PG 70-28 binder (Figure 3-3). Figure 3-2 also indicates that the binders met the PG specification (for the high temperature properties) consistent with the prescribed $G^*/\text{Sin } \delta \geq 2.20$ kPa threshold for Superpave performance-graded binders (Asphalt Institute, 1996).

ASPHALT MIXTURE TEST SPECIMENS

Three types of test specimens for the asphalt mixtures were fabricated and tested. These test specimens consisted of:

1) laboratory gyratory-molded samples from the raw materials (binders, aggregates, and other additives) denoted as "Lab specimens" or "Lab samples"; these were molded to the gradation and binder content specified in the approved mix design report;
2) field-molded samples from the plant mix at the time of placement using the TTI mobile laboratory denoted as "Plant Mix specimens" or "Plant Mix samples"; and
3) specimens cut from field-extracted cores denoted as "Core specimens" or "Core samples."

These three types of test specimens were utilized to enable comparison of the mix quality and compaction among the laboratory-prepared mix, plant mix, and field-extracted cores. Comparative characterizing of the material properties allowed for checking the plant mix quality, construction quality, and prediction of performance. The test specimens are discussed in detail in the subsequent text. Note that a minimum of two replicate specimens were used for each mixture type/layer per test.

Laboratory (Lab) Molded Samples

TTI's Servopac Superpave Gyratory Compactor (SGC) was utilized for molding samples in the laboratory from the same raw materials (binders, aggregates, and other additives) that the contractor had used on the SH 114 perpetual pavements. As recommended by TxDOT mix verification procedures, all the test specimens were molded to a target AV content of 7±0.5 percent. Figure 3-4 shows one of TTI's gyratory compactors.

Figure 3-4. Gyratory Compactor.

The SGC compaction parameters were 1.25° compaction angle and 87 psi vertical pressure at a rate of 30 gyrations/minute. Prior to compaction, all the loose asphalt mixtures were subjected to 4 hr short-term oven aging at 275 °F consistent with the AASHTO PP2 standard aging procedure for Superpave mixture performance testing (AASHTO, 1994). For each mixture type, different mixing and compaction temperatures were utilized consistent with the binder PG grade (TxDOT, 2006b).

Field-Molded Samples (Plant Mix) Using the TTI Mobile Lab

TTI's mobile laboratory, shown in Figure 3-5, is equipped with a Pine Gyratory compactor and an oven for fabricating specimens in the field from the ready hot-mix on site. The mobile lab facilitates molding of test specimens at similar field hot-mix conditions and therefore allows for checking the plant mix and construction quality, respectively.

Figure 3-5. TTI Mobile Lab.

Like the laboratory-molded samples, all the field samples were molded to 7±0.5 percent AV. For each mixture/layer type, the molding compaction temperature was similar to the field placement temperature of the plant mix. Essentially, the loose hot-mix was scooped just before compaction after being placed on the pavement surface from the truck. Thermocouple probes were used for measuring the temperatures of both the plant mix on the pavement surface and the samples being molded. The electric oven mounted inside the mobile lab is used to maintain the sample temperature if molding with the gyratory compactor is not done immediately after scooping the loose hot-mix. Like the Servopac SGC, the Pine SGC has a capacity of 87 psi vertical pressure, 30 gyrations/minute, and 1.25° compaction angle.

Note, however, that the TTI mobile lab was not available during placement of the Superpave mixes; it was used during the placement of the conventional mixes only (on the Conventional section).

Field-Extracted Core (Core) Samples

Cores were also extracted in the field from which test specimens were sawn to characterize the material layer properties and among others to assess the construction quality and predict performance. The cores were extracted from both the wheelpaths (mostly in the outside lane and outside wheelpath) and the untrafficked shoulders. TTI's 6″ diameter coring rig (with a coring depth of up to 30″) was used to accomplish this task. Figure 3-6 shows TTI's coring rig with an example of a 20″ long field-extracted core.

Figure 3-6. TTI's Coring Rig and Field-Extracted Core.

From the field-extracted cores, a minimum of two test specimens were cut for each layer/material type. The specimens were labeled accordingly and the air voids determined. The measured AVs for the field-extracted cores are shown in Table 3-1.

Table 3-1. Air Voids Measured from Field-Extracted Core Specimens.

Layer	Material	Measured AV		Design Density
		Range	Average	
FW 01: Superpave Section				
Layer 1	½″ HDSMA	-	-	96.0%
Layer 2	¾″ SFHMAC	6.1 – 9.3%	7.6%	96.0%
Layer 3	1″ SFHMAC	7.4 – 10.6%	7.8%	96.0%
Layer 4	¾″ SFHMAC (RBL)	5.2 – 7.6%	7.1%	97.0%
FW 02: Conventional Section				
Layer 1	½″ HDSMA	-	-	96.0%
Layer 2	Type C	5.8 – 7.7%	7.0%	96.0%
Layer 3	Type B	5.3 – 8.2%	6.7%	96.0%
Layer 4	Type C (RBL)	4.8 – 6.9%	6.4%	97.0%

In general, the Conventional section appears to have been better compacted than the Superpave section, at least based on the AV data in Table 3-1. With the lower binder content and coarse aggregate gradations discussed in Chapter 2, it may not be surprising that the 1″ SFHMAC layer was compacted to the least density (highest AV content). The higher number of compactive rolling passes may account for TxDOT Type B's better AV content, i.e., improved compaction rolling pattern (Chapter 2). As would be expected, both sections indicate the highest density for the RBL, albeit that the Conventional section (lower AV) appears to be better. Note that the 0.5″ HDSMA layer (Layer 1) had not been placed at the time of field coring, so it was not included in the specimen matrix for the field-extracted cores.

THE HAMBURG WHEEL TRACKING TEST

The HWTT is a test device used for characterizing the rutting resistance of asphalt mixtures in the laboratory including stripping susceptibility assessment (moisture damage potential). The loading configuration consists of a repetitive passing load of 158 lb-force (705 N) at a wheel speed of 52 passes per minute and a test temperature of 122 °F in a controlled water bath. The HWTT test specimens are 2.5″ thick by 6″ diameter, with one trimmed edge. Figure 3-7 shows the Hamburg test device with a specimen setup.

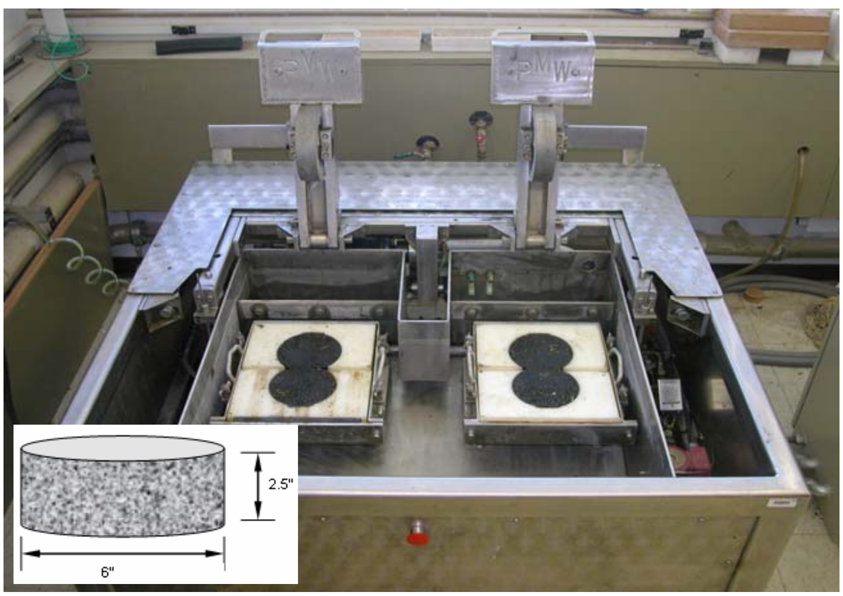


Figure 3-7. The Hamburg Test Device and Test Specimen.

During HWTT testing, the measurable parameters include the applied load, temperature, number of load passes, and vertical permanent deformation (rutting). The HWTT terminal rutting failure criterion is 12.5 mm rut depth ($Rut_{HWTT} \leq$ 12.5 mm [0.5″]) and is listed in Table 3-2 per binder type. The HWTT test results are summarized in Table 3-3. Note that the results in Table 3-3 should be analyzed with respect to both the rut magnitude and the corresponding number of load passes indicated in parentheses. It is also important to review the binder content of the Superpave mixes; in some instances, the field samples had substantially different binder contents than that proposed in the mix design.

Table 3-2. HWTT Terminal Rutting Failure Criterion.

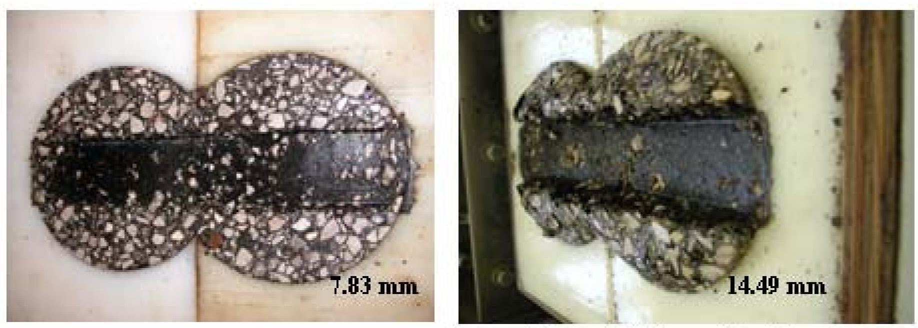
Rut_{HWTT}	Number of Passes	Mixture with Binder Type
≤ 12.5 mm (0.5″)	10,000	PG 64-XX
≤ 12.5 mm (0.5″)	15,000	PG 70-XX
≤ 12.5 mm (0.5″)	20,000	PG 76-XX

Table 3-3. HWTT Laboratory Test Results.

Layer	Material	Binder	HWTT ($Rut_{HWTT} \leq$ 12.5 mm [0.5″]) (mm)		
			Lab	Plant Mix	Core
FW 01: Superpave Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	5.18 (@20,000)	2.36 (@20,000)	-
Layer 2	¾″ SFHMAC	4.2% PG 76-22 *(5.23%)*	11.89 (@20,000)	-	13.38 (@12,151)
Layer 3	1″ SFHMAC	4.0% PG 70-22 *(3.35%)*	3.12 (@20,000)	-	7.83 (@20,000)
Layer 4	¾″ SFHMAC (RBL)	4.2% PG 64-22	14.10 (@8,450)	-	13.18 (@2,951)
FW 02: Conventional Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	5.18 (@20,000)	2.36 (@20,000)	-
Layer 2	Type C	4.4% PG 70-22	13.13 (@15,000)	-	12.34 (@15,000)
Layer 3	Type B	4.5% PG 64-22	13.62 (@9640)	13.38 (@8100)	12.52 (@7000)
Layer 4	Type C (RBL)	5.3% PG 64-22	12.78 (@4875)	13.63 (@6301)	14.49 (@10,000)

( ) = extracted binder content from field cores.
Except for Layer 1 (igneous/granite), all aggregate type was limestone.

Table 3-3 shows that the 1″ SFHMAC layer performed satisfactorily and passed the HWTT test ($Rut_{HWTT} < 12.5$ mm). By contrast, the TxDOT Type B failed the HWTT test ($Rut_{HWTT} > 12.5$ mm) indicating that this layer may be susceptible to rutting. This rutting may be attributed to the use of the softer PG 64-22 binder contrary to the recommended PG 76-22 binder in Figure 1-1 (Chapter 1). Note also that during the initial laboratory mix design process, the Type B mix was not checked in the HWTT test. According to the area engineer, there was no specification requirement to run the HWTT tests on the conventional dense-graded Type B and C mixes during mix-design. As expected, all the fatigue-resistant layers (Layer 4 - RBL) performed poorly ($Rut_{HWTT} > 12.5$ mm). Figure 3-8 shows a pictorial comparison of rutting for a rut-resistant and fatigue-resistant layer, respectively. Clearly, there is more rutting in the RBL, about twice that found in the rut-resistant layer. Note that the flexible RBLs are not specifically designed to be rut-resistant, and it is, therefore, not surprising that they failed the HWTT test.



Layer 3, Rut-resistant layer (1″ SF HMAC), Rut depth < 12.5 mm

Layer 4, Fatigue-resistant layer (RBL, Type C), Rut depth > 12.5 mm

Figure 3-8. HWTT Rutting on 1″ SFHMAC and TxDOT Type C (RBL).

In general, the Superpave section appears to exhibit better laboratory rutting resistance characteristics than the Conventional section. Some of the possible contributing factors to the Conventional section's poor lab performance in comparison to the Superpave section could be the use of the softer PG binders and higher binder contents. Also, considering that the Superpave section is more than a year older and has been subjected to conventional traffic, densification under traffic compaction could be one contributing factor to its field cores' better rut resistance compared to the newer Conventional section placed in 2006. Note that while the cores from the Conventional section were extracted just after 4 weeks of placement (i.e., constructed in

May 2006), the Superpave section was over 1 year older at the time of core extraction and testing (i.e., constructed successive layers/lifts in Dec 2003/2004/2005).

However, the ¾″ SFHMAC layer (Layer 2, core) with PG 76-22 binder and relatively coarse aggregate gradation performed poorly (about 13.38 mm at 12,151 load passes) in contrast to the lab-molded specimens (about 11.89 mm at 20, 000 load passes). The reasons for this difference are being investigated. However, binder extraction tests with the Troxler Ignition Oven revealed a binder content of 5.23 percent (versus 4.2 percent design) and an average AV content of 7.6 percent for the specific cores tested. This difference in binder content could have been a contributing factor but is yet to be checked with other cores from different highway locations. For all the samples tested, the ½″ HDSMA exhibited superb rutting performance as would be typically expected for SMA mixtures.

THE OVERLAY TESTER

The Overlay Tester is a simple performance test for characterizing the cracking potential of asphalt mixtures in the laboratory at an ambient (room) temperature of 77 °F. The test loading configuration consists of a cyclic triangular displacement-controlled waveform at a maximum horizontal displacement of 0.025″ and a loading rate of 10 s per cycle (5 s loading and 5 s unloading). Typical OT test specimens are 6″ total length, 3″ wide, and 1.5″ thick that can be conveniently cut by trimming a lab molded (SGC) specimen or a 6″ diameter highway core. Figure 3-9 shows the OT setup and a test specimen.

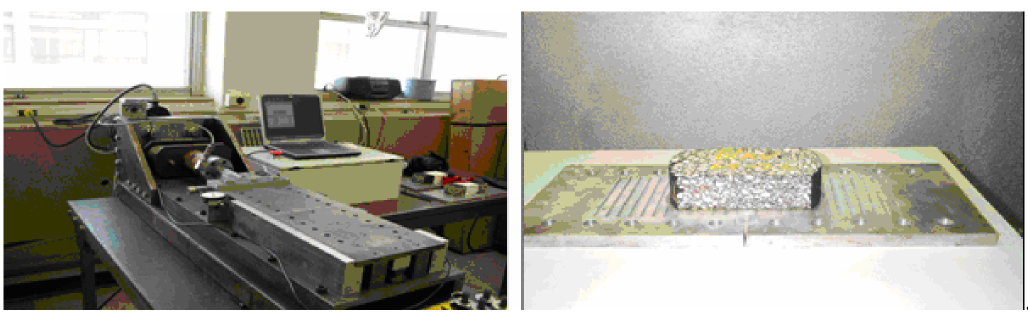


Figure 3-9. The Overlay Tester and Specimen Setup.

During OT testing, the measurable parameters include the applied load (stress), opening displacement (fixed at 0.025″), time, number of load cycles, and the test temperature. Details of the OT are published elsewhere (Scullion, 2006; Zhou et al., 2005). For surfacing mixes, the proposed cracking failure criterion is 300 load cycles (i.e., $N_{OT} \geq 300$) at a stress reduction of 93 percent, and 750 cycles has been proposed for RBL type layers (Zhou and Scullion 2005).

The OT results for the SH 114 mixes are summarized in Table 3-4. However, no OT criteria are available for the rut-resistant layers. With regard to the anticipated modes of cracking, either bottom up or top down, it is clearly recommended to have good crack-resistant mixes at the top and bottom of the perpetual pavement structure.

Table 3-4. OT Laboratory Test Results.

Layer	**Material**	**Binder**	**OT (N_{OT})**		
			Lab	**Plant Mix**	**Core**
FW 01: Superpave Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	900+	643	-
Layer 2	¾″ SFHMAC	4.2% PG 76-22 *(5.23%)*	153	-	206
Layer 3	1″ SFHMAC	4.0% PG 70-22 *(3.35%)*	108	-	74
Layer 4	¾″ SFHMAC (RBL)	4.2% PG 64-22	768	-	652
FW 02: Conventional Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	900+	643	-
Layer 2	Type C	4.4% PG 70-22	324	-	106
Layer 3	Type B	4.5% PG 64-22	175	86	122
Layer 4	Type C (RBL)	5.3% PG 64-22	900+	436	550

( ) = extracted binder content from field cores.
Except for Layer 1 (igneous/granite), all aggregate type was limestone.

Table 3-4 shows that both sections are crack resistant based on the RBL layer (Layer 4: ¾″ SFHMAC and TxDOT Type C, respectively), which is designed to be fatigue resistant. Consequently, no serious bottom-up fatigue cracking can theoretically be expected from these two sections.

Notice also from Tables 3-3 and 3-4 that the ½″ HDSMA (Layer 1) performed very well under both HWTT and OT testing. Theoretically, this is not surprising because SMA mixtures are generally designed to be both fatigue- and rut-resistant based, among other factors, on their high binder content (minimum 6 percent), and their use of higher quality aggregates. Layers 2 and 3 are generally not designed to be fatigue-resistant, but the values obtained are thought reasonable for limestone aggregate mixes with relatively low binder contents.

However, cracking resistance is not considered critical for the rut-resistant mixes (1″ SFHMAC and TxDOT Type B in Layer 3). Tables 3-3 and 3-4 also show that the plant mix and field core results for the Conventional section are fairly comparable. These results are remarkable and show the potential of the TTI mobile lab for relating to on-site conditions, suggesting that the plant mix and construction quality were consistent.

THE DYNAMIC MODULUS TEST

Dynamic Modulus (DM) testing is an AASHTO standardized test method for characterizing the visco-elastic properties of asphalt mixtures, measured in terms of the complex modulus |E*| (AASHTO, 2001). In this project, the DM test was also used for generating the PerRoad, VESYS5, and MEPDG Level 1 input data for the asphalt mixtures; see Chapter 5 of this interim report. A typical DM test is performed over a range of different temperatures and loading frequencies. In this project, the DM test was conducted at five test temperatures of 14, 40, 70, 100, and 130 °F and six loading frequencies of 25, 10, 5, 1, 0.5, and 0.1 Hz for each test temperature, respectively (AASHTO, 2001). DM is a stress-controlled test involving application of a repetitive sinusoidal dynamic compressive-axial load (stress) to an unconfined specimen. Figure 3-10 shows TTI's Universal Testing Machine (UTM-25) setup that was used for conducting the DM test and includes the loading configuration and test specimens.

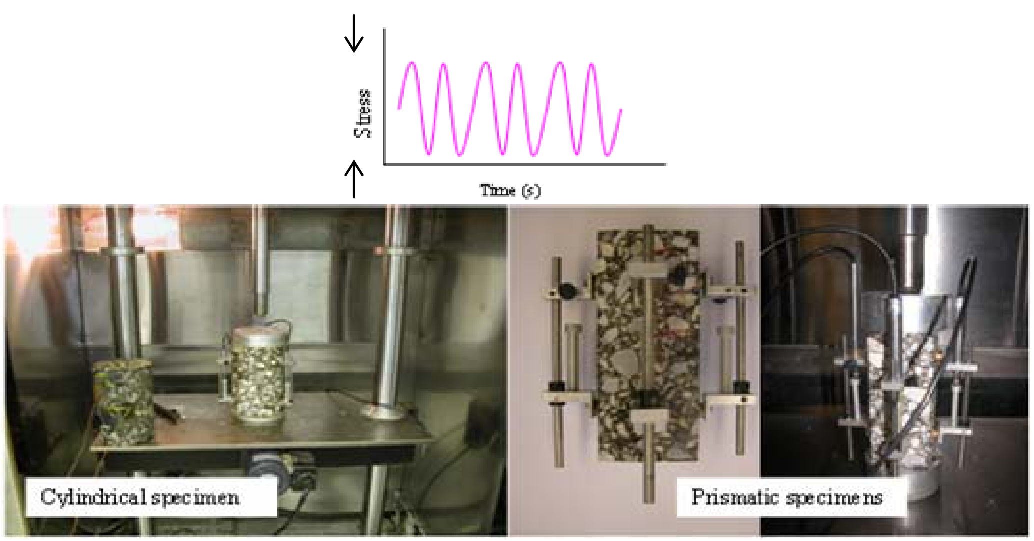


Figure 3-10. DM Test Setup and Test Specimens (Cylindrical and Prismatic).

DM Test Specimens and Testing Procedure

The standard DM test specimen is cylindrically shaped with dimensions of 4″ diameter (ϕ) by 6″ in height (h). However, for the field cores with thinner layers (< 6″) like the RBL, the TxDOT Type C, or the ¾″ SFHMAC, prismatic specimens as shown in Figure 3-10 were used. These prismatic specimens were cut consistent with the procedure suggested by Dr. Jacob Uzan (Project Consultant) in Report 0-4822-1 (Scullion, 2006). The minimum specimen dimensions were 2″ breadth by 2″ width by 5″ in length to ensure at least a minimum 1.5 aspect ratio and coverage of the nominal aggregate size. Although reasonably promising results have been obtained thus far, utilization of the prismatic specimens is still under investigation in the ongoing research to validate their applicability, in particular, evaluation of the anisotropic effects. Note, however, that all the lab and plant mix specimens including the core specimens for the rut-resistant layers (Layer 3) that are more than 6″ thick were standard cylindrically shaped (4″ ϕ by 6″ h).

The stress level for conducting the DM test was chosen to maintain the measured resilient strain (recoverable) within 50 to 150 microstrain consistent with the AASHTO TP 62-03 test protocol (AASHTO, 2001). The order for conducting each test sequence was from the lowest to the highest temperature and the highest to the lowest loading frequency at each temperature to minimize specimen damage. For each test sequence, the test terminates automatically when a preset number of load cycles have been reached. During DM testing, the measurable parameters include the applied load (stress), loading frequency, temperature, vertical axial deformations, phase angle, and the dynamic modulus.

DM Test Results

The typical parameters that are computed from DM testing are the complex modulus ($|E^*|$) and the phase angle (δ) that characterizes the visco-elastic properties of the asphalt mixtures. The $|E^*|$ data are then used for generating master-curves for pavement performance prediction in the MEPDG program (Level 1 analysis). A typical set of the $|E^*|$ data from DM testing is included in Appendix B for all the asphalt layers/materials. A comparative plot of the $|E^*|$ master-curves for the rut- and fatigue-resistant layers is shown in Figure 3-11 based on lab test specimens, at a reference temperature of 70 °F.

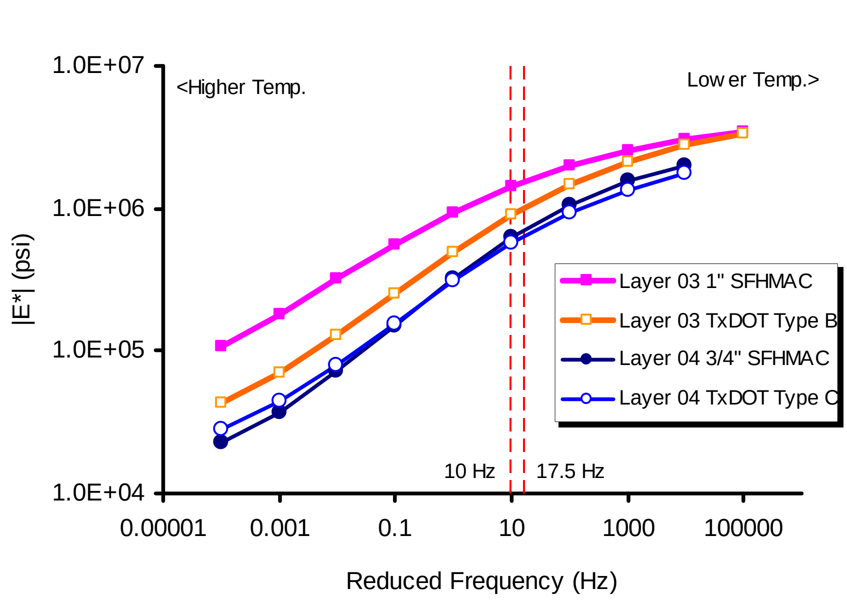


Figure 3-11. |E*| Master-Curves at 70 °F (Lab Specimens).

The above |E*| master-curves were generated using the time-temperature superposition signomoidal model shown in Equation 3-1 (Pellinen and Witczak, 2002):

$$Log(|E^*|) = \delta + \frac{\alpha}{1+e^{\beta-\gamma\log(\xi)}} \qquad \text{Equation (3-1)}$$

where:

|E*|	=	Dynamic modulus (psi)
ξ, δ	=	Reduced frequency (Hz) and minimum modulus value (psi)
α, β, γ	=	Span of modulus and shape parameters

The expected variation in stiffness of the rut-resistant and fatigue-resistant (flexible) layers is clearly evident in Figure 3-11 based on the higher and lower |E*| magnitudes, respectively. This response trend was theoretically expected and is consistent with the mix-design characteristics and structural design expectations. Notice also that as expected (see Chapter 2), the 1″ SFHMAC is considerably stiffer than the TxDOT Type B mixture especially at the lower reduced loading frequencies (i.e., ≤ 100 Hz), which also corresponds to the higher temperature domain.

Table 3-5 provides a summary of the |E*| results at 70 °F, 10 Hz. Detailed |E*| results are listed in Appendix B.

Table 3-5. Summary |E*| Results at 70 °F, 10 Hz.

Layer	Material	Binder	Measured |E*| (ksi) @ 70 °F, 10 Hz		
			Lab	Plant Mix	Core
FW 01: Superpave Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	592	639	-
Layer 2	¾″ SFHMAC	4.2% PG 76-22 *(5.23%)*	1062	-	690
Layer 3	1″ SFHMAC	4.0% PG 70-22 *(3.35%)*	1364	-	1684
Layer 4	¾″ SFHMAC (RBL)	4.2% PG 64-22	605	-	651
FW 02: Conventional Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	592	639	-
Layer 2	Type C	4.4% PG 70-22	647	-	839
Layer 3	Type B	4.5% PG 64-22	892	1051	1166
Layer 4	Type C (RBL)	5.3% PG 64-22	542	550	527

( ) = extracted binder content from field cores.
Except for Layer 1 (igneous/granite), all aggregate type was limestone.

With a few exceptions, the Superpave section generally appears to be much stiffer than the Conventional section based on the higher |E*| values in magnitude. Based on these results, the Superpave section would theoretically be expected to be more rut-resistant than the Conventional section (lower binder PG grades with high binder contents), which is consistent with the HWTT rutting performance predictions observed previously. Also, the |E*| values from the field core specimens, in particular for the rut-resistant layers (Layer 3), are higher than those of the corresponding lab values. Densification under traffic compaction could be a contributing factor. The one exception is the ¾″ SFHMAC (Layer 2) where the lab specimen was measured at 1062 ksi, whereas the field core was 690 ksi. The cause of this was probably the higher field core binder content (5.23 percent as compared with 4.2 percent); see Table 2-2 in Chapter 2.

Similar to Table 3-5, Table 3-6 is a summary of the |E*| results at 77 °F, 17.5 Hz. These modulus results at 77 °F, 17.5 Hz were approximated from the data shown in Appendix B, based on linear interpolations. This was to enable comparison with the FWD measurements (in Chapter 4), which are typically conducted at 17.5 Hz and the data normalized to 77 °F.

Table 3-6. Summary |E*| Results at 77 °F, 17.5 Hz.

Layer	Material	Binder	Interpolated |E*| (ksi) @ 77 °F, 17.5 Hz		
			Lab	Plant Mix	Core
FW 01: Superpave Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	630	650	-
Layer 2	¾″ SFHMAC	4.2% PG 76-22 *(5.23%)*	1050	-	600
Layer 3	1″ SFHMAC	4.0% PG 70-22 *(3.35%)*	1150	-	1500
Layer 4	¾″ SFHMAC (RBL)	4.2% PG 64-22	625	-	600
FW 02: Conventional Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	630	650	-
Layer 2	Type C	4.4% PG 70-22	630	-	620
Layer 3	Type B	4.5% PG 64-22	800	910	1100
Layer 4	Type C (RBL)	5.3% PG 64-22	545	570	560

( ) = extracted binder content from field cores.
Except for Layer 1 (igneous/granite), all aggregate type was limestone.

The trend of the results listed in Table 3-6 is essentially similar to that observed in Table 3-5, albeit that the general change in the |E*| magnitude (comparing Tables 3-5 and 3-6) is not very significant or consistent. This change is attributed to the fact that the effect of the increase in temperature (70 to 77 °F) on the mixture stiffness is compensated for by the increase in frequency (10 to 17.5 Hz). In general, while an increase in temperature is typically associated with a decline in mixture stiffness (low |E*| values), the opposite is true for an increase in loading frequency. Overall, the results in Tables 3-5 and 3-6 are within theoretical expectations consistent with the mix-design characteristics discussed in Chapter 2; the RBLs have the lowest |E*| values (more flexible), and the rut-resistant layers are considerably stiffer with the highest |E*| values in magnitude ranging from 800 ksi (lab) to about 1684 ksi (cores).

THE REPEATED LOAD PERMANENT DEFORMATION TEST

The Repeated Load Permanent Deformation (RLPD) test was utilized to characterize the permanent deformation properties of asphalt mixtures, under repeated compressive Haversine loading to supplement the HWTT and DM test results. Note that the RLPD test is also one of the laboratory test methods Zhou and Scullion (2004) have recommended for generating input data (mixture properties) for the VESYS5 software. In this project, the RLPD test was conducted consistent with the procedures outlined in TxDOT Report 9-150-01-4 by Zhou and Scullion (2004).

RLPD is a stress-controlled test involving repetitive application of a Haversine-shaped compressive-axial load (stress) to an unconfined specimen, at a frequency of 1 Hz with 0.1 s loading time and 0.9 s rest period, respectively, for up to 5000 load cycles. Although the RLPD tests were conducted at two stress levels (30 and 20 psi) and two test temperatures (77 and 104°F), this chapter's emphasis is on the test results at 104 °F and 20 psi. This is because 104 °F is considered a closer representation of the Texas high temperatures (compared to 77 °F) at which rutting may be critical for in-service asphalt pavements, although temperatures over 104 °F are not uncommon in Texas especially in summer (see Figures 5-6 and 5-7 in Chapter 5). As with the DM test, TTI's UTM-25 setup was used for conducting the RLPD test, and the loading configuration is shown in Figure 3-12.

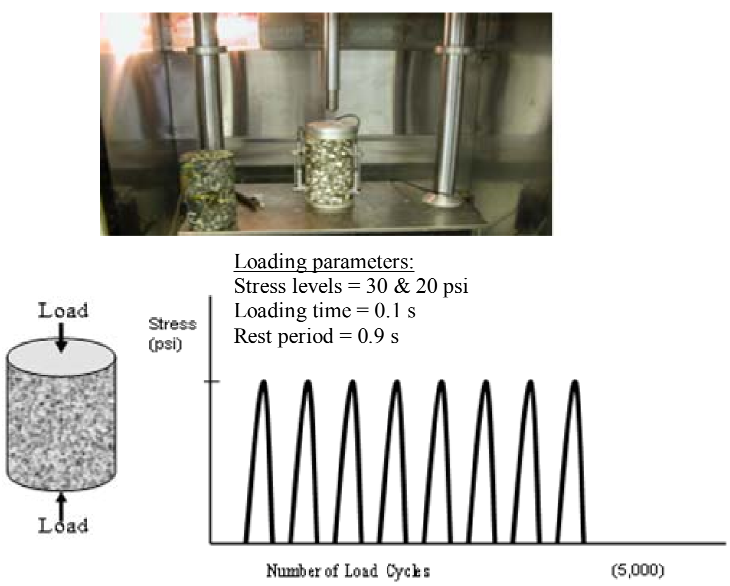


Figure 3-12. RLPD Loading Configuration.

For this project, the RLPD test was preset to terminate automatically either at 5001 load cycles or 25,000 microstrain (≅ 0.1″ permanent deformation), whichever came first. During RLPD testing, the measurable parameters include the applied load (stress), test temperature, frequency and time, number of load cycles, axial permanent deformation, and strains, respectively. All the RLPD test specimens were cylindrically shaped (4″ ϕ by 6″ h) using lab-molded samples only. As shown in Figure 3-13 (semi-log plots), only results for selected layers/materials are presented to supplement the HWTT and DM test results.

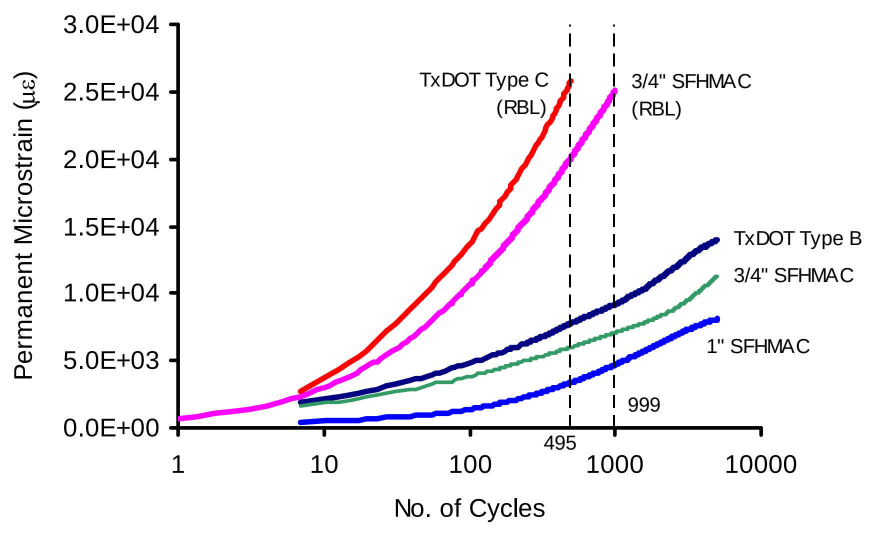


Figure 3-13. RLPD Accumulated Permanent Microstrain at 104 °F, 20 psi.

As expected, Figure 3-13 shows that the fatigue-resistant layers (RBLs) accumulated more permanent deformation in terms of the measured microstrain than the rut-resistant layers (1″ SFHMAC and TxDOT Type B). In fact, the RBLs had accumulated over two times more microstrain in less than 1000 load repetitions (about 25,000 με at 495 and 999 load repetitions for the TxDOT Type C and ¾″ SFHMAC, respectively). For the rut-resistant layers, the accumulated microstrain were still less than 25,000 με (in fact, about 8000 and 11,000 με, respectively) even after 5000 load repetitions, indicating that these layers are considerably more resistant to permanent deformation.

In concurrence with the HWTT and DM test results, Figure 3-13 further shows that the Superpave section is more resistant to permanent deformation than the Conventional section, based on the relatively lower microstrain magnitudes (i.e., ¾″ SFHMAC (RBL) < TxDOT Type C (RBL) and 1″ SFHMAC < TxDOT Type B, respectively). In fact, the Superpave section Layer 2 (¾″ SFHMAC) had accumulated less microstrain than the conventional TxDOT Type B mixture. At such a relatively high temperature (104 °F), the binder PG grade definitely plays a significant role. Note that the ¾″ SFHMAC (Layer 2) used a stiffer PG 76-22 binder, while the TxDOT Type B mixture used PG 64-22.

RLPD Permanent Deformation Parameters, Alpha (α) and Gnu (μ)

From a plot of the accumulative axial permanent microstrain versus load repetitions on a log-log scale, permanent deformation parameters ε_r, a, b, alpha (α), and gnu (μ) were determined consistent with the procedure described by Zhou and Scullion (2004). These parameters constitute the VESYS5 rutting input parameters (μ and α) for asphalt mixtures (see Chapter 5) and are defined as follows:

- ε_r = axial resilient microstrain measured at the 100th load cycle.
- a and b = intercept and slope of the linear portion of the permanent microstrain curve (log-log scale).
- alpha (α) = rutting parameter computed as $\alpha = 1 - b$.
- gnu (μ) = rutting parameter computed as $\mu = {ab}/{\varepsilon_r}$.

More details of these parameters can be found elsewhere (Zhou and Scullion, 2004). In theory, the smaller the μ value and the larger the α value, the greater the resistance to permanent deformation the mixture is. Consequently, stiffer mixtures like the 1″ SFHMAC are expected to have smaller μ values and larger α values than conventional dense-graded mixes. This phenomenon would also be expected for any given HMA mixture evaluated at lower test temperatures when compared to results measured at higher temperatures. Table 3-7 is a list of the μ and α parameters determined for the laboratory-molded mixtures (Lab) at 104 °F (20 psi) and 77 °F (30 psi), respectively.

Table 3-7. RLPD Permanent Deformation Parameters (Lab Mixes).

Layer	Material	Binder	α		μ	
			77 °F	104 °F	77 °F	104 °F
FW 01: Superpave Section						
Layer 1	½″ HDSMA	6.8% PG 70-28	0.809	0.611	0.210	0.219
Layer 2	¾″ SFHMAC	4.2% PG 76-22	0.765	0.594	0.221	0.232
Layer 3	1″ SFHMAC	4.0% PG 70-22	0.887	0.678	0.182	0.192
Layer 4	¾″ SFHMAC (RBL)	4.2% PG 64-22	0.721	0.565	0.250	0.281
FW 02: Conventional Section						
Layer 1	½″ HDSMA	6.8% PG 70-28	0.809	0.611	0.210	0.219
Layer 2	Type C	4.4% PG 70-22	0.747	0.586	0.242	0.251
Layer 3	Type B	4.5% PG 64-22	0.819	0.659	0.203	0.228
Layer 4	Type C (RBL)	5.3% PG 64-22	0.693	0.544	0.261	0.283
Except for Layer 1 (igneous/granite), all aggregate type was limestone.						

Table 3-7 shows that the rut-resistant layers have the highest α and lowest μ values, respectively, thus indicating resistance to permanent deformation. The RBLs, on the other hand, have the lowest α and highest μ values, respectively. Also, while the μ variation is not very pronounced, α exhibits an increasing trend with a decrease in temperature, suggesting an increasing resistance to permanent deformation with a decrease in temperature as would be theoretically expected.

SUMMARY

The following list summarizes the important findings from this chapter:

- As per DSR testing, the binder rheological properties are within theoretical expectations. All the binders met the PG specification consistent with the high temperature properties for Superpave performance-graded binders.
- Like the top ½″ HDSMA layer, the 1″ SFHMAC on the Superpave section passed the HWTT test ($Rut_{HWTT} < 12.5$ mm) with superior laboratory rutting resistant properties. However, the TxDOT Type B on the Conventional section failed the HWTT test ($Rut_{HWTT} > 12.5$ mm), thus exhibiting a potential for rutting. In the field, however, this Type B material will be under 5″ of more rut resistant HMA

layers (Type C and HDSMA). Because of this cover, no major rutting problems are anticipated with the Type B material. The poor HWTT rutting performance of the ¾″ SFHMAC cores (Layer 2 with a stiffer PG 76-22 binder) needs to be further investigated with cores extracted from different highway locations.

- Both the Superpave and Conventional sections exhibit no potential for bottom-up fatigue cracking based on OT testing of the RBLs (¾″ SFHMAC and TxDOT Type C). Like the top ½″ HDSMA layer, the measured number of OT load cycles was over 300, a failure criterion proposed for surfacing asphalt mixtures.
- In concurrence with the HWTT results, the DM and RLPD tests indicated that the Superpave section was generally stiffer and more resistant to permanent deformation/rutting than the Conventional section. The modulus values and accumulated permanent microstrain measured on the Superpave section were considerably higher and lower than on the Conventional section, respectively.
- As indicated by the DM test results, the rut-resistant layers, in particular the 1″ SFHMAC, are considerably stiffer mixtures than traditional dense-graded mixtures, with modulus values over 800 ksi.
- The differences in the laboratory-predicted performance is due among other factors to the differences in the mix-design characteristics. In general, the Superpave section used relatively higher PG binder grades, lower binder contents, and coarser aggregate gradations compared to the Conventional section. This difference may account for its superior permanent deformation/rutting performance under laboratory RLPD and HWTT testing, respectively.
- The good correlation observed between the lab and plant mix results suggests that the TTI mobile lab is promising as a means to perform quality assurance by assessing the plant-mix delivered to the project site.
- The use of prismatic specimens for DM testing and possibly RLPD needs to be further investigated, especially the anisotropic effects and specimen geometry.
- An appropriate OT failure criterion needs to be investigated and established for the intermediate and lower asphalt layers. The current $N_{OT} \geq 300$ failure criterion was proposed with surfacing asphalt mixtures in mind. Similarly, it is also felt that the HWTT criterion is best suited for asphalt mixtures in the upper layers.

CHAPTER 4

FIELD TESTING, RESULTS, AND ANALYSES

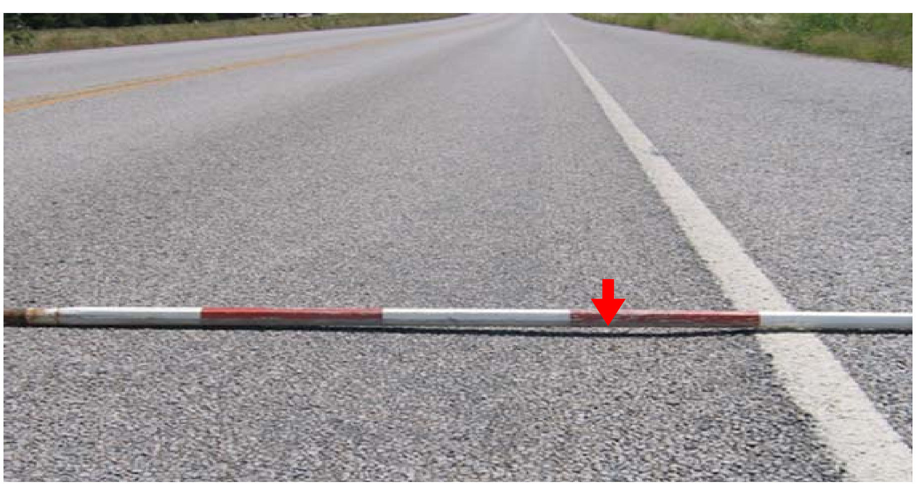
Initial field testing and performance evaluation of the SH 114 highway/pavement structures was categorized into four tasks:

1) visual surveys and pavement surface profile measurements,
2) non-destructive measurements with the ground penetrating radar,
3) falling weight deflectometer measurements, and
4) forensic evaluations of field-extracted cores.

These tasks together with the associated results are discussed in this chapter, followed by a summary of the important observations and findings.

VISUAL SURVEYS AND PAVEMENT SURFACE PROFILES

Visual surveys and pavement surface profile measurements indicated that some rutting was apparent on the top of the ¾″ SFHMAC layer prior to placement of the final ½″ HDSMA layer on the Superpave section. Figure 4-1 shows rutting of up to about 0.5″ in the outside wheel path on top of the ¾″ SFHMAC layer after one year of service. Figure 4-2 is a graphical plot of the surface rut measurements at various longitudinal locations.



0.5" rut depth in outside wheel path, on top of 3/4″ SFHMAC

Figure 4-1. Surface Rutting on Top of ¾″ SFHMAC Prior to ½″ HDSMA.

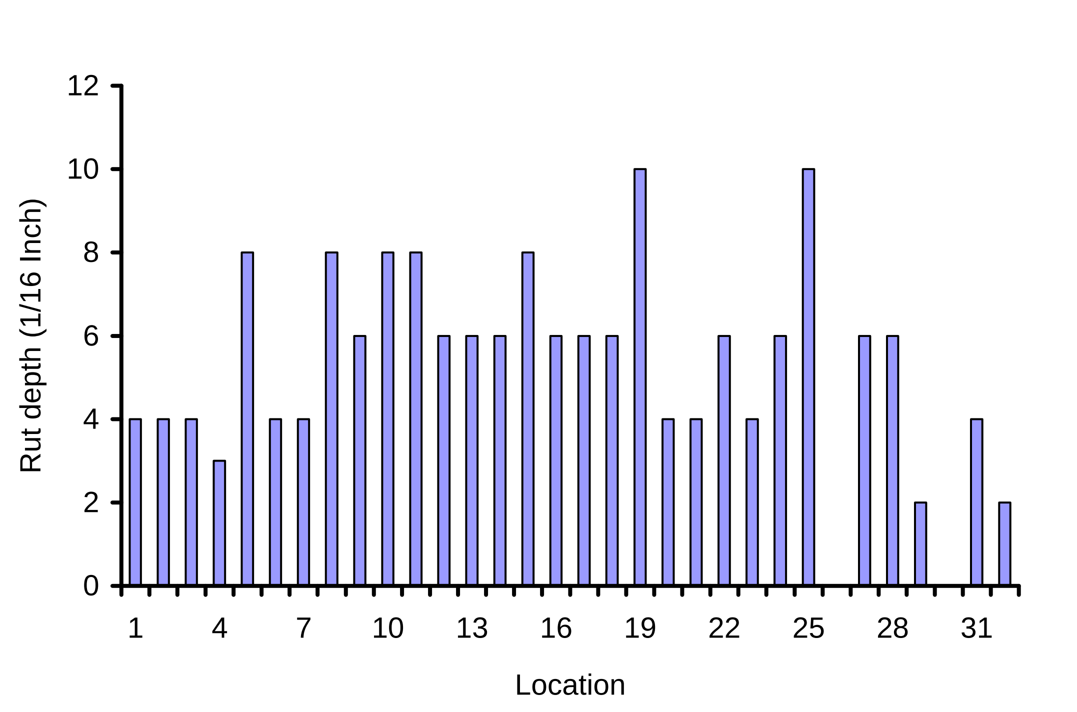


Figure 4-2. Rut Depths Measured on the Superpave Section (Prior to Placing the ½″ HDSMA).

The ¾″ SFHMAC was used as the driving surface for over a year prior to placement of the top (final) ½″ HDSMA in July 2006. Theoretically, this rutting would corroborate the laboratory test results reported for the ¾″ SFHMAC (Layer 2) core specimens in Chapter 3. However, this rutting was thought to be associated with the fact that there is a longitudinal construction joint in the outside wheelpath of the outside lane. This joint was staggered by only a small amount for each lift of HMA during construction. The 1″ SFHMAC was reported to be difficult to compact, and the longitudinal joints, particularly the free edge, were problematic. Subsequent coring, as will be described later in this section, indicated that the HMA in the rutted location was not well compacted.

Thus far, visual surveys of the finished pavement structures after placing the top and final ½″ HDSMA surfacing (in July 2006) indicated no major surface defects on both sections. The sections are now open to traffic, and performance monitoring will determine if the rutting found in the lower layers on the Superpave section will continue to occur.

GROUND PENETRATING RADAR MEASUREMENTS

TTI's 1-GHz air-coupled GPR was used for non-destructive evaluation of the SH 114 pavement structures both during and after construction. This GPR has a maximum operable speed of 70 mph with a potential to capture pavement data up to a depth of 2 ft (Scullion, 2006). Figure 4-3 shows TTI's GPR system setup.

Figure 4-3. TTI's GPR System Setup.

TTI's GPR is utilized to characterize: (1) pavement layer densities (AV), (2) pavement layer thicknesses, and (3) presence of free moisture. The measurements are based on electromagnetic wave principles and dielectric characteristics (function of moisture content and density) of the pavement layer materials. Details of the GPR are documented elsewhere (Scullion, 2006).

As reported elsewhere (Scullion, 2006), GPR measurements were first conducted by the Fort Worth District staff in 2004 (Wimsatt, 2003). Large areas of trapped moisture were found within the Superpave section. This observation was verified with field coring conducted by the Fort Worth District later in 2004. This and other observations led to the initiation of a construction field change to this project where edge drains (see Chapter 2) were installed to drain the moisture trapped within the Superpave section and the placement of a surface seal to prevent further water ingress. Subsequent GPR testing showed that these corrective measures were working well.

In the summer of 2006, GPR measurements were taken immediately after placement of the final ½″ HDSMA surfacing, both as a means of quality control and in an attempt to understand the reasons for the premature rutting in the Superpave section. The quality control application was discussed earlier. In these most recent GPR tests, no moisture was detected in either section. Some density variations were, however, observed on the Superpave section but with no evidence of major trapped moisture. Figures 4-4 to 4-6 show some of the GPR data for the Superpave and Conventional sections, respectively.

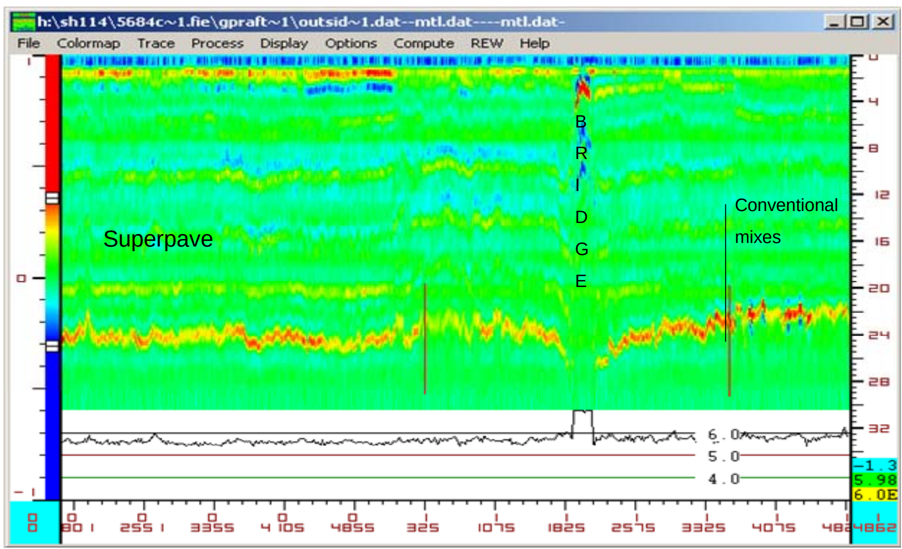


Figure 4-4. GPR Data after Construction (Superpave and Conventional).

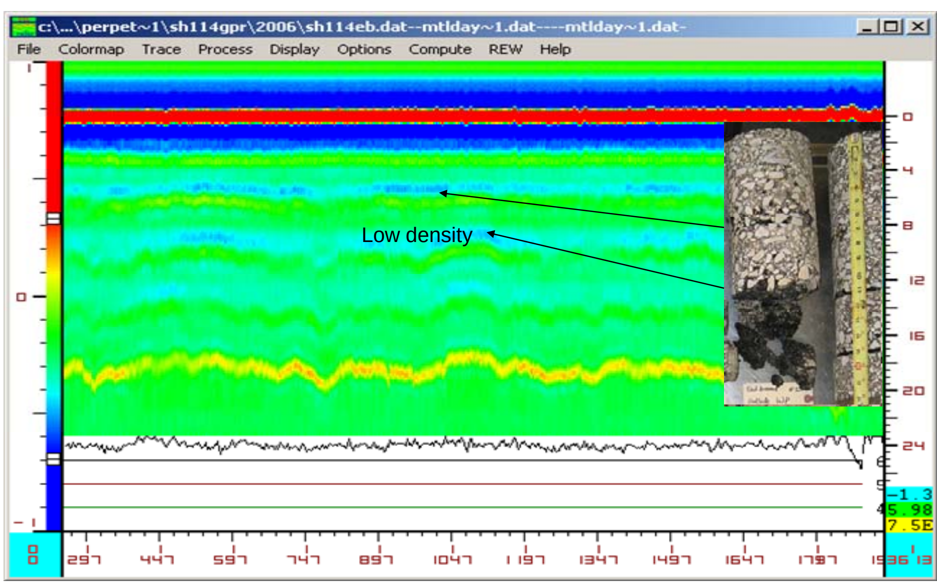


Figure 4-5. Low Density Areas on the Superpave Section.

Figure 4-5 shows some low density areas that were detected on the Superpave section but with no evidence of moisture ingress. This low density problem was primarily centered around the longitudinal construction joints that apparently appear not to have been significantly staggered. The GPR data in Figures 4-4 and 4-5 were collected after placement of the edge drains and a surface seal. These data should be compared with the GPR interpretation shown in Figure 4-6. The strong marked reflections are within the HMA layer typically at layer interfaces. From these analyses, the Superpave section clearly had indications of trapped moisture problems.

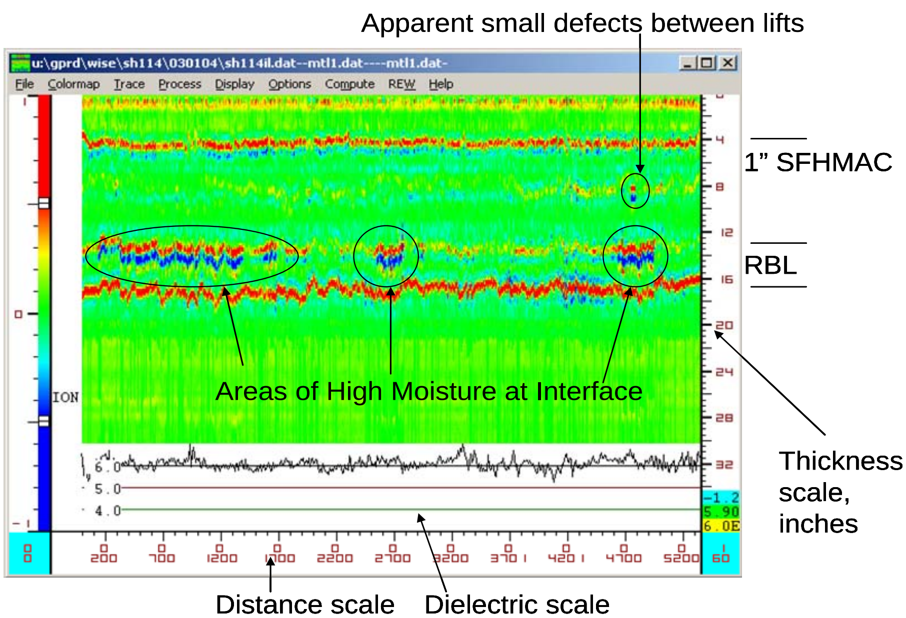


Figure 4-6. Moisture Problems on the Superpave Section (Wimsatt, 2004).

FALLING WEIGHT DEFLECTOMETER MEASUREMENTS

The FWD is a non-destructive test device used to characterize the pavement material properties in terms of the elastic modulus (Scullion, 2006). The FWD measurements are surface deflections measured at offsets from a load plate. Dynamic load impulses of up to 16,000 lb are applied to the surface of the pavement. In this project, FWD deflection measurements were collected at 300 ft intervals on the Superpave section and 50 foot intervals on the Conventional section.

Numerical back-calculation software, Modulus 6.0, was used to process the FWD raw data (Scullion and Liu, 2001). The Modulus 6.0 software is capable of backcalculating layer moduli for up to three layers on top of the subgrade. The sections on SH 114 have five layers on top of the subgrade, so it was necessary to combine the layers to do the backcalculation analysis. For this analysis, the pavement was modeled as 5″ of HMA (½″ HDSMAC and ¾″ SFHMAC) over 17″ of HMA (1″ SFHMAC plus 4″ of RBL) over 8″ of lime-treated subgrade. Other options are available for combining the layers, but during site testing and coring, it was found that:

- the temperatures acquired for use in the backcalculation analysis showed that the top 6″ of HMA was substantially hotter than the lower HMA layers. The average temperature was 105 °F for the top layer and 95 °F for the 1″ SFHMA layer.
- the lime-stabilized layer was present and providing good support to the succeeding HMA layers.

The results obtained for both sections using the cited layer combinations are shown in Appendix C. The results obtained for the Conventional section only are shown in Figure 4-7. The results shown in Figure 4-7 are reasonable, with average moduli values of 285 and 640 ksi for the two HMA layers and 69 and 11.9 ksi for the treated and natural subgrade layers, respectively. The subgrade modulus shown in Appendix C of 28 ksi for the Superpave section is a lot higher than the value obtained for the Conventional section of 11.9 ksi. This difference is because of the influence of shallow bedrock in the FWD data collected on the Superpave section. These FWD data were difficult to process; in many instances, the bedrock comes to the surface. With reference to the FWD data shown in Appendix C, 11 of the first 29 deflection bowls had deflections measured at 24″ from the center of the load plate (W3) of less than 1 mil. This is a clear indication of shallow bedrock.

The next step in the FWD data analysis is to assign backcalculated moduli values for each layer to provide input values to be used in the FPS design system. However, these values also have to be temperature corrected to 77 °F prior to use in the FPS 19 design system. The results for the composite 17″ layer are also shown graphically in Figure 4-8.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

District:2 (Fort Worth)
County :249 (WISE)
Highway/Road: SH 114

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	5.00	20,000	470,000	H1: v = 0.35
Base:	17.00	50,000	2,000,000	H2: v = 0.35
Subbase:	8.00	10,000	150,000	H3: v = 0.35
Subgrade:	72.14(by DB)	15,000		H4: v = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
9504.000	8,953	4.74	2.69	2.17	1.74	1.41	1.08	0.94	244.1	541.3	126.4	12.6	1.01	112.6 *
9557.000	9,001	4.37	2.90	2.43	2.02	1.68	1.27	1.12	331.4	804.9	27.4	10.2	0.97	102.2
9605.000	8,917	4.23	2.85	2.40	2.03	1.68	1.33	1.18	323.8	984.1	10.5	10.1	0.74	140.0
9653.000	8,949	4.41	3.01	2.55	2.12	1.73	1.33	1.16	399.2	654.7	49.1	9.2	0.53	115.0
9704.000	8,989	5.49	3.43	2.82	2.29	1.82	1.40	1.20	254.0	512.8	46.4	9.7	0.32	123.5
9754.000	8,862	6.30	3.79	3.09	2.44	1.90	1.39	1.15	230.8	347.6	50.4	9.6	0.70	100.0
9804.000	8,969	4.89	3.04	2.50	1.98	1.66	1.20	1.01	316.6	459.0	102.2	10.6	1.49	89.9
9851.000	8,921	4.60	2.75	2.32	1.90	1.51	1.12	0.94	273.5	609.3	94.7	11.4	1.11	94.6
9901.000	8,905	4.85	2.95	2.41	1.94	1.54	1.13	0.98	304.8	452.6	94.7	11.4	0.84	92.8
10000.000	8,897	4.20	2.38	1.96	1.58	1.26	0.95	0.81	255.8	725.2	85.7	14.3	0.35	99.6
10050.000	8,913	4.21	2.38	1.98	1.59	1.27	0.95	0.82	249.6	772.9	68.5	14.3	0.40	96.5
10100.000	8,886	4.04	2.35	1.88	1.45	1.13	0.85	0.71	359.3	469.4	120.8	15.9	0.58	101.5
10185.000	8,870	4.61	2.42	1.94	1.59	1.29	0.99	0.84	173.7	994.3	17.7	15.1	1.86	107.7
Mean:		4.69	2.84	2.34	1.90	1.53	1.15	0.99	285.9	640.6	68.8	11.9	0.84	102.1
Std. Dev:		0.62	0.43	0.36	0.30	0.24	0.18	0.16	60.4	205.3	38.4	2.3	0.45	11.6
Var Coeff(%):		13.21	15.09	15.36	15.62	15.75	15.90	16.49	21.1	32.0	55.8	19.4	54.15	11.4

Legend: in = inch; DB = depth to bedrock

Figure 4-7. Modulus 6.0 Results for the Conventional HMA Section on SH 114.

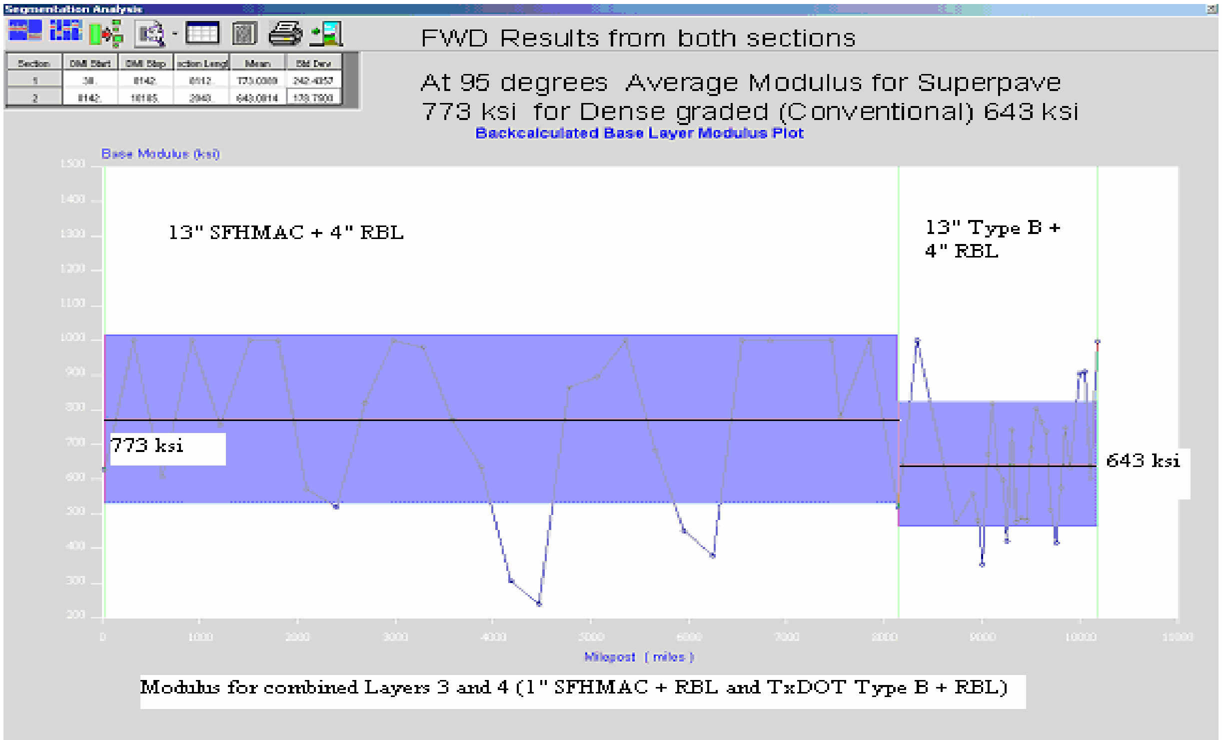


Figure 4-8. Composite Moduli Values Computed for Layers 3 and 4 of SH 114.

From the FWD results presented, the moduli values were assigned to the composite HMA layers as shown in Table 4-1. Conservative values were assigned to the subgrade and the lime-treated subbase layer. The modulus of the rut-resistant layers for the Superpave section is higher than that obtained for the Conventional section. This observation is also consistent with the DM results discussed in Chapter 3 (Tables 3-5 and 3-6). No significant differences were observed in the average moduli value for the composite 5″ surface HMA layer.

Table 4-1. Average Backcalculated FWD Moduli Values.

Layer/Material	**Average E Value (ksi)**	
	Superpave	**Conventional**
HMA Layers 1 and 2 ([SMA + ¾″ SFHMAC] or [SMA + Type C])	285 @ 105°F	285 @ 105°F
HMA Layers 3 and 4 ([1″ SFHMAC + RBL] or [Type B + RBL])	773 @ 95°F	643 @ 95°F
Base (6% lime-treated subgrade)	57	
Subgrade	11.9	

To compute design values for the FPS 19, it is necessary to apply temperature-corrected factors to the backcalculated moduli values. Temperature correction for the asphalt layers was accomplished through the following equation (Scullion, 2006):

$$TCF = T^{2.81} \ / 200{,}000 \qquad \text{Equation (4-1)}$$

Where *TCF* is the asphalt modulus temperature correction factor to 77 °F, and *T* is the temperature of the HMA at the time the FWD data were collected. For the FWD test temperatures of 95 and 105 °F, the computed TCF values were 1.80 and 2.39, respectively. To use the computed values within FPS, the next step is to assign temperature-corrected design moduli to each layer in the pavement structure. Based on the information generated in this report, Tables 4-2 and 4-3 are proposed for the Superpave and Conventional sections. These tables compare the moduli values initially assigned in the design process (Design), with the FWD backcalculated values (Field), with the values obtained from the lab (Lab).

Table 4-2. Comparison of Moduli Values Obtained for the Superpave Section.

Layer/Material	Average Modulus @ 77 °F (ksi)		
	Design	**Field (FWD)**	**Lab**
SMA + ¾″ SFHMAC	500	826	840
1″ SFHMAC	750	1394	1150
RBL	17	500	625
Lime-treated Subgrade	17	57	-
Subgrade	9.1	11.9	-
Depth to Bedrock (inch)	200	72	-

In developing Table 4-2, the Lab values were obtained from the lab-molded samples from Table 3-6 presented earlier. The value for the 1″ SFHMAC (Field) was set conservatively as that obtained in the composite backcalculation where the SF layer was combined with the RBL layer, and the value for the RBL modulus (Field) was assigned based on past experience with mixes using PG 64-22 binders.

As described in Chapter 2, the Design values were those used in the original FPS 19 design. In the next chapter of this report, the impact of using the "Field" moduli values on the recommended design thickness will be described. Table 4-3 is a comparison of the moduli values for the Conventional section.

Table 4-3. Comparison of Moduli Values Obtained for the Conventional Section.

Layer/Material	Average Modulus @ 77 °F (ksi)		
	Design	Field (FWD)	Lab
SMA + Type C	NA	826	630
Type B	NA	1157	800
RBL	NA	500	545
Lime-treated Subgrade	NA	57	-
Subgrade	NA	11.9	-
Depth to Bedrock (inch)	NA	72	-

For the Conventional section, no structural design was completed. The layer thicknesses were assigned to be the same as those used in the Superpave section. The moduli value from the lab is somewhat lower than that obtained from the field (FWD). However, with reference to Table 3-6 (Chapter 3), a lab moduli value of 1100 ksi was measured on field cores obtained from the Conventional section.

Thus far, FWD measurements (Table 4-1) on the Superpave section indicate that both the base (6 percent lime-treated subgrade) and subgrade are in very good condition. The lime-treated layer, as shown in Figure 4-9, was found from field coring to be a stiff non-moisture susceptible layer. The layer was cored with no deterioration, which is unusual for a lime-treated layer. Laboratory seismic density measurements of the base also indicated a base modulus value of 66.1 ksi, which does not differ significantly from the FWD backcalculated value in Table 4-1 (57 ksi).

Figure 4-9. Field-Extracted Cores with the Base at the Bottom.

From Figure 4-9, the fact that 90 percent of the base thickness (about 7″) was extracted in an intact state is a clear indication that the 6 percent lime-treated subgrade is still in very good condition.

FORENSIC EVALUATION OF FIELD-EXTRACTED CORES

Forensic evaluations of field-extracted cores indicated problems for the Superpave section after over a year of exposure to conventional traffic, before placement of the top ½″ HDSMA layer. As evident in Figures 4-10 and 4-11, a majority of cores from this section broke within the HMA layers and layer interfaces, exhibiting vertical segregation and severe debonding, particularly at the bottom of the compacted lifts of the rut-resistant 1″ SFHMAC layer. As shown in Figure 4-11b, Scullion (2006) has also reported similar problems on other Texas perpetual pavement projects. The probable cause is mix-design (low binder content and coarse aggregate gradations) and construction (compaction). The newer Conventional section (Figure 4-11a, middle core) with cores extracted just after 4 weeks of placement did not show any defects, suggesting good construction practices (i.e., improved rolling pattern and number of passes).

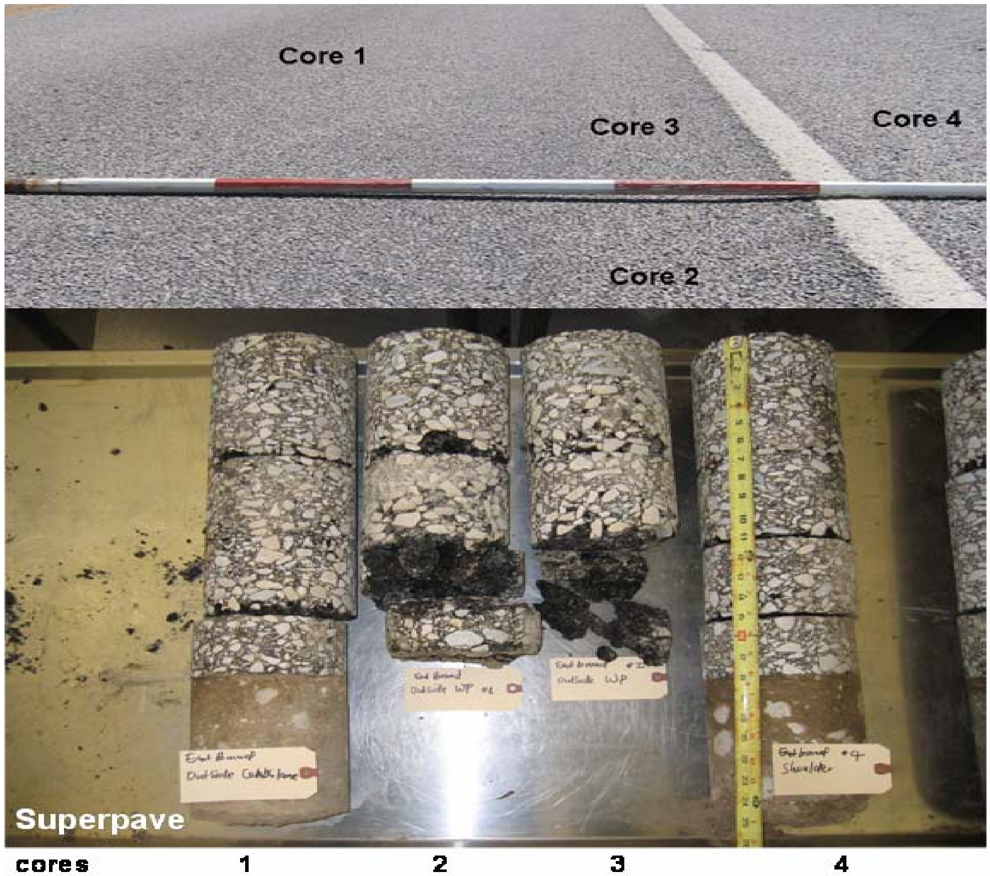
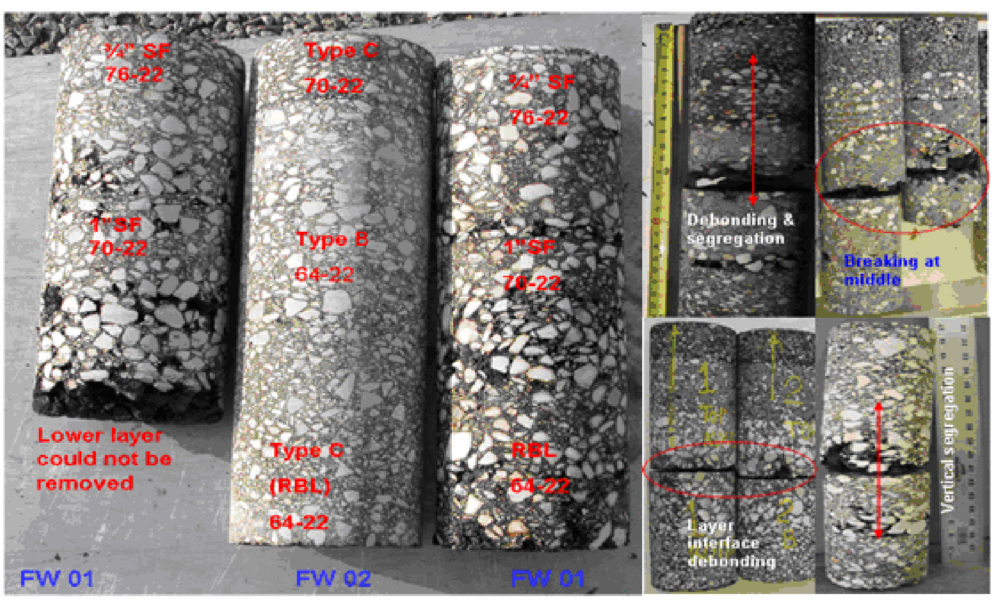


Figure 4-10. Superpave Cores.



(a) SH 114 Project

(b) IH 35 Laredo & Waco Projects

Figure 4-11. Forensic Evaluation of Field-Extracted Cores.

Note that because of the coarse aggregate gradation, the rut-resistant layers have proved to be difficult to compact and, as a result, may be prone to vertical segregation. They easily debond and may be susceptible to moisture damage in the long-term. In particular, the 1″ SFHMAC on the Superpave section had a relatively coarser aggregate gradation with lower fines, lower binder content, and was compacted using fewer rolling passes (Chapter 2) compared to the TxDOT Type B layer on the Conventional section. Consequently, the Superpave section exhibits vertical segregation and debonding problems as evident in Figures 4-10 and 4-11, which was not the case for the intact cores from the Conventional section.

However, Figure 4-10 also shows that the cores from the untrafficked shoulder (core 4) and in between the wheelpath (core 1) are in a considerably better condition than the cores from the trafficked wheelpath (cores 2 and 3). Apart from the interfaces, the untrafficked cores show very little evidence of debonding problems compared to the trafficked cores. This observation suggests that traffic may have also contributed to the debonding problems on the Superpave section.

SUMMARY

Significant observations and findings from field testing and forensic investigations are summarized as follows:

- Radar measurements indicated no evidence of major surface density and thickness variations or presence of moisture on the Conventional section. This lack of evidence of defects is indicative of good construction practices. However, density variations especially within the 1″ SFHMAC layer, were observed on the Superpave section, suggesting constructability problems.
- The base ($E \cong 57$ ksi) and subgrade ($E \cong 11.9$ ksi) appear to be in reasonably good condition based on FWD measurements. However, the FWD has been known to give higher than laboratory measured modulus values; therefore, these results should be interpreted with caution.
- The FWD calculated composite modulus values for the asphalt layers indicated that the Superpave section was relatively stiffer than the Conventional section. This observation supports the lab results reported in Chapter 3.

- Forensic evaluations of field-extracted cores, in particular the 1″ SFHMAC layer, showed severe vertical segregation and debonding with a high potential for moisture damage on the Superpave section. This problem was attributed to mix-design (coarse aggregate gradation and low fines) and compactability related problems. Cores from the newer Conventional section were found to be intact. Thus, both the mix-design and construction procedures for the 1″ SFHMAC mixtures need to be reviewed.
- It was difficult to process the FWD data collected on these sections because of the presence of the very shallow bedrock on the Superpave section. In addition, the pavement has five layers over the subgrade, whereas the current version of Modulus 6.0 will only permit the use to calculate moduli values for three layers plus the subgrade. To perform the analysis, Layers 1 and 2 and Layers 3 and 4 were combined.
- Based on the assumptions stated in this chapter, it appears that reasonable moduli values were found from the FWD analysis. The backcalculated values were substantially higher than those used in the thickness design process, but they were somewhat similar to those obtained in the Dynamic modulus tests reported in Chapter 3.

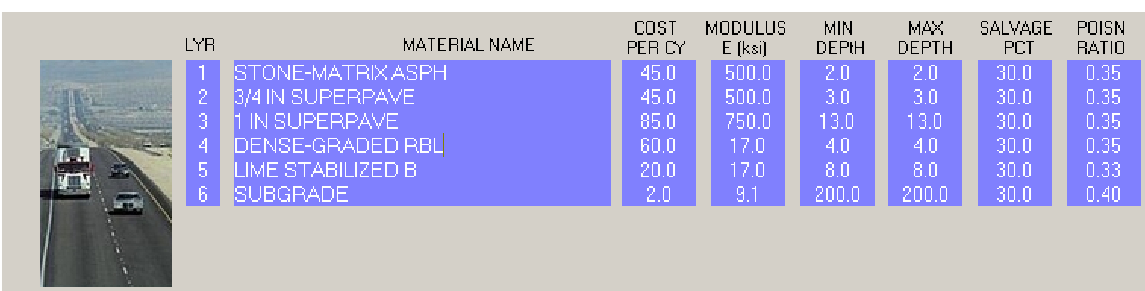
CHAPTER 5

COMPUTATIONAL SIMULATIONS, RESULTS, AND ANALYSES

Computational simulations and numerical analyses of the SH 114 pavement structures were accomplished with the FPS, PerRoad, VESYS5, and MEPDG software. This chapter presents the processing of the input data, structural analyses, performance predictions, and subsequent interpretation of the output data. As most of the materials data from the laboratory has only recently become available, the analysis presented in this chapter is viewed as preliminary. More analyses will be completed on these data sets and reported in later reports.

THE FPS SOFTWARE

The FPS software requires layer moduli as the main structural input. These moduli are typically obtained from analysis of FWD data collected in the field (often normalized to a reference temperature of 77 °F). Table 4-2 provided a comparison of the moduli values used in the initial thickness design with those obtained from the FWD and from lab testing. In an attempt to determine the impact of different moduli values on the required FPS pavement thickness, a prototype upgrade to the FPS 19 was used. This proposed program has all of the features of the existing FPS 19, but it can permit the use of more input layers. The original design assumptions used by the Fort Worth designers are shown in Figure 5-1.



LYR	MATERIAL NAME	COST PER CY	MODULUS E (ksi)	MIN DEPtH	MAX DEPTH	SALVAGE PCT	POISN RATIO
1	STONE-MATRIX ASPH	45.0	500.0	2.0	2.0	30.0	0.35
2	3/4 IN SUPERPAVE	45.0	500.0	3.0	3.0	30.0	0.35
3	1 IN SUPERPAVE	85.0	750.0	13.0	13.0	30.0	0.35
4	DENSE-GRADED RBL	60.0	17.0	4.0	4.0	30.0	0.35
5	LIME STABILIZED B	20.0	17.0	8.0	8.0	30.0	0.33
6	SUBGRADE	2.0	9.1	200.0	200.0	30.0	0.40

Figure 5-1. Original Design Moduli Input to the Proposed FPS 19 Upgrade.

The 20-year traffic estimate of 37.2 million 18-kip ESALs with 27 percent trucks was used in the analysis. The FPS pavement predictions for the designed structure are shown in Figure 5-2.

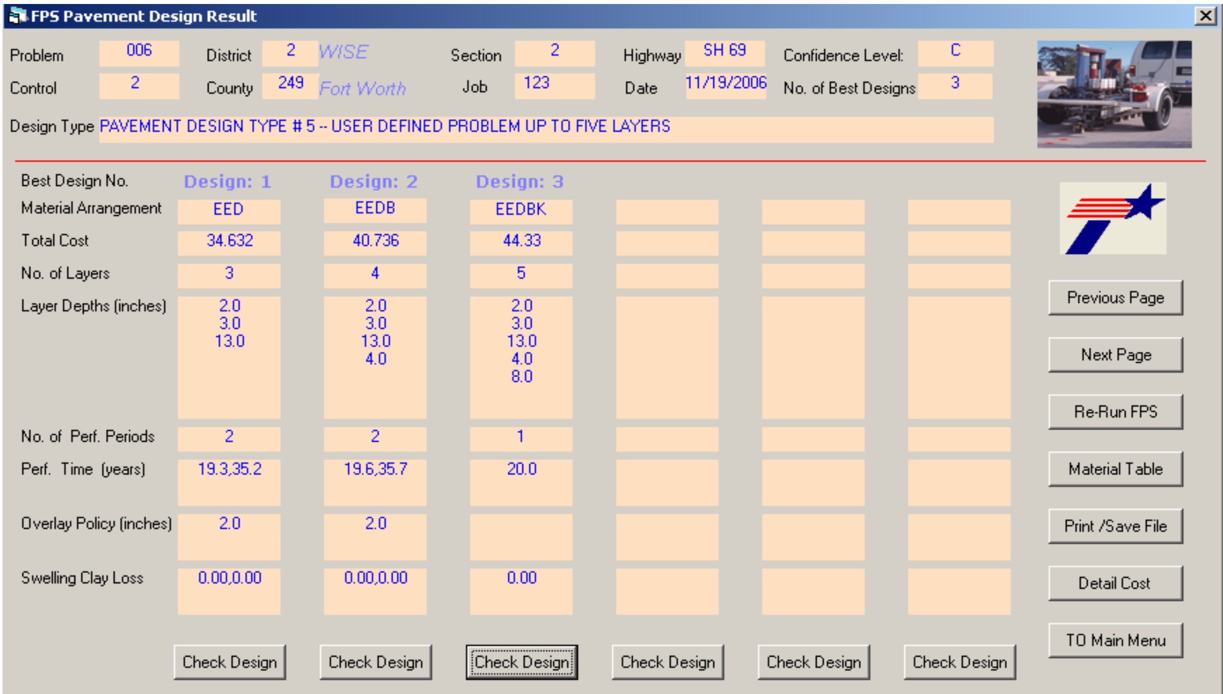


Figure 5-2. Pavement Life Predictions for the As-Designed Pavement Structure.

The as-designed pavement structure is shown in "Design 3" in Figure 5-2; this pavement is predicted to last 20 years without requiring an overlay. The next step in the FPS process is to perform a mechanistic check to ensure that the critical strains and predicted life are within the acceptable range. The results from the mechanistic check of the proposed structure are shown in Figure 5-3.

The computed tensile strain at the bottom of the RBL layer, which controls fatigue cracking (bottom-up), was computed to be 29.1 microstrain, well below the generally accepted limit for perpetual pavements of 70 microstrain. The vertical compressive strains at the top of the subgrade (which control subgrade rutting) was 79.4 microstrain, again well below the generally accepted criterion of 200 microstrain.

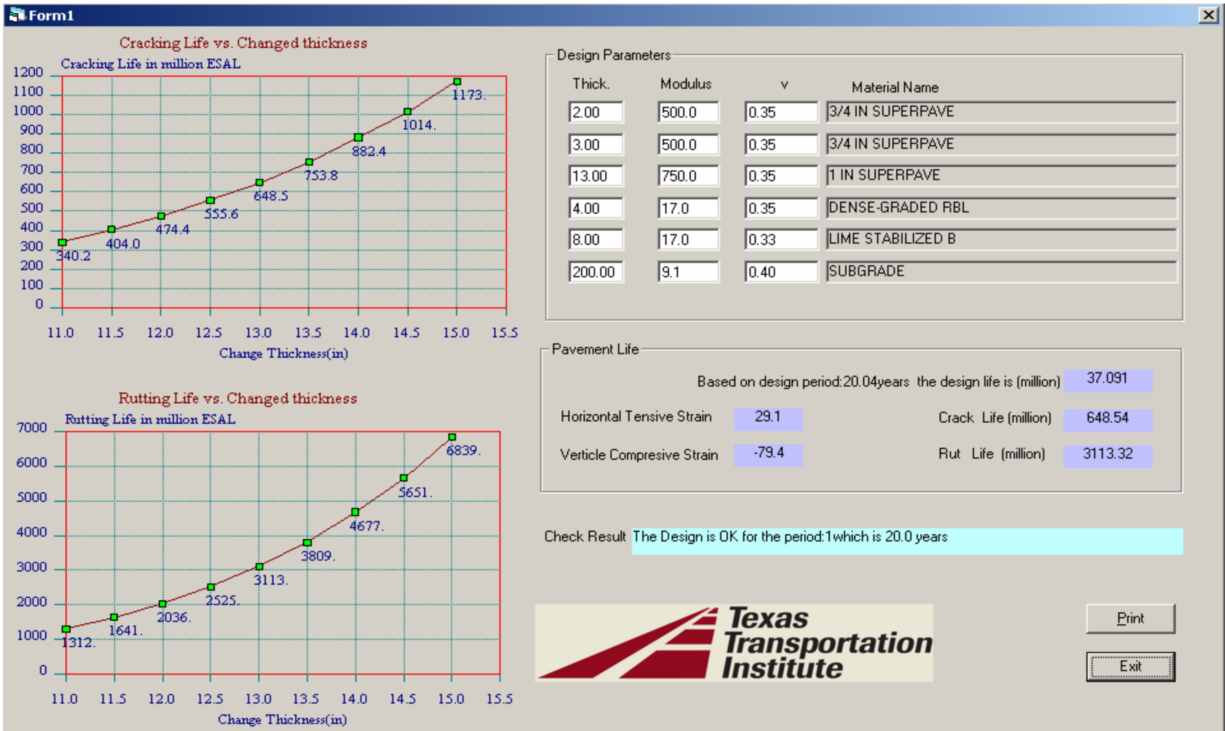
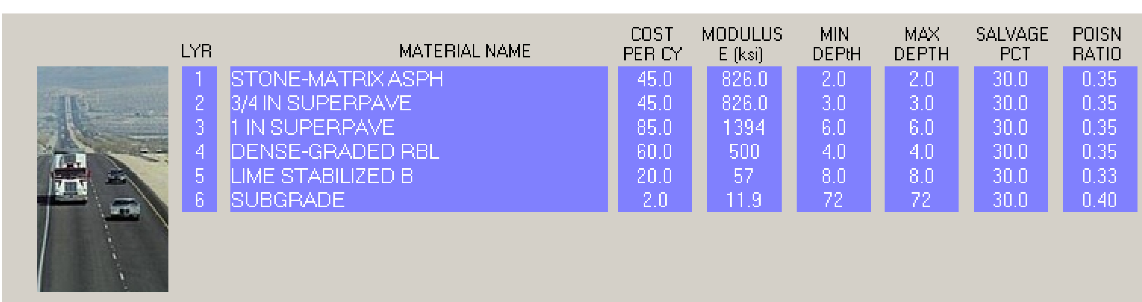


Figure 5-3. Mechanistic Check of the As-Designed Pavement Structure.

The next step in the analysis is to repeat the analysis with the moduli values obtained from FWD interpretation. This was done, and the only layer thickness allowed to vary was the thickness of the 1″ SFHMAC layer, which was originally designed as 13″ thick. Keeping all other input variables constant, the new thickness of the 1″ SFHMA layer was computed to be 6″, a reduction of 7″ over the original design. The FWD-based moduli values used in the revised design are shown in Figure 5-4, and the mechanistic design check results are shown in Figure 5-5. The new design was predicted to last 23.5 years before requiring an overlay.



LYR	MATERIAL NAME	COST PER CY	MODULUS E (ksi)	MIN DEPtH	MAX DEPTH	SALVAGE PCT	POISN RATIO
1	STONE-MATRIX ASPH	45.0	826.0	2.0	2.0	30.0	0.35
2	3/4 IN SUPERPAVE	45.0	826.0	3.0	3.0	30.0	0.35
3	1 IN SUPERPAVE	85.0	1394	6.0	6.0	30.0	0.35
4	DENSE-GRADED RBL	60.0	500	4.0	4.0	30.0	0.35
5	LIME STABILIZED B	20.0	57	8.0	8.0	30.0	0.33
6	SUBGRADE	2.0	11.9	72	72	30.0	0.40

Figure 5-4. Thickness Required from the Proposed FPS 19 Upgrade Using the Field Moduli Values.

In Figure 5.5, the critical strain levels are now 35.4 and 98.5 microstrain, still well below the generally accepted criteria (i.e., $\varepsilon_t \leq 70$ με and $\varepsilon_v \leq 200$ με).

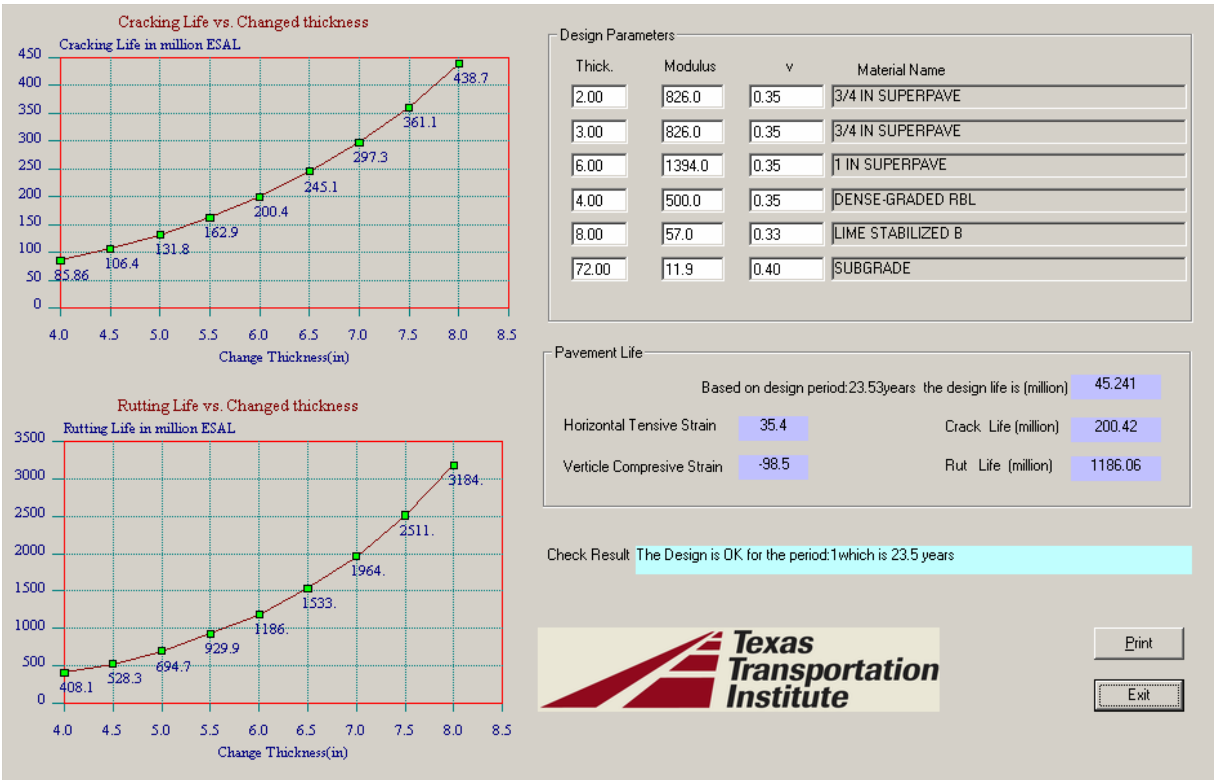


Figure 5.5. Mechanistic Check Results for the Structure Proposed Using Field Moduli Values.

As a final step, a third design was run using a moduli value of 750 ksi for the SMA and stone-filled HMA mixes. This value is the value recommended in the latest version of TxDOT's online design recommendations. Using these values and 500 ksi for the RBL layer, the required thickness of the 1″ SFHMA was computed to be 8″. This design also passed the mechanistic check (i.e., $\varepsilon_t \leq 70$ με and $\varepsilon_v \leq 200$ με).

THE PERROAD SOFTWARE

PerRoad is a simple M-E based numerical software for the structural thickness design and response (stress, strain, and deflection) checking of perpetual pavement structures. During execution, the PerRoad computes the worst case pavement response using a five layer linear-elastic program, WESLEA (Timm, 2004). If the PerRoad computed response (i.e., stresses, strains, and/or deflections) exceeds the specified mechanistic response thresholds, then the pavement design thicknesses and/or material properties need to be adjusted accordingly. The current M-E design procedure for perpetual pavements is based on two main response-limiting criteria, namely:

- horizontal tensile microstrain at the bottom of the lowest asphalt layer (ε_t): ≤ 70 με, and
- vertical compressive microstrain at the top of the subgrade layer (ε_v): ≤ 200 με.

The principle theory behind the PerRoad program is that fatigue- and rut-resistant designed full-depth asphalt pavements or perpetual pavements should have no fatigue cracking or rutting problems during their design life. For given traffic loading and environmental conditions, a pavement structure is theoretically considered a perpetual pavement if the above strain response thresholds are met; otherwise, the layer thicknesses and/or material properties need to be modified accordingly. In this project, the PerRoad Version 2.4 software was used for the structural analysis and evaluation to determine whether the SH 114 pavement structure met the above prescribed M-E response criteria.

PerRoad Input Data

Like any other pavement design and analysis software, the required input data for the PerRoad program include the pavement structure, environment, material properties, and traffic loading. These input data are discussed in the following sections.

Pavement Structures

Since the current PerRoad software is limited to only a five-layered pavement structure, the seven-layered SH 114 pavement structure was reduced to a five-layered pavement structure as shown in Table 5-1.

Table 5-1. PerRoad Reduced Five-Layered Pavement Structures.

Original Layer	⇒	Reduced Pavement Structure		
		Layer	**Thickness**	**Material**
Layer 1 Layer 2 Layer 3	→	Layer 1	18″ (composite modulus)	1) ½″ HDSMA + ¾″ SFHMAC + 1″ SFHMAC or 2) ½″ HDSMA + Type C + Type B
Layer 4	→	Layer 2	4″	Fatigue-resistant (RBL) = ε_t @ bottom
Layer 5	→	Layer 3	8″	Base (stabilized subgrade)
Subgrade	→	Layer 4	≅200″	Subgrade = ε_v on top
		Subgrade	∞	Subgrade

With the PerRoad software, the main layers of structural interest for M-E strain response analysis are the RBL (at the bottom) and subgrade (on top), respectively. To obtain reasonable results, it was observed in this project that the subgrade had to be divided into two sub-layers of similar modulus values, with the top portion assigned a thickness of 200″ and the rest, infinite (Table 5-1). Provided it is equal to or greater than 1″, the thickness of the top portion of the subgrade sub-layer has no significant bearing on the computed strains (i.e., similar results were obtained for 1″, 72″, and 200″, respectively). So 200″ was used through out the analysis.

As shown in Table 5-1, all the top asphalt layers above the RBL were combined into one composite asphalt layer (denoted as Layer 1) with a total thickness of 18″ and one composite modulus value based on the average sum of the original individual layers. This use of a total of only five layers (including the subgrade) is the limitation with the current version of this program. Since the critical layer in terms of thickness is the rut-resistant layer, the thickness of this composite layer should be varied while the rest of the structure remains fixed. This means that the RBL should not be combined with other asphalt layers, since it is where the pavement response of interests will be computed.

Environment

In the current PerRoad software, the environment is characterized in terms of yearly seasonal subdivisions as a function of temperature variations (Timm, 2004). Each season is further categorized in terms of the number of weeks per year, the total being 52 for the entire year. In this project, the Fort Worth environment was subdivided into five seasons: summer, fall, winter, spring1, and spring2, respectively. Table 5-2 provides a list of these seasons together with the associated duration (in weeks) and pavement surface and subsurface temperatures.

As will be described, these temperatures were generated using the Enhanced Integrated Climatic Model (EICM). These seasonal subdivisions are a close representation of the Texas environment.

Table 5-2. Fort Worth Seasonal Subdivisions.

Season	Summer	Fall	Winter	Spring1	Spring2
Duration, weeks	10 (19.2%)	16 (30.8%)	2 (3.8%)	18 (34.6%)	6 (11.5%)
Representative mean pavement surface temperature (°F)	115	87	40	77	55
Mean temperature at 11.5″ depth (Rut-resistant layer) (°F)	103	85	48	68	58
Mean temperature at 20″ depth (RBL layer) (°F)	100	84	53	68	60

The MEPDG's (EICM) software was used to generate temperature profiles at various depths within the SH 114 pavement structures including the surface (AASHTO, 2006). Figure 5-6 shows the yearly pavement surface temperature variation based on the Fort Worth Alliance Airport. Figure 5-7 is a plot of the temperature-frequency distribution as a function of the pavement depth and seasonal subdivision.

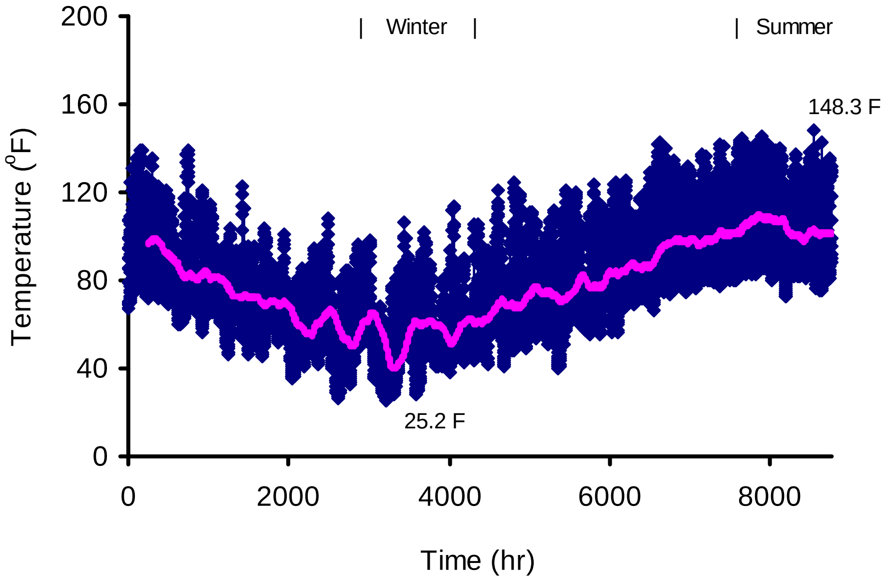


Figure 5-6. Yearly Pavement Surface Temperature Variations.

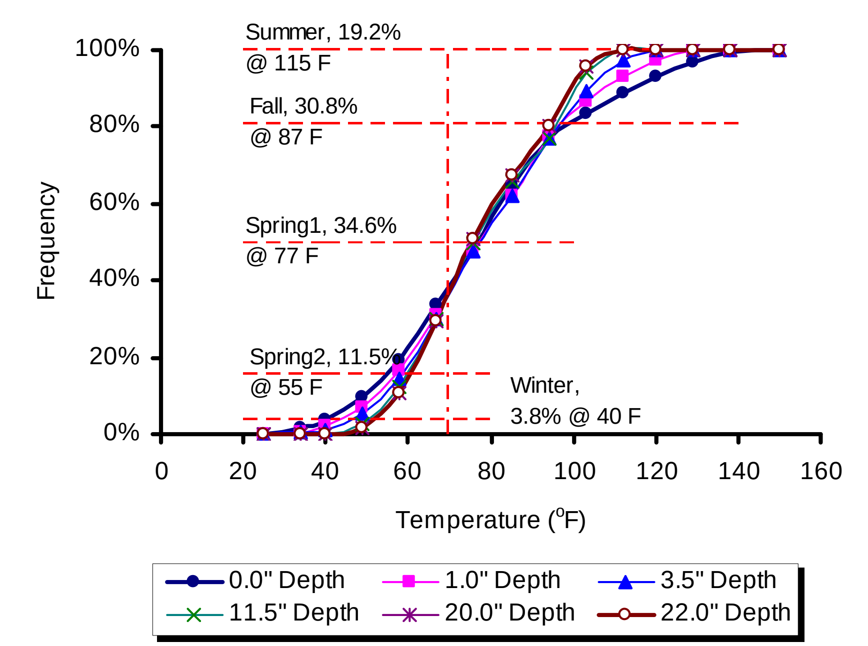


Figure 5-7. Temperature-Frequency Distribution and Seasonal Subdivisions.

Material Properties

The material properties required for PerRoad analysis are the elastic modulus (E) and the Poisson's ratio (ν). In the PerRoad software, the year is subdivided into seasons, and the elastic layer modulus must be specified for each season. For this analysis and considering the temperature-sensitivity of the asphalt mixtures, the modulus values of the asphalt layers were input and varied as a function of seasonal temperature variations. These modulus values were determined from laboratory DM testing at 10 Hz and various test temperatures discussed in Chapter 3 (see also Appendix B). Figure 5-8 shows an example of the plot of the various asphalt layer modulus values as a function of temperature at 10 Hz DM testing based on lab-molded specimens. Note that Figure 5-8 is an extract from the tabulated DM results in Appendix B.

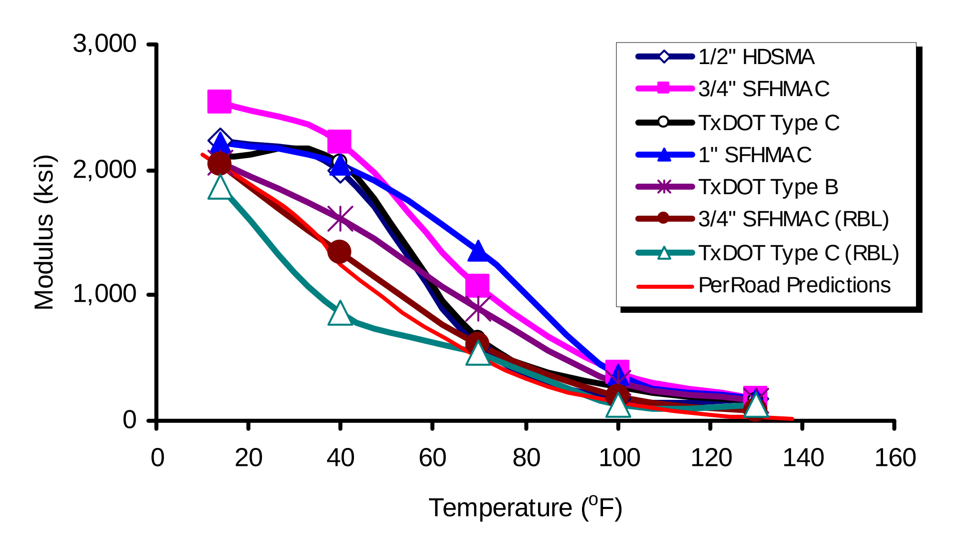


Figure 5-8. Dynamic Modulus Results at 10 Hz (Lab Specimens).

For each season (summer, fall, winter, spring1, and spring2), individual modulus values (i.e., from Figure 5-8) were input into the PerRoad software for each asphalt layer. In considering that temperature varies with pavement depth, the modulus values of each asphalt layer and season were determined at different reference temperatures using Figure 5-7. Notice also the typical visco-elastic nature of asphalt mixtures in Figure 5-8 (i.e., the modulus is exhibiting a decreasing trend with an increase in temperature as expected).

Note, however, that if seasonal modulus values are unavailable, the PerRoad program does provide an option to computationally predict the modulus values from an asphalt mixture-temperature model in-built in the program (Timm, 2004). However, utilization of this model requires knowledge of the reference modulus value and average pavement surface temperatures for each season as well as the material and environmentally dependent coefficients, Q_i. As utilized in this project, the researchers recommend the former approach, unless otherwise local Q_i coefficients for typical Texas materials and environmental conditions are established. Another conservative approach would be to assume the modulus values based on past experience.

Unbound materials are sensitive to moisture variations, and the modulus values will generally vary seasonally, typically lowest during the wet season and highest under dry and/or frozen conditions. In this project, the modulus values for the base and subgrade materials were not varied. Instead, the initial design values discussed in Chapter 2 (i.e., 17 ksi for the treated subgrade (base) and 9.1 ksi untreated subgrade) were used for all the seasons based on the lab design analysis. The backcalculated moduli (FWD) values of 57 ksi (base – treated subgrade) and 11.9 ksi for the subgrade were used based on the field analysis. Nonetheless, a sensitivity analysis of this effect on the pavement structural response is ongoing, and findings will be presented in future reports. An example of the PerRoad input data for the pavement structure and material properties is shown in Appendix D.

Traffic Loading

In the PerRoad software, the traffic may be entered as a specific axle (i.e., single axle dual wheels with 16 to 18 kips load) or a spectrum of axle load distribution. The later approach was used in this project, and the tandum loading configuration was utilized. For conservative purposes, a rural interstate traffic distribution as in-built in the PerRoad software was assumed (Timm, 2004). The distributive number of axles associated with each loading configuration and axle weight is shown in Appendix D. The total daily axles in the design lane were taken as 4914 (27.3 percent of 18,000 ADT) at annual axle growth rate of 4.5 percent. In the PerRoad analysis, all tires are assumed to be inflated to 100 psi pressure. If the traffic data are unavailable, the default load spectra derived from the literature and the Long-Term Pavement Performance (LTPP) database that comes with the PerRoad software may be used (Timm, 2004).

PerRoad Results and Output Data

During analysis, the PerRoad software runs through all the possible input seasons, material properties, and traffic-loading conditions. The final output results represent the worst-case scenario with maximum responses in terms of stress, strain, and/or deflections. The PerRoad results are summarized in Table 5-3 based on an M-E deterministic analysis. An example of the PerRoad output screen is also included in Appendix D.

Table 5-3. PerRoad Results.

Performance Criteria	Threshold	Lab		Plant Mix/Cores	
		Superpave	Conventional	Superpave	Conventional
Vertical surface deflection	≤ 20 milli-inch	23.19	23.47	18.72	19.20
ε_t @ bottom of RBL	≤ 70 με	60.02 με	59.96 με	50.94 με	56.85 με
ε_v on top of Subgrade	≤ 200 με	145.80 με	148.82 με	135.73 με	144.92 με

Table 5-3 shows that both the pavement structures met the M-E strain response criteria (ε_t and ε_v) prescribed for perpetual pavements, indicating that no major rutting or bottom-up fatigue cracking problems would theoretically be expected during their service lives. If these pavement structures had however failed the M-E strain response criteria, the first probable recommendation would have been to increase the thickness of the rut-resistant layers (i.e., the composite Layer 1, see Table 5-1). The second option would have been to modify the material properties (i.e., the modulus values), which would literally mean improving the mix-design characteristics especially for the lab-molded specimens, among other measures.

Compared to the FPS analyses discussed previously, the strain magnitudes (ε_t and ε_v) in Table 5-3 are considerably higher. This difference is partly due to the fact that the FPS does not take into account the seasonal temperature variations and its effects on the HMA layer moduli. Instead, FWD backcalculated moduli values at a fixed reference temperature of 77 °F are often used for each HMA layer. The PerRoad, on the other hand, takes into account environmental/climatic effects and allows for seasonal variations in the layer moduli, with the computed strain responses representing the worst-case scenario.

Note in Table 5-3 that the vertical deflections predicted based on the lab-molded specimens are slightly over the specified 20 milli-inch threshold. This is attributed to the generally lower moduli values of the lab-molded specimens (Chapter 3 and 4). However, these deflections are considerably low if a 50 or 75 milli-inch threshold consistent with the MEPDG analyses discussed in the subsequent sections is used. In general, the PerRoad vertical deflection predictions (Table 5-3) represent the worst-case scenario, in particular the high summer temperatures associated with the lowest HMA moduli values. Nonetheless, vertical deflection is not a criterion for meeting the current M-E design requirement for perpetual pavements.

Based on the PerRoad probabilistic analyses and the associated assumptions made, a structural thickness design check indicated that a 9″ thick rut-resistant layer was sufficient for the 30-year design period. This analysis is shown in Figure 5-9 and equates to a total HMA layer thickness of 18″. A total HMA layer thickness of 20″ (i.e., 11″ thick of the rut-resistant layer) indicated a service life of up to 53 years, which falls within the 50-year perpetual pavement concept.

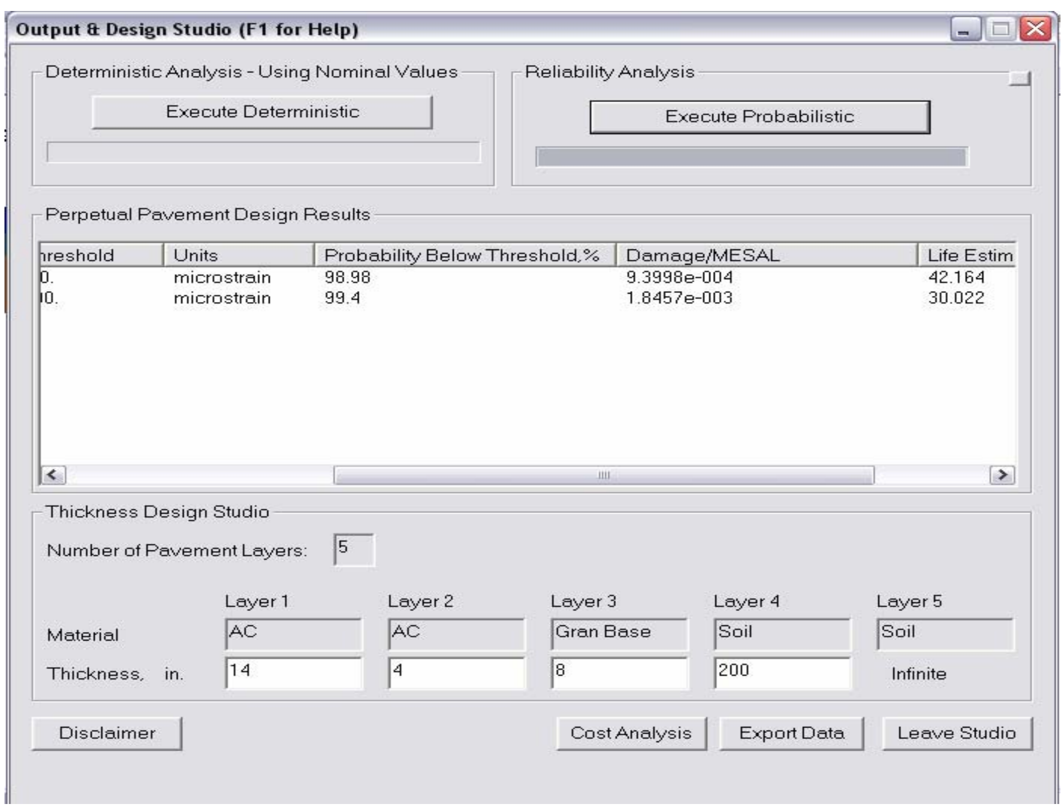


Figure 5-9. PerRoad Analysis.

THE VESYS SOFTWARE

VESYS5 is a probabilistic and mechanistic analysis computer program for flexible pavement design and performance prediction. It is based on the elastic model of layered homogeneous material in half-infinite space with viscoelastic-plastic theory. Full details of the VESYS5 can be found elsewhere (Zhou et al., 2005). It predicts the asphalt pavement performance (rutting, fatigue cracking, present serviceability index [PSI], etc.) with time.

VESYS Input Data

VESYS5 is a multi-layered linear elastic program with the capacity of up to seven layers: three asphalt layers (one surfacing, one rut-resistant, and one fatigue-resistant), two base layers, two subbase layers, and the subgrade. To accommodate the three asphalt layer requirements, the two top asphalt layers were combined into one composite asphalt layer with a total thickness of 5″ and one composite modulus value based on the average sum of the original individual layers. Table 5-4 shows the reduced pavement structures for VESYS5 analysis.

Table 5-4. VESYS5 Reduced Pavement Structures.

Original Layer	⇒	Reduced Pavement Structure: Layer	Thickness	Material
Layer 1 Layer 2	→	Layer 1	5″ (composite modulus)	1) ½″ HDSMA + ¾″ SFHMAC 2) ½″ HDSMA + Type C
Layer 3	→	Layer 2	13″	Rut-resistant
Layer 4	→	Layer 3	4″	Fatigue-resistant (RBL)
Layer 5	→	Layer 4	8″	Base (stabilized subgrade)
Subgrade			∞	Subgrade

Environment and Climate

In terms of environment, the VESYS5 allows seasonal subdivisions to account for both temperature and moisture variations. Like PerRoad, five seasonal subdivisions with pavement surface temperature profiles based on the EICM model were utilized. These seasonal subdivisions are shown in Figure 5-10.

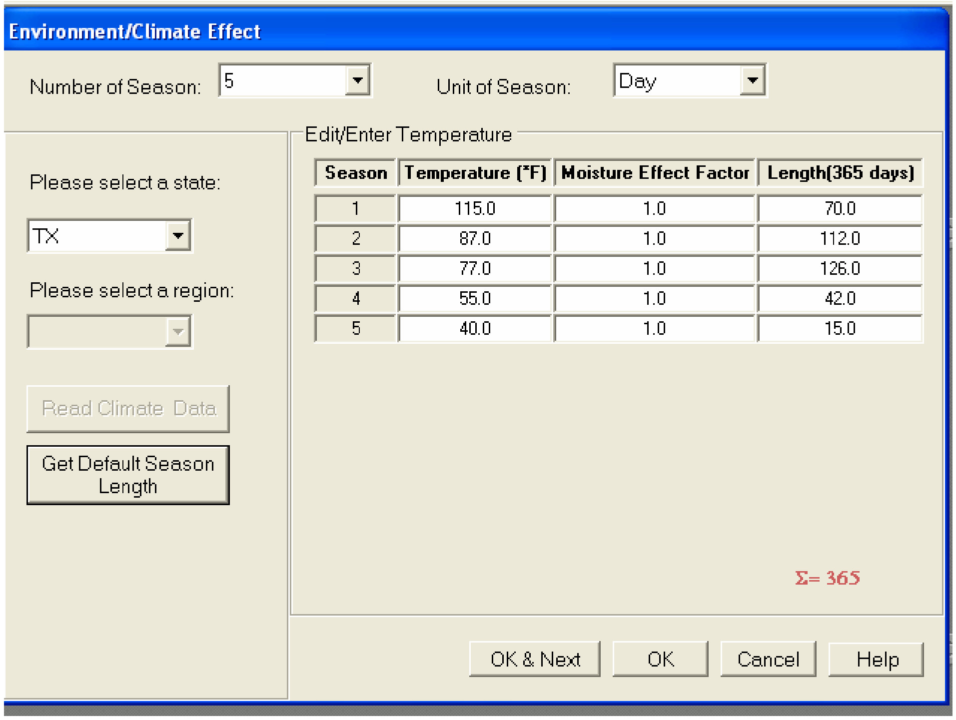


Figure 5-10. Environmental/Climatic Effect for VESYS5 Analysis.

In the analysis, the base and subgrade modulus values were fixed at 17 ksi and 9.1 ksi (initial design values). A moisture effect factor of 1.0 was used for these modulus values, which remained fixed for each season. The mean pavement surface temperatures were varied seasonally, as shown in Figure 5-10. The total seasonal duration should sum to a year (or 365 days), as shown in Figure 5-10.

Material Properties

The required input material properties are the elastic modulus, Poisson's ratio, and the rutting parameters, α and μ. The elastic moduli were obtained from the DM test (10 Hz) (Chapter 3) based on lab-molded specimens and were varied seasonally as a function of temperature. The rutting parameters were determined from the RLPD test (Chapter 3), also based on lab-molded specimens. However, since testing was conducted only at two temperatures (77 and 104 °F), the rutting parameters (α and μ) at other temperatures were interpolated/extrapolated in reference to typical values recommended by Zhou and Scullion (2004). Figure 5-11 shows an example of the modulus input screen for the composite Layer 1.

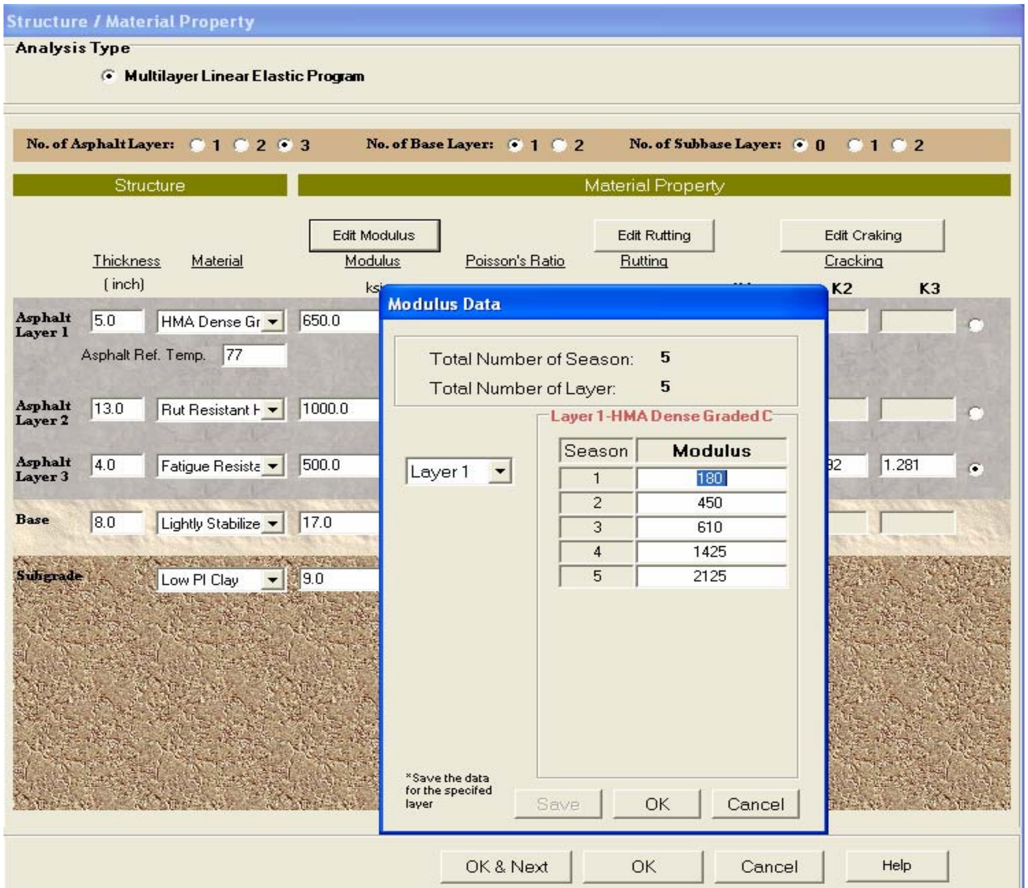


Figure 5-11. Input Screen for Material Properties (VESYS5).

The reference temperature for the analysis was 77 °F. Typical cracking parameters k_i consistent with recommendations by Zhou et al. (2005) were used. The base and subgrade moduli were fixed at 17 ksi and 9.1 ksi, respectively. Under typical design analyses, the thickness of the rut-resistant layer would be varied until acceptable performance is predicted. The analysis period was 20 years, the maximum the software can handle.

Traffic

A daily traffic repetition of 4914 (27.3 percent of 18,000 ADT) with a growth rate of 4.5 percent over a 20-year design period was used. The axle loading configuration was tandem with a 100 psi tire pressure. According to Zhou and Scullion (2004), the “Axle Weight/2” in the VESYS5 software is entered as 8.5 kip (i.e., 34/2/2 = 8.5 kip) for a 34 kip tandem load. An example of the traffic input screen together with the input parameters used for this analysis are shown in Appendix D.

VESYS Output Data and Results

Table 5-5 is a summary of the rutting results. Graphical plots as a function of time are included in Appendix D.

Table 5-5. VESYS5 Rutting Results after 20 Years.

Layer	Rutting (Inches)	
	Superpave	**Conventional**
HMA surface layers	0.20	0.23
Rut-resistant	0.16	0.28
Fatigue-resistant (RBL)	0.46	0.48
Base (lime-treated subgrade)	0.02	0.02
Subgrade	0.14	0.14
Total rutting	0.98	1.15

While the predicted total rutting is around 1″, about 42 percent of the total rutting is contributed by the flexible RBLs on both sections. This was attributed to the fact that the VESYS5 software uses the same reference temperature for all the asphalt layers in the pavement structure. Another contributing factor could be the input data, in particular the RLPD rutting parameters (α and μ), which were obtained only at two test temperatures. These factors will be investigated in the ongoing research work. A similar amount of rutting is evident in the base and subgrade, probably due to similar material properties. On a comparative basis, the Conventional section appears to be more rut susceptible than the Superpave section, similar to the HWTT and RLPD results discussed previously in Chapter 3.

No fatigue cracking was predicted in either section. Graphical results are shown in Appendix D. This observation is in agreement with the OT, FPS, and PerRoad results discussed previously. However, both sections show potential for surface roughness problems. In fact, the PSI had dropped to about 1.8 for the Superpave section and 1.2 for the Conventional section by the 20th year of service life. The PSI plots are shown in Figures 5-12 and 5-13, respectively, and suggest that at least one surface treatment would be required in the first 20 years of service to restore the pavement surface quality. In practice, this means that at least one overlay should be placed before the 20th year is reached.

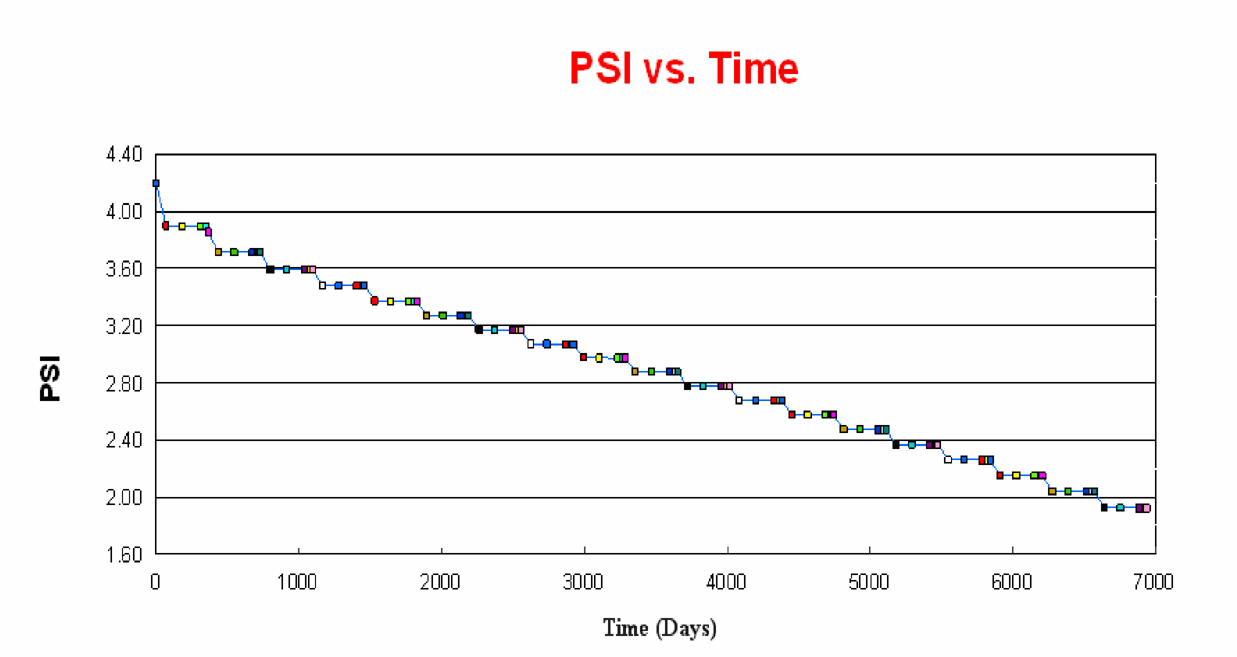


Figure 5-12. PSI Plot for the Superpave Section.

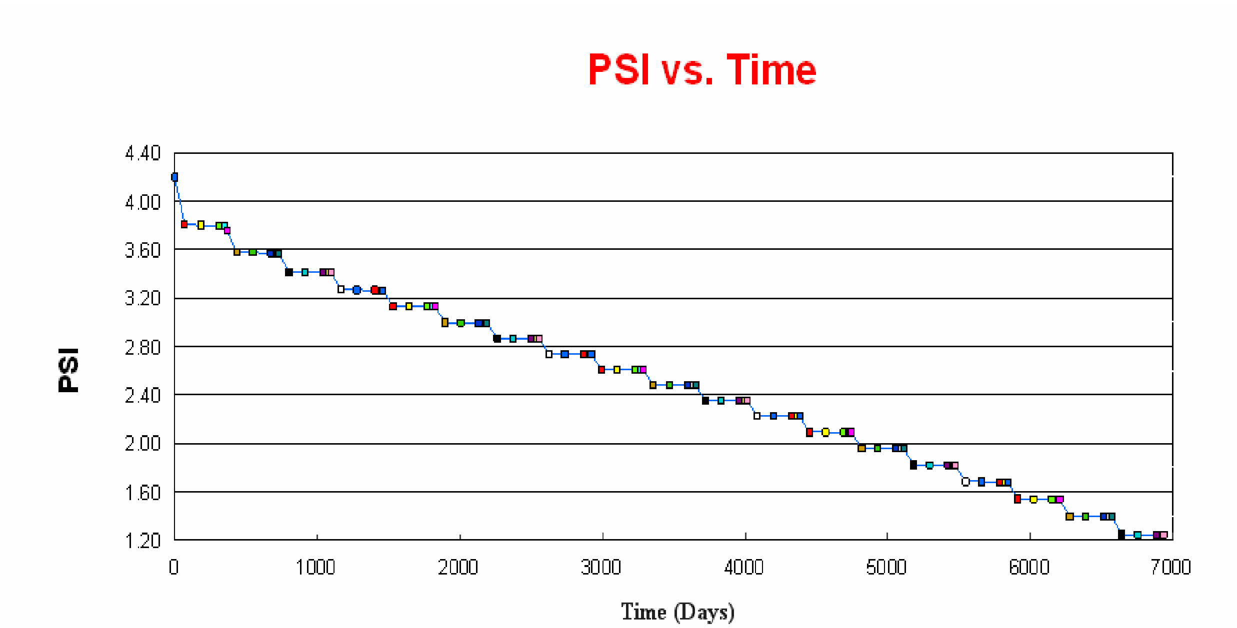


Figure 5-13. PSI Plot for the Conventional Section.

Note that although reasonable results were obtained, this is only a preliminary analysis; more work remains to be done on the VESYS5 software, including validation and sensitivity analyses. The current VESYS5 program is very sensitive to the α and μ rutting parameters (RLPD test), and its applicability for perpetual pavements continues to be investigated.

THE MEPDG SOFTWARE

Numerical performance predictions in terms of cracking, rutting, and surface roughness (international roughness index [IRI]) were accomplished with the MEPDG software Version 0.910. Details of the MEPDG software can be found elsewhere (AASHTO, 2006). The MEPDG is an M-E based numerical software for pavement structural design analysis and performance prediction, within a given service period (AASHTO, 2006). The MEPDG adopts two major aspects of M-E based material characterization: pavement response properties and major distress/transfer functions. Pavement response properties are required to predict states of stress, strain, and deformation within the pavement structure when subjected to external wheel loads and thermal stresses. These properties for assumed elastic material behavior are the elastic modulus (E) and Poisson's ratio (ν). The major MEPDG distress/transfer functions for HMAC pavements are load-related fatigue fracture, permanent deformation, rutting, and thermal cracking.

MEPDG Input Data

In terms of the input data, the MEPDG utilizes a hierarchical system for both material characterization and analysis (AASHTO, 2006). This system has three input levels. Level 1 represents a design philosophy of the highest achievable reliability, and Levels 2 and 3 have successively lower reliability, respectively. In addition to the typical volumetrics, Level 1 input requires laboratory measured binder and asphalt mixture properties such as the shear and dynamic modulus, respectively; whereas Level 3 input requires only the PG binder grade and aggregate gradation characteristics. Level 2 utilizes measured binder shear modulus properties and aggregate gradation characteristics.

The binder complex shear modulus is determined from DSR testing of a rolling thin film-oven short-term aged binder sample, often measured at 10 rad/s, and includes the phase angle and various representative test temperatures as the MEPDG input data (Chapter 3). These binder data are used in the MEPDG software to predict asphalt mixture aging during analysis (Chapter 3). For the asphalt mixtures, the actual DM input data for MEPDG Level 1 analysis are the test temperatures, the test loading frequencies, and the respective measured |E*| values.

Summarized, the basic MEPDG input data include the general project information, traffic, climate (environment), pavement structure (structural design and material properties), distress failure limits, pavement design life, and a design reliability level (AASHTO, 2006).

MEPDG Analysis and Output Data

During execution, the MEPDG software predicts performance at any age of the pavement for a given pavement structure and traffic level under a particular environmental location (AASHTO, 2006). The MEPDG predicted performance is then matched against predefined performance criteria at a given reliability level and design life. If the predefined performance criteria or analysis parameters are not met, the following options are feasible:

- reviewing/modifying the input data including the pavement structure (thicknesses), materials, traffic, environment, reliability level, pavement design life, and analysis parameters (distress failure limits); or
- changing the HMAC mix-design and/or the material types.

In this project, the MEPDG software Version 0.910 was used to predict performance of the SH 114 pavement structures at a 95 percent reliability level. This new MEPDG Version 0.910 has the capability to handle more than 5 layers over a 50-year analysis period, which is advantageous for analyzing perpetual pavements.

An average annual daily traffic of 18,000 with a traffic growth rate of 4.5 percent (compound growth) was utilized. The truck composition was taken as 27.3 percent in the design direction and 100 percent in the design lane. The Level 1 MEPDG input was used for characterizing the material properties. Environmental characterization was based on climatic data from the Alliance Airport in Fort Worth, Texas. Typical distress failure criteria consistent with TxDOT tolerable limits were used (TxDOT, 2003).

MEPDG (Level 1) Results

The MEPDG Level 1 results are summarized in Table 5-6 and include both the distress and reliability predictions. Detailed results of the MEPDG permanent deformation and rutting analyses are included in Appendix D.

Table 5-6. MEPDG Level 1 Distress Analysis.

Performance Criteria		Distress Target	Reliability Target	Distress Predicted	
				Lab	Plant Mix/Core
FW 01: Superpave Section					
1	Terminal IRI (in/mi)	172	95	210.8 (80%)	225.4 (86%)
2	AC Surface Down Cracking (Long. Cracking) (ft/500)	1000	95	3.4 (4%)	56 (22%)
3	AC Bottom-Up Cracking (Alligator Cracking) (%)	25	95	0 (0%)	0.2 (0%)
4	AC Thermal Fracture (Transverse Cracking) (ft/mi)	1000	95	1 (6%)	1 (6%
5	Permanent Deformation (AC Only) (in)	0.50	95	0.42 (11%)	0.35 (36%)
6	Permanent Deformation (Total Pavement) (in)	0.75	95	0.59 (10%)	0.61 (18.2%)
FW 01: Conventional Section					
1	Terminal IRI (in/mi)	172	95	215.2 (82%)	222 (85%)
2	AC Surface-Down Cracking (Long. Cracking) (ft/500)	1000	95	7.7 (9%)	41.6 (18%)
3	AC Bottom-Up Cracking (Alligator Cracking) (%)	25	95	0 (0%)	0.1 (0%)
4	AC Thermal Fracture (Transverse Cracking) (ft/mi)	1000	95	1 (6%)	1 (6%)
5	Permanent Deformation (AC Only) (in)	0.50	95	0.51 (53%)	0.39 (38%)
6	Permanent Deformation (Total Pavement) (in)	0.75	95	0.69 (13%)	0.56 (10.1%)

in = inches; mi = mile; ft = feet; AC = asphalt concrete;
Long. Cracking = longitudinal cracking;
The numbers in parentheses, i.e., (0%) or (80%) represent reliability predictions.

In Table 5-6, the reliability predictions (in parentheses) represent the probability percentage of the pavement not performing to expectations (i.e., the percentage chance of the distress exceeding the target threshold). For instance, there is 0 percent probability that both pavement sections will have major bottom-up fatigue-related problems during their service life. Or in other words, there is 0 percent probability that bottom-up fatigue cracking will exceed the

25 percent threshold at 95 percent design reliability. The 95 percent design reliability implies that 5 percent chance of failure or exceeding the target (distress) threshold is allowable.

With the exception of IRI, no major distresses were predicted on both pavement structures. In fact, Table 5-6 shows no evidence of bottom-up fatigue cracking (0 percent probability), which supports the OT results discussed in Chapter 3. Also, the total pavement rutting is acceptably within the 0.75″ design limit. However, the asphalt layers on the Conventional section, predominantly in the top HMA layers (Appendix D), appear to exhibit a potential for permanent deformation. As shown in Table 5-6, the predicted permanent deformation (0.51″) slightly exceeds the 0.5″ design limit with about a 53 percent chance of occurrence. On the Superpave section, Appendix D also shows that Layer 2 (¾″ SFHMAC) accumulated more permanent deformation based on the core specimens compared to the corresponding lab specimens, which incidentally concurs with the HWTT results (Chapter 3) and field visual observations (Chapter 4). This comparatively high deformation was attributed to the core specimens' lower modulus values (Chapter 3), possibly arising from the higher binder content discussed in Chapter 2. Evidently, these results emphasize the fact that the top layers should be equally stiff to minimize surface rutting. On both sections, however, no permanent deformation was predicted in the 13″ thick rut-resistant layers. Also, unlike with the VESYS5 software, no significant permanent deformation was predicted in the RBLs, presumably due to load shielding by the upper HMA layers (18″ total thickness) and better environmental (temperature) modeling by the MEPDG software.

As seen in Table 5-6, both sections failed the IRI distress criterion with over 80 percent probability of the predicted distress exceeding the 172 in/mi threshold. In fact, reliability predictions indicate that the IRI will reach critical levels approximately in the 23rd year of service life, suggesting that at least one overlay should be done before this time. In practice, this means that at least one overlay would be required within the first 23 years of service. Figure 5-14 shows these IRI results graphically with the Superpave and Conventional sections almost overlapping each other for both reliability and IRI predictions.

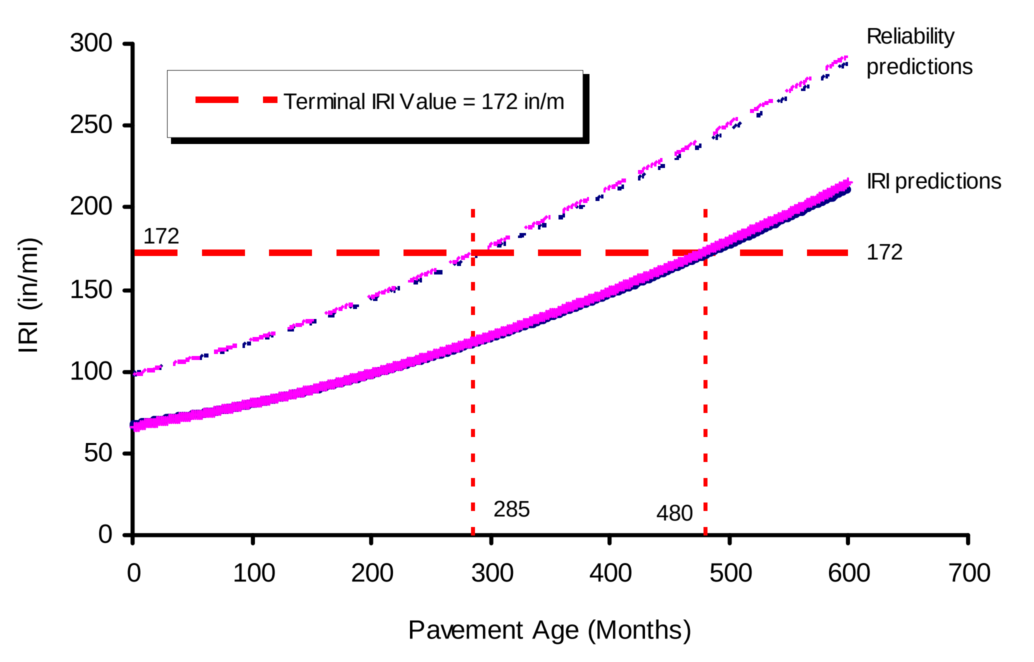


Figure 5-14. MEPDG Level 1 IRI Predictions.

SUMMARY

The findings from this chapter are summarized as follows:

- Both pavement sections met the perpetual pavement criteria based on the FPS and PerRoad analysis. The computed tensile and compressive microstrain at the bottom of both of the RBLs and on top of the subgrade were less than 70 and 200 $\mu\varepsilon$, respectively.
- Based on the VESYS5 and MEPDG Level 1 analyses, both pavement sections indicated no evidence of serious bottom-up fatigue cracking during their service life. Although the total pavement rutting was within acceptable levels and in fact no permanent deformation was predicted in the rut-resistant layers, the top asphalt layers in particular on the Conventional section (with softer PG binder grades) exhibited potential for permanent deformation.
- While no major significant distress was predicted, and bearing in mind that actual field performance may in fact be different, the VESYS5 and MEPDG pavement surface roughness analyses indicated that at least one overlay will be required

within the first 20 years of service. This finding was also essentially corroborated by the FPS analysis, which indicated an overlay within the first 23 years of service.

- With respect to the material properties, these researchers recommend using laboratory and/or field determined modulus values. If unavailable, the modulus values can be estimated from the literature and/or experience.
- While perpetual pavements are designed to be rut and fatigue resistant, the VESYS5 and MEPDG analyses suggest that proper account should also be taken of other potential distresses such as surface roughness, longitudinal (surface down) cracking, and transverse (thermal) cracking through, among other measures, appropriate materials selection and mix-designs.
- The proposed upgrades to the FPS 19W software offer greater potential for both thickness design and structural analysis of perpetual pavements. The FPS upgrade will incorporate a mechanistic check, triaxial check, stress-strain response analysis, and pavement life prediction (rutting and cracking). PerRoad is a mechanistic software for the thickness design and structural analysis of perpetual pavements. It incorporates a mechanistic response check including strain analysis and pavement life prediction (rutting and fatigue). It also incorporates a more elaborate environmental/climatic model with respect to the material properties, but it is limited to evaluating a total of five layers (including the subgrade) only.
- The MEPDG software can analyze pavement structures over a period of 50 years and can accommodate multiple layers. This software has the potential for modeling and analyzing perpetual pavements (including performance prediction), where the intermediate/lower layers typically have expected service lives in excess of 50 years. The MEPDG also incorporates more comprehensive traffic and environmental/climatic models. However, investigation of the applicability of both the MEPDG and VESYS5 for analyzing perpetual pavements through sensitivity analyses and field validation in the ongoing performance monitoring program should continue.

CHAPTER 6
DISCUSSION AND SYNTHESIS OF RESULTS

This chapter provides a comparative overview of the data and results presented in this interim report. The discussions include 1) design and constructability, 2) material properties, 3) computational analysis, and 4) overall comparison of the results. A summary is then presented to wrap up the discussions.

DESIGN AND CONSTRUCTABILITY

Constructability in terms of workability and compactability is one of the critical issues associated with Texas's FDAP structures, particularly for the rut-resistant 1″ SFHMAC layers. This section relates the design aspects to the constructability of these thick rut-resistant layers. Consistent with design recommendations, the compaction lift thickness for the conventional 13″ rut-resistant layer (Layer 3) was 5″ + 5″ + 3″. As many as eight compactive rolling passes for the intermediate roller were needed to attain the 96 percent target density. Figure 6-1(a) shows an intact full length core (middle) from this section extracted four weeks after placement, suggesting good workability and compactability of the mixture.

Like many other Texas perpetual pavement projects, the compacted lift thickness on the Superpave section was 4″ for the 1″ SFHMAC layer with a compaction rolling sequence consisting of two vibratory passes for the breakdown roller, three pneumatic passes for the intermediate roller, and one vibratory pass plus one static pass for the finishing roller. As on other Texas perpetual pavements projects, Figure 6-1 shows unsatisfactory results for this section where a majority of the extracted cores broke. These cores exhibited severe debonding and vertical segregation problems within the 1″ SFHMAC lifts. Figure 6-1(a), outside cores, clearly shows these design and constructability related problems on the Superpave section. Construction measures such as reducing the lift compaction thickness or increasing the number of compactive rolling passes (as on the Conventional section) may have improved the results. As seen in Figure 6-1(c), one project (IH 35 Waco [Hillsboro]), utilized 3″ compaction lift thickness with satisfactory results. Similarly, increased compactive effort (i.e., more compactive rolling passes) yielded satisfactory results on the Conventional section, which is evident in Figure 6-1(a), middle core.

(a) SH 114 Project

(b) IH 35 Laredo & Waco Projects

(c) IH 35 Hillsboro Project
(3″ compaction lift thickness)

(d) IH 35 San Antonio Project
(few days after construction)

Figure 6-1. Forensic Comparison of Field-Extracted Cores.

Aggregate Characteristics versus Compactability

In an effort to improve their rutting resistance, the rut-resistant layers are typically designed with a coarse aggregate gradation and relatively low binder content (Chapter 2). This mix-design philosophy apparently appears to compromise the mixture workability and compactability, resulting in vertical segregation and debonding problems seen in Figure 6-1. In particular, the 1″ SFHMAC mixtures have a relatively high percentage of coarse aggregates and are generally low in fines. As a result, they are difficult to compact. For these mixtures, the average aggregates retained on the ¾″ sieve is about 13 percent while the average percentage passing the No. 8 sieve is about 23.3 for a specification range of 19 to 45 percent (No. 8). As shown in Figure 2-2 (Chapter 2), the amount of aggregates retained on the ¾″ sieve for the 1″ SFHMAC was actually 15.1 percent extracted versus the 10.7 percent design. And the actual cumulative passing the No. 8 sieve was 20 percent (extracted) instead of the design 23.2 percent. Compared to the 25.4 percent passing the No. 10 sieve for the TxDOT Type B layer on the Conventional section, the difference is quite significant. Additionally, Table 2-3 (Chapter 2) also shows a greater percentage of the coarser rock in the aggregate blend proportion than the fines. The coarseness of the gradation appears to cause workability and compactability related problems resulting in substandard in-place materials shown in Figure 6-1(a).

Binder Content versus Compactability

In general, laboratory work with the SGC has shown an inverse relationship between binder content and compactability. A comparative evaluation of three binder contents (4.1, 4.4, and 4.7 percent) for one 1″ SFHMAC mixture indicated that the required compactive effort increased with a decrease in the binder content. About 2.5 times more gyrations were required for the 4.1 percent versus the 4.7 percent mixture and 1.5 times more for the 4.4 percent versus 4.7 percent mixture for the same binder type, target height (2.5″), compaction temperature, AV, and aggregate type (limestone). These data imply that the 4.1 percent binder content mixture may exhibit greater compactability problems in the field (i.e., the mixture would not be easy to compact to the required density).

The rut-resistant layers (1″ SFHMAC mixtures) have design binder contents typically around 4.2 percent with an observed range of 3.7 to 5.3 percent across the various existing Texas perpetual pavement projects. Combined with a coarse aggregate gradation, they are prone to compactability problems and would require more compactive rolling passes to attain the target density. In fact, although the 1″ SFHMAC on the Superpave section was designed at 4 percent, the extracted binder content based on field cores was about 3.35 percent (Chapter 2). While verification with cores from different highway locations may be necessary and not ruling out aggregate absorption (limestone), this low binder content definitely does indicate potential for workability and compactability related problems.

Figure 6-2 compares the laboratory compactability of the SH 114 mixtures using the SGC and shows the greater compactive effort for the 1″ SFHMAC mixtures. While the average number of gyrations for most of the mixtures to attain the 2.5″ target height was about 76, it was 160 for the 1″ SFHMAC mixture. For the TxDOT Type C (RBL) mixture, the number of gyrations was as low as 25, indicating better compactability characteristics. In fact, none of the Texas perpetual pavements has exhibited any constructability related problems with the RBLs thus far.

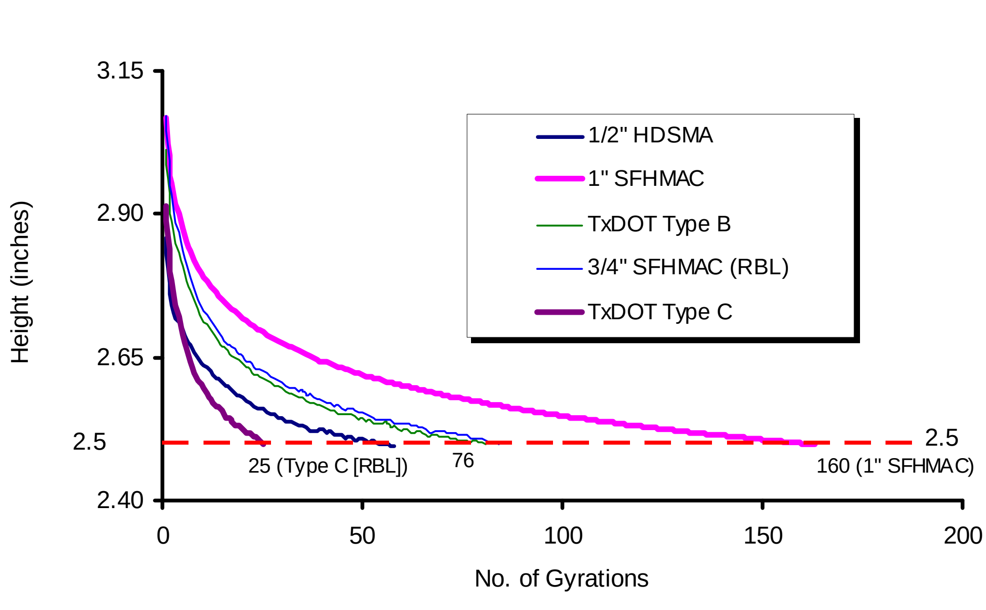


Figure 6-2. Laboratory Comparison of Mixture Compactability.

Overall, these observations suggest that the currently recommended compacted lift thickness, rolling pattern control strip, and mixture design parameters (aggregate gradation and blending, design binder content) may need to be revisited. This means that while optimizing the mix-designs to ensure rut- and fatigue-resistance, proper account should also be taken of the constructability aspects including workability and compactability.

MATERIAL PROPERTIES

As pointed out by Scullion (2006), the laboratory test results in Chapter 3 also indicated that the rut-resistant layers (in particular the 1″ SFHMAC) are very stiff mixtures with high modulus values, significantly higher than those typically assumed for design. While the majority of the original pavement designs were based on values around 500 to 750 ksi, Chapter 3 (Table 3-5) showed a modulus range of 800 to 1700 ksi at 70 °F, 10 Hz. Table 6-1 is a comparative summary of the laboratory (dynamic modulus) and field (FWD) modulus results at 77 °F, 17.5 Hz.

Table 6-1. Comparative Summary of Modulus Results at 77 °F, 17.5 Hz.

Layer	Material	Binder	Lab Testing, |E*| (ksi)		FWD (ksi)
			Lab Mixes	Plant/Cores	
FW 01: Superpave Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	630	650	826
Layer 2	¾″ SFHMAC	4.2% PG 76-22 *(5.23%)*	1050	600	
Layer 3	1″ SFHMAC	4.0% PG 70-22 *(3.35%)*	1150	1500	1394
Layer 4	RBL	4.2% PG 64-22	625	600	500
FW 02: Conventional Section					
Layer 1	½″ HDSMA	6.8% PG 70-28	630	650	826
Layer 2	Type C	4.4% PG 70-22	630	620	
Layer 3	Type B	4.5% PG 64-22	800	1100	1157
Layer 4	RBL	5.3% PG 64-22	545	560	500

( ) = extracted binder content from field cores.
Except for Layer 1 (igneous/granite), all aggregate type was limestone.

The modulus values in Table 6-1 do not differ significantly (i.e., lab vs. plant/cores vs. FWD) and show that the rut-resistant HMA layers are stiff mixes with an average modulus value of around 1100 ksi. The RBLs, on the other hand, revolve around 550 ksi. This trend has also been observed on other Texas perpetual pavements (Scullion, 2006). Dynamic modulus values for other Texas perpetual pavements at 70 °F, 10 Hz are included in Appendix B.

LABORATORY TESTING

With respect to laboratory testing, more work is still required with prismatic specimens cut from field cores where the pavement layer thickness is less than 6″ (e.g., the SMAs and RBLs). Although reasonable results were obtained in this project, more verification testing is still required, in particular to better account for the anisotropic effects and specimen geometry. To maintain an aspect ratio and nominal aggregate-size coverage of at least 1.5, the recommended minimum specimen dimensions are 2″ breadths by 2″ width by 5″ long.

Also, the RLPD test is under review to better represent field conditions, in particular with respect to the test temperatures. The current RLPD test is run only at two test temperatures of 77 and 104 °F, respectively, for all the HMA mixtures/layers. For an environmental/climatic model with five seasonal subdivisions as was used in this project, the material properties had to be interpolated/extrapolated. For the VESYS5 software, which is very sensitive to the RLPD rutting parameters (α and μ), this interpolation/extrapolation aspect could be a potential source of errors. Consequently, determination of the α and μ parameters must equally be done with extreme care to get reasonable VESYS5 results.

In general, proper processing of the laboratory test data is very critical for both interpretations of the material properties and performance prediction as well as generating the appropriate input data for the software analyses. For the software analyses, having the right and correct input data are very critical to obtaining good results.

COMPUTATIONAL ANALYSES

Consistent with theoretical expectations, the FPS and PerRoad analyses indicated that the pavement structures satisfied the perpetual pavement requirements. The predicted mechanistic strain responses were below the thresholds currently prescribed for perpetual pavements ($\varepsilon_t \leq 70$ με [RBL] and $\varepsilon_v \leq 200$ με [subgrade]).

Equally, the VESYS5 and MEPDG indicated no evidence of bottom-up fatigue cracking, and the total predicted pavement rutting was within the design threshold limits. However, although the MEPDG predicted no permanent deformation in the intermediate rut-resistant layers, there was an indication of potential for permanent deformation in the top asphalt layers, particularly on the Conventional section, emphasizing the fact that the top layers must be equally designed with considerable stiffness to prevent surface rutting.

Pavement Structure and Life Prediction

Re-evaluation of the structural designs without consideration of the PSI/IRI analyses indicated that a thickness of about 10″ rather than 13″ for the rut-resistant layer would have been adequate. The results from the software analyses are summarized in Table 6-2.

Table 6-2. Pavement Structural Analysis – without PSI/IRI Consideration.

Layer	Material	Thickness Design (inches)					
		TxDOT Recom.	Initial FPS design	Proposed FPS 19 Upgrade (FWD)	PerRoad 2.4	VESYS5	MEPDG V 0.910
1	SMA	2	2	2	2	2	2
2	RRL-Transition	3	3	3	3	3	3
3	RRL	8	13	6	9	10	10
4	RBL	4	4	4	4	4	4
5	Base	8	8	8	8	8	8
Total HMA thickness		17	22	15	18	19	19
Total pavement thickness		25	30	23	26	27	27
Life prediction prior to first surface treatment/overlay in years				23.5	30	20	24

TxDOT Recom. = TxDOT recommendations based on HMA moduli values published in the TxDOT online design guide.
RRL = rut-resistant layer; RBL = rich bottom layer; Base = 6% lime-treated subgrade
V = Version

Table 6-2 suggests that a total HMA thickness of 19″ (versus the current 22″) would have been structurally adequate for the SH 114 project, resulting in an overall savings of 3″ HMA and close to the structural thickness derived using moduli values from TxDOT's online design guide. It is also important to note that the software analyses and life predictions in Table 6-2 are comparable. This information is remarkable and emphasizes the point that for the same pavement structures, traffic, environment, and materials, similar results should theoretically be obtained provided that all influencing factors, including the software differences (i.e., analysis methodologies), are appropriately accounted for. The key to obtaining good results is the right/correct input data, assuming the software models are correct.

Overall, the software analyses indicate that at least one overlay would be required within the first 30 years of service to restore the functional serviceability of the highway. Note that 30 years was initial design life for these pavement structures. Actual performance will, however, be influenced by other parameters not accounted for in analytical models, especially as related to variability in construction. These computational results nonetheless provide an analytical indication of the progression of critical distresses and the expected performance. So these results are not unreasonable. An ongoing field performance monitoring program should be a basis for supplementing and validating these results.

Software Comparison

Table 6-3 is a summary comparison of the software as utilized and observed in this project.

Note that both the VESYS5 and MEPDG pavement surface roughness predictions appeared pessimistically unreasonable. This being a preliminary analysis, it is still too premature to conclusively ascertain whether the results are correct or incorrect, especially considering that the two softwares indicated similar predictions. It could be that 1) both the VESYS5 and MEPDG use a similar (or related) PSI/IRI model presumably not ideal for perpetual pavements, 2) the input data was incorrect, or 3) the predictions are correct and that is how the pavement would theoretically be expected to perform. Nonetheless, numerical investigations with respect to PSI/IRI analysis, including verification of the input data, are ongoing.

Table 6-3. Software Comparison.

Software	Advantage/Disadvantage
Proposed FPS 19W upgrade	Has the capacity to handle 7 layers (including the subgrade). TTI enhancements to the FPS 19W offer greater potential for analyzing perpetual pavements. Predicts pavement life with respect to rutting and cracking. Environmental/climatic effects not well accounted for.
PerRoad 2.4	Software is limited to a total of 5 layers including the subgrade. For satisfactory results, the pavement structures were configured as follows: 1) 2 asphalt layers (1 composite for the surfacing layers including the RRL and 1 RBL). 2) 1 base layer (lime-treated subgrade). 3) Dividing the subgrade into two sub-layers (top portion with thickness ≥ 1″ and bottom portion with infinite thickness). Predicts pavement life with respect to rutting and fatigue (bottom-up cracking).
VESYS5	Can accommodate numerous layers with an analysis period of up to 20 years; up to 3 HMA layers (1 composite surfacing layer, 1 RRL, and 1 RBL). Predicts performance as a function of time. Although in general reasonable results were obtained, the following deficiencies were observed: 1) Assumes one reference temperature for all the asphalt layers. 2) Interpolation software required to tie lab input data to environmental conditions. 3) Laboratory RLPD rutting parameters determined only at 2 temperatures. 4) Software very sensitive to the lab data (i.e., RLPD rutting parameters). 5) Suspicious/unsatisfactory pavement surface roughness (PSI) prediction.
MEPDG Version 0.910	Can accommodate multiple layers with an analysis period of over 50 years, which is ideal for perpetual pavements with the expected service life for the intermediate/lower layers of over 20 years. Environmental/climatic (temperature and moisture) effects well accounted for. Predicts performance as a function of time. Although in general reasonable results were obtained, suspicious/unsatisfactory pavement surface roughness (IRI) predictions were observed (just like with the VESYS5 software).

COMPARISON OF RESULTS

With the exception of forensic evaluation, which indicated mix-design and constructability related problems (segregation and debonding) for the 1″ SFHMAC layer, there was generally a good correlation among the laboratory testing, field testing, and computational results. The data presented in this report have shown that these approaches (laboratory testing, field testing, and computational analysis) strongly complement each other and should be engaged simultaneously in studies of this nature. These approaches constitute a critical tool for obtaining comprehensive and broad-based results. Long-term field performance is also necessary to validate performance; to date, only initial field performance has been conducted for the SH 114 FDAP project. Where there were inconsistent results, however (such as with the ¾″ SFHMAC field cores [Layer 2]), the data need to be harmonized and the source of differences/errors investigated. One way to achieve this, is re-sampling from different (if possible multiple) highway locations, or in the future establishing core sampling points based on GPR measurements that would indicate good/defective spots prior to actual coring.

SUMMARY

The major conclusions derived from the discussions in this chapter are summarized as follows:

- The constructability (workability and compactability) related problems associated with the 1″ SFHMAC mixtures is predominantly mix-design related and not necessarily contractor related in terms of quality control and assurance. Proposed remedial measures to address this problem include:
  (1) Mix-design – increasing the binder content, increasing the fines in the aggregate fractions, and/or reducing the coarse aggregate proportion in the blend, but without compromising other performance characteristics such as rut and fatigue resistance caused by poor structural integrity.
  (2) Construction – reducing the compaction lift thickness within allowable limits and/or increasing the compactive effort (rolling pattern/number of rolling passes).
- The 1″ SFHMAC mixtures are considerably stiff mixtures with modulus values ranging from 800 ksi to about 2000 ksi.

CHAPTER 7
SUMMARY AND RECOMMENDATIONS

This chapter is a summary of the findings and recommendations drawn from this interim report and includes ongoing and future works.

FINDINGS AND RECOMMENDATIONS

The major findings/observations and recommendations are:

- The pavement sections indicated no predisposition toward bottom-up cracking based on the overlay test, FPS, PerRoad, VESYS5, and MEPDG analyses. Although no permanent deformation was predicted in the rut-resistant layers, the MEPDG analysis indicated that the surface layers should also be of reasonable stiffness to minimize surface rutting. Overall, computational analyses indicated that at least one overlay would be required within the first 20 years of service to restore the functional aspects of the pavement.
- While perpetual pavements are designed to be rut- and fatigue-resistant, the VESYS5 and MEPDG analyses indicated that proper account should also be taken of other potential distresses such as surface roughness, top-down cracking, and thermal fracture through, among other measures, appropriate materials selection and mix-designs.
- The mix-design and construction procedures for the 1″ SFHMAC mixtures should be revisited. These mixtures have constructability (workability and compactability) related problems such as segregation and debonding due to their coarseness and generally low binder content. The occurrence of this problem on most of the Texas perpetual pavement projects suggests that this is not necessarily a contractor-related problem in terms of quality control and assurance. Therefore, optimal mix-design procedures that do not compromise both constructability and pavement structural integrity should be sought.

- The 1″ SFHMAC mixtures have been found to be stiff mixtures with relatively higher design moduli values than currently assumed, in the range of 800 ksi to 2000 ksi.
- With regard to the pavement thickness, the design assumptions on SH 114 were conservative. The total HMA thickness at 22″ could have been reduced by at least 3″. Using the current TxDOT recommendations for layer moduli, the thickness could have been reduced by 5″.

ONGOING AND FUTURE WORK

Currently, monitoring and performance evaluation of all the Texas perpetual pavements is ongoing, including field testing, laboratory testing, forensic evaluation of cores, and computational analyses. The results will form a basis for supplementing and validating the findings of this interim report as well as Report 0-4822-1 (Scullion, 2006). However, based on this report's finding, it is recommended to include intermittent extra coring (for lab testing and forensic evaluation) in the periodic performance monitoring program. In particular, this extra coring should be targeted to pavement sections where previous coring had exhibited unsatisfactory or inconclusive results. Supplemental coring locations, preferably based on GPR measurements (to locate good/defective spots), should be used. Additionally, aggregate absorption measurements should be incorporated in the scope of work, particularly for pavement sections where binder extraction tests had shown considerably lower binder contents than originally designed.

Other ongoing work should include data collection for the project database that incorporates both planned construction and existing perpetual pavement sections. With respect to computational modeling and numerical analyses, some of the ongoing and future planned work include:

- Review of the processing and assemblage of input data for all the software used in the project. This review includes both laboratory and field raw data. Having the right and correct input data are critical to obtaining appropriate results.
- A further review of the PerRoad software is recommended; however, its main limitation is that it can only handle two HMA layers including the RBL, one base

layer, and the subgrade. This review will also include sensitivity analysis (material properties, temperatures, and traffic) and consultation with the developer of the PerRoad software.

- Further review and sensitivity analyses of the VESYS5 software with respect to the input data, the environment, and surface roughness (PSI) performance predictions are required. The current VESYS5 version assumes one reference temperature for all asphalt layers; it is very sensitive to the laboratory (rutting) input parameters that are typically determined at only two test temperatures, and the PSI predictions appeared to be pessimistically unreasonable.
- Sensitivity analysis of the MEPDG software with respect to the pavement surface roughness (IRI) predictions. This work will include review of the input data, IRI model, and subsequent consultations with the developers of the MEPDG software.

In general, however, the majority of Texas perpetual pavements are performing satisfactorily to date.

PRODUCT DELIVERABLES

The product deliverables contained in this interim report are located on the following pages:

- Chapter 2, page 2-23;
- Chapter 3, pages 3-20 to 3-21;
- Chapter 4, pages 4-13 to 4-14;
- Chapter 5, pages 5-22 to 5-23;
- Chapter 6, pages 6-1 to 6-10; and
- Chapter 7, pages 7-1 to 7-3.

REFERENCES

AASHTO. *Pocket Facts*: MEPDG Version 0.910, *NCHRP 1-37A, Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures,* http://www.trb.org/mepdg, Accessed October 2006.

AASHTO. *AASHTO Designation: TP 62-03, Standard Method of Test for Determining Dynamic Modulus of Hot Mix Asphalt Concrete Mixtures*, *AASHTO Standards,* Washington D.C., 2001.

AASHTO. *AASHTO: TP5-98, Standard Test Method for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer*, *AASHTO Standards,* Washington, D.C., 1998.

AASHTO. *AASHTO Designation: PP2. Standard Practice for Short and Long Term Aging of Hot Mix Asphalt, AASHTO Provisional Standards*, Washington, D.C., June Edition, 1994.

APA. *Asphalt Pavement Alliance: Perpetual Pavements – A Synthesis*, APA 101. Maryland, 2002.

Asphalt Institute. *Superpave Mix Design – Superpave Series No. 2 (SP-2)*, Lexington, Kentucky, 1996.

Newcomb, D.E., Buncher, M., and Huddleston, I.J. "Concepts of Perpetual Pavements," Transportation Research Circular Number 503. Perpetual Bituminous Pavements, Transportation Research Board (TRB), Washington D.C., December 2001, pp 4-11.

Pellinen, T.K. and Witczak, M.W. "Stress Dependent Master Curve Construction for Dynamic (Complex) Modulus," *Annual Meeting of Asphalt Paving Technologist*, AAPT, Colorado Springs, Colorado, 2002.

Scullion, T. "Perpetual Pavements in Texas: The State of the Practice," Report FHWA/TX-05/0-4822-1, Texas Transportation Institute, College Station, Texas, 2006.

Scullion, T. and Hilbrich, S. "Determining the Optimal Lime Content for the Lime Stabilized Subgrade on SH 114 in the Fort Worth District"; Study 4822 Internal Report, Texas Transportation Institute, College Station, Texas, July 2004.

Scullion, T. and Liu, W. "Flexible Pavement Design System FPS 19W: User's Manual," Technical Report 0-1869-1. Texas Transportation Institute, College Station, Texas, 2001.

Scullion, T. and Liu, W. "MODULUS 6.0 for Windows: User's Manual," Technical Report 0-1869-2. Texas Transportation Institute, College Station, Texas, 2001.

Sebesta, S. and Scullion, T. "Using Infrared Imaging and Ground-Penetrating Radar to Detect Segregation in Hot-Mix Overlays," Report 4126-1, Texas Transportation Institute, College Station, Texas, 2002.

Timm, D.H. "A Guide to PerRoad 2.4 for PerRoad Perpetual Pavement Software," National Center for Asphalt Technology (NCAT), Auburn University, Alabama, 2004 (August).

Timm, D.H. and Newcomb, D.E. "Perpetual Pavement Design for Flexible Pavements in US," *International Journal of Pavement Engineering (IJPE*), Vol. 7. No. 2, June 2006, pp. 111-119.

TxDOT. "Condition of Texas Pavements. PMIS Annual Report, FY 2001 – 2003," Texas Department of Transportation (TxDOT), Construction Division, Materials & Pavement Section, Austin, Texas, 2003.

TxDOT. "Memorandum on Full-depth Asphalt Pavements, Flexible Pavement Design Task Force Implementation," Texas Department of Transportation, Austin, Texas, April 23, 2001.

TxDOT. *Pocket Facts:* Monthly Estimate Reports (TxDOT Expressway). *http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/cisrpts/cis02.htm*, Accessed November 2006, 2006a.

TxDOT. *Pocket Facts:* TxDOT Test Specification Manuals. http://manuals.dot.state.tx.us, Accessed May 2006, 2006b.

TxDOT. *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges*, Austin, Texas, 2004.

Wimsatt, A.J. "Pavement Design Report for Wise County State Highway SH 114," CSJ: 0353-01-017. Internal Design Report, Texas Department of Transportation (TxDOT), Fort Worth District, Fort Worth, Texas, 2003 (February 20).

Zhou, F., Hu, S., and Scullion, T. "Integrated Asphalt (Overlay) Mix Design with Balancing Both Rutting and Cracking Requirements," Draft Report FHWA/TX-05/5123-1, Texas Transportation Institute, College Station, Texas, 2005.

Zhou, F. and Scullion, T. "Overlay Tester: A Rapid Performance Related Crack Resistance Test," FHWA/TX-05/0-4467-2, Texas Transportation Institute, College Station, Texas, 2005, pp. 85.

Zhou, F. and Scullion, T. "Input Parameters of Enhanced VESYS5," Implementation Report 9-1502-01-4, Texas Transportation Institute, College Station, Texas, 2004.

APPENDIX A: AGGREGATE GRADATIONS

FW 01 = Superpave Section, FW 02=Conventional Section

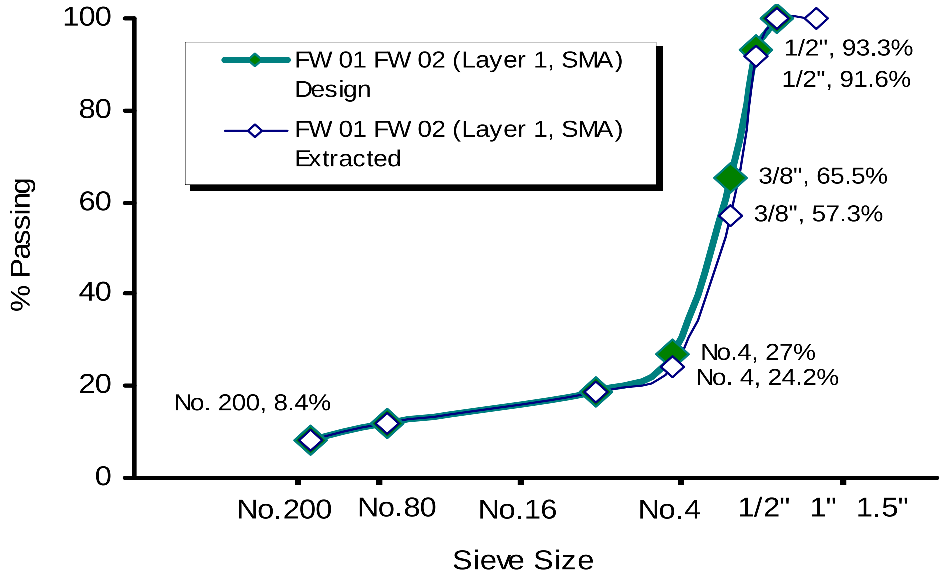


Figure A-1. ½″ HDSMA (Layer 1, Superpave and Conventional Sections).

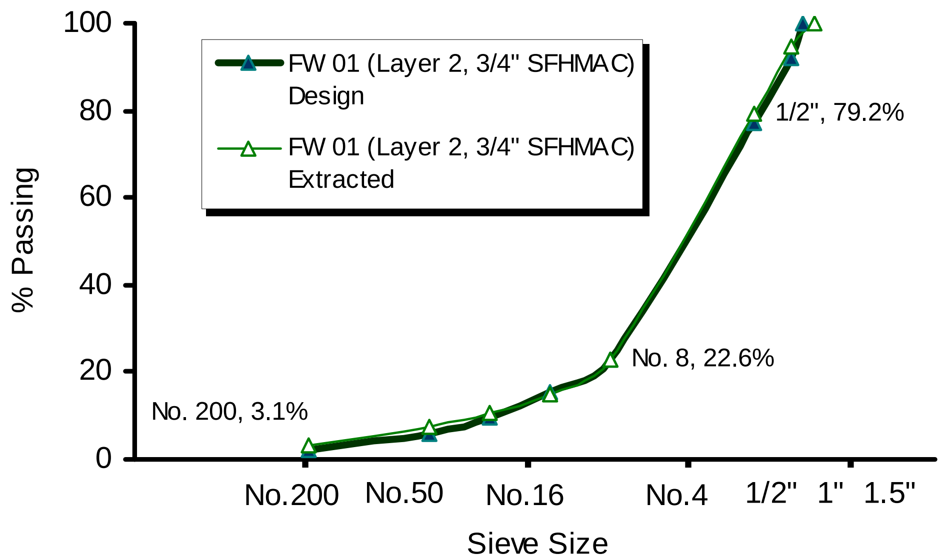


Figure A-2. ¾″ SFHMAC (Layer 2, Superpave Section).

FW 01 = Superpave Section, FW 02=Conventional Section

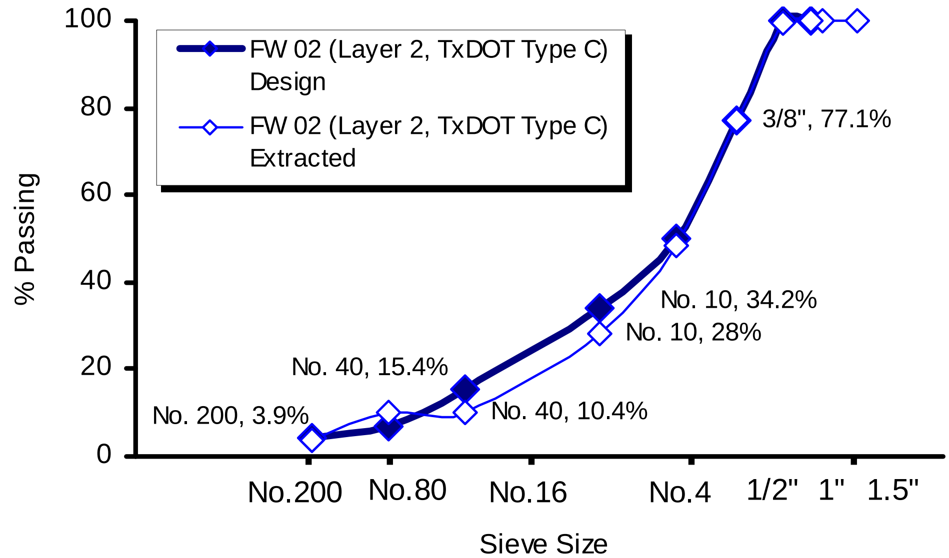


Figure A-3. TxDOT Type C (Layer 2, Conventional Section).

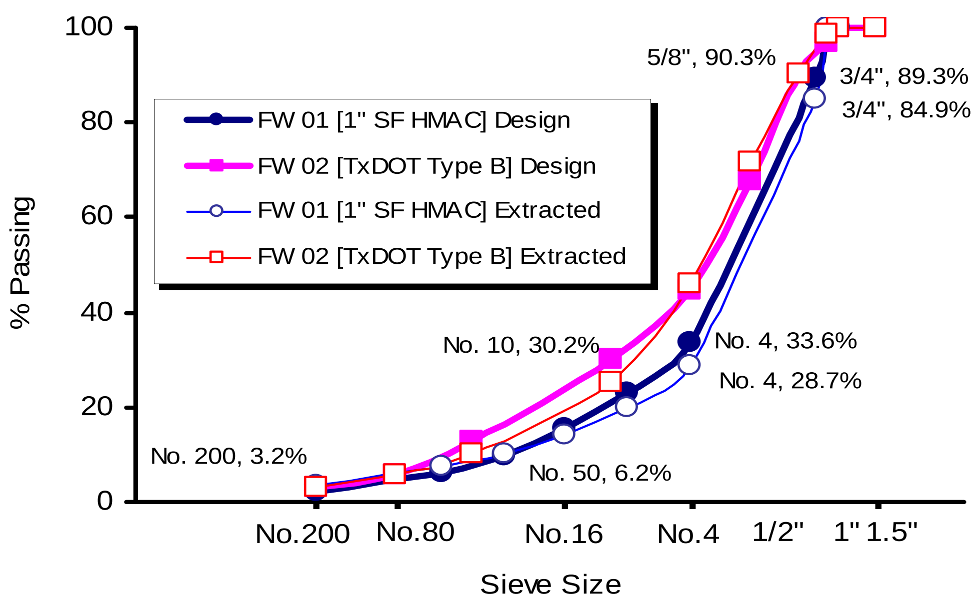


Figure A-4. 1″ SFHMAC (Layer 3, Superpave Section) and TxDOT Type B (Layer 3, Conventional Section).

FW 01 = Superpave Section, FW 02=Conventional Section

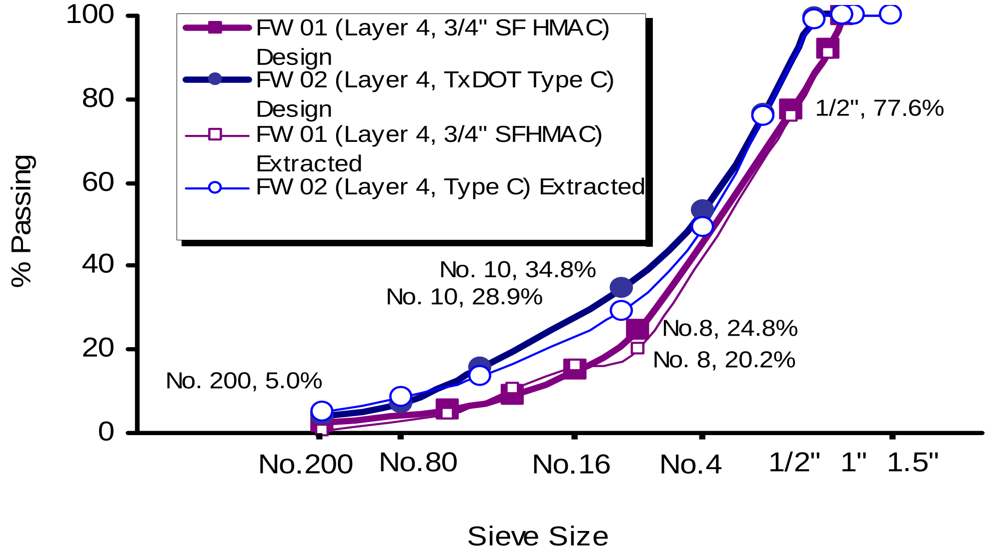


Figure A-5. ¾″ SFHMAC (Layer 4, Superpave Section) and TxDOT Type C (Layer 4, Conventional Section).

APPENDIX B: MATERIAL PROPERTIES

Table B-1. Binder PG Properties.

Binder Type	Temperature (°F)	G* (Pa)	Phase Angle, δ (°)	G*/Sin δ (kPa)
PG 64-22	122	16491.0	80.1	16.49
PG 64-22	136	4536.3	80.7	5.46
PG 64-22	**147**	**2320.5**	**83.0**	**2.40**
PG 64-22	158	953.0	83.7	1.06
PG 70-22	122	34327.0	70.0	44.36
PG 70-22	136	10730.8	70.2	12.13
PG 70-22	147	5791.7	70.8	5.83
PG 70-22	**158**	**2781.6**	**71.1**	**3.04**
PG 70-22	169	876.6	73.2	1.08
PG 70-28	122	38260.8	68.1	45.04
PG 70-28	136	12632.1	68.2	15.94
PG 70-28	147	4260.7	69.8	6.73
PG 70-28	**158**	**2215.9**	**70.1**	**2.66**
PG 70-28	169	991.5	70.7	0.99
PG 76-22	122	49400.5	63.7	64.73
PG 76-22	136	20819.4	64.1	21.81
PG 76-22	147	9606.9	64.8	10.42
PG 76-22	158	5155.1	67.7	5.22
PG 76-22	**169**	**2713.7**	**68.1**	**3.19**
PG 76-22	180	1582.8	70.6	1.59

PG pass criterion: G*/Sin δ ≥ 2.20 kPa

Table B-2. Dynamic Modulus – Lab Molded Specimens.

TEMP. & LOADING FREQ.		|E*| (ksi)						
Temperature	Frequency	Layer 01	Layer 02		Layer 03		Layer 04	
(°F)	(Hz)	½″ HDSMA	¾″ SFHMAC	TxDOT Type C	1″ SFHMAC	TxDOT Type B	¾″ SFHMAC	TxDOT Type C
14	25	2193	2757	2458	3446	3524	2805	2002
14	10	2031	2532	2302	3215	3191	2647	1865
14	5	1853	2353	2164	3027	2994	2504	1725
14	1	1470	2014	1831	2547	2456	2132	1407
14	0.5	1316	1888	1697	2335	2244	1930	1271
14	0.1	975	1459	1344	1802	1740	1511	969
40	25	2471	2283	1788	2469	2170	1476	1009
40	10	1987	2047	1605	2225	2051	1343	859
40	5	1615	1896	1454	2072	1888	1248	767
40	1	1075	1572	1122	1699	1575	997	553
40	0.5	900	1429	984	1540	1436	901	470
40	0.1	596	1126	696	1176	1116	681	325
70	25	743	1421	833	1520	1144	804	745
70	**10**	**592**	**1062**	**647**	**1364**	**892**	**605**	**542**
70	5	371	929	545	1236	749	490	445
70	1	229	727	380	954	497	313	303
70	0.5	177	594	315	836	390	251	246
70	0.1	126	377	207	592	259	167	170
100	25	291	577	388	359	422	311	248
100	10	178	384	273	244	297	186	136
100	5	139	297	210	197	228	134	96
100	1	77	163	126	126	137	73	60
100	0.5	70	131	104	112	113	61	52
100	0.1	55	82	71	88	77	44	40
130	25	253	234	292	290	213	130	174
130	10	169	182	152	182	163	77	122
130	5	111	141	120	142	121	58	93
130	1	75	94	79	108	84	40	60
130	0.5	73	81	86	90	69	34	57
130	0.1	48	70	75	80	54	29	46

Table B-3. Dynamic Modulus – Plant Mix and Field Core Specimens.

TEMP. & LOADING FREQ.		|E*| (ksi)						
Temperature	Frequency	Layer 01	Layer 02		Layer 03		Layer 04	
(°F)	(Hz)	½″ HDSMA	¾″ SFHMAC	TxDOT Type C	1″ SFHMAC	TxDOT Type B	¾″ SFHMAC	TxDOT Type C
14	25	3901	3975	2423	4864	4800	1368	1542
14	10	3242	3778	2180	4624	4496	1257	1450
14	5	2346	3065	1997	4392	4211	1170	1381
14	1	1863	2613	1580	3859	3739	958	1214
14	0.5	1664	2435	1425	3585	3486	877	1147
14	0.1	1242	1962	1055	2915	2835	696	973
40	25	2366	2131	2033	3896	3909	1041	1200
40	10	1789	1887	1795	3546	3523	937	1,058
40	5	1291	1703	1622	3284	3285	859	957
40	1	938	1325	1219	2675	2691	677	761
40	0.5	771	1197	1068	2431	2446	608	681
40	0.1	481	865	753	1894	1844	448	509
70	25	1061	876	843	2043	1460	790	736
70	**10**	**639**	**690**	**839**	**1684**	**1166**	**651**	**527**
70	5	468	576	689	1449	989	546	446
70	1	368	375	424	1003	686	359	311
70	0.5	270	303	332	864	573	290	261
70	0.1	216	169	183	580	369	176	173
100	25	589	450	417	1138	963	327	392
100	10	381	299	257	559	725	221	272
100	5	289	250	188	505	546	166	220
100	1	166	143	101	301	327	96	129
100	0.5	145	122	82	265	258	81	110
100	0.1	109	80	50	224	188	54	71
130	25	409	252	184	828	766	189	117
130	10	244	161	106	328	311	106	78
130	5	179	121	76	263	209	80	57
130	1	108	74	42	209	163	48	35
130	0.5	105	67	38	198	162	44	31
130	0.1	85	54	29	158	134	34	21

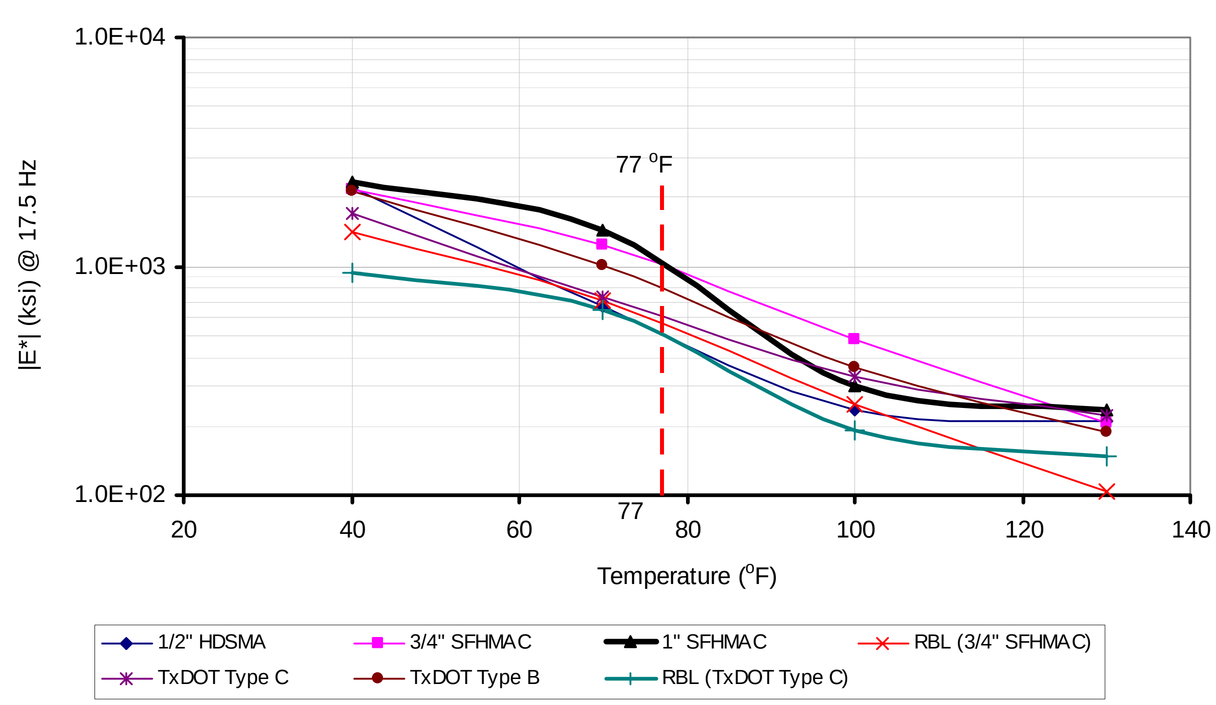


Figure B-1. Dynamic Modulus Results at 17.5 Hz – Lab Mixes.

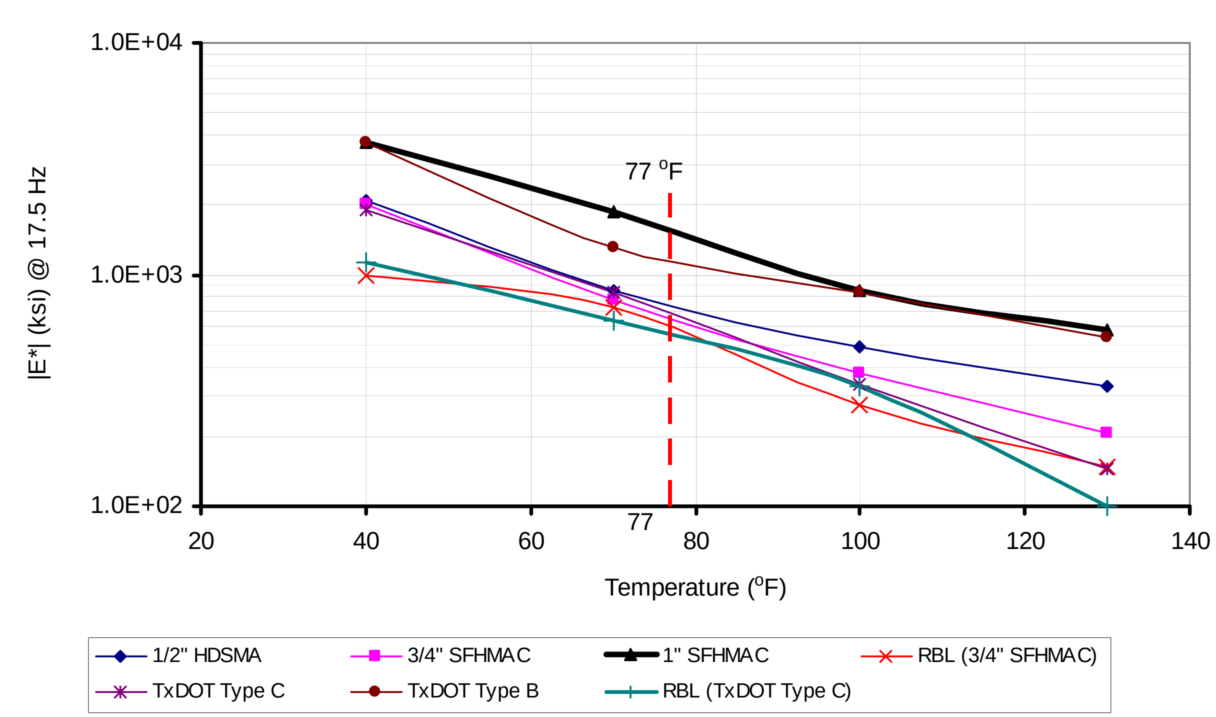


Figure B-2. Dynamic Modulus Results at 17.5 Hz – Plant Mix/Cores.

Table B-4. Comparison of Dynamic Modulus Values for the Rut-Resistant Layers (Field Cores).

Location	Mixture	Average |E*| @ 70 °F, 10 Hz
SH 114, Fort Worth	TxDOT Type B	1166 ksi
SH 114, Fort Worth	1″ SFHMAC	1684 ksi
IH 35, Laredo, Gilbert	1″ SFHMAC	1912 ksi
IH 35, Laredo, Zumwalt 01	1″ SFHMAC	1072 ksi
IH 35, Waco, Hillsboro	1″ SFHMAC	1140 ksi
IH 35, Waco, McLennan	1″ SFHMAC	1101 ksi
IH 35, New Braunfels	1″ SFHMAC	1267 ksi
Average		1335 ksi

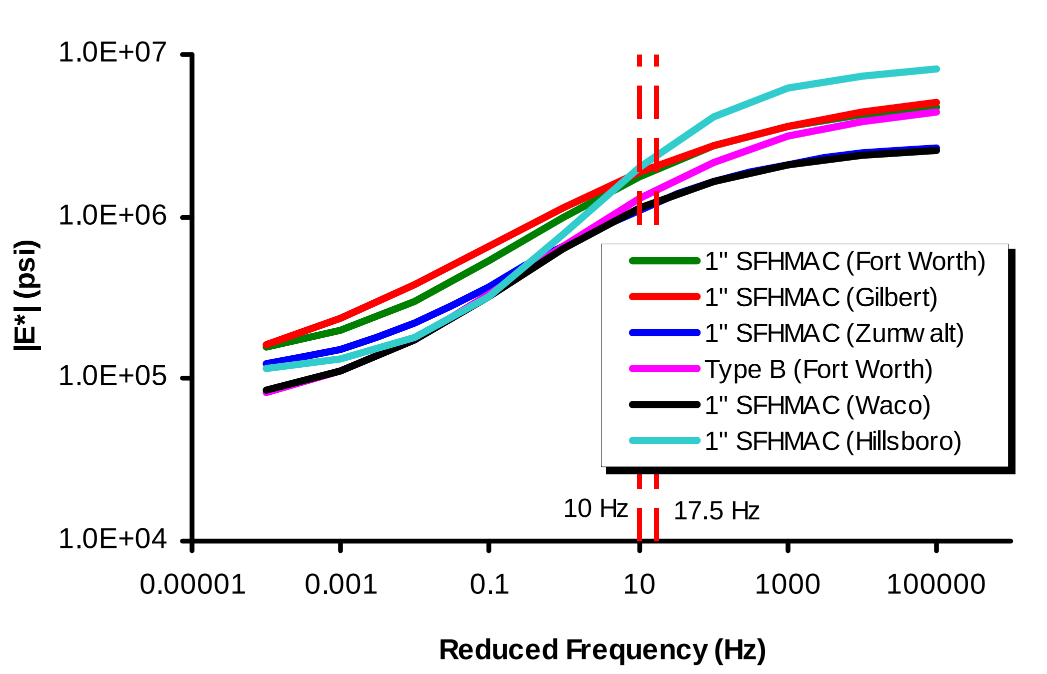


Figure B-3. Dynamic Modulus Master-Curves for the Rut-Resistant Layers (Field Cores).

APPENDIX C: FWD DATA ANALYSIS

Table C-1. TTI Modulus 6.0 Software Analysis - Summary Report.

			MODULI RANGE (psi)		
District: 2 (Fort Worth)					
County: 249 (WISE)		Thicknesses (in)	Minimum	Maximum	Poisson Ratio Values
Highway/Road: SH0114	Pavement:	5.00	160,000	720,000	H1: v = 0.35
	Base:	17.00	50,000	1,000,000	H2: v = 0.35
	Subbase:	8.00	25,000	75,000	H3: v = 0.35
	Subgrade:	58.88 (by DB)	15,000		H4: v = 0.40

Station	Load	Measured							Calculated Moduli Values (ksi)				Absolute	Depth to	
	(lbs)	R1	R2	R3	R4	R5	R6	R7	SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	ERR/Sens	Bedrock	
30	9609	3.95	1.72	1.17	0.87	0.65	0.43	0.31	218.6	627.1	75	33.2	3.46	64.3	*
326	9450	4.28	2.24	1.76	1.44	1.2	0.94	0.84	205.4	1000	75	13.1	3.28	118.7	*
619	9358	4.22	2.23	1.62	1.25	0.99	0.73	0.61	247.4	606.1	75	17.9	3.46	87.7	*
919	9462	2.96	0.89	0.51	0.4	0.29	0.22	0.18	195.9	1000	68.6	65.3	16.61	300	*
1213	9303	3.49	1.52	1.02	0.76	0.57	0.41	0.34	228.8	753.7	75	36.2	4.84	75	*
1512	9330	3.87	1.78	1.3	1.04	0.84	0.65	0.57	195.9	1000	46.4	21.6	5.14	102.3	*
1807	9219	3.28	1.46	0.99	0.74	0.6	0.48	0.42	225	1000	68.6	33	7.45	133.7	*
2097	9164	3.45	1.58	1.07	0.78	0.56	0.38	0.3	275.3	570.4	75	36	2.94	67.2	*
2399	9116	4.3	2.25	1.61	1.26	0.95	0.69	0.54	248.3	517	75	18.4	2.55	82.4	*
2691	9140	3.71	1.95	1.46	1.15	0.9	0.69	0.59	248.8	819.1	75	18.1	2.91	101.8	*
2987	9299	3.24	1.46	1.08	0.87	0.7	0.54	0.48	255	1000	68.6	26.3	5.98	94.9	*
3285	9183	3.87	1.84	1.37	1.09	0.87	0.66	0.56	190	978	75	18.7	3.47	93.7	*
3585	9183	3.37	1.15	0.71	0.52	0.37	0.25	0.19	185.3	770.4	75	58.3	5.87	62.1	*
3878	9140	3.31	1.19	0.71	0.51	0.35	0.23	0.17	209.1	634.1	75	61	5.45	60	*
4179	9025	4.05	2.04	1.38	0.96	0.63	0.46	0.34	370.8	306.4	75	27.9	2.37	73.7	*
4469	8925	5.16	2.73	1.89	1.37	0.95	0.66	0.49	321.8	237.9	75	18.1	0.79	79.4	*
4770	9060	4.38	1.95	1.46	1.17	0.93	0.68	0.59	160	864.2	75	18	3.18	81.6	*
5065	9005	3.59	1.45	1	0.75	0.57	0.42	0.36	186.2	894.3	75	34.4	4.87	81.1	*
5361	9076	2.55	0.81	0.44	0.3	0.2	0.13	0.1	249	1000	75	83	18.36	55.9	*
5656	8973	2.26	0.72	0.33	0.21	0.15	0.11	0.06	395.9	683.3	75	132	19.78	63.2	*
5951	8945	3.18	1.18	0.66	0.44	0.31	0.17	0.13	267.2	452.1	75	73.9	5.52	51.8	*
6250	8905	3.5	1.19	0.68	0.42	0.26	0.16	0.09	233.4	379.1	75	77.8	3.62	57.2	*
6544	8973	3.45	1.48	1.11	0.91	0.78	0.62	0.56	220.1	1000	75	22.4	8.43	117.9	*
6839	9005	3.92	1.57	1.12	0.9	0.76	0.6	0.57	177	1000	43.3	25.2	8.77	114.4	*

Table C-1. TTI Modulus 6.0 Software Analysis - Summary Report (Continued).

7551	9068	4.21	1.82	1.26	0.93	0.77	0.56	0.5	166	778.8	75	24.9	5.98	300	*
7849	9088	4.19	1.4	1.12	0.95	0.74	0.59	0.52	160	1000	25	30.8	10.21	116.2	*
8142	9064	3.08	0.77	0.45	0.36	0.24	0.21	0.19	307.2	519.2	75	102.4	20.43	300	*
8335	8965	1.32	0.84	0.55	0.43	0.33	0.24	0.2	720	1000	75	87	18.87	66.5	*
8735	9005	5.07	2.53	2	1.51	1.01	0.71	0.53	193	472.6	25.3	18	1.67	81.7	
8908	9029	5.11	3.02	2.46	2.02	1.59	1.19	1.01	232.7	557.7	73.5	8.9	0.51	99.1	
8960	8957	4.99	3.14	2.57	2.03	1.56	1.12	0.91	309.3	482	25	9.9	0.81	88.5	*
9005	8937	4.82	3.18	2.55	1.96	1.45	1.06	0.8	495.3	353.2	25.6	10.7	1	96.3	
9054	8969	4.24	2.63	2.17	1.74	1.34	1.01	0.87	316.7	673.5	29	11.1	0.22	102.3	
9105	9048	3.65	1.94	1.52	1.19	0.93	0.69	0.59	263.6	816.3	75	17.2	1.54	88.6	*
9156	8925	3.26	2.02	1.56	1.23	0.95	0.72	0.56	486.2	633.5	75	15.8	1.28	92.6	*
9206	9025	3.61	2.34	1.9	1.46	1.15	0.82	0.66	512.2	597.7	29	14	1.02	82.4	
9256	8985	3.61	2.44	1.87	1.41	1.06	0.74	0.62	720	420.7	35.5	15.4	1.21	78.2	*
9304	8997	3.73	2.23	1.84	1.44	1.13	0.83	0.67	338.5	742.2	48.1	13.5	0.53	89.4	
9354	8993	4.37	2.89	2.34	1.83	1.41	1.04	0.85	458.7	477	25	10.9	0.33	96.2	*
9404	8925	4.64	2.98	2.42	1.91	1.5	1.07	0.87	362.6	489.5	28.9	10.2	0.81	85.5	
9456	8981	4.65	2.81	2.25	1.76	1.36	0.98	0.83	312.8	481	48.8	11.4	0.43	87.5	
9504	8953	4.74	2.69	2.17	1.74	1.41	1.08	0.94	213.5	688	75	10.2	1.49	112.6	*
9557	9001	4.37	2.9	2.43	2.02	1.68	1.27	1.12	324.5	805.5	31.1	8.1	0.9	102.2	*
9605	8917	4.23	2.85	2.4	2.03	1.68	1.33	1.18	391.4	763.7	34.1	7.6	1.47	140	*
9653	8949	4.41	3.01	2.55	2.12	1.73	1.33	1.16	366.1	738.1	27.2	7.6	0.35	115	
9704	8989	5.49	3.43	2.82	2.29	1.82	1.4	1.2	245.9	509.9	52.3	7.5	0.45	123.5	
9754	8862	6.3	3.79	3.09	2.44	1.9	1.39	1.15	204.5	415.1	28.1	7.9	0.33	100	
9804	8969	4.89	3.04	2.5	1.98	1.66	1.2	1.01	271.8	575.6	52.1	8.7	1.38	89.9	
9851	8921	4.6	2.75	2.32	1.9	1.51	1.12	0.94	244.7	747.2	45.3	9.3	0.61	94.6	
9901	8905	4.85	2.95	2.41	1.94	1.54	1.13	0.98	250.6	633.9	25.5	9.9	0.63	92.8	
10000	8897	4.2	2.38	1.96	1.58	1.26	0.95	0.81	230.8	904.1	34.2	11.9	0.82	99.6	
10050	8913	4.21	2.38	1.98	1.59	1.27	0.95	0.82	229.9	910.2	36	11.7	0.52	96.5	
10100	8886	4.04	2.35	1.88	1.45	1.13	0.85	0.71	305.8	599.7	67.9	13.4	0.84	101.5	
10185	8870	4.61	2.42	1.94	1.59	1.29	0.99	0.84	172.3	997.1	39.6	11.4	1.9	107.7	
Mean:		4.01	2.1	1.61	1.27	0.99	0.73	0.62	285	706.8	57	28.2	4.46	88.9	
Std. Dev:		0.83	0.77	0.71	0.59	0.48	0.36	0.31	121.6	220.4	20.6	27.1	5.56	26	
Var Coeff (%):		20.6	36.7	44.1	46.22	48.11	48.9	50.76	42.7	31.2	36.1	96.2	124.68	29.2	

Legend: in = inch; DB = depth to bedrock; SURF = surface; SUBB = subbase; SUBG = subgrade; ERR = error, R1 to R7 = Radius 1 to 7

APPENDIX D: COMPUTATIONAL ANALYSIS

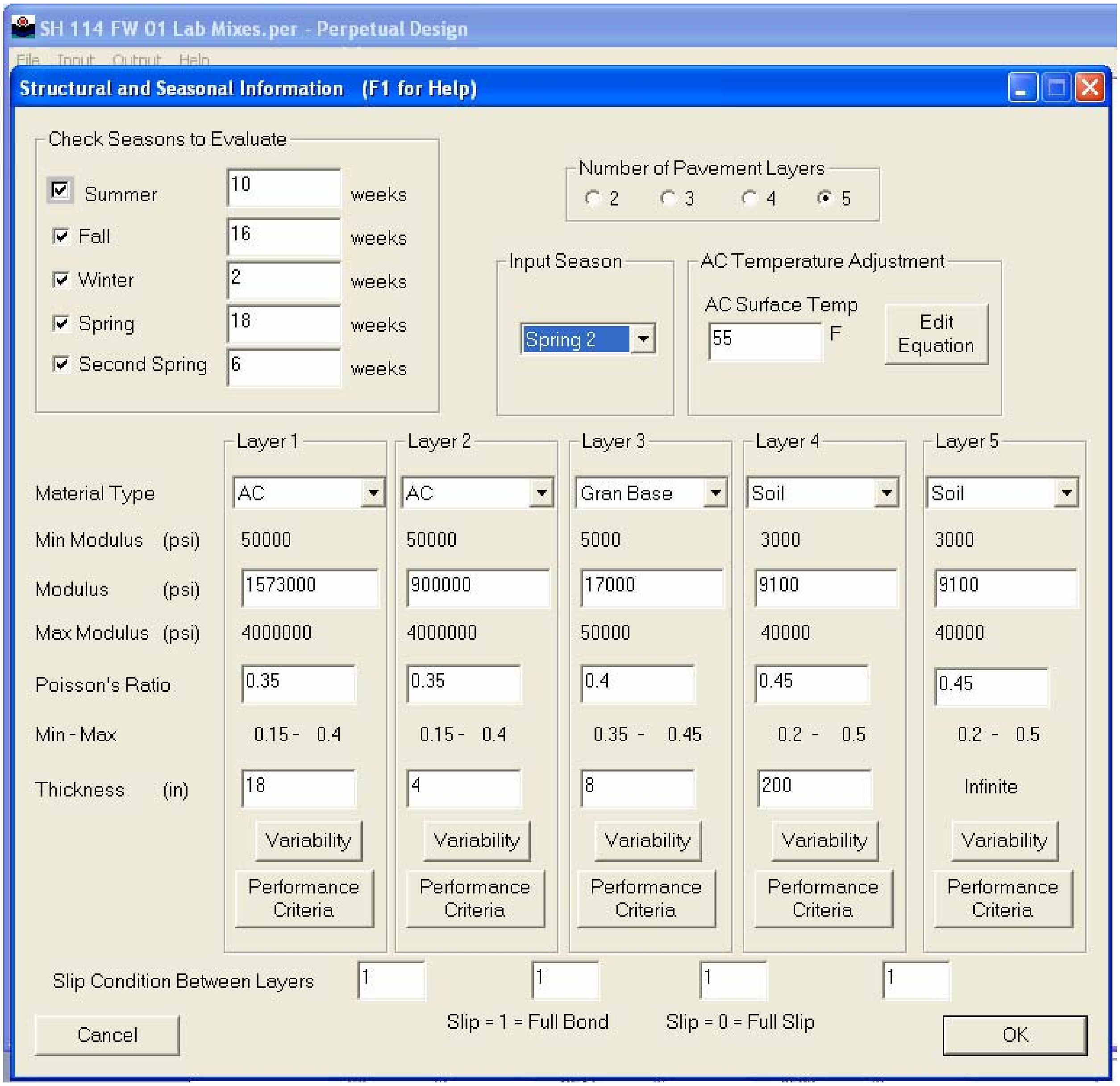


Figure D-1. PerRoad Input Data – Pavement Structure and Material Properties.

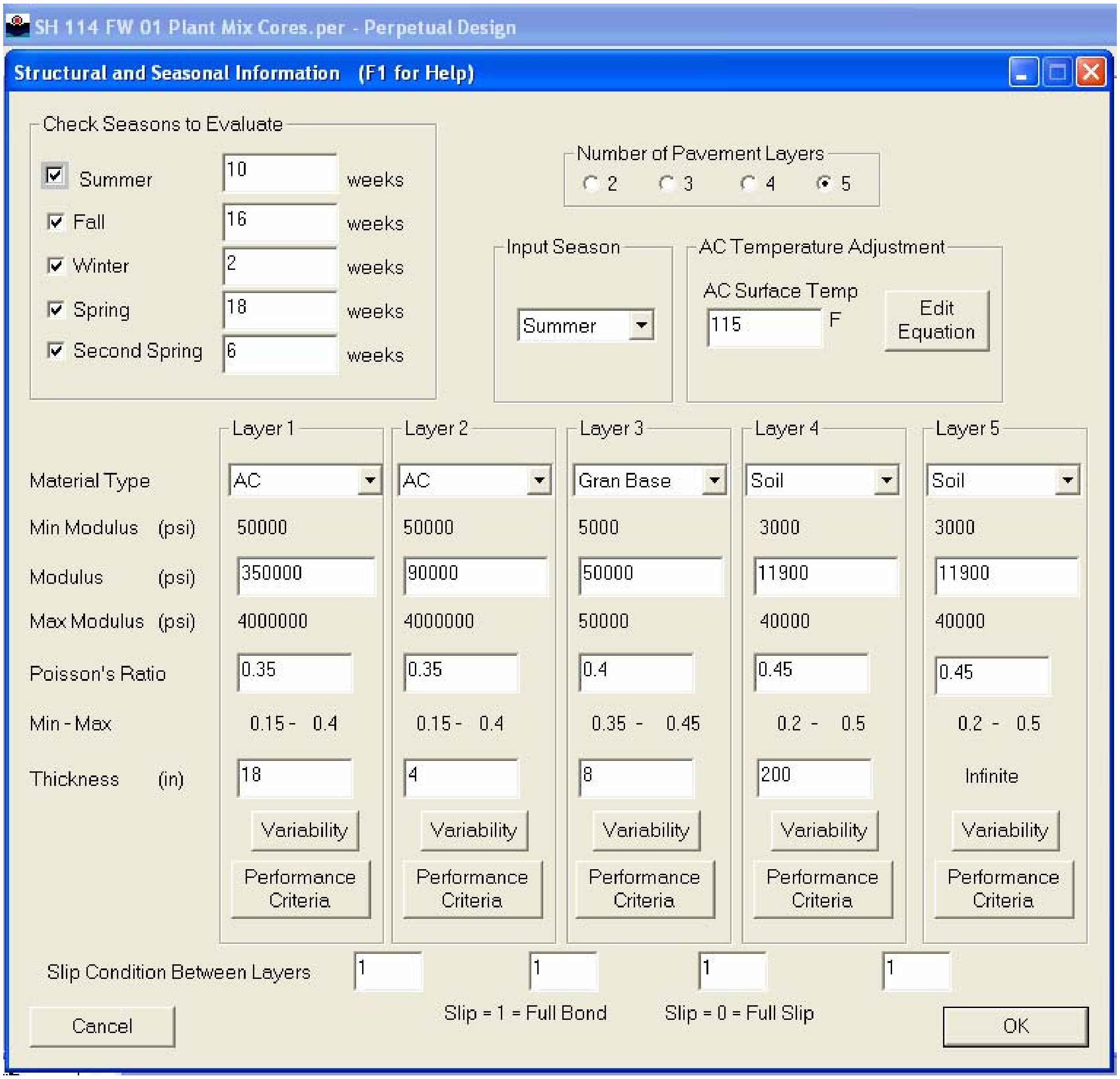


Figure D-2. PerRoad Input Data – Pavement Structure and Material Properties.

Figure D-3. PerRoad Input Data – Load Spectra.

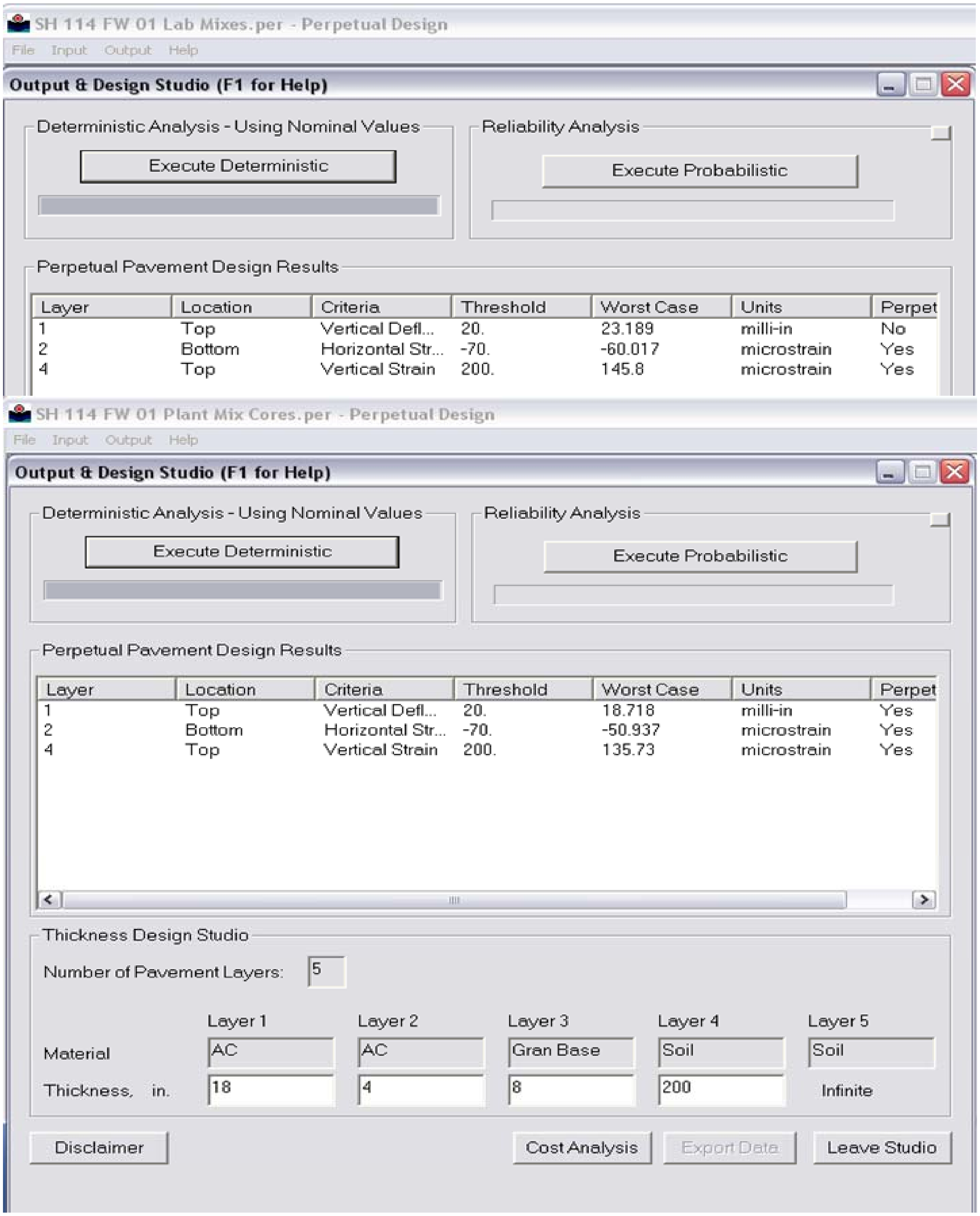


Figure D-4. PerRoad – Output Data (Superpave Section).

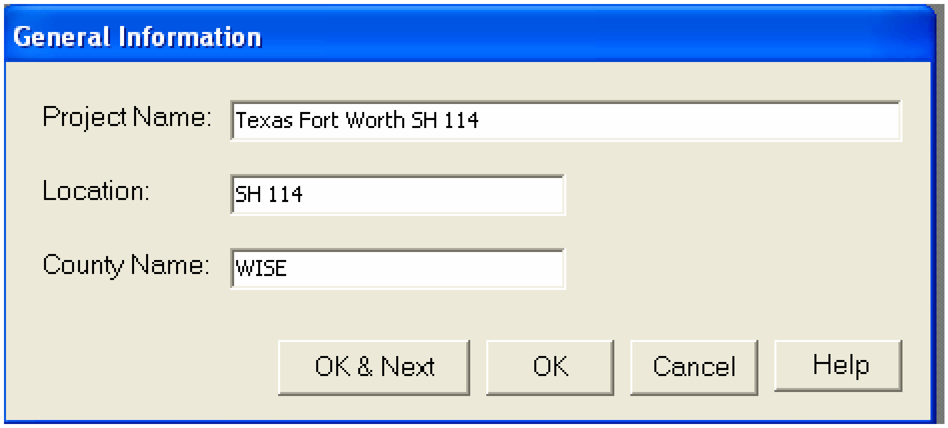


Figure D-5. VESYS5 Input Data – General Information (Superpave and Conventional Section).

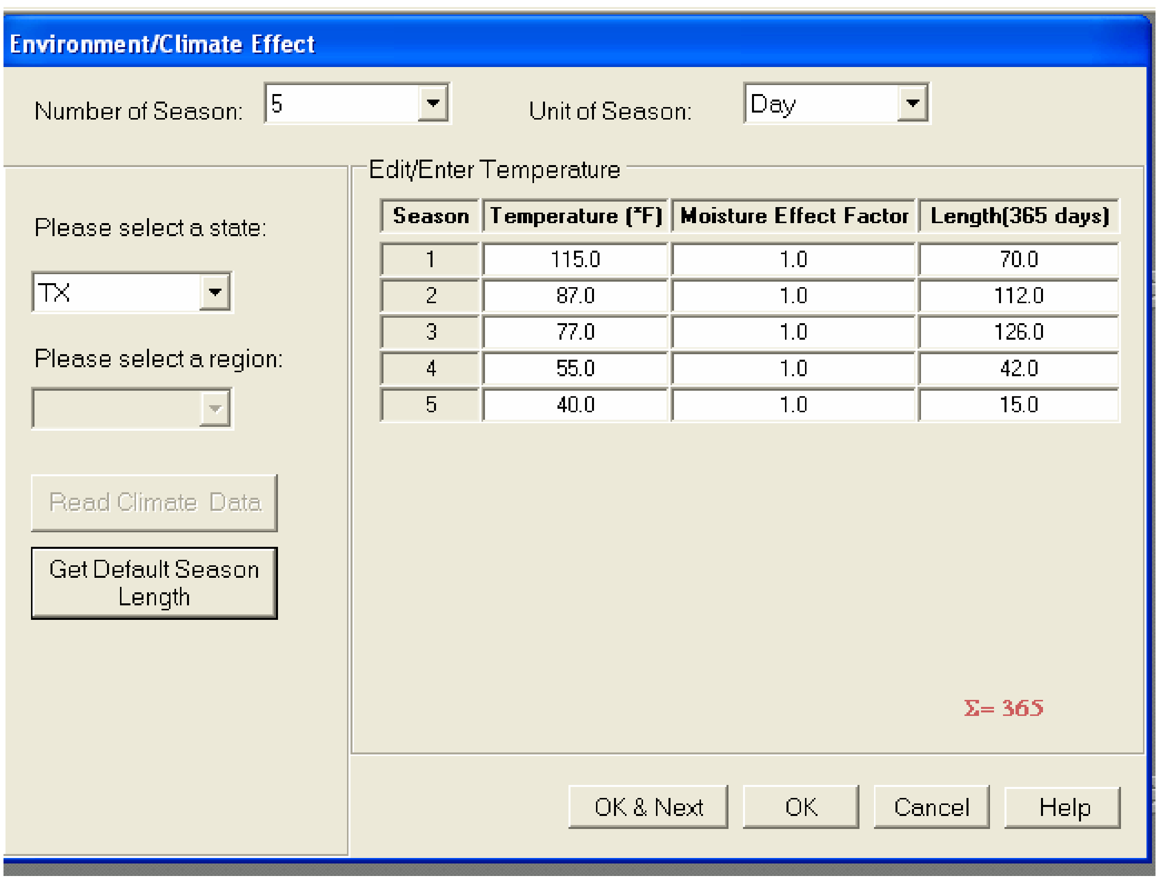


Figure D-6. VESYS5 Input Data – Environment/Climate (Superpave and Conventional Section).

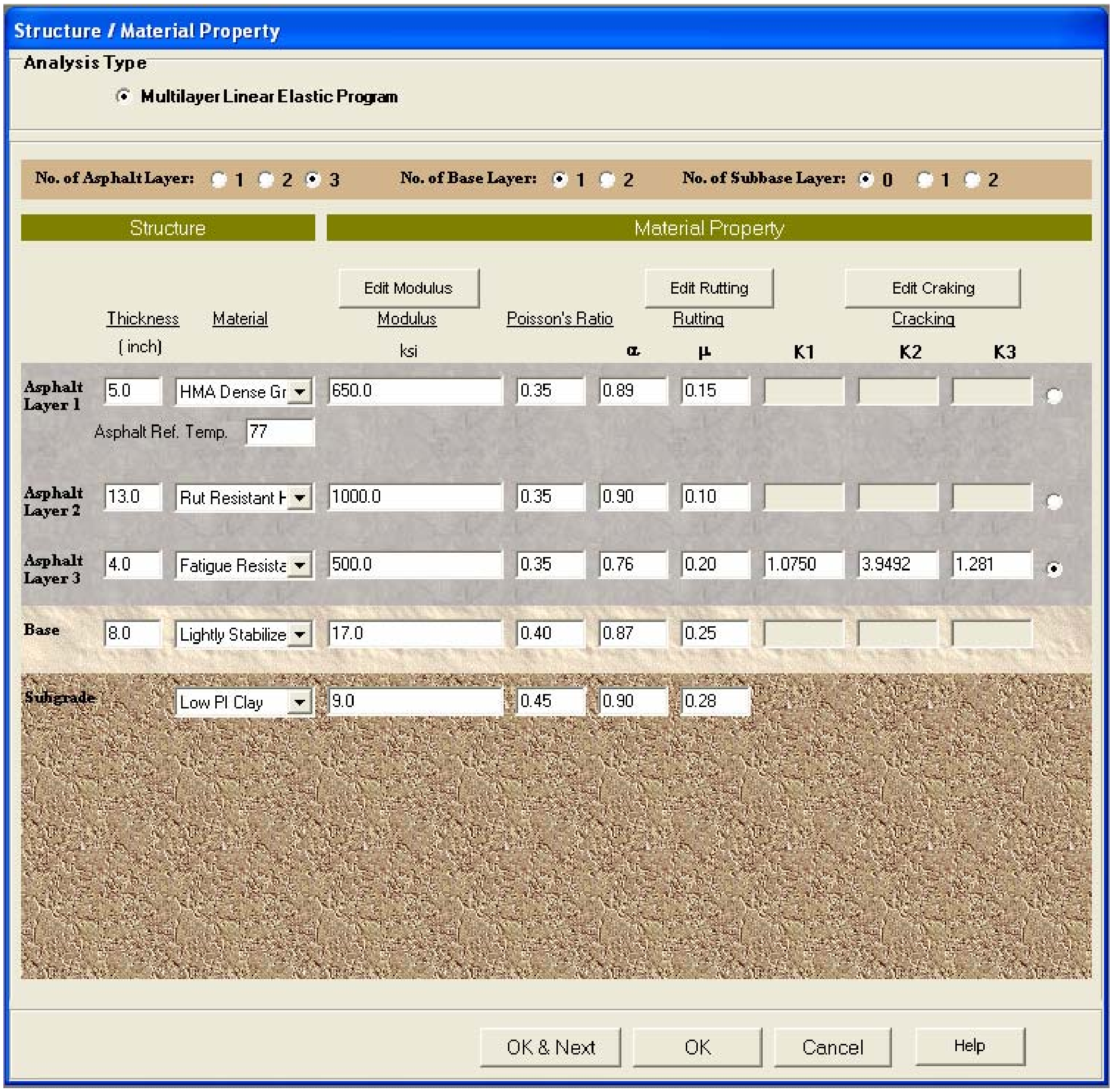


Figure D-7. VESYS5 Input Data – Pavement Structure and Materials (Superpave Section).

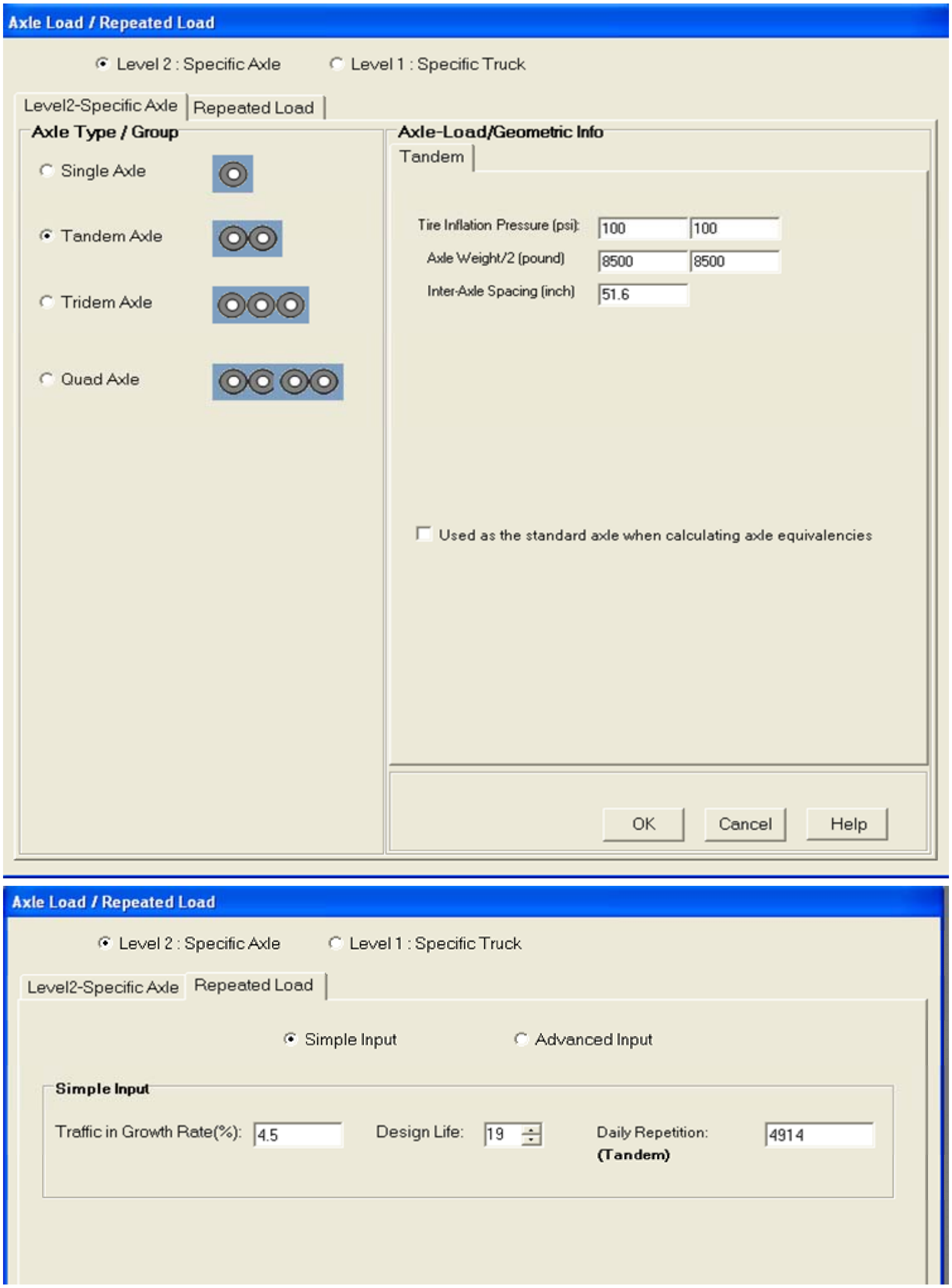


Figure D-8. VESYS5 Input Data – Traffic Loading (Superpave and Conventional Section).

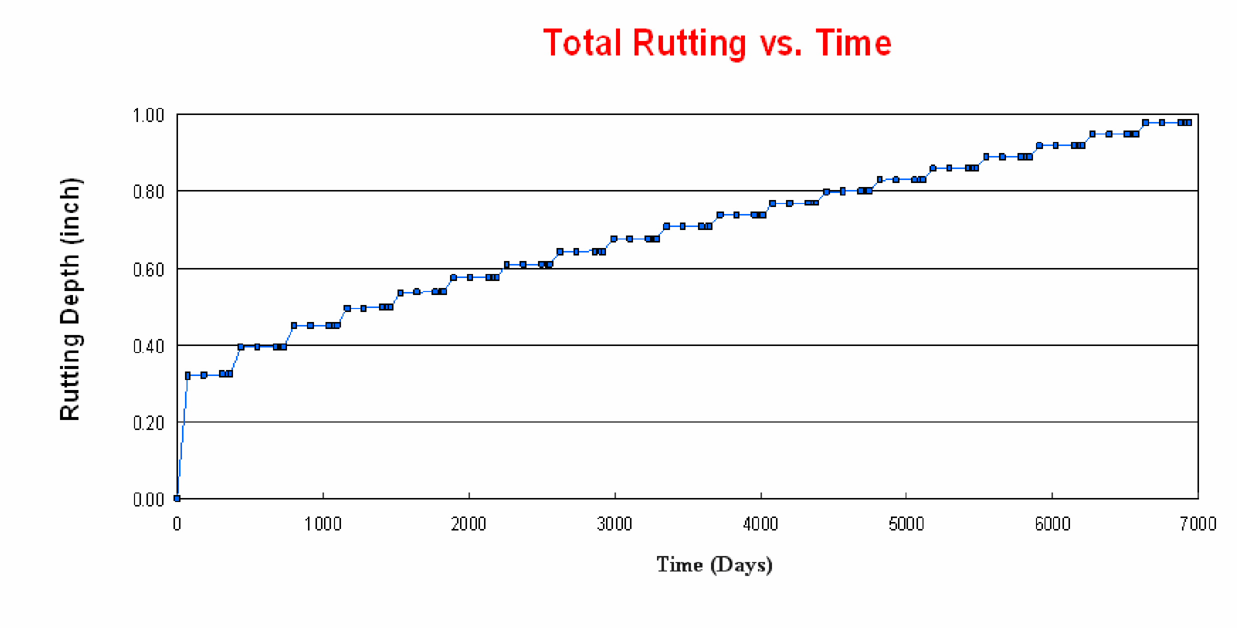


Figure D-9. VESYS5 Results – Rutting (Superpave Section).

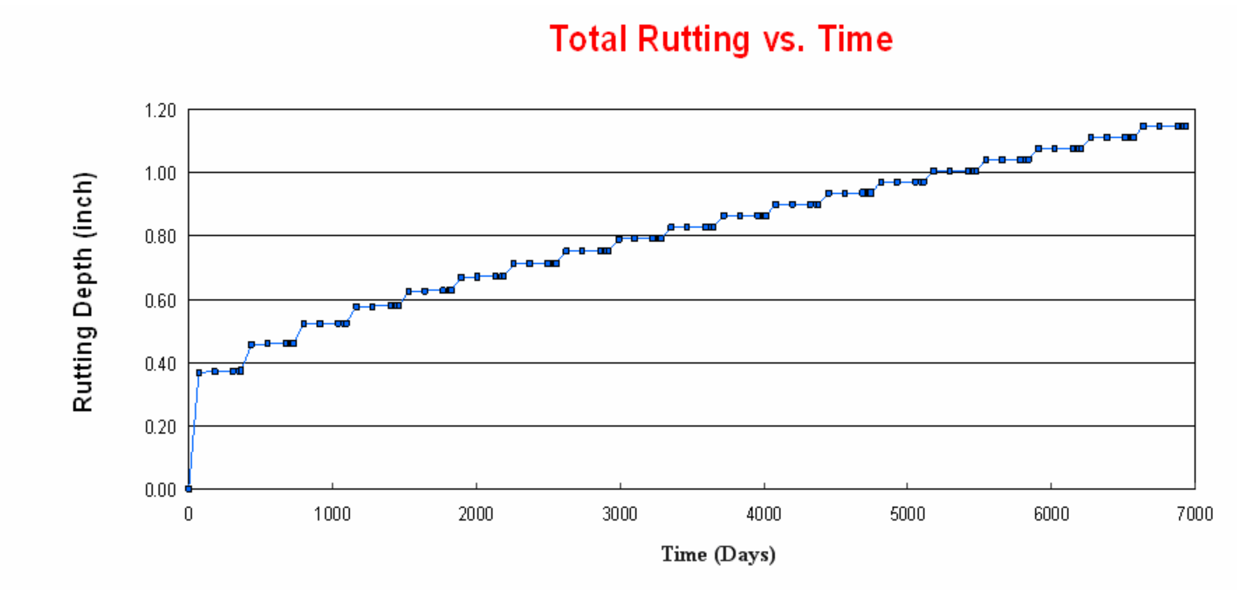
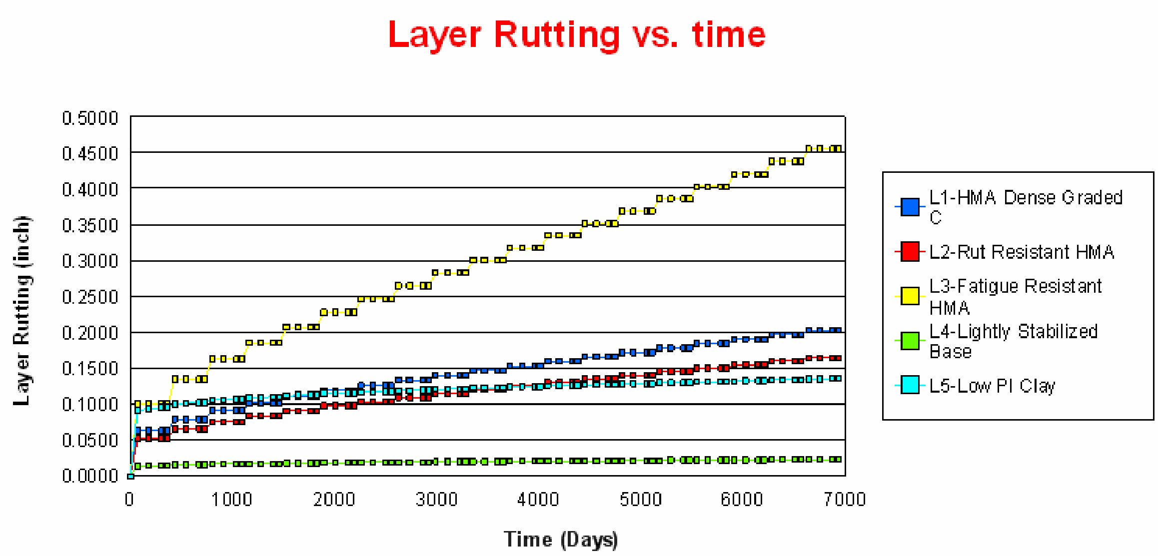


Figure D-10. VESYS5 Results – Rutting (Conventional Section).



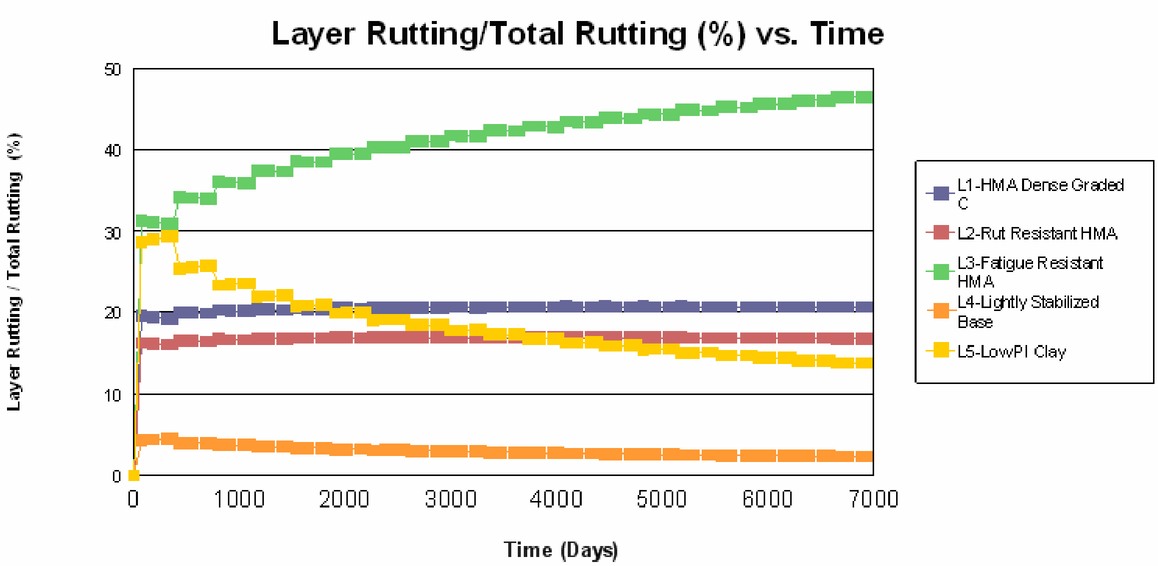
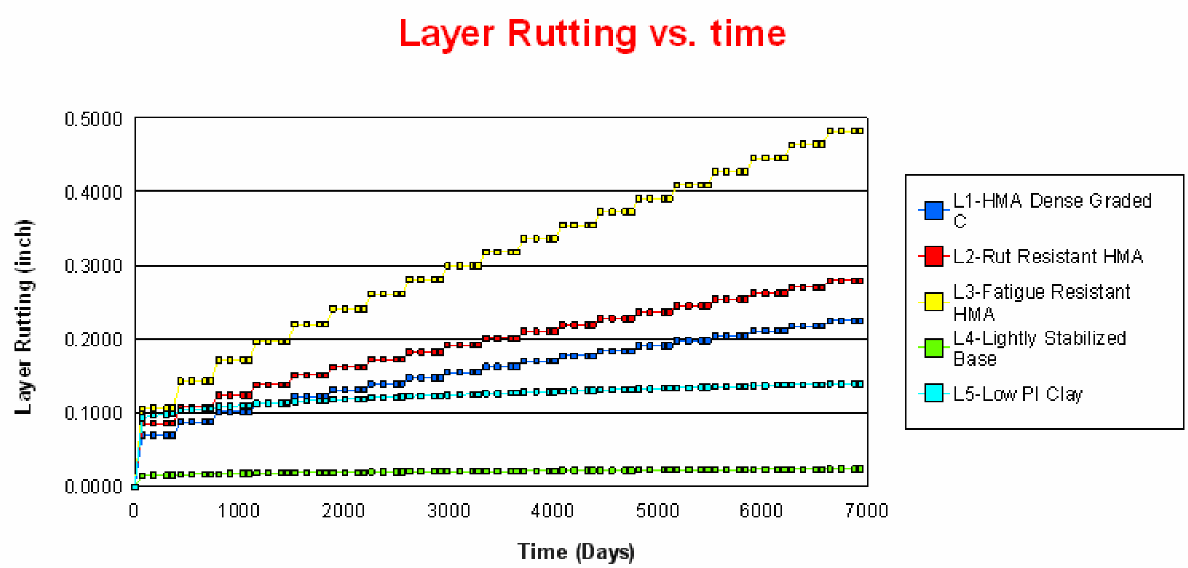


Figure D-11. VESYS5 Results – Layer Rutting (Superpave Section).



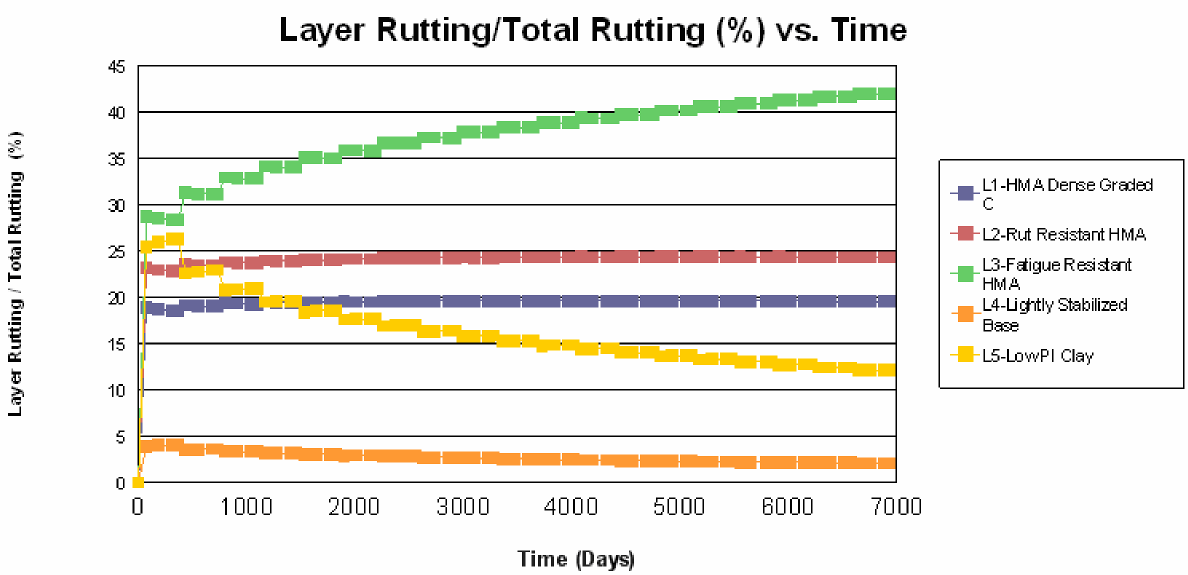


Figure D-12. VESYS5 Results – Layer Rutting (Conventional Section).

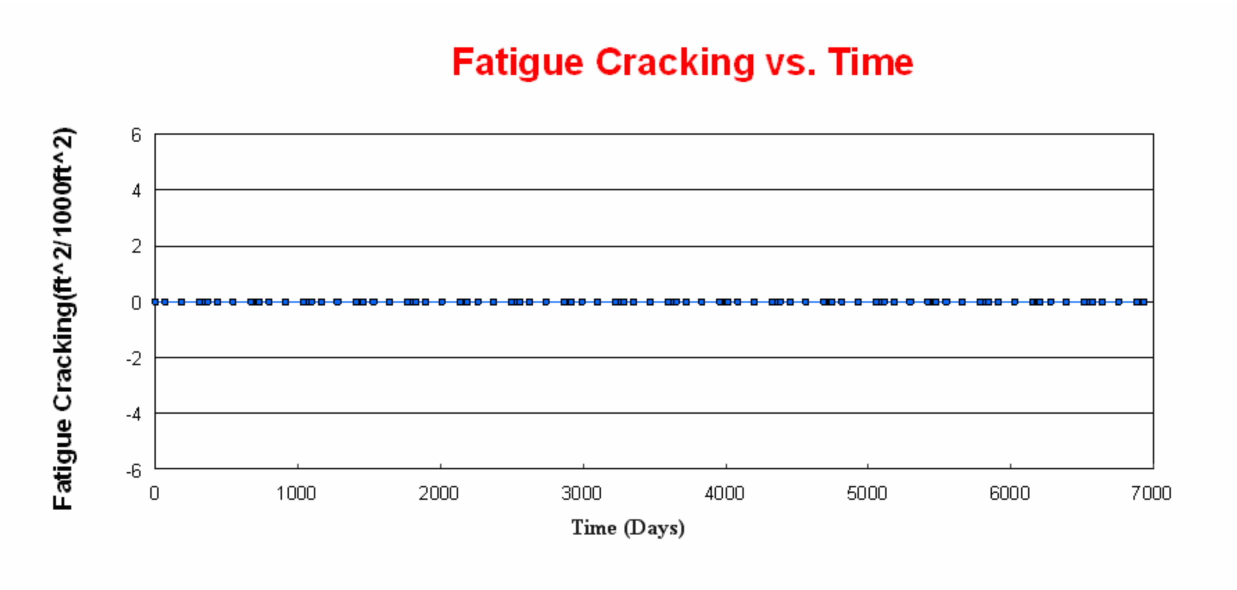


Figure D-13. VESYS5 Results – Fatigue Cracking (Superpave Section).

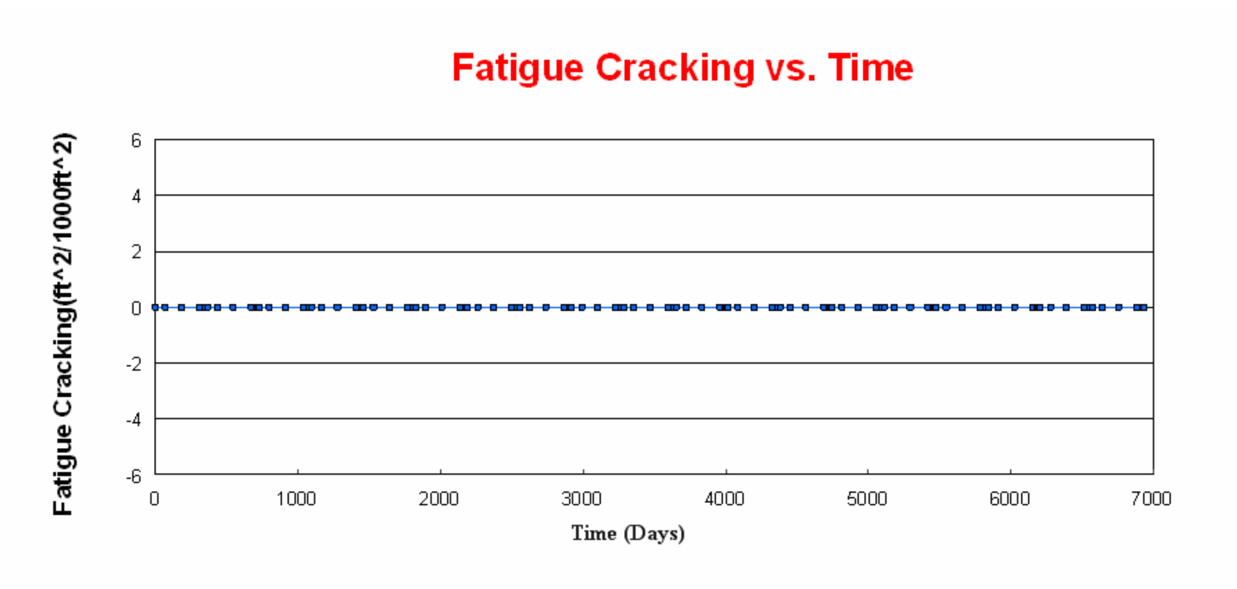


Figure D-14. VESYS5 Results – Fatigue Cracking (Conventional Section).

Table D-1. MEPDG Level 1 Analysis – Layer Permanent Deformation and Rutting.

Layer and Material Type		Distress Target	PD/Rutting Predictions (inches)	
			Lab	Plant Mix/Core
FW 01: Superpave Section				
Layer 1	½″ HDSMA	0.5″	0.29	0.14
Layer 2	¾″ SFHMAC	0.5″	0.13	0.31
Layer 3	1″ SFHMAC	0.5″	0.00	0.00
Layer 4	¾″ SFHMAC	0.5″	0.00	0.00
Layer 5	Flex Base (6% Lime treated)	-	0.01	0.01
Subgrade		-	0.16	0.15
Total for AC layers		0.5″	0.42	0.45
Total		0.75″	0.59	0.61
% Contribution of AC layers			71%	74%
% Contribution of base & subgrade			29%	26%
FW 02: Conventional Section				
Layer 1	½″ HDSMA	0.5″	0.30	0.14
Layer 2	TxDOT Type C	0.5″	0.21	0.25
Layer 3	TxDOT Type B	0.5″	0.00	0.00
Layer 4	TxDOT Type C	0.5″	0.00	0.00
Layer 5	Flex Base (6% Lime treated)	-	0.02	0.01
Subgrade		-	0.16	0.16
Total for AC layers		0.5″	0.51	0.39
Total		0.75″	0.69	0.56
% Contribution of AC layers			74%	70%
% Contribution of base & subgrade			26%	30%

PD = permanent deformation; AC = Asphalt concrete; Notation ″ represents inch