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Sustainable Perpetual Asphalt Pavements and Comparative Analysis of Lifecycle Cost to Traditional 20-Year Pavement Design: Technical Report

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Cooperative Research Program

TEXAS A&M TRANSPORTATION INSTITUTE
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16. Abstract

Since 2001, Texas has been designed and constructed 10 sections of perpetual pavement (PP) on heavily trafficked highways. With the oldest section having a service life of over 14 years, there is an opportunity to review the existing PP design and construction practices with a view of enhancing the design procedures and recommending the best construction practices for cost-effectiveness. A critical review of field performance is warranted with recommendations on how to make these pavements cost competitive with both conventional 20 years flexible pavements and rigid concrete pavements. The research methodology and scope of work mainly included:

- Extensive reviews of existing PP design and construction practices and global PP data.
- Case studies on in-service PPs with field performance and life-cycle cost (LCC) comparisons with conventional flexible and rigid pavements.
- Recommendation of best design methods and construction practices.

The field performances of Texas PP sections are still in good condition to date, with no major structural distresses. Also, from the comparative performance and LCC evaluations with conventional pavements, the PPs exhibited superior performance- and cost-effectiveness during their 50-year life cycle. To enhance the PP mechanistic-empirical design procedure, default EL criteria were developed and incorporated into the enhanced TxME system. Due to current conservative Texas PP designs, recommendations were provided for structural and material design for future cost-effective Texas PPs.

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DISCLAIMER

This research was performed in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the FHWA or TxDOT. This report does not constitute a standard, specification, or regulation.

This report is not intended for construction, bidding, or permit purposes. The researcher in charge of the project was Lubinda F. Walubita.

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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CHAPTER 1. INTRODUCTION

Since 2001, Texas has been designing and constructing perpetual pavements (PPs) on heavily trafficked highways where the estimated 18-kip equivalent single axle loads (ESALs) exceed 30 million after a 20-year design period. By definition and unlike conventional flexible pavements, PP, also commonly known as full-depth asphalt pavements, are pavement structures designed not to have major structural rehabilitation or reconstruction work, but require only minor periodic surface renewals for at least 50 years. To date, there are 10 PP sections in service within Texas. With the oldest section having a service life of over 14 years, there is an opportunity to review the existing PP design and construction practices with a view of modifying the design procedures and recommending the best construction practices to meet current traffic demands. The concern on these PPs was cost, as they were excessively thick and their multiple lifts of different mixes made PPs difficult and expensive to construct.

PROJECT OBJECTIVES

A critical review of field performance is warranted with recommendations on how to make the PPs cost competitive with both conventional 20 years flexible pavements and rigid concrete pavements. Thus, this study was initiated with the following goals:

- To synthesize case studies on in-service PPs with life cycle cost (LCC) comparison with conventional flexible and rigid pavements.
- To recommend modification to the current PP design and enhance existing mechanistic-empirical (M-E) design software by incorporating the developed PP design procedure.
- To recommend the best practice for construction of Texas PP.

The work plan includes an extensive review of existing PP design and construction practice through in-service case studies followed by development of new design methods and recommendation of the best construction practice.

RESEARCH TASK AND WORK PLAN

Figure 1 summarizes the associated research tasks and scope of works to accomplish the objectives aforementioned. Each task was designed to specifically address the following key aspects of the project:

- Task 1—Literature review and data collection. Through Task 1, researchers identified potential issues in current Texas PP practice.
- Task 2—Performance evaluation and life cycle cost analysis (LCCA) of in-service PP. This aspect was to complete case studies comparing the in-service PPs with traditional flexible and rigid pavements for performance and LCC.
- Task 3—Recommendation of new Texas PP design method. This aspect is to identify the endurance limit (EL) determination approach and enhance M-E software for Texas PP.
- Task 4—Recommendation of best practices for Texas PP construction and maintenance.
- Task 5—Development of Texas PP design specification and construction guideline.

- Task 6—Recommendation for implementation plan of new design construction procedure.
- Task 7—Workshop and demonstration case studies.

Task 1	**Literature Review & Data Collection** • Review of Texas Perpetual Pavement practices, related studies & databases • Survey of TxDOT Districts and other national agencies • Review available LCC analysis methods and tools for pavements
Task 2	**In-Service Perpetual Pavement Case Studies & LCC Comparisons** • Field performance assessment of current in-service Perpetual Pavements • Life cycle cost comparison with conventional pavements
Task 3	**Development of New Texas Perpetual Pavement Design Methods** • Evaluation of the VECD and other models for determining the endurance limits • Enhancement of existing Perpetual Pavement M-E design software
Task 4	**Best Construction & Maintenance Practices for Texas Perpetual Pavements** • Enhancement of Texas Perpetual Pavement construction best practices • Formulate QC/QA tools and rehabilitation/maintenance protocols
Task 5	**Development of Texas Perpetual Pavement Design Spec. & Construction Guidelines** • Texas Perpetual Pavement structural design spec. and construction guidelines • Texas Perpetual Pavement rehabilitation/maintenance guidelines
Task 6	**Recommendations for Implementation of the New Design & Construction Procedures** • M-E software calibration and validation • Implementation plans for new design and construction procedures • Polling of TxDOT Districts for potential interest in future Perpetual Pavement construction • Formulation of Texas Perpetual Pavement trial sections and field monitoring plans
Task 7	**Workshop & Demonstration Case Studies** • Software demonstration with 1 or 2 design examples • Life-cycle cost comparison (old versus new design) • Perpetual Pavement specification/guideline demonstration
Task 8	**Synthesis & Close Out** • Reports and products

Figure 1. Work Plan and Research Tasks.

REPORT CONTENT AND ORGANIZATION

This report consists of seven chapters including this one (Chapter 1), which provides the background, research objectives, methodology, and scope of work. Chapters 2 through 6 are the main backbone of this research report and cover the following key items:

- Chapter 2 – Literature review and data collection.
- Chapter 3 – Performance of Texas PP sections.

- Chapter 4 – LCCA of PP.
- Chapter 5 – Enhancement of M-E design for Texas PP.
- Chapter 6 – Best practice of Texas PP design and construction.

Chapter 7 summarizes the report with a list of major findings and recommendations. Some appendices containing important data are also included at the end of the report.

SUMMARY

This introductory chapter discussed the background and the research objectives. The research methodology and scope of work were then described, followed by a description of the report contents. Specifically, this final report provides documentation of the work accomplished throughout the whole project period.

CHAPTER 2. LITERATURE REVIEW AND DATA COLLECTION

Chapter 2 provides the review of Texas PP design and construction procedures and the study of global data related perpetual and full depth pavement practices covering the state, national, and international levels. Also, a brief discussion of M-E design packages is presented for use to enhance Texas PP design principles.

OVERVIEW OF TEXAS PP

PP, especially appropriate for heavily trafficked highways, is defined as a long-lasting thick hot mix asphalt (HMA) pavement structure with a service life in excess of 50 years without major structural rehabilitation and/or reconstruction activities (in particular the intermediate and bottom layers). Deep seated structural distresses such as bottom-up fatigue cracking and/or full-depth rutting are considered unlikely, or if present, are very minimal. However, they are subject to periodic surface maintenance and/or renewal in response to surface distresses in the upper layers of the pavement (*1*). With these pavement structures, distresses and rehabilitation activities are confined to the easily accessible and replaceable surface portions of the pavement. So, when surface distresses reach undesirable levels, an economical solution is often to replace or simply overlay the top layers. These rehabilitation considerations are especially significant on heavily trafficked highways where lane closures/user-delays may be cost prohibitive.

PP Design Concept

The PP concept was derived from a mechanistic principle that thickly designed HMA pavements with the appropriate material combinations, if properly constructed, will structurally outlive traditional design lives while simultaneously sustaining high traffic volumes/loads. The PP design philosophy is such that the pavement structure must:

- Have enough structural strength to resist structural distresses such as bottom-up fatigue cracking and permanent deformation (rutting).
- Be durable enough to resist damage due to traffic forces (abrasion) and environmental effects (e.g., moisture damage).

The PP mechanistic design principle consists of providing enough total pavement thickness and flexibility in the lowest HMA layer to avoid bottom-up fatigue cracking and enough stiffness in the upper pavement layers to prevent rutting. The principal approach to PP design focuses on pavement response related to both distresses (fatigue cracking and rutting), and the following limiting strain criteria are used as mechanistic benchmarks:

- Tensile strain at the bottom of composite HMA layer: < 70 micro-strains (for limiting bottom-up fatigue cracking).
- Compressive strain at the top of subgrade: < 200 micro-strains (for limiting full-depth rutting).

Also, special attention is required in designing a durable foundation to provide long-term support to the pavement structure/traffic loading and to reduce seasonal support variation due to environmental effects (e.g., freeze-thaw and moisture changes). Figure 2 shows a generalized PP design.

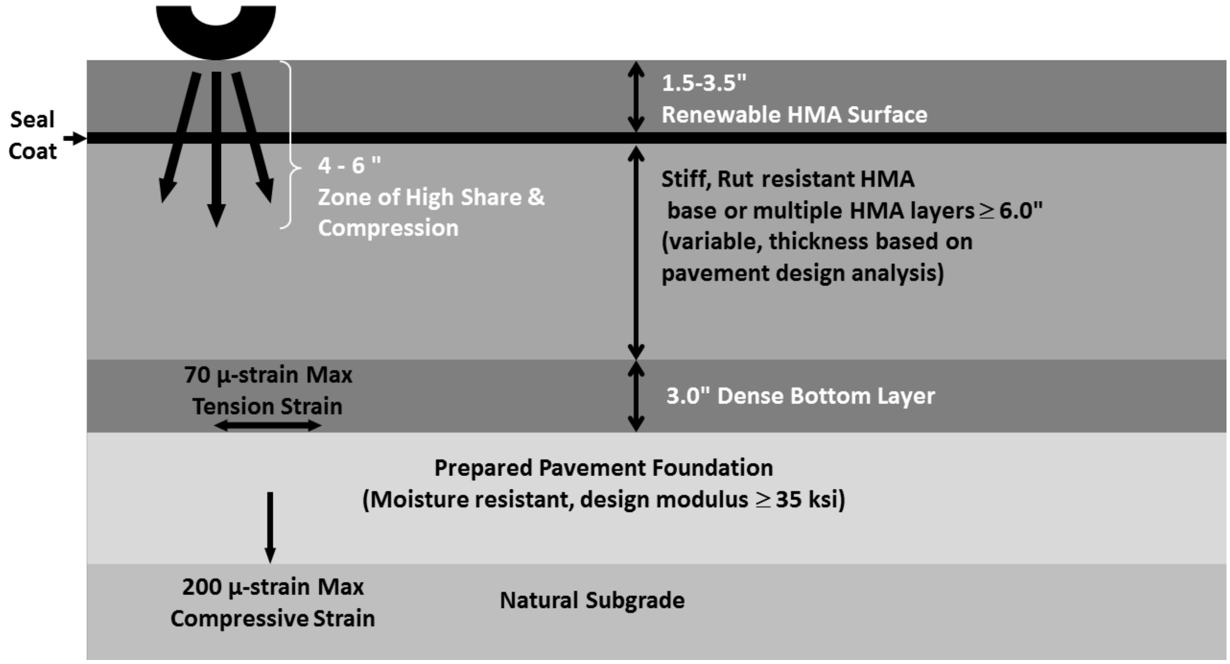


Figure 2. Generalized PP Design.

Texas PP Design and Construction Practices

The Texas PP concept was initially proposed based on the Texas Department of Transportation (TxDOT) 2001 memorandum recommending the use of full-depth asphalt pavements on heavy truck trafficked highways where the 20-years estimate of 18-kip ESALs is in excess of 30 million including the material-layer type and the proposed minimum layer thickness (*2*, *3*). The material-layer type and proposed minimum layer thickness in the memorandum was used to build 10 existing Texas PP sections located in IH 35 and SH 114, as presented in Table 1.

Table 1. Typical Texas PP Structure.

Layer No.	Mixture/Material	Thickness (in.)	Function
1	PFC	1.0–1.5	Optional layer on high traffic and rainfall areas
2	SMA	2.0–3.0	Renewable HMA surface
3	¾-in. SFHMA	2.0–3.0	Load transitional layer (LTL)
4	1-in. SFHMA	≥ 8.0	- Rut-resistant layer (RRL) - Main structural load-carrying layer
5	½-in. SFHMA	2.0–4.0	- Rich bottom layer (RBL) - Fatigue resistant - Impermeable layer
6	Stabilized base/subgrade	≥ 6.0	Providing stable foundation at the stage of construction
7	Subgrade		

Legend: PFC = permeable friction course; SMA = stone matrix asphalt; SFHMA = stone-filled hot mix asphalt

As a surface layer, Layer 1 and 2 are intended to improve the resistance to oxidation/weathering, thermal cracking, and rutting. The PFC is recommended to be placed on top of the SMA layer in locations where overall traffic volume is high and average rainfall is at least 25 in. per year. The renewable surface lift will need periodic replacement. Layer 3 is a transitional load-carrying layer composed of SFHMA mix with a nominal maximum aggregate size of ¾ in. Layer 4 is the main structural load-carrying and stiff RRL with a minimum thickness of 8 in. to ensure adequate structural capacity in terms of the load spreading capability. A 1 in. SFHMA mix has been typically used for this layer (*4*).

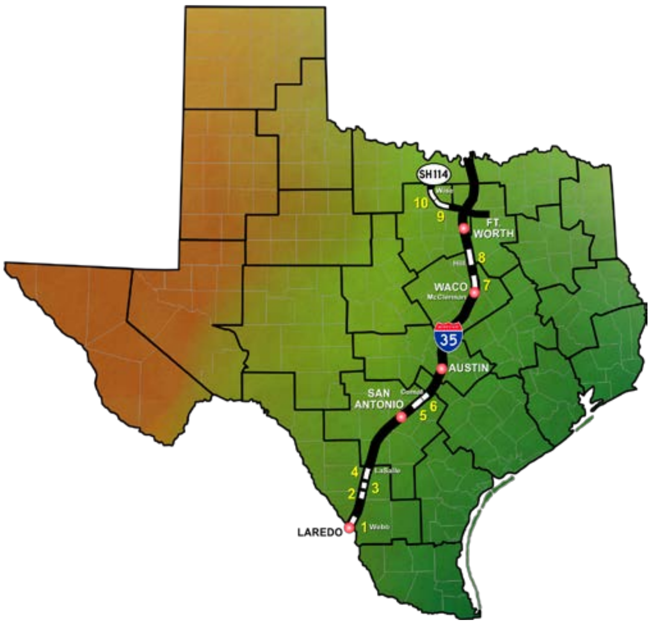
The primary purpose of the RBL in Layer 5 is to establish a fatigue resistant bottom to the overlaying HMA composite mass as a stress relieving layer. This layer represents the flexible and typically high asphalt-binder content (AC) fatigue resistant layer with 2.0 to 4.0 in. thickness. Layer 6 is placed with a treated subgrade material, typically 3.0 to 6.0 percent lime treatment to provide the working platform during construction and the stable pavement foundation. However, 2.0 percent cement treated layer has also been placed on one in-service PP section.

Existing In-Service PP Sections

To date, 10 of Texas PP sections had been constructed since 2001 in different districts as:

- Laredo: 4 sections on IH 35.
- San Antonio: 2 sections on IH 35.
- Waco: 2 sections on IH 35.
- Fort Worth: 2 sections on SH 114.

Figure 3 presents the location and information of Texas PP section. While the in-service PP sections were constructed in 2003 and 2008, it is likely that the pavements are still in good condition without major structural maintenance and rehabilitation (M/R). The field performance evaluation of each section was conducted in this study and described in Task 3.



No.	HWY	District	Length (mile)	Const. Year
1	IH 35	LRD	6.00	2007
2	IH 35		4.00	2005
3	IH 35		7.36	2003
4	IH 35		5.44	2004
5	IH 35	SAT	1.74	2005
6	IH 35		1.30	2006
7	IH 35	WAC	2.20	2003
8	IH 35		3.25	2008
9	SH 114	FTW	2.20	2006
10	SH 114		1.74	2006
Avg.			3.50	

Legend: LRD = Laredo; SAT = San Antonio; WAC = Waco; FTW = Fort Worth

Figure 3. Texas In-Service PP Sections.

GLOBAL PERPETUAL PAVEMENT DATA

Since the PP concept varies between states and countries, it is important to look at the practices of other agencies for perpetual or full-depth pavements. Thus, the global data related to PP design and construction were assembled and compared with the practice of Texas PPs. The data were collected for other states and international agencies through reviewing existing literature, online publications, and databases on perpetual and full-depth pavements, including the following:

- Design factors including traffic, design life, strain criteria, etc.
- Pavement layer and thickness details.
- Pavement material selections.
- Construction and maintenance practices.
- Number of PP sections built (in United States), etc.

The global data were collected from a total of 16 states and 15 countries (at least one country for each continent). Table 2 presents the design factors, the required number and thickness of asphalt layers, and the number of in-service PP sections of some states and countries. As shown in Table 2, while Texas has the most number of in-service PP highway sections, Texas PP design requires the thickest asphalt layers (22 in.) using premier mixtures among the states and countries reviewed. However, those PP sections using thinner HMA layers than Texas show good field performance. It is an evident fact for current Texas PP to need possibly significant improvement in material quality and thickness reduction for cost-effectiveness. In this regard, the researcher proposed an alternative Texas PP design having thinner thickness of HMA layers than the current in-service PP structures in Chapter 6. Appendix I presents the comparative summary of the PP data collected from other states and countries.

Table 2. Global Data Related to PPs.

State/Country	Design Factors			Number of Asphalt Layers	Total Thickness of Asphalt Layers (in.)	In-Service Sections
	Traffic	Life (Years)	Strain Criteria			
Texas	ADT > 100K (ESAL ≥ 30M)	50	εt ≤ 70με εv ≤ 200με	4	≈ 22	10
California	ADT > 100K	40	εt ≤ 70με εv ≤ 200με	3	≈ 13	4
New Mexico	ESAL > 32M	30	εt ≤ 60με -	3	≈ 15	1
Kentucky	ADT > 100K	40	εt ≤ 70με -	2	≈ 11	2
Michigan	ESAL > 30M	40	εt ≤ 65με -	4	≈ 14	3
Mexico	ESAL > 67M	50	εt ≤ 120με εv ≤ 250με	4	≈ 12.5	7
India	MSA > 200	50	εt ≤ 70με εv ≤ 200με	3	≈ 15	-
UK	ESAL > 80M	40	- -	4	≈ 15	1
South Africa	ESAL > 30M	50	- -	-	-	-

Legend: ADT = Average Daily Traffic; εt = Tensile Strain; εv = Vertical Strain; K = × 1000; M = × 1,000,000; MSA = million standard axles; με = micro-strains

M-E DESIGN AND ANALYSIS APPLICATION

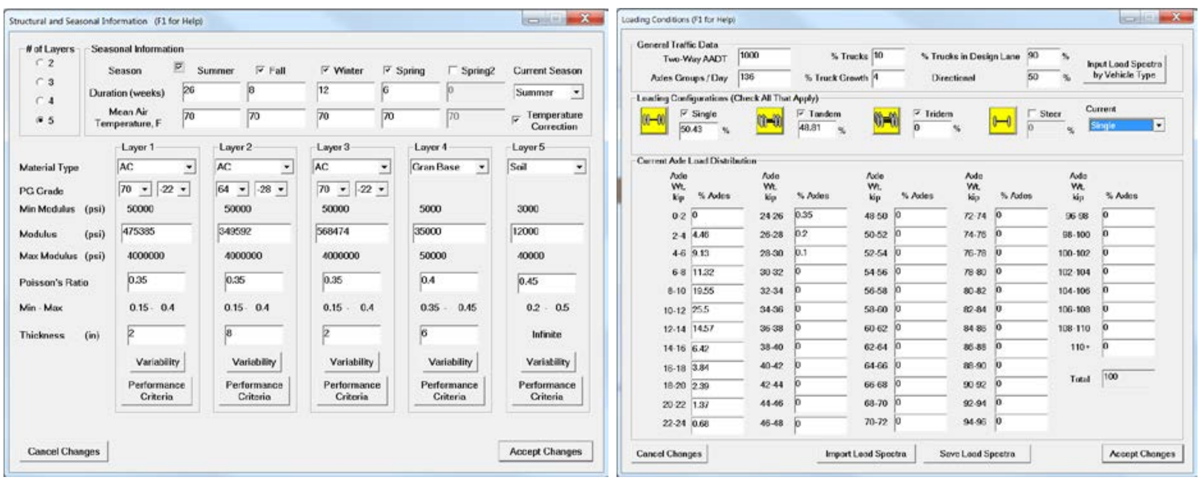
There are several design applications incorporating the M-E design approach that are applicable to PP design and analysis, including AASHTOWare Pavement M-E Design, PerRoad, flexible pavement design system (FPS), and Texas mechanistic-empirical flexible pavement design system (TxME). All the software were comparatively evaluated in this study with a focus on the FPS and TxME. Brief discussions of each of these design packages are presented below.

AASHTOWare Pavement M-E Design

The AASHTOWare Pavement M-E Design, formerly Darwin-ME, is an M-E based analytical software for pavement structural design analysis and performance prediction within a given service period. This design procedure is primarily based on pavement performance predictions of increased levels of distress over time. The performance predictions include permanent deformation, rutting, cracking (bottom up and top down), thermal fracture, and surface roughness. Because of its comprehensive performance analysis models, this software has potential to be used for PP design and performance analysis. However, application of the AASHTOWare Pavement M-E Design software for Texas PP design requires local calibration to the Texas environmental conditions and materials to obtain realistic results.

PerRoad

PerRoad, developed at Auburn University in conjunction with the Asphalt Pavement Alliance (APA), uses the M-E design philosophy. The program couples layered elastic analysis with a statistical analysis procedure (Monte Carlo simulation) to estimate stresses and strains within a pavement structure. The user needs to specify the number of pavement layers, material types and properties, variability, performance criteria, seasonal durations, and load spectra expected for the pavement structure. Then, PerRoad calculates the worst-case pavement response using a five-layer linear-elastic program: WESLEA. In the deterministic design mode, the trial design is judged to be non-perpetual if any of the prescribed PP performance criteria (particularly rutting and fatigue cracking) have been exceeded. If this is the case, changes in the design thicknesses should be made until the pavement responses are below the threshold (*1*). Figure 4 shows the PerRoad 4.3 input screens of the pavement structure and loading conditions.



(a) Structure Input **(b) Load Spectra Input**

Figure 4. PerRoad Input Screen.

FPS 21

FPS 21 is a structure design software developed and used routinely by TxDOT for:

1) Pavement structural (thickness) design.
2) Overlay design.
3) Stress-strain response analysis.
4) Pavement life prediction (rutting and cracking).

The design approach is based on a linear-elastic analysis system, and the key material input is the back-calculated falling weight deflectometer (FWD) modulus values of the pavement layers. The FPS design system is comprised of the trial pavement structure development and thickness design and the design checks including performance prediction. The FPS system has an embedded performance function relating the computed surface curvature index of the pavement to the loss in ride quality. Since the design check is principally based on the mechanistic design concepts, users can ensure if a PP design meets the limiting strain response criteria as illustrated in Figure 5. Since the FPS 21 is traditionally used for conventional flexible HMA pavement design in TxDOT, allowing for up to seven layers to be considered, and can sufficiently accommodate PPs, it was explored in this study.

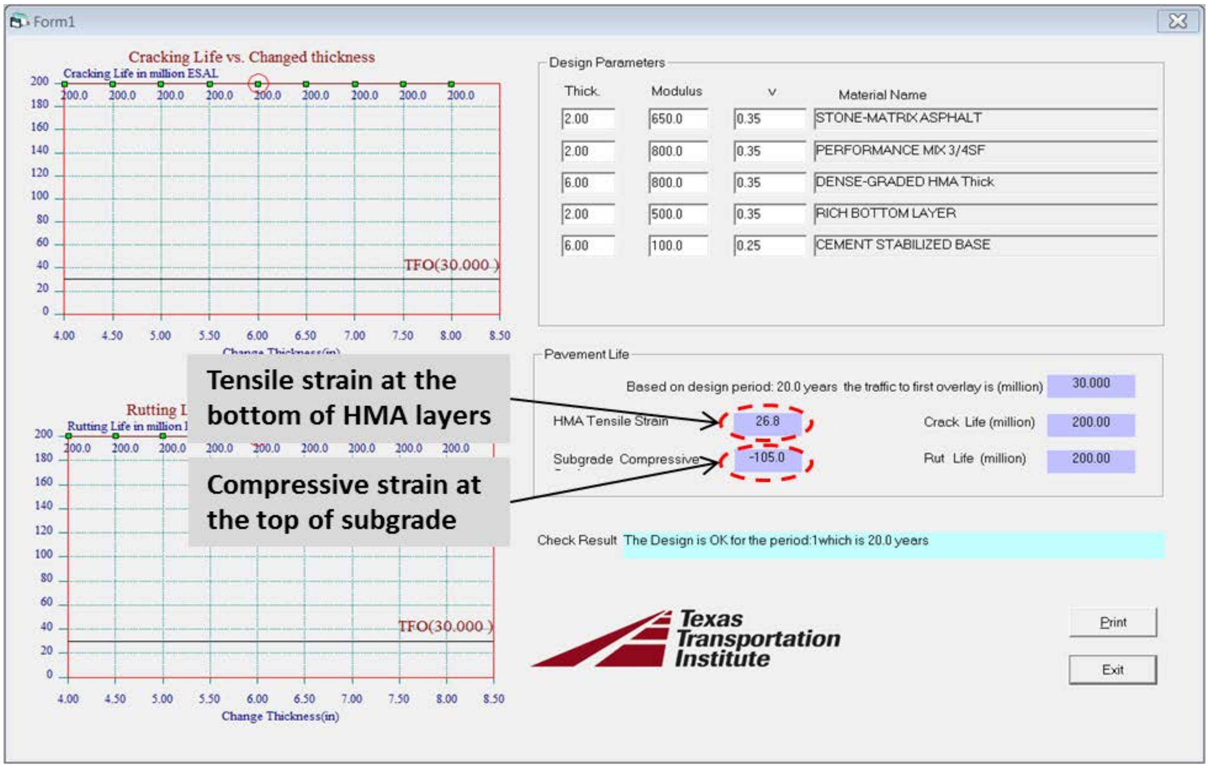


Figure 5. Mechanistic Check Output Screen.

TxME

Similar to the FPS, the TxME flexible pavement design system was developed by TxDOT and Texas A&M Transportation Institute (TTI) to enable designers to make more economical, reliable designs based on M-E modeling and performance-based material characterization. It is used for performance prediction of asphalt concrete thermal and fatigue cracking, AC and subsurface rutting, and stabilized base fatigue cracking. Three types of flexible pavement structures can be designed in the TxME, including (a) surface treated, (b) conventional or thin HMA, and (c) PP. The TxME provides connection with FPS 21 to conduct the performance check for each FPS 21 recommended design option (*5*). For any type of pavement design and analysis, there are four categories of input: (a) pavement structure and associated material properties; (b) traffic, including ESALs and load spectrum; (c) climate, enhanced integrated climate model (EICM) incorporated; and (d) reliability-related input, including performance criteria and variability.

In terms of PP performance prediction, TxME can predict rutting, ELs, and low temperature cracking. Two levels of ELs are considered in TxME:

- Level 2: When traffic input is ESALs, 18-kip axle load will be applied at the equivalent annual temperature. The tensile strain at the bottom of asphalt layer will be determined and compared to the single EL value (mix and binder type related).
- Level 1: When traffic input is load spectra, then maximum tensile strains at the bottom of the asphalt layer under different load levels and different temperature conditions will be determined and the corresponding strain distribution will be evaluated and then compared with the user-defined strain distribution criteria.

If the pavement meets the EL criteria, it is perpetual and no fatigue cracking prediction is needed. Otherwise, fatigue cracking will be predicted following the same models as for the conventional pavement. Chapter 5 presents a detailed description on enhancement of TxME for Texas PP design. Figure 6 shows the pavement structure information screen of the TxME software.

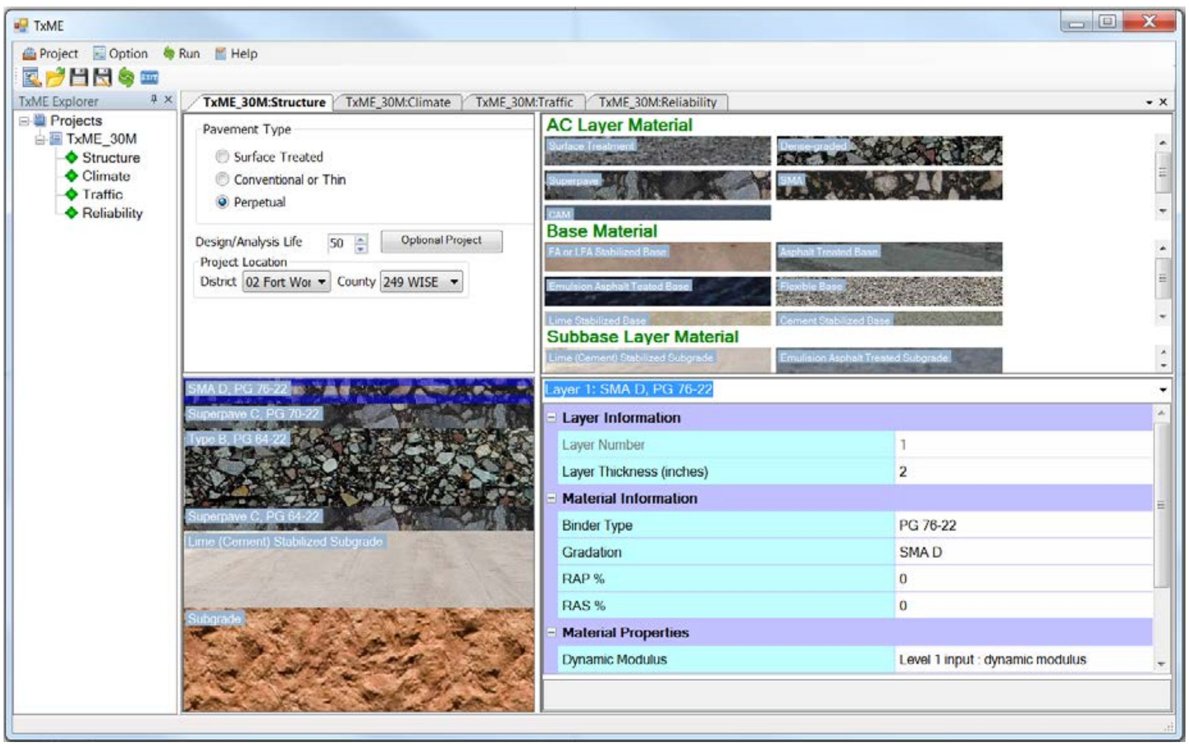


Figure 6. TxME Pavement Structure Information Screen.

Table 3 lists a detailed comparative evaluation of all pavement design packages. In this study, the FPS 21 and TxME were used as the software for Texas PP design and analysis.

Table 3. Comparison of M-E Design Software on PPs.

Item	AASHTOWare	PerRoad 4.3	FPS 21	TxME
Design Approach	Linear-elastic analysis	Layered elastic analysis	Linear-elastic analysis (layer thickness design & analysis)	Linear-elastic analysis + fracture mechanics for cracking analysis
Reliability analysis	No	Yes (Monte Carlo method)	No	Yes (Rosenblueth method)
Running time	Long (> 10 min.)	Long (depends on simulation number)	Short (< 10 sec.)	Medium (< 2 min.)
Max no. layers	7	5	7	7 (9 for perpetual)
Input	Comprehensive	Simple	Simple	Comprehensive
Output	Monthly distress or performance prediction	Years to certain damage	Recommended design alternative	Monthly distress or performance prediction
Analysis period	> 20 years	> 20 years	> 20 years	> 20 years
EL	Compares determined single strain value with user input	Determines the strain distribution and compared with user input (single or distribution)	No	Determines single strain (for ESALs) or strain distribution (for load spectrum) and compares with user input
Applications	- PP - Flexible/rigid PVMNT - New & Overlay	PP only	- PP - Flexible PVMNT - New & Overlay	- New only - PP - Flexible PVMNT - Surface treated
Application to PP structures	Requires calibration	Requires calibration	Requires calibration	Requires calibration
Calibration option	Yes	No	No	Yes
Software modification availability	No	No	Yes (limited to ESALs input and single value)	Yes

Legend: PVMNT = pavement

SUMMARY

In this chapter, researchers evaluated the PP design and construction practices used for Texas in-service PPs and collected the global PP data including design factor, pavement layer and thickness details, material selections, etc. From the literature review, the following summaries could be drawn:

- The PP design theory is based on the Asphalt Institute PP design philosophy for heavily trafficked highways without major structural rehabilitation and/or reconstruction activities up to 50 years of service life.
- The general PPs have enough structural strength to mitigate bottom-up fatigue cracking and rutting by minimizing horizontal tensile strain (< 70 micro-strains) at the bottom of composite HMA layer and compressive strain (< 200 micro-strains) at the top of subgrade, respectively.
- The global PP data showed that Texas PP design requires the thickest asphalt layers (22 in.) using premier mixtures among the states and countries reviewed while those PP

sections using thinner layers show good field performance. Thus, current Texas PP design procedures need significant improvement in material quality and thickness reduction for cost-effectiveness.

- Through the evaluation of currently available design applications, the FPS 21 and TxME were selected as a design package to be incorporated with enhanced Texas PP design principle.

CHAPTER 3. PERFORMANCE OF TEXAS PP SECTIONS

To date, 10 PP sections had been constructed since 2001 in four TxDOT districts, including Fort Worth, Laredo, San Antonio, and Waco. In this chapter, the field performance of in-service PP was evaluated and also compared with conventional flexible and rigid pavements for defensible performance-effectiveness justifications.

IN-SERVICE TEXAS PP SECTIONS

Figure 7 and Table 4 provides a map layout of Texas PPs and a summary of location details in terms of the reference marker and global positioning system (GPS) coordinate, respectively. All PP sections have been constructed on IH 35 that is the primary north-south highway in Texas, except for two sections (SH 114). Nevertheless, the sections on both IH 35 and SH 114 have a 20-year traffic design estimate of over 30 million 18-kip ESALs.

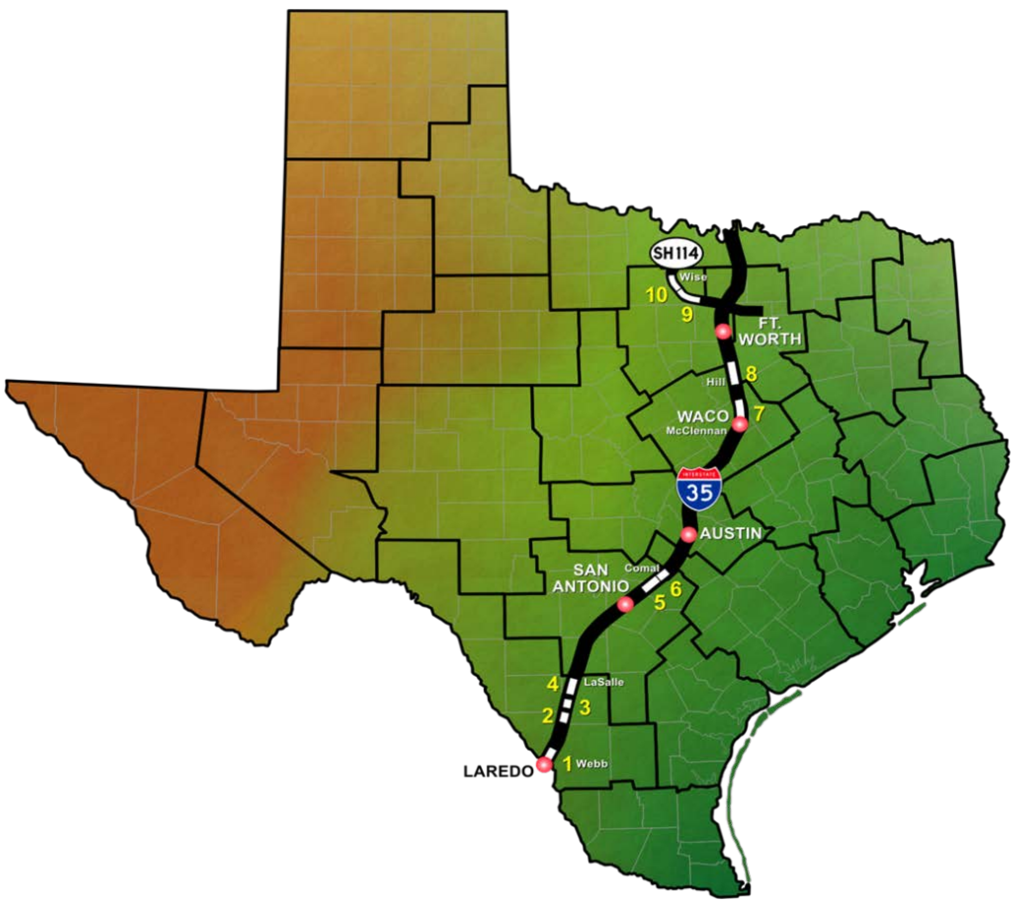


Figure 7. Location of In-Service PP Sections.

Table 4. Texas PP Location Details.

No.	HWY	CSJ	District	Reference Marker		Length (mile)	Const. Year (Completion)	Comment
				Begin	End			
1	IH 35	0018-05-062	Laredo	08+0.403	13+0.828	6.000	2008	
2	IH 35	0018-02-049		49+0.431	53+0.427	4.000	2005	Overlay (2011)
3	IH 35	0018-01-063		58+0.000	65+0.362	7.362	2003	Overlay (2014)
4	IH 35	0017-08-067		69+0.439	74+0.003	5.442	2004	Overlay (2014)
5	IH 35	0016-04-091	San Antonio	188+0.774	190+0.368	1.740	2007	
6	IH 35	0016-04-094		190+0.368	191+1.015	1.300	2007	
7	IH 35	0015-01-164	Waco	340+0.052	342+0.622	2.200	2003	
8	IH 35	0048-09-023		368+0.724	371+0.916	3.250	2008	
9	SH 114	0353-01-026	Fort Worth	580+0.804	583+0.500	2.200	2006	Conventional dense-graded
10	SH 114	0353-01-026		583+0.500	586+0.200	1.740	2006	Superpave (SP) SFHMA mixes

In-Service PP Structures

Table 5 presents the design materials and thickness of in-service Texas PP sections. From Table 5, the majority of the PP structures are conservatively thicker than minimum thickness presented in Table 1, with a total HMA layer and base thicknesses averaging 21 and 8 in., respectively. Thus, a typical in-service Texas PP is about 30 in. total thickness, comparatively more conservative than the PPs in other states and countries (Table 2).

Table 5. Structure and Layer Materials of Texas PPs.

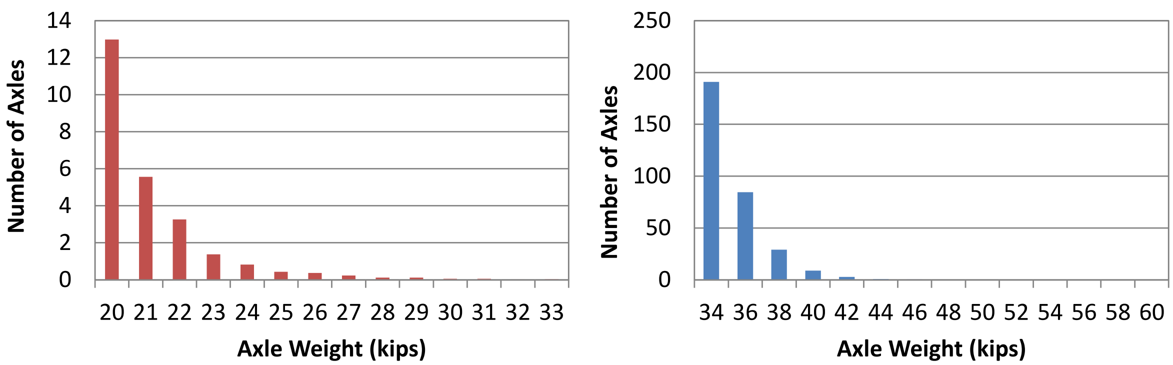
Layer No.	Material	Layer Thickness (In.)											
		Sec #	1	2	3	4	5	6	7	8	9	10	AVG.
		Design Spec.	IH 35 Laredo				IH 35 San Antonio		IH 35 Waco		SH 114 Fort Worth		
6	PFC	1.0-1.5	-	-*	-**	-**	1.5	1.5	2	1.5	-	-	1.6
5	SMA	2.0-3.0	3	3	3	3	2	2	3	2	2	2	2.5
4	¾" SF	2.0-3.0	3	3	3	3	2	2	3	3	3	3	2.8
3	1" SF	≥ 8.0	8	8	12	8	12	12	10	12	13	13	10.8
2	RBL	3.0-4.0	2	3	2	4	4	4	4	4	4	4	3.5
1	Base	6.0-12	8	8	8	8	6	8	14	8	8	8	8.4
0	Subgrade	-	Natural in-situ soil material										
Total HMA thickness (in.)		≥ 14.0	16	17	20	18	21.5	21.5	22	22.5	22	22	21
Total Pavement thickness (in.)		≥ 20.0	24	25	28	26	27.5	29.5	36	30.5	30	30	29

* Type D overlay in 2011
** Re-surfaced in 2013–2014

As shown in Table 5, the PP sections on IH 35 in Cotulla were overlaid with 1.5 to 2.0 in. of HMA (Type D) due to high surface rutting in the wheel path of the outside lane, as shown in

Figure 8. The surface distress resulted from illegal overweight traffic caused by oil activities in the Cotulla energy sector zone. In the recent years, IH 35 in Cotulla has experienced high illegal overweight traffic, especially Class 9 overloaded oil trucks, based on the analysis of TxDOT permanent weigh-in motion (WIM) data installed on IH 35 near Cotulla (Figure 9). Researchers believe that illegal overweight truck traffic in these areas of sustained elevated temperatures brings about surface distress such as rutting failure. It is also understood that the current 18-kips ESALs on these IH 35 sections is about three times more than the initial design estimate.

Figure 8. High Surface Rutting in IH35 Cotulla.



(a) Single Axle **(b) Tandom Axle**

Figure 9. Daily Overweight Axle Load Distribution in Cotulla Section (2015).

Traffic Data Collection

To effectively evaluate the performance of in-service Texas PP sections, accurate traffic loading should be incorporated in the evaluation process. Researchers collected traffic data on two in-service sections: IH 35 (Cotulla) and SH 114 (Fort Worth). The traffic data were obtained from the WIM facilities near the respective sections and from the pneumatic traffic tube counters. Table 6 summarizes the processed traffic data for both PP sections.

Table 6. Traffic Data of IH 35 (Cotulla, Laredo) and SH 114 (Fort Worth).

Traffic Data		IH 35 (Cotulla)	SH 114 (Fort Worth)
Traffic Volume	ADT	6,600	4,579
	ESAL (million)	29.54	16.58
Vehicle Speed (mile/hr)		75	70
% Truck		42.2 percent	29.2 percent
Axle Weight and Load Distribution		From WIM station	- From WIM station - Converting data from traffic counter (pneumatic tube)
Growth Factor		2.68 percent	1.79 percent

Figure 10 shows the axle load spectra data from the WIM station near IH 35 (Cotulla, Laredo District) for single and tandem axles. The PP section on IH 35 (Cotulla) is in the middle of the energy sector zone, so the pavement has experienced higher illegal overweight traffic, especially overloaded oil trucks (Figure 9). The PP sections on IH 35 in Cotulla (Laredo District) were resurfaced with 1.5 to 2.0 in. with a Type D mix due to high surface rutting (averaging about 0.42 in.) in the wheel path of the outside lanes.

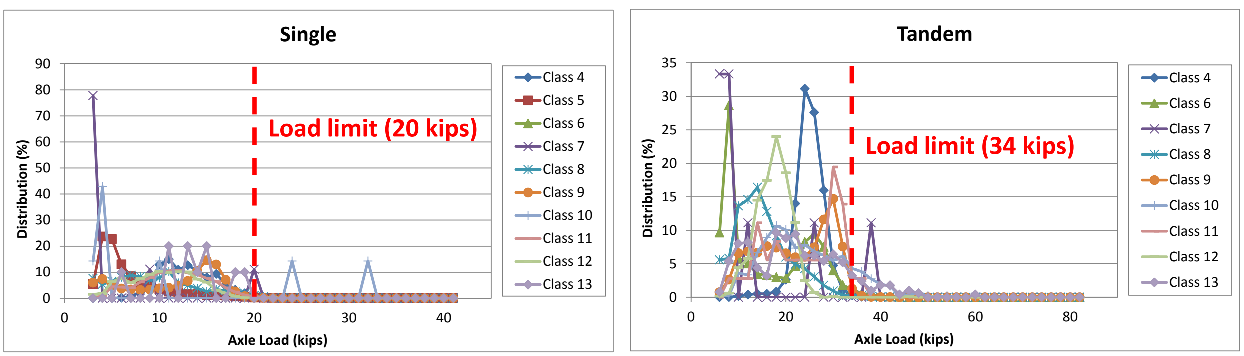


Figure 10. Single and Tandem Axle Load Spectra from WIM Station (IH 35 Cotulla).

Along with the WIM, the traffic data on SH 114 were collected from the pneumatic traffic tube counting system that was used as the primary method of field traffic data collection for Project 0-6658 (*6*, *7*). Axle load spectra and axle load distribution factors, typically determined from WIM data are the Level 1 traffic data inputs for the M-E design approach. However, due to the limited number of available WIM stations, it is not feasible to obtain complete axle load spectra data from all the desired highway sections. Therefore, as an alternative, a simple analysis method was developed to estimate the axle load spectra data from the pneumatic tube counters using the cluster analysis method (*6*, *7*). That is, the axle load spectra of each axle type can be estimated using the vehicle class distribution collected by the pneumatic tube traffic counters by means of cluster analysis. Figure 11 presents the single and tandem axle load spectra converted from the pneumatic traffic tube data. Appendix II provides all the axle load spectra data collected from the WIM stations (IH 35 and SH 114) and converted from the pneumatic traffic tube data (SH 114).

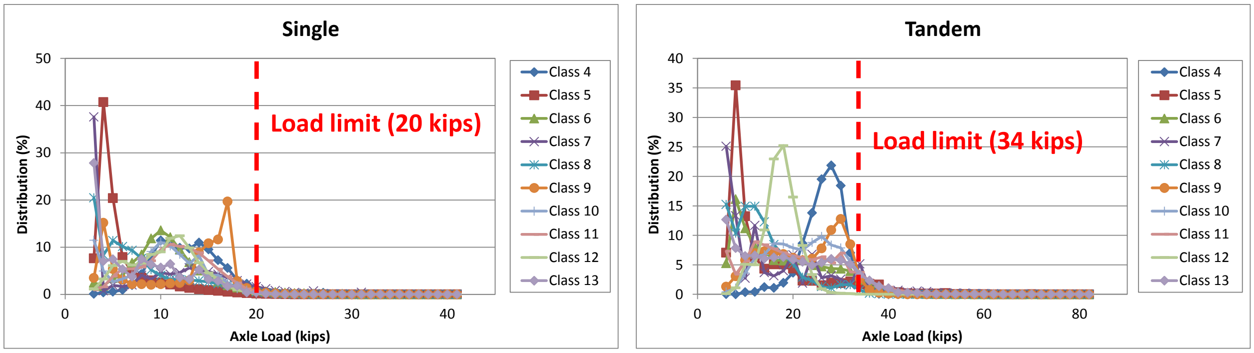


Figure 11. Single and Tandem Axle Load Spectra from Traffic Tube Data (SH 114).

Field Performance of In-Service Texas PP Sections

The field performance of in-service PP sections, listed in Table 4, was evaluated in conjunction with the Texas flexible pavement database from Project 0-6658. The evaluation was performed based on the field performance data collected from the 500-ft test sections of each in-service Texas PP, including the rutting survey, visual surface crack survey, and surface roughness (International Roughness Index [IRI]) and pavement serviceability index (PSI) measured using Profiler, conducted for Project 0-6658 (*7*). While the in-service PPs were constructed between 2003 and 2008, it is likely that they are still in good condition, as shown in Figure 12 and Figure 13, without major structural M/R except for the sections near Cotulla in the Laredo District. Appendix III present the performance evaluation of each existing PP section, including pavement structure, section map, and latest pictures.

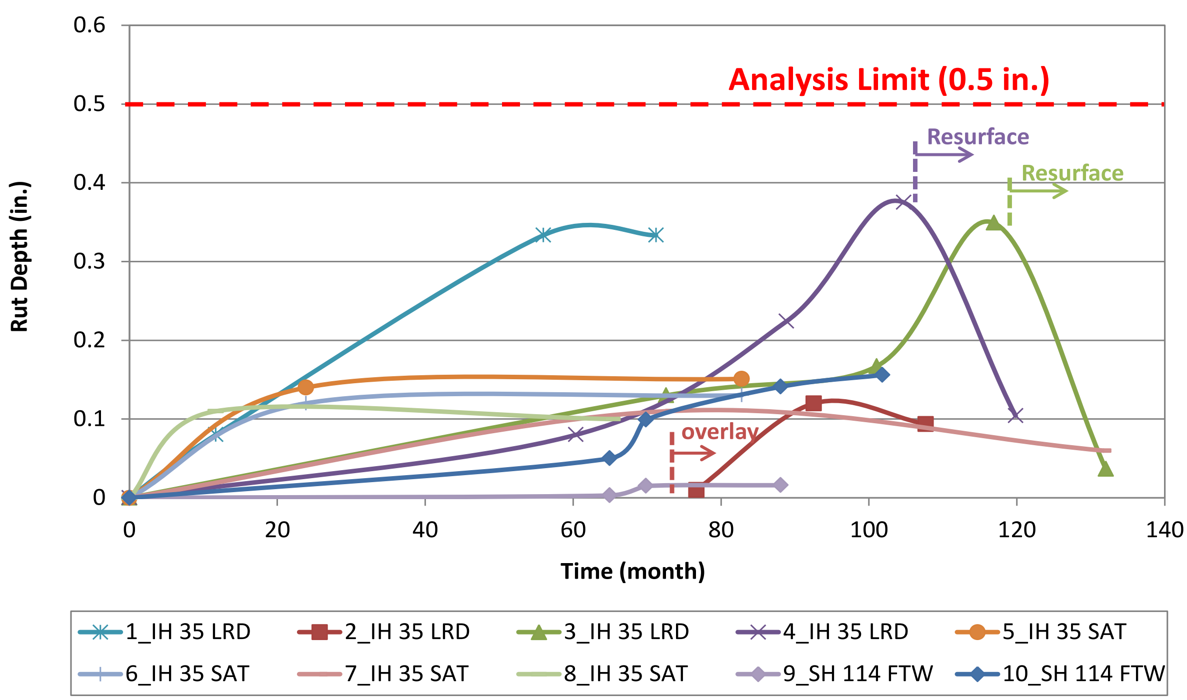


Figure 12. Surface Rutting History of Texas PPs.

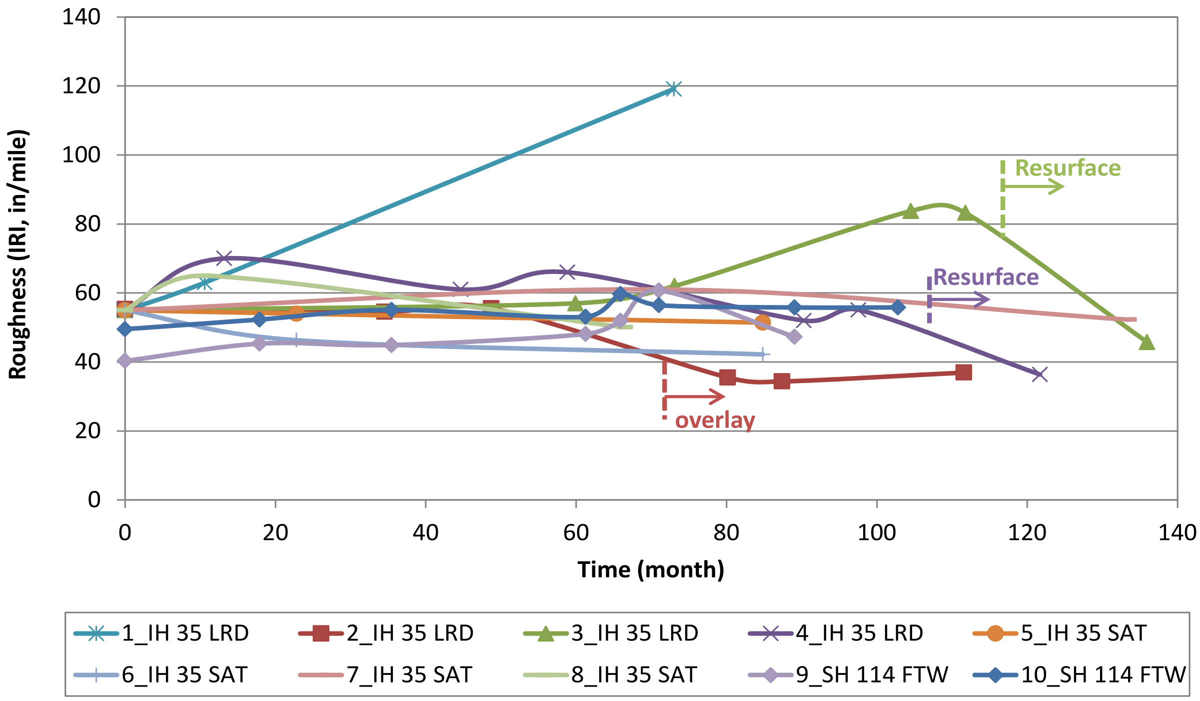


Figure 13. Roughness History of Texas PPs.

COMPARATIVE FIELD PERFORMANCE EVALUATION

For defensible performance-effectiveness justifications of PPs, researchers comparatively evaluated the performance prediction of all the in-service Texas PPs over conventional flexible and rigid pavements under the same traffic loading and climatic conditions.

PP versus Conventional Flexible Pavement

The TxME software developed by TxDOT was used to predict performance of perpetual and conventional flexible pavements, including thermal cracking, asphalt concrete fatigue cracking, and rutting failures. While the PP structures and material properties data required for TxME were collected from each in-service PP section, the structure of conventional flexible pavements was assumed as 6 in. HMA (Type D) surface, 6 in. flexible base, 4 in. lime treated subbase, and subgrade as illustrated in Figure 14. The performance predictions for both pavement systems were conducted under the same traffic loading and climatic conditions at each PP location. Also, to evaluate the life cycle of each pavement, the performance criteria (analysis limit) was used for each distress type, which are presented by the TxME software, as listed in Table 7.

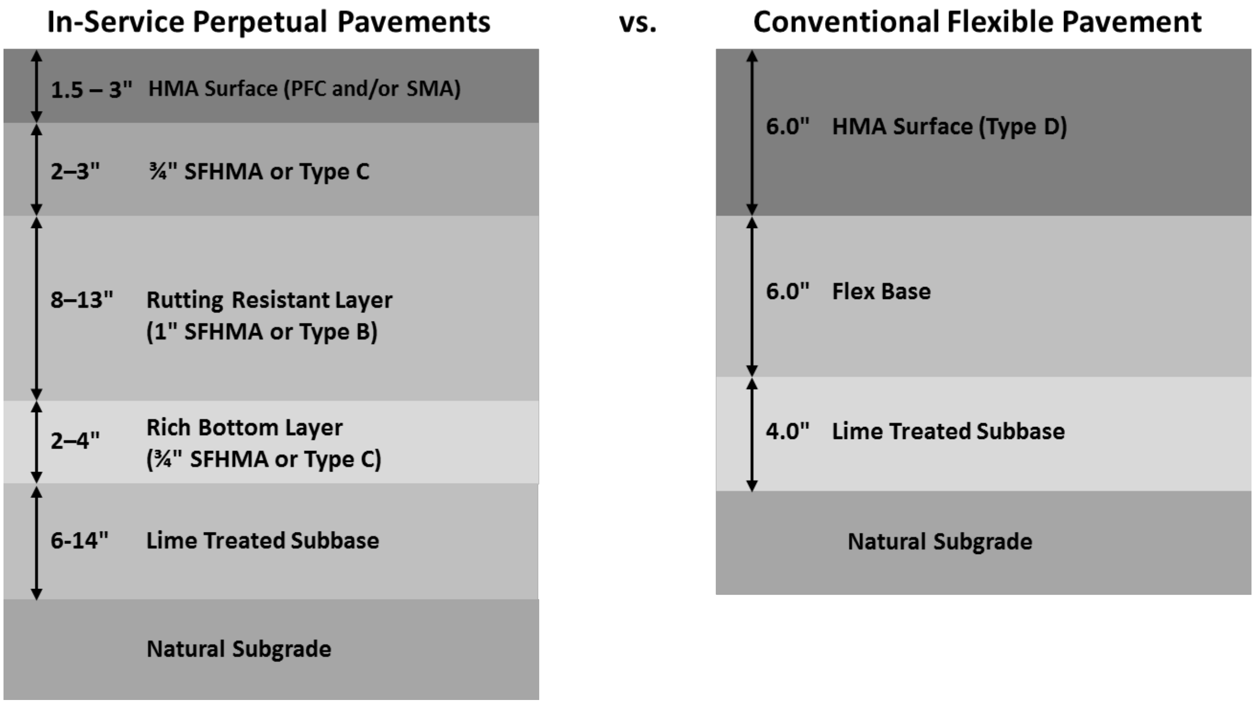


Figure 14. Pavement Structure for Performance Predictions (Flexible Pavement).

Table 7. Performance Criteria of Flexible Pavement System.

Pavement Performance (Distress)	Limit
Thermal Cracking	2,112 ft/mile
Fatigue Cracking of AC Layer	50 percent
Rutting	0.5 in.

Figure 15 shows an analytical example of performance predictions of both perpetual and conventional flexible pavements under the same traffic loading and climatic conditions at the IH 35 Cotulla (Laredo) location. From the evaluation of all performance predictions, researchers found that the PPs show superior performance to conventional flexible pavements under the same traffic loading and climatic conditions. While the TxME software predicted the PPs to last mainly for the design life (i.e., up to 50 years) without significant structural failures, the conventional flexible pavements were predicted to fail earlier with shorter service lives as follows:

- Thermal cracking (2,112 ft/mile): over 50 years.
- Fatigue cracking (≥ 50 percent): 3–15 years.
- Rutting (≥ 0.5 in.): 9–15 years.

Appendix IV presents all performance predictions of the perpetual and conventional flexible pavements evaluated in this study.

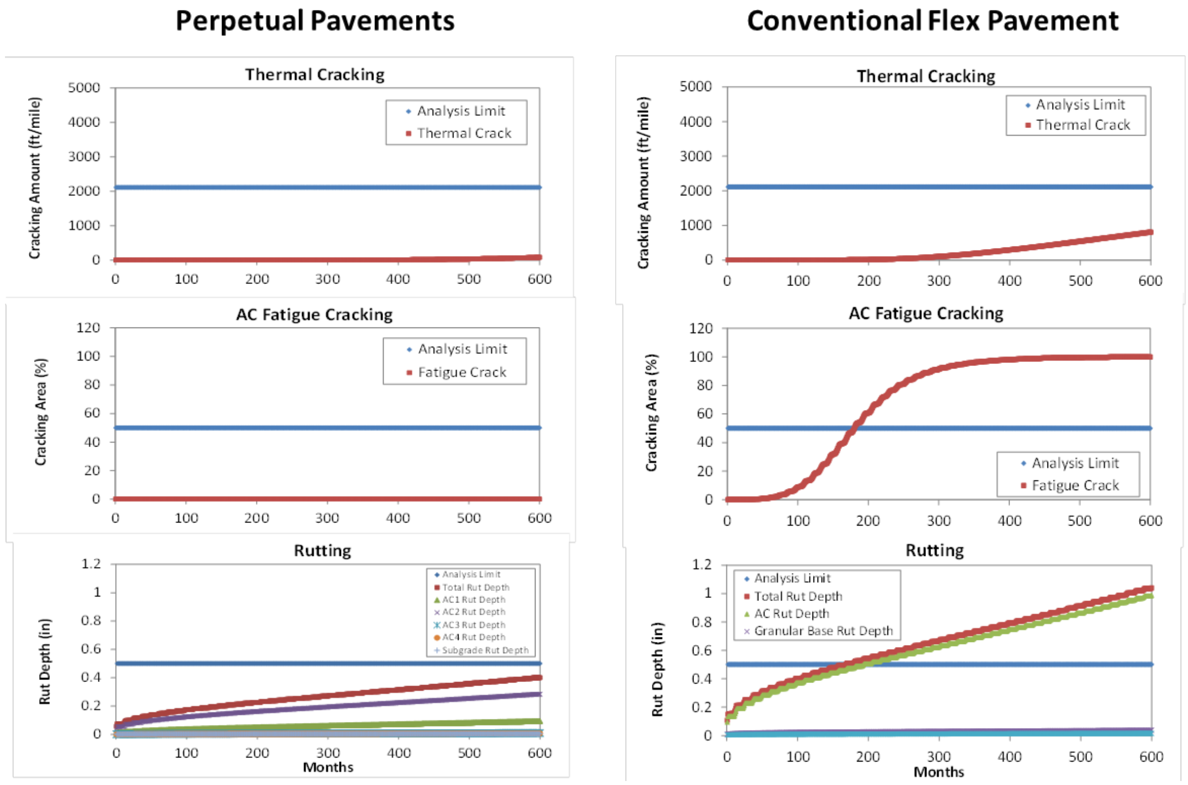


Figure 15. Performance Predictions; PP versus Flexible Pavement (IH 35 Cotulla, Laredo District).

PP versus Conventional Rigid Pavement

Researchers comparatively evaluated the performance prediction of the in-service PPs over conventional rigid pavement under the same traffic loading and climatic conditions. The AASHTOWare® Pavement M-E Design software was used to predict the distresses of rigid pavement including punchout and load transfer efficiency (LTE). As the same with the evaluation of flexible pavement, while the structures and material properties inputs for PPs were employed from each in-service section, the structure for conventional rigid pavements was a typical Texas structure, assumed as 11 in. of continuously reinforced concrete pavement (CRCP) surface on asphalt treated base, as illustrated in Figure 16. The analysis limit of punchout and LTE are based on the performance criteria recommended by AASHTOWare Pavement M-E Design software as follows:

- CRCP Punchouts: 10 per mile.
- Minimum LTE: 80 percent.

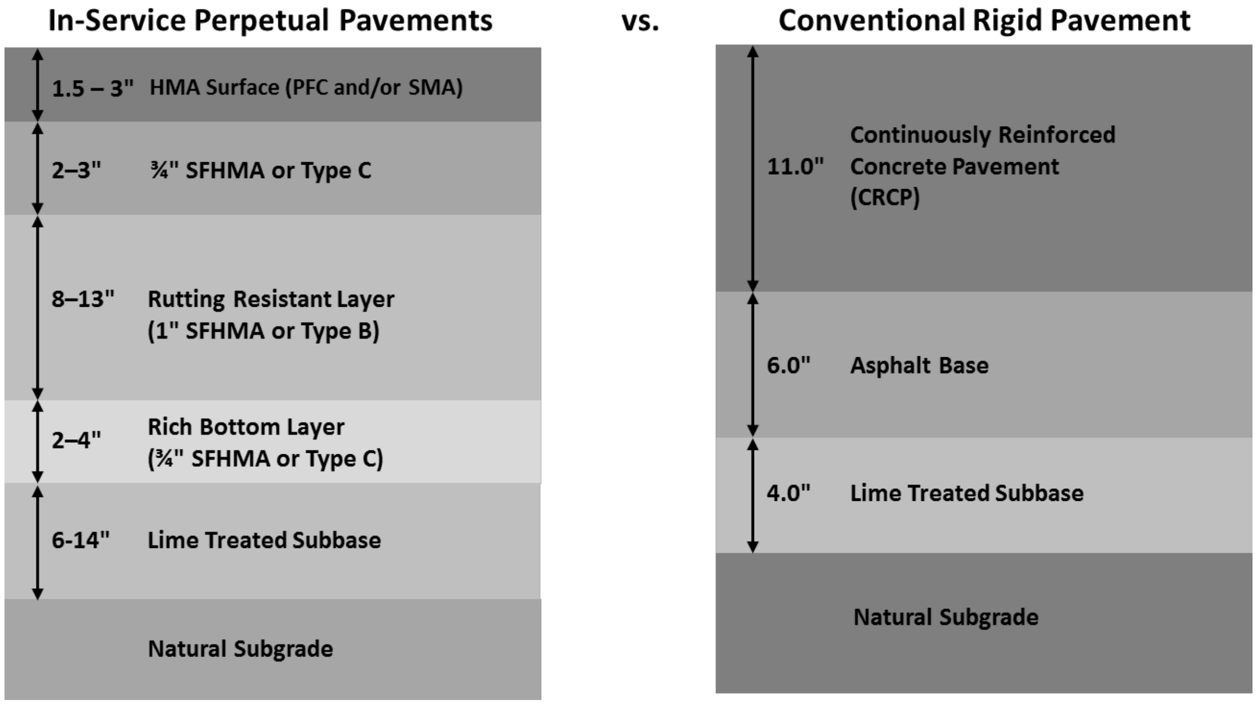


Figure 16. Pavement Structure for Performance Predictions (Rigid Pavement).

Figure 17 shows an analytical example of performance predictions of both pavement systems under the same traffic loading and climatic conditions on IH 35 in Cotulla (Laredo District). From the evaluation of all performance predictions, it was found that the PPs show comparable performance to conventional rigid pavements under the same traffic and climatic conditions. That is, while M-E design software predicted the PPs to last mainly for the 50 years of design life without significant structural failures, some conventional rigid pavements (CRCP) were predicted to reach the analysis limits earlier with relatively shorter service lives as follows:

- Punchout (10/mile): 20–50 years.
- LTE (80 percent): 20–30 years.

Appendix IV presents all comparative performance predictions of the perpetual and conventional rigid pavements.

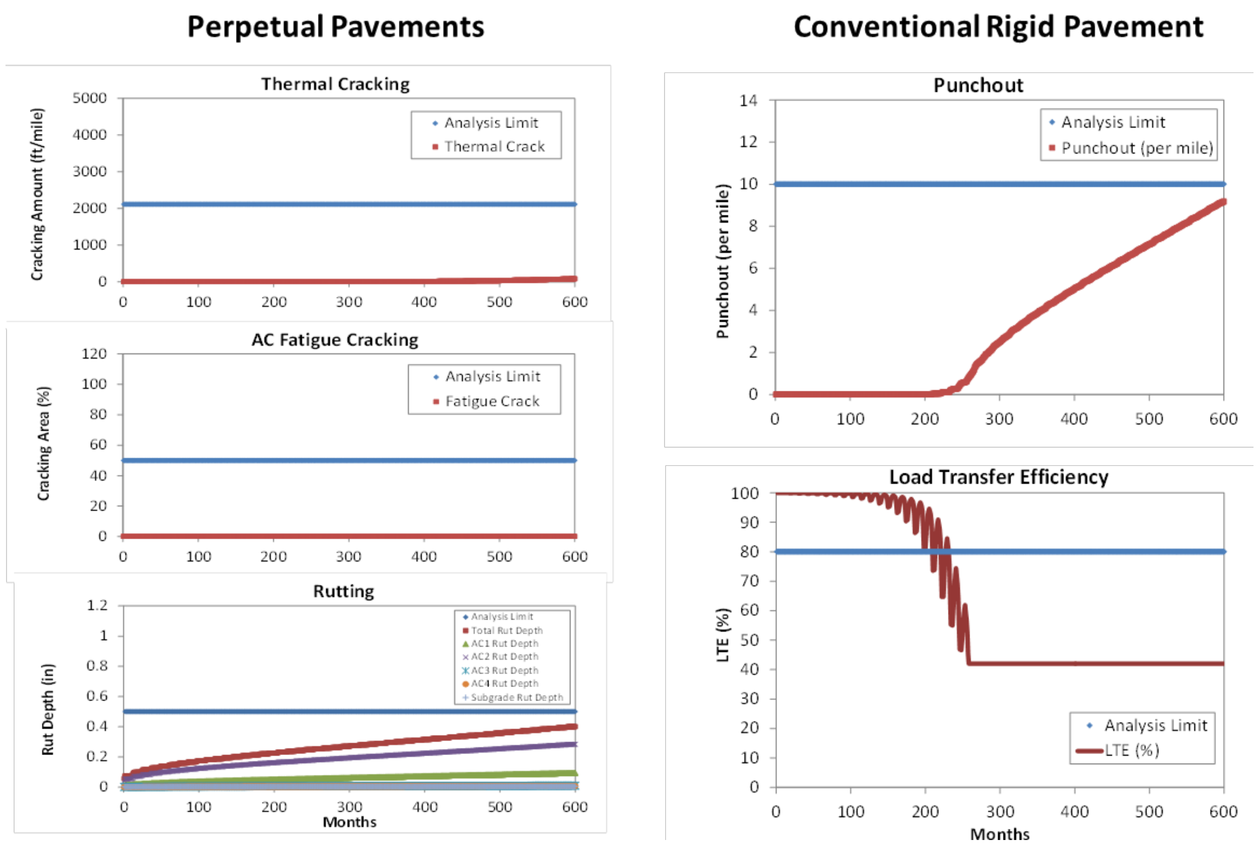


Figure 17. Field Performance Predictions: PP versus CRCP (IH 35 Cotulla, Laredo District).

SUMMARY

The findings from this chapter are summarized as follows:

- In Texas, 10 PP sections had been constructed since 2001 in four TxDOT districts, including Fort Worth, Laredo, San Antonio, and Waco. The majority of the pavement structures are conservatively thicker than minimum thickness with a total HMA layer and base thicknesses averaging 21 and 8 in., respectively.
- The PP section in IH 35 in Laredo District were overlaid after 6 to 10 years of service life due to illegal overweight traffic caused by oil activities in the Cotulla energy sector zone.
- The field performance of each Texas in-service PP section was evaluated using the Texas flexible pavement database from Project 0-6658. The PP sections' field performance data including rutting, surface cracks, and IRI/PSI are under analysis limit and still in good condition.
- The comparative performance prediction between the in-service PP and conventional pavements were conducted for defensible performance-effectiveness justifications of PP. From the comparative evaluation, the PPs showed superior performance to conventional flexible and rigid pavements under the same traffic loading and climatic conditions.

CHAPTER 4. LCCA OF PP

One of the key objectives of this research project is comparative LCCA of PPs versus conventional pavements to provide economic justification for the use of PPs. The LCCA is defined as a tool used to evaluate design alternatives to identify the one that may be the most cost effective to build and maintain. Thus, the LCCA can be used to compare the total agency (expenditures) and user costs of PPs against competing project alternatives such as traditional flexible and rigid pavements throughout the analysis period. The costs that are considered in LCCA are typically agency costs and user costs, described as:

- Agency costs: all expenditures the agencies pay within the project life (i.e., initial construction cost, M/R cost, and reconstruction cost).
- User costs: estimated costs of delaying the traffic during each activity (i.e., construction, maintenance) within the project life (i.e., value of user time [$/hour]).

PROCEDURE OF LCCA

The LCCA procedure consists of five steps, as illustrated in Figure 18. The process begins with the development of alternatives and then defines the schedule of each activity, such as initial construction, M/R, and reconstruction of each alternative. Next, the agency and user costs of these activities are estimated. The computation of life-cycle cost using the economic technique known as discounting is conducted to convert the costs into present dollars summed for each alternative. Finally, the agency can determine which alternative is the most cost-effective. The steps are ordered so that the analysis builds upon information gathered in prior steps (*8*).

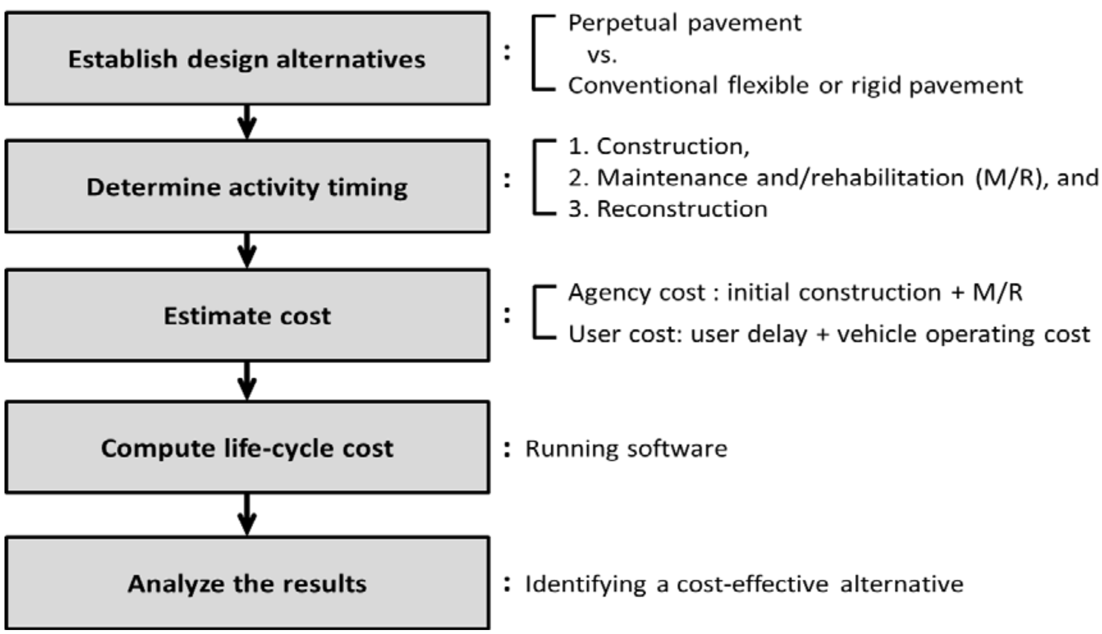


Figure 18. Life-Cycle Cost Analysis Step.

Step 1: Establish Design Alternatives

Alternative A and B pavements are established as project design alternatives.

Step 2: Determine Activity Timing

After the design alternatives have been established, M/R plan of each alternative should be developed. This plane is to schedule when the future M/R activities will occur and when agency funds will be expended (*8*). Figure 19 illustrates an example for the cycles of initial construction and M/R of two different alternatives.

- Alternative A: activities (rehabilitation) at t_{Ai}.
- Alternative B: activities (rehabilitation) at t_{Bi}.

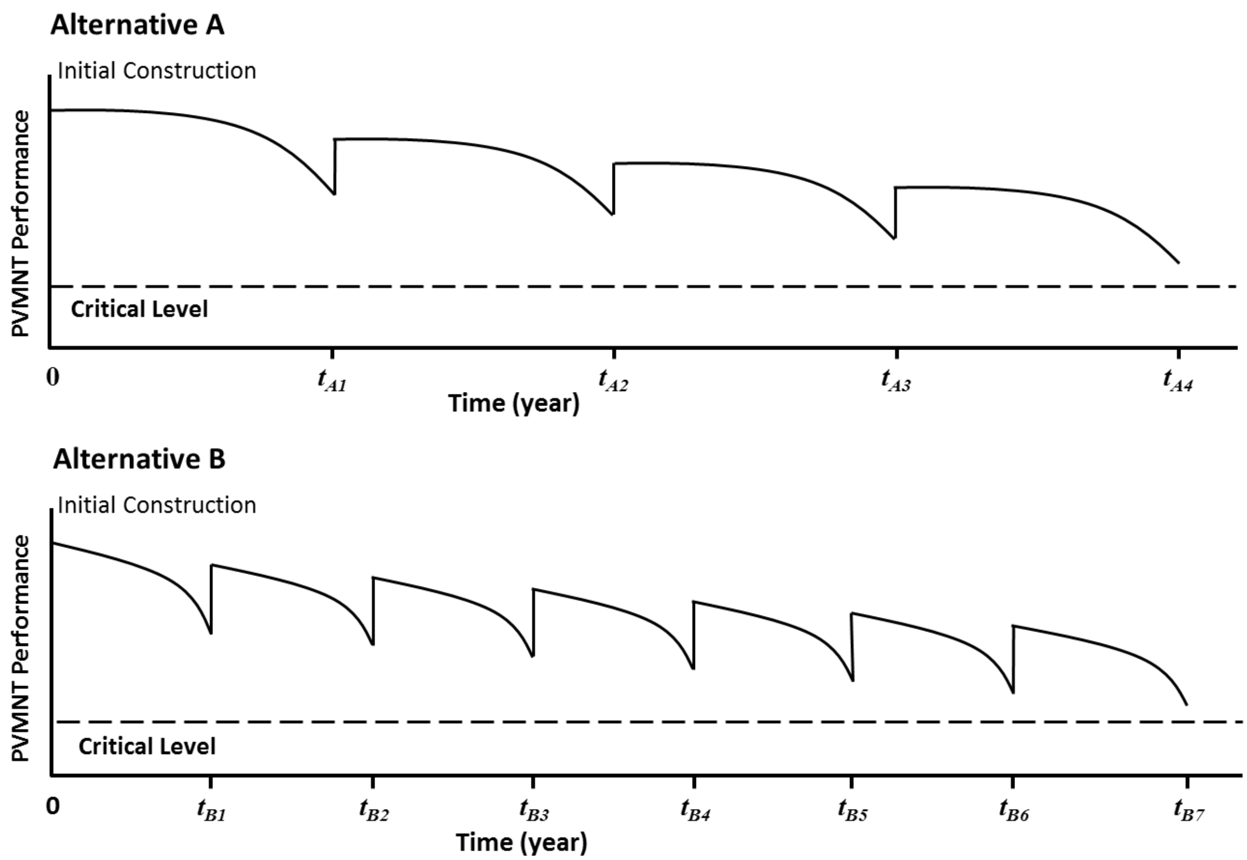


Figure 19. Activity Timing of Alternatives.

Step 3: Estimate Cost

The agency cost (initial construction, M/R, and reconstruction costs) of each alternative is determined based on the construction cost estimated using the historical cost data (i.e., bid prices of agency). The user cost is determined based on the value of time and road user costs by the agency.

Step 4: Compute Life-Cycle Cost

With the determined activity timing and costs, the total LCCs for each alternative were calculated using an LCCA tool or software.

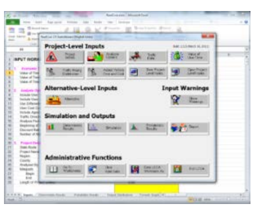
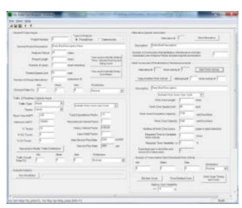
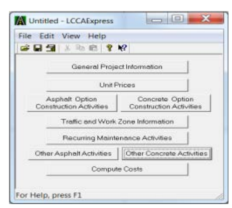
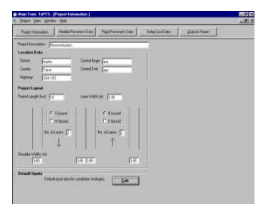
Step 5: Analyze Results

With the deterministic or probabilistic LCCs calculated, the present values of the differential costs are compared across competing alternatives.

LCCA SOFTWARE FOR PP DESIGN

As shown in Figure 18, an LCCA tool or software is needed to calculate the total LCC of each alternative so that they may be directly compared. However, since money spent at different times have different present values, the projected activity costs for an alternative cannot be simply added together to calculate the total LCC of that alternative (*8*). Hence, LCCA software needs to compute the present value of each alternative automatically. There are several LCCA tools that incorporate the LCCA methodology as it applies to pavement projects, including RealCost, LCCA Original, LCCAExpress, and Texas pavement type selection program (TxPTS). The software converts anticipated future costs to present dollar values so that the lifetime costs of pavement alternatives can be directly compared. To substantiate the LCCs of PPs having many activities during 50-year design life, researchers used the RealCost software developed by the Federal Highway Administration (FHWA) in comparison with conventional flexible and rigid pavements. The RealCost allow users to input the largest number of activities to each design alternative among the software. Table 8 presents the comparative evaluation of the LCCA software applicable for pavement project.

Table 8. Comparison of LCCA Software for Pavements.

Software	**RealCost**	**LCCA Original**	**LCCAExpress**	**TxPTS**
Interface Screen				
Institute	FHWA	APA	APA	TxDOT
Maximum No. of alternative & activities	- 6 alternative - 24-activity	- 4-alternative - 10-activity	- 2-alternatives (flexible vs. rigid) - 5-activity	Multi-alternatives - flexible vs. rigid) - 6-activity
Agency cost input option	Total agency cost	Total agency cost	Quantity of materials & unit cost used in each activity	Quantity of materials & unit cost used in each activity
Applicability for PP	Yes	Yes	No	Yes

COLLECTION OF CONSTRUCTION COST DATA FOR LCCA

With the assistance of TxDOT Construction Division, researchers collected the unit costs for construction and maintenance and estimated the construction cost required to perform LCCA of the PP and conventional flexible and rigid pavements.

HMA Materials

The unit price of HMA materials were obtained from the 2017 TxDOT average low bid unit prices. However, due to the lake of cost data, the unit costs of SFHMA materials were estimated through the historical bid prices averaged from 2004 to 2010, provided by TxDOT Construction Division. Using the unit price (dollar per ton) of each material, the construction cost required to place 1 in. thick per 1 mile (dollar/1 in./mile) were calculated with theoretical maximum specific gravity (commonly referred to as rice gravity) and target density. The rice gravity and target density of each HMA material were obtained from the available data from studies 0-4822 and 0-6658 and the TxDOT standard specification, respectively (*4*, *7*). These construction costs were also used for the M/R activities including overlay and reconstruction.

Table 9 presents the construction costs estimated for HMA mixes.

Table 9. Construction Cost of HMA Materials.

Material Type	PG	Avg. Bid Price		Rice Value	Target Density	HMA Density	Construction Cost
		$/ton	$/lb	pcf	%	pcf	$/1" thick./mile
PFC	PG 76-22	97.98	0.0444	143.58	80	114.87	26,955
SMA	PG 76-28	106.53	0.0483	149.83	96	143.83	36,697
	PG 76-22	102.49	0.0465				35,306
3/4" SFHMAC*	PG 76-22	36.31	0.0165	148.00	96	142.08	12,355
	PG 70-22	38.30	0.0174				13,033
1" SFHMAC*	PG 76-22	36.31	0.0165	148.00	96	142.08	12,355
	PG 70-22	39.00	0.0177				13,271
½" SP	PG 70-22	77.70	0.0352	148.00	96	142.08	26,440
	PG 64-22	71.80	0.0326				24,432
Type D	PG 70-22	82.11	0.0372	149.83	97	145.33	28,580
	PG 64-22	75.94	0.0344				26,432
Type C	PG 70-22	64.20	0.0291	149.83	97	145.33	22,346
	PG 64-22	65.00	0.0295				22,624
Type B	PG 64-22	60.12	0.0273	154.82	97	150.18	21,623

*Data from 2004 to 2010
Legend: PG = performance grade

Concrete Materials (CRCP)

The agency cost information for conventional rigid pavement was estimated based on the TxDOT average bid unit price of CRCP. The agency costs for CRCP consists of the construction (Item 360) and the concrete pavement repair (Item 361) that is full-depth repair. Table 10 lists the agency cost information including initial construction and repair costs. The construction cost listed in Table 10 is only for the surface of rigid pavement (CRCP).

Table 10. Construction Cost of CRCP.

Thickness (in.)	Construction (Item 360)		Full Depth Repair (Item 361)	
	Avg. Bid Price ($/SY)	Construction Cost ($/mile/lane)	Avg. Bid Price ($/SY)	Repair Cost ($/mile/lane)
7	85.31	600,580	215.00	1,513,600
8	42.07	296,181	291.10	2,049,344
9	39.43	277,556	310.00	2,182,400
10	47.99	337,860	259.99	1,830,330
11	48.23	339,545	189.80	1,336,192
12	52.27	367,972	301.67	2,123,757
13	48.83	343,756	314.49	2,214,010
14	37.34	262,878	245.00	1,724,800
15	58.91	414,705	377.57	2,658,093

Base Materials

The initial construction cost for base and subbase layers were also estimated with the average bid prices provided by TxDOT. To calculate the amount of loose material required in the pavement structure, the maximum density was assumed as 134 pcf for flexible and lime or cement treated base materials and 145 pcf for asphalt treated base material. Table 11 and Table 12 present the construction costs of the flexible and lime- and cement-treated base and the asphalt treated base materials, respectively.

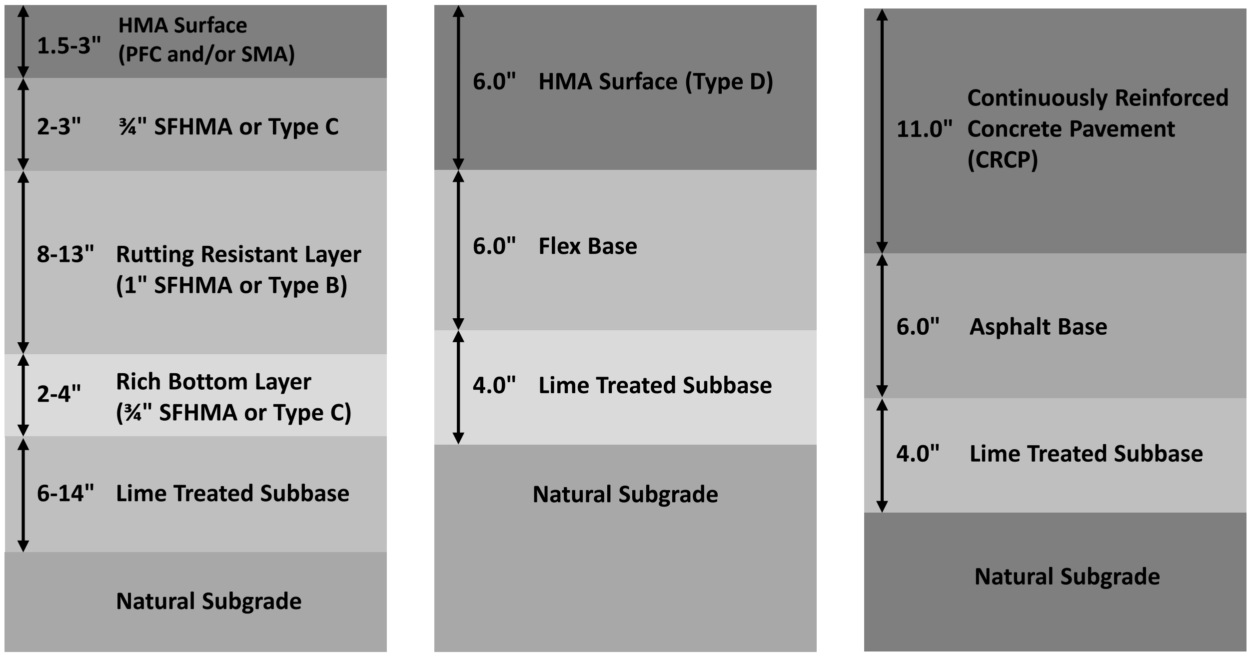
Table 11. Construction Cost of Flexible and Lime- and Cement-Treated Base.

Material Type	Additive Content	Avg. Bid Price		Density	Construction Cost
		Aggregate	Additive		
	%	$/ton	$/ton	pcf	$/1" thick/mile
Flexible	-	28.00	-	134	8,986
Cement	2	28.00	125	134	9,788
	3				10,189
	4				10,591
Lime	2	28.00	152.09	134	9,962
	3				10,450
	4				10,938
	6				11,914

Table 12. Construction Cost of Asphalt-Treated Base (Item 292).

Material Type	HMA Density	Avg. Bid Price		Construction Cost
	(pcf)	($/ton)	($/lb)	$/1" thick/ mile
Grade 1_PG64	145.0	90.94	0.041	31,581
Grade 2_PG64		64.86	0.029	22,523
Grade 4_PG64		59.54	0.027	20,675
Grade 4_AC 1.5		180.00	0.082	62,509

Using the construction costs presented from Table 9 to Table 12, all agency costs including initial construction cost and M/R cost of the pavement alternatives were calculated based on material type and thickness of each pavement layer illustrated in Figure 20.



(a) PP **(b) Flexible Pavement** **(c) Rigid Pavement (CRCP)**

Figure 20. Pavement Structures for LCCA.

ACTIVITY TIMING OF PAVEMENTS

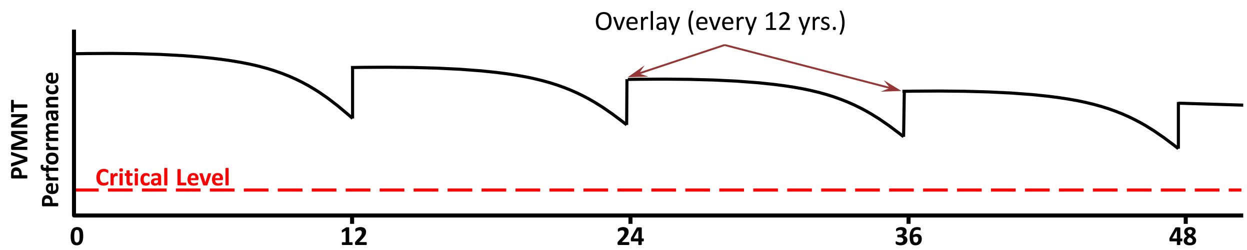
After initial construction of perpetual, flexible, and rigid pavements, the M/R plans were developed. While the timing of M/R activities should be determined based on evaluation of performance condition of each alternative, the judgment of experienced engineers can be used when actual data are unavailable or not applicable (*8*). Thus, in this study, the activity plans such as overlays or repairs after initial construction were defined based on the recommendation of TxDOT engineers while the analysis period of the pavements was set to 50 years. For the conventional flexible pavement, the activity plans were set to two scenarios: 1) 2 in. overlay every 4 years and 2) 2 in. overlay 8 years. Because the surface of conventional flexible pavement is affected by traffic load and/or climate condition, researchers made the two scenarios that the pavement in Scenario 1 is damaged quickly with higher traffic load and/or severe weather condition and the damage of Scenario 2 pavement occurs slowly. The reconstruction will be applied every 20 years for both scenarios. On the other hand, for PP and CRCP alternatives, the M/R is applied with 2 in. overlay every 12 years and full-depth repair every 30 years, respectively. The activity timings of all pavement alternatives are as follows:

- PP: 2 in. overlay every 12 years and reconstruction every 50 years.
- Conventional flexible pavement.
  - Scenario 1: 2 in. overlay every 4 years and reconstruction every 20 years.
  - Scenario 2: 2 in. overlay every 8 years and reconstruction every 20 years.
- Conventional rigid pavement (CRCP): full-depth repair every 30 years.

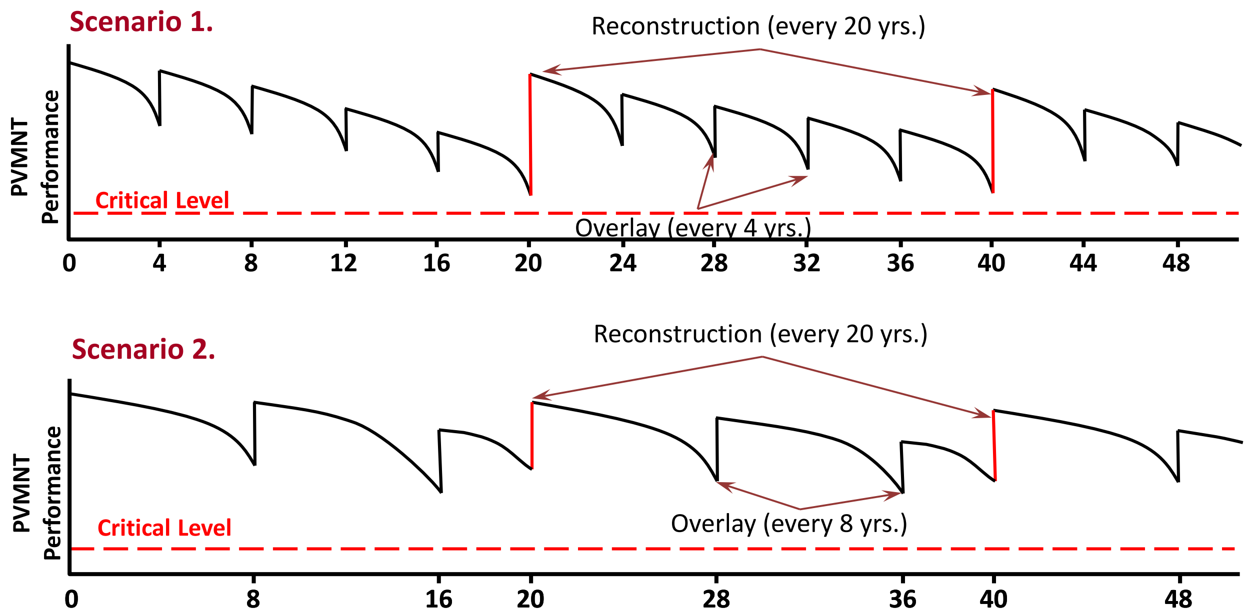
Table 13 and Figure 21 present all parameters used for LCCA, including analysis period, activities plans, and cost data and the cycle of construction and M/R activities of each pavement alternative, respectively.

Table 13. Input Parameters for LCCA.

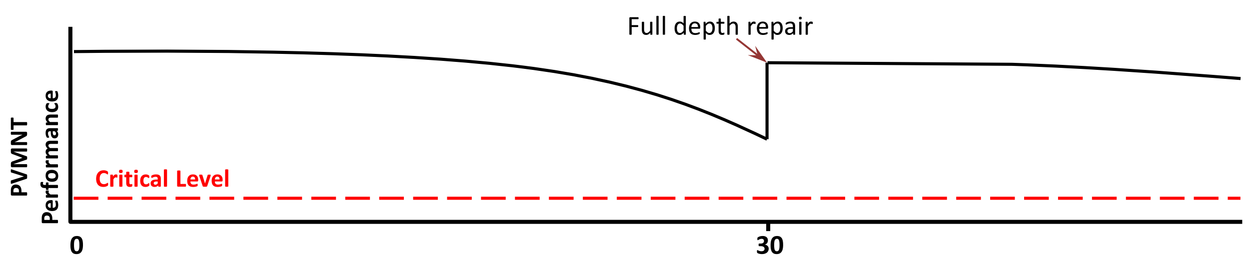
Items		**Alternative**			
		Perpetual	**Flexible**		**Rigid (CRCP)**
			Scenario 1	**Scenario 2**	
Analysis period		50 years			
Interval of activities	Maintenance/ Rehabilitation	12 years (2 in. overlay)	4 years (2 in. overlay)	8 years (2 in. overlay)	30 years (full-depth repair)
	Reconstruction	50 years	20 years	20 years	-
Number of activities	M/R	4	10	5	1
	Reconstruction	0	2	2	0
Construction and M/R cost		Estimated based on TxDOT average bid price			
User cost	Value of user time	Passenger car: \$22.09/hour, Trucks: \$32.26/hour			
Traffic data		Traffic data collected at each section			
Discount rate		4.0%			



(a) PP



(b) Conventional Flexible Pavement



(c) Conventional Rigid Pavement

Figure 21. Activity Plans of Pavement Alternatives.

COMPARATIVE LCCA

With the deterministic LCC calculated using the RealCost software developed by FHWA, the present values of the perpetual and conventional pavements were compared across competing alternatives and activity timing of pavements.

Scenario 1: Overlay Every 4 Years for Conventional Flexible Pavement

In Scenario 1, the conventional flexible pavement was set to 2 in. overlay every 4 years, assuming that the pavement is damaged quickly due to higher traffic load and/or severe weather condition. As an example (IH 35 Cotulla, Laredo District) shown in Figure 22, the PP has higher initial construction costs due to its thicker, multiple HMA layers but lower total agency and user costs than the conventional flexible pavement. On the other hand, the PP has lower agency costs and comparable user costs to the conventional rigid (CRCP) pavement, because the rigid pavement has the highest initial construction and M/R cost even though it has only one M/R activity during the analysis period (50 years). Appendix V presents the LCCA results for comparing the existing PPs to the conventional flexible (overlaid every 4 years) and rigid pavements.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,583.00*	*$ 1,310.79*	*$ 5,161.00*	*$ 4,617.58*	*$ 7,628.00*	*$ 1,288.91*
Present Value	**$ 2,036.63**	**$ 1,076.02**	**$ 2,605.88**	**$ 1,751.41**	**$ 3,930.96**	**$ 1,085.96**

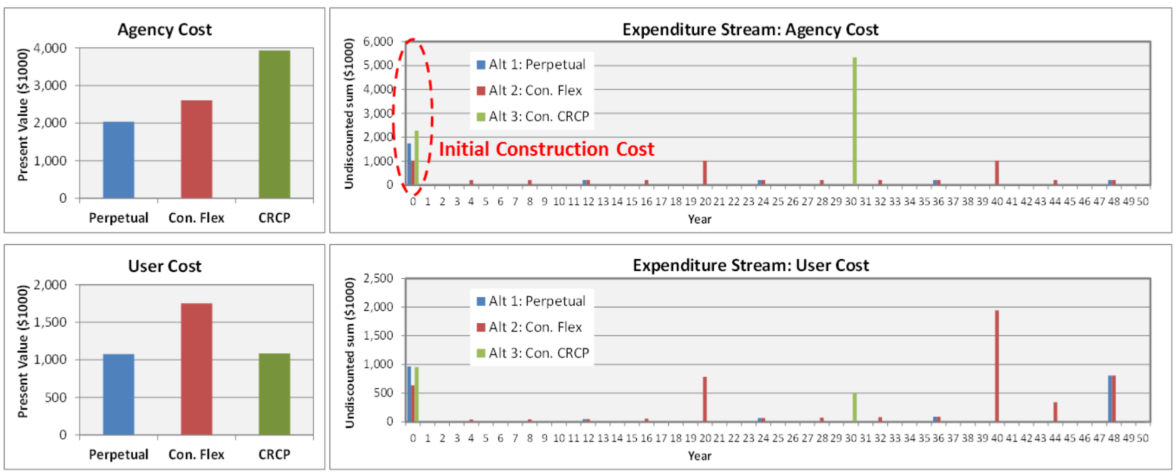


Figure 22. LCCA: Perpetual vs. Flexible (Overlaid Every 4 Years) vs. Rigid Pavements (IH 35 Cotulla).

Table 14 and Figure 23 show the cost comparison of each PP section with conventional pavements and the percentage-wise comparison using the conventional flexible pavement as a reference, respectively. From these comparisons, it is indicated that the PPs have higher cost-effectiveness than conventional flexible (overlaid every 4 years) and rigid pavements have during their life cycle (50 years) due to lower agency and user costs. The cost level of these pavements can be compared as follows:

- Initial construction cost: conventional rigid > perpetual > conventional flexible.
- Total agency cost: conventional rigid > conventional flexible > perpetual.
- User cost: conventional flexible > perpetual > conventional rigid.

Table 14. LCCA of Pavement Alternatives (Scenario 1).

Section No.	Initial Cost ($1,000)			Total Agency Cost ($1,000) (Initial + M&R + Recon)			User Cost ($1,000)		
	PP	Flexible	Rigid	PP	Flexible	Rigid	PP	Flexible	Rigid
1	2,270	1,526	3,425	2,717	3,912	5,897	1,487	2,509	1,500
2	1,627	1,017	2,283	1,925	2,606	3,931	1,160	1,759	1,182
3	1,739	1,017	2,283	2,037	2,606	3,931	1,076	1,751	1,086
4	1,722	1,017	2,283	2,020	2,606	3,931	1,240	2,356	1,147
5	2,733	1,526	3,424	3,180	3,912	5,896	253,064	434,953	211,419
6	2,902	1,526	3,424	3,349	3,912	5,896	253,064	434,953	211,419
7	3,577	1,526	3,424	4,024	3,912	5,896	636,323	646,532	640,142
8	3,930	2,034	4,567	4,527	5,216	7,863	23,203	34,561	22,307
9	2,429	1,017	2,283	2,721	2,606	3,931	536	707	547
10	1,721	1,017	2,283	2,019	2,606	3,931	536	707	547
AVG	2,465	1,322	2,968	2,852	3,389	5,110	117,169	156,079	109,130

Legend: Flexible = conventional flexible pavement; Rigid = conventional rigid pavement (CRCP); Recon = reconstruction; AVG = average

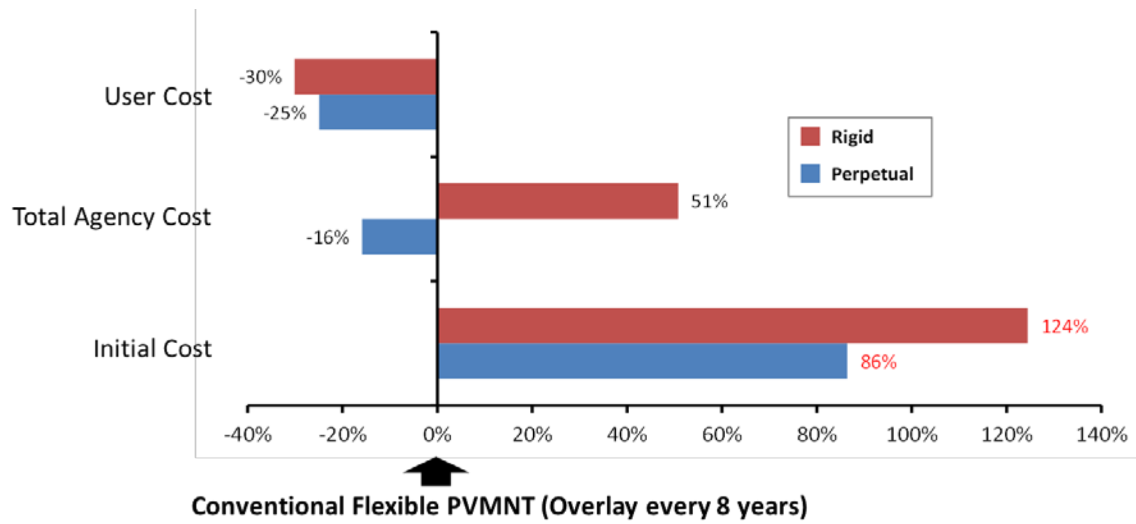


Figure 23. Comparison of LCCA Based on Flexible Pavement (Overlaid Every 4 Years) as Reference Base.

Scenario 2: Overlay Every 4 Years for Conventional Flexible Pavement

In Scenario 2, the activity of overlaying every 8 years was applied to the convention flexible pavement, assuming that the damage on the surface occur slowly due to comparatively lower traffic load and/or a moderate weather condition. As compared to Scenario 1 (Figure 22), the LCCA of Scenario 2 indicated, as illustrated in Figure 24, that the PP has a similar total agency cost to the conventional flexible pavement due to less M/R activities of the conventional pavements overlaid every 8 years. However, the user cost of PP is still lower than the flexible pavement. Appendix VI presents all LCCA results comparing the existing PPs to the conventional flexible (overlaid every 8 years) and rigid (CRCP) pavements.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,583.00*	*$ 1,310.79*	*$ 4,106.00*	*$ 3,831.50*	*$ 7,628.00*	*$ 1,288.91*
Present Value	**$ 2,036.63**	**$ 1,076.02**	**$ 2,113.70**	**$ 1,545.95**	**$ 3,930.96**	**$ 1,085.96**

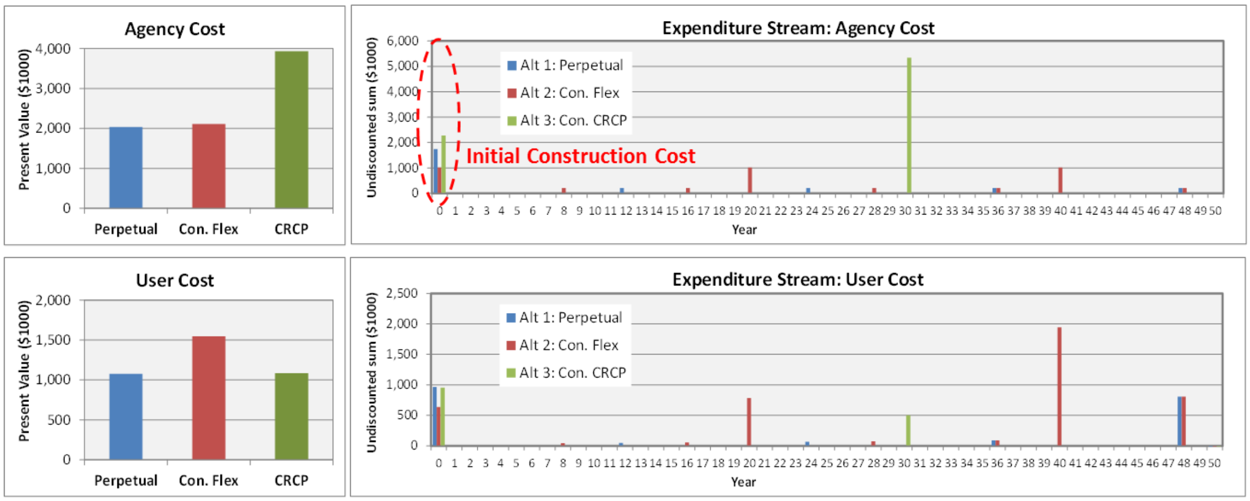


Figure 24. LCCA: Perpetual vs. Flexible (Overlaid Every 8 Years) vs. Rigid Pavements (IH 35 Cotulla).

Table 15 and Figure 25 show the cost comparisons of all PP sections with conventional pavement alternatives. The comparisons present that averaged total agency cost of PPs is 5 percent higher than that of conventional flexible pavements. This is due to lower M/R costs of the conventional flexible pavement overlaid every 8 years. However, the user cost of PPs is much lower than ones of conventional flexible and rigid pavements even in Scenario 2. The cost level of these pavements can be compared as follows:

- Initial construction cost: conventional rigid > perpetual > conventional flexible.
- Total agency cost: conventional rigid > perpetual > conventional flexible.
- User cost: conventional flexible > perpetual > conventional rigid.

Table 15. LCCA of Pavement Alternatives (Scenario 2).

Section No.	Initial Cost ($1,000)			Total Agency Cost ($1,000) (Initial + M&R + Recon.)			User Cost ($1,000)		
	PP	Flexible	Rigid	PP	Flexible	Rigid	PP	Flexible	Rigid
1	2,270	1,526	3,425	2,717	3,172	5,897	1,487	2,235	1,500
2	1,627	1,017	2,283	1,925	2,114	3,931	1,160	1,564	1,182
3	1,739	1,017	2,283	2,037	2,114	3,931	1,076	1,546	1,086
4	1,722	1,017	2,283	2,020	2,114	3,931	1,240	2,066	1,147
5	2,733	1,526	3,424	3,180	3,172	5,896	253,064	354,569	211,419
6	2,902	1,526	3,424	3,349	3,172	5,896	253,064	354,569	211,419
7	3,577	1,526	3,424	4,024	3,172	5,896	636,323	715,112	640,142
8	3,930	2,034	4,567	4,527	3,807	7,863	23,203	31,431	22,307
9	2,429	1,017	2,283	2,721	2,114	3,931	536	644	547
10	1,721	1,017	2,283	2,019	2,114	3,931	536	644	547
AVG	2,465	1,322	2,968	2,852	2,706	5,110	117,169	146,438	109,130

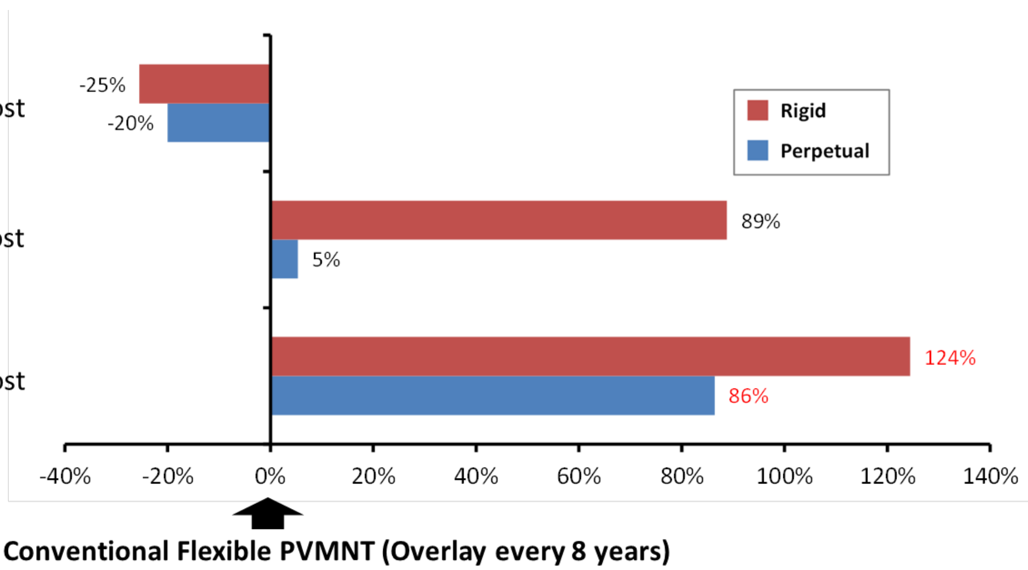


Figure 25. Comparison of LCCA Based on Flexible Pavement (Overlaid Every 8 Years) as Reference Base.

SUMMARY

In this chapter, the LCCA was conducted to compare the total agency (expenditures) and user costs of PPs against competing project alternatives such as traditional flexible and/or rigid pavements under the same traffic condition. For the comparisons, the activity plans of conventional flexible pavement were set two scenarios of overlay ever 4 and 8 years. The LCCA for both scenarios shows:

- The LCCA of conventional rigid (CRCP) pavement indicated the highest agency cost during the analysis period due to expensive initial construction and repair costs while it has lowest user cost.
- The LCCA of conventional flexible pavement indicated that its agency cost was higher in Scenario 1 and comparable in Scenario 2 to the PP. However, the user cost was the highest due to frequent M/R activities for both scenarios.
- The LCCA shows higher cost-effectiveness of PP compared to the conventional flexible and rigid pavements during their life cycle (50 years) due to lower and/or comparable agency and user costs.

CHAPTER 5. ENHANCEMENT OF M-E DESIGN FOR TEXAS PP

To date, there are more than 10 in-service PP sections that are currently being monitored and evaluated for data population in the Texas flexible pavements and overlays database (*6*, *7*). Also, the TxDOT routinely collects traffic data using permanent WIM stations on some of the PP sections. Thus, it is possible to validate and calibrate the EL concept and enhance PP design method using the actual measured performance and traffic data along with representative material properties and climatic conditions.

As outlined in the Project 0-6856 work plans, the primary goal of Task 3 was to identify a candidate EL determination approach and enhance the existing PP M-E design software. With the activities, this chapter provides an update of the work completed in Task 3 of this study. As documented in this chapter, the following outcomes were generated:

- Comprehensive review of EL for PP.
- Recommendation of EL determination approach and test protocol.
- Default EL values for Texas typical mixtures.
- Documentation of the enhancement of existing PP M-E design system.

COMPREHENSIVE REVIEW OF EL FOR PP

For PPs, it is expected that bottom-up fatigue cracking does not occur if the strain level is below the HMA fatigue EL. Therefore, additional pavement thickness, greater than that required for keeping strains below the EL, would not provide additional life. This concept has significant design and economic implications.

EL and Lab Study

The concept of an EL is widely recognized in many areas of materials science, especially that of ferrous metals. Barret et al. described the EL for metals as being a stress below which for un-cracked materials, the plot of stress versus cycles to failure becomes essentially horizontal and fatigue does not occur (*9*). Although this limit has been extensively studied and defined in metal and other material areas, relatively less work was done for HMA, a typical viscoelastic material. For PPs, there is a belief that bottom-up fatigue cracking does not occur if the strain level is below the HMA fatigue EL. Monismith and McLean first demonstrated the existence of a fatigue EL below which asphalt mixtures tend to have an extraordinarily long fatigue life and proposed an EL of 70 micro-strains for asphalt pavements (*10*). The log-log relationship between strain and bending cycles converged below 70 micro-strains at approximately 5 million cycles as shown in Figure 26. Maupin and Freeman noted a similar convergence (*11*). Nunn in the United Kingdom (UK) and Nishizawa et al. proposed concepts for long-life pavements for which classical bottom-up fatigue cracking would not occur (*12*, *13*). Nishizawa et al. reported an EL of 200 micro-strains based on the analysis of in-service pavements in Japan (*13*). Similarly, strain levels at the bottom of the asphalt layer of between 96 and 158 micro-strains were calculated based on backcalculated stiffness data from the FWD for a long-life pavement in Kansas (*14*). Other engineers proposed that one should limit the strain anywhere from 60 to 100 micro-strains based upon laboratory testing (*15*). Another experimental pavement project allowed PP design to reach the less conservative value of 125 micro-strains (*16*).

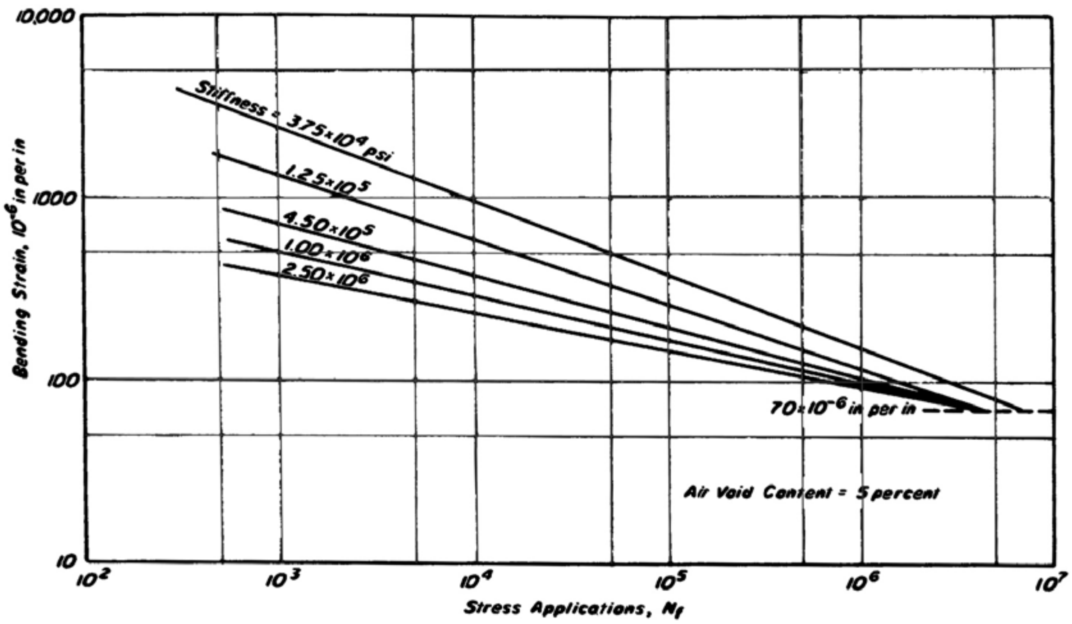


Figure 26. Strain vs. Stress Applications to Failure Relationships (*10*).

The National Center for Asphalt Technology (NCAT) has led a research effort for National Cooperative Highway Research Program (NCHRP) Project 9-38 to investigate the EL for HMA (*17*). This study involved conducting fatigue tests for a number of mixtures over a wide range of strain levels. Tests have been conducted that have required up to 50 million cycles to failure. The Asphalt Institute has also been involved in the portion of the work to test samples having fatigue lives up to 50 million cycles. The primary objectives of that study were to determine if HMA mixtures do have an EL and to provide guidance on determining this limit for various mixture types. The results indicated that since the EL varies with HAM mix types, there is not just one limit that can be used for all mixes.

Most recently, NCHRP Project 09-44A further identified that the EL varies with mixture properties, temperature, and pavement design conditions with the following findings (*18*):

- The EL varies depending on binder grade, binder content, air voids, temperature, and the rest period between load applications.
- Mixtures using softer binders exhibit higher ELs than mixtures using stiffer binders. High binder contents and low air voids produced high EL values compared to low binder contents and high air voids, which showed low ELs.
- EL values were higher at high temperatures, which correspond to soft mixtures compared to low temperatures that correspond to stiff mixtures.
- HMA stiffness (modulus) was found to be an excellent surrogate property that takes into account all of the primary mix variables: binder grade, binder content, air voids, and temperature. This concept, however, needs to be used carefully since air voids and binder content can counteract each other and create the same stiffness but may have different ELs.

EL and Field Measured Strain Distribution

Although laboratory testing showed that HMA mixtures may have an EL, further verification and validation was still needed from field strain measurements. In this respect, a study at the NCAT Pavement Test Track provided very interesting insights. The NCAT Pavement Test Track is comprised of 46 experimental test sections in Opelika, Alabama. In 2000, all 46 sections were built with a minimum thickness of 23 in. of bituminous material to help control the potential for bottom-up fatigue cracking (*19*). At the conclusion of the first experiment (10 million ESALs), no fatigue cracking had been observed at any of the 46 sections.

After the 2000 test cycle, many sections were rebuilt to cater to other investigative needs. When the 2003 NCAT Test Track experiment began, many of the original test sections were left in-place to receive another 10 million ESALs of traffic. The additional traffic did not prove detrimental to the pavement structure in terms of fatigue cracking, which was still not observed after 20 million ESALs of traffic. Compare to the 2000 Test Track, the eight sections from the 2003 Test Track were considerably thinner ranging from 5 to 9 in. of total HMA (Figure 27).

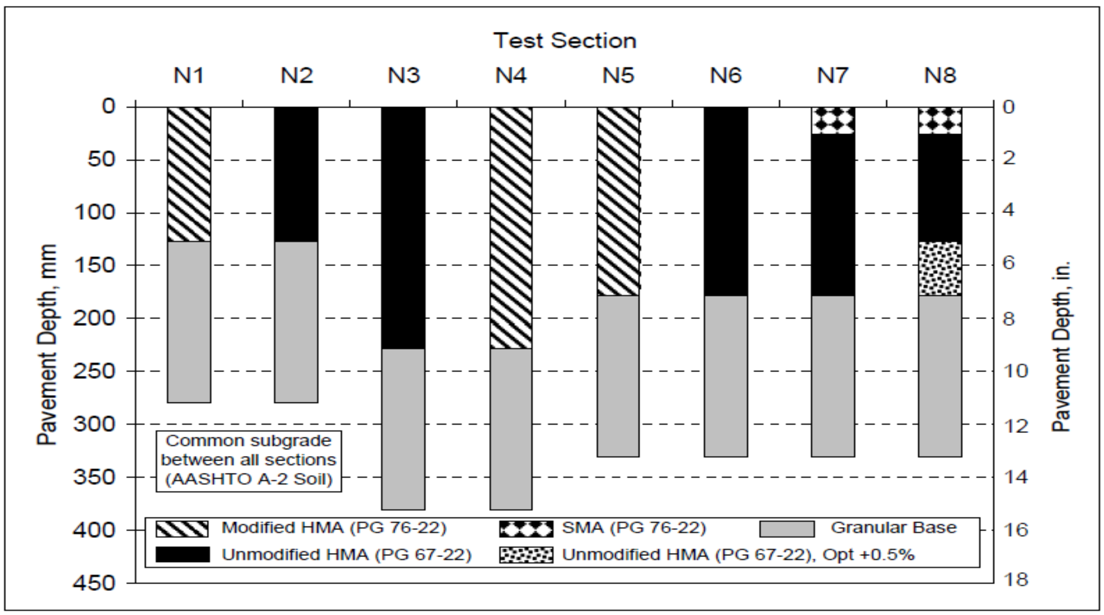


Figure 27. Structural Sections at the 2003 NCAT Test Track (*19*).

Figure 28 shows the cumulative distributions of the estimated strain values for 2003 NCAT structural sections. Section N8 was originally designed to cater to other investigating needs and was excluded. While five sections experienced fatigue cracking: N1, N2, N5, N6, and N7, other two sections (N3 and N4) did not show signs of fatigue cracking.

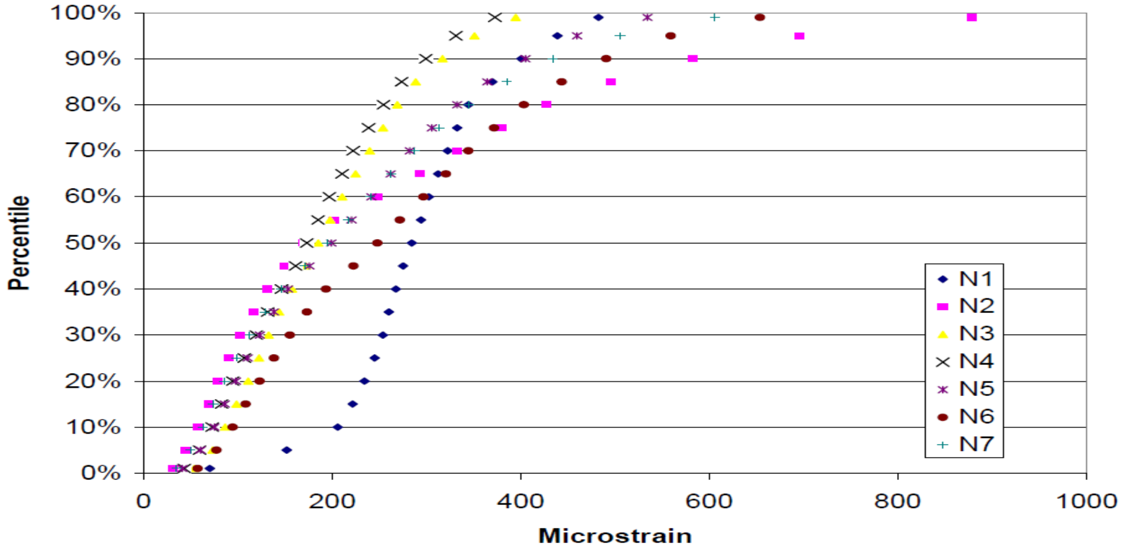


Figure 28. Cumulative Distribution of Strains for 2003 NCAT Sections (*19*).

The third experiment at the NCAT Test Track began to traffic the pavement on November 10, 2006. At this point in time, only eight of the original 2000 Test Track sections remained in-place. Of those sections, as of December 4, 2008, 30 million ESALs had trafficked over these eight test sections and signs of fatigue cracking have yet to be witnessed. Figure 29 shows the strain cumulative distributions.

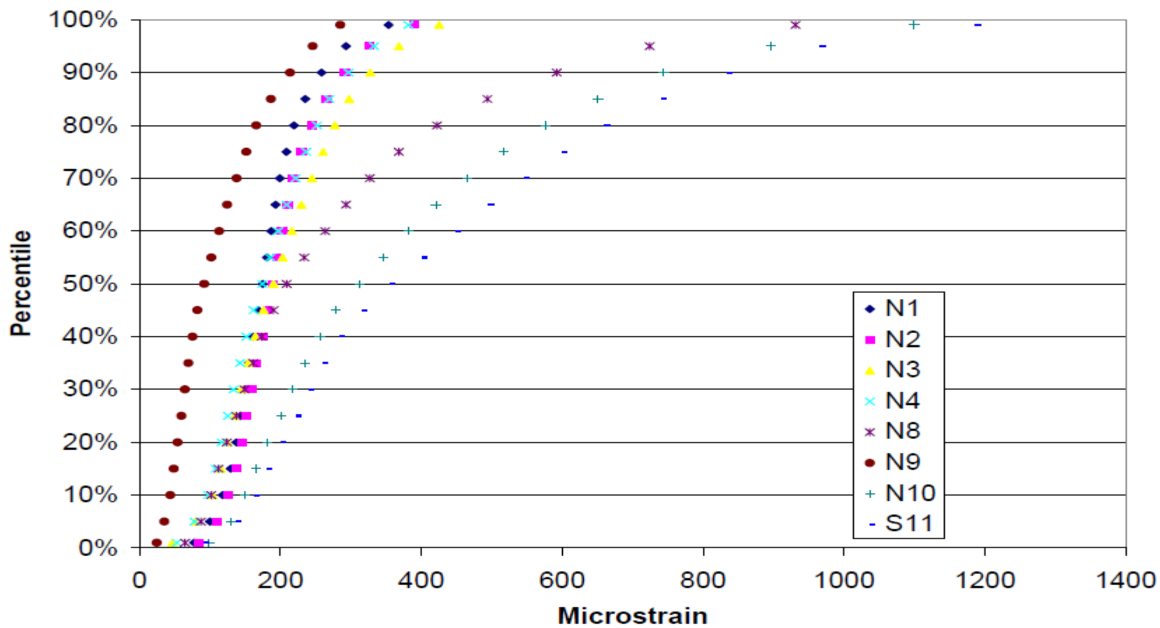


Figure 29. Cumulative Distribution of Strains for 2006 NCAT Sections (*19*).

Among the analyzed 2003 and 2006 sections, N3 and N4 were able to withstand 19 million ESALs without fatigue cracking. The strains seen in these two sections were much higher than those seen from the previous Test Track cycle; therefore, the combination of higher strains and extended trafficking without cracking made them ideal for consideration in the development of strain criteria for PPs.

Criteria Based on Strain Distribution

Using four strain profiles developed for Sections N3 and N4, an average strain distribution was calculated. Previous research had found gauge precision at the NCAT Test Track to be approximately 30 micro-strains between duplicate strain gauges (*19*). When gauge variability (±15 micro-strains) was considered, all four profiles fell within the gauge tolerance of the average strain distribution. Therefore, the average strain profile was determined to be an upper bound for strain criteria in flexible PP design, as seen in Figure 30. Based on measured strains from the NCAT Test Track from sections that have not experienced fatigue cracking, Willis proposed a cumulative frequency distribution of allowable strains for PPs design. Table 16 lists the exact values for each percentile (*19*).

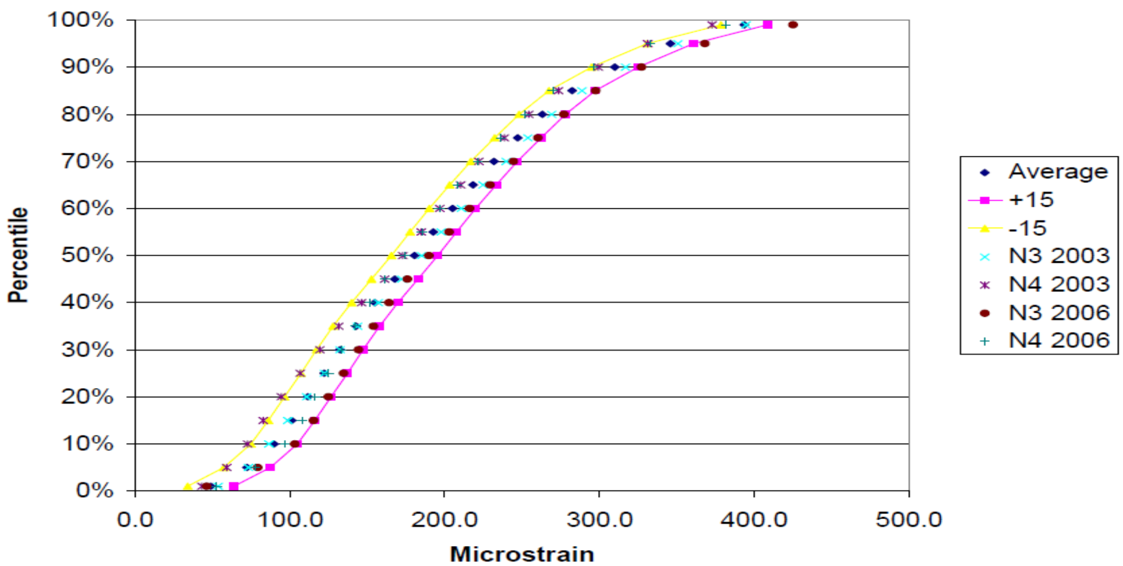


Figure 30. Average Strain Distribution with Confidence Bands (*19*).

Table 16. Strain Criteria for PPs (*19*).

Percentile	Fatigue Limit	Percentile	Fatigue Limit
99%	394	45%	168
95%	346	40%	155
90%	310	35%	143
85%	282	30%	132
80%	263	25%	122
75%	247	20%	112
70%	232	15%	101
65%	218	10%	90
60%	205	5%	72
55%	193	1%	49
50%	181		

Methodology for Incorporating the EL into M-E Design Procedures

Based on the preceding discussions, several considerations should be given when incorporating EL into M-E design procedures:

1) EL should be mixture-dependent. Different asphalt mixture should have different ELs since the asphalt-binder grade, AC, and gradation type are different.
2) Temperature effects on the EL should be considered. A pavement section in a cold climatic area should have a lower EL value than that in a hot climatic area even when the pavement structural thickness and layer materials are same.
3) The third consideration should be given to whether the EL is really best represented by a single value or not.

In view of the above considerations, researchers envisioned two levels of potential methods when incorporating the EL into the TxME design system, namely:

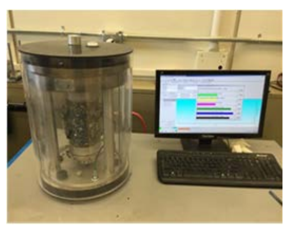
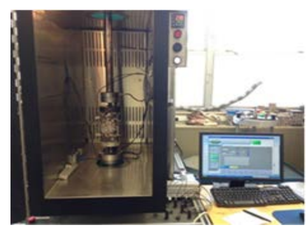
- Level 2: When traffic input is simply the ESAL, the 18-kip axle load will be applied at the equivalent annual temperature. The tensile strain at the bottom of the asphalt layer will be determined and compared to a single EL value. The single EL should be dependent on the mixture type, asphalt-binder type, and climatic condition.
- Level 1: When traffic input is the load spectra, then the maximum tensile strains at the bottom of the asphalt layer under different load levels and different temperature conditions will be determined and the corresponding strain distribution will be evaluated and then compared with the user defined strain distribution criteria.

RECOMMENDATION OF EL DETERMINATION APPROACH AND TEST PROTOCOL

Traditionally, the four-point beam fatigue test was conducted to determine the EL of asphalt mixtures. Recently, two other laboratory tests, called the simplified viscoelastic continuum damage (S-VECD) and the repeat direct tension (RDT) tests, respectively, have been reported to be able to determine the EL parameter, too. Appendix VII describes the test procedures and corresponding data analysis associated with these approaches/methods.

In this study, researchers tried both the S-VECD and RDT tests to determine the EL for a typical TxDOT mixture and found that the results are comparable. However, while use of S-VECD can obtain the EL values at different temperatures by testing at only one temperature, the RDT needs more sets of tests for certain temperatures. In addition, only the S-VECD has a standard test procedure and corresponding data analyzing software. Table 17 summarizes and compares the features of each test method. Based on this comparison, the S-VECD test was identified to be the suitable EL test method for the typical Texas mixtures in this project.

Table 17. The EL Tests Comparisons.

Item	Beam fatigue	S-VECD	RDT
Test machine	Beam Fatigue Apparatus	Asphalt Mixture Performance Tester (AMPT)	Material Testing System (MTS)
Sample size	380 mm × 50 mm × 63 mm	100 mm Dia. × 130 mm tall	100 mm Dia. × 150 mm tall
No. of samples	> 4	3	4
Analysis program	Excel template	Alpha-F Software	Excel template
Standard test procedure	Yes, AASHTO T321	Yes, AASHTO TP107	No
Advantage	Result is straightforward	- Simple to run - Easy calculation of EL at different temperature	- Simple to run
Limitation	- Need long test time - Difficult sample fabricate	- Need dynamic modulus (DM) result	- Need to run different sets of test for temperatures
Test setup			

AASHTO=American Association of State Highway and Transportation Officials

DEFAULT EL VALUES FOR TYPICAL TEXAS MIXTURES

HMA Mixtures Evaluated

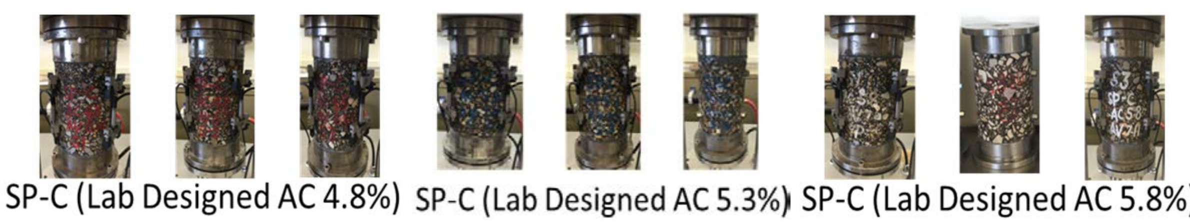
To determine the default EL values for typical Texas mixtures, five plant-mixtures and eight laboratory designed mixtures were selected to be tested in the project as listed in Table 18. For each mixture, at least three replicates were made for S-VECD testing. Also, three replicates were made for DM test since the S-VECD model needs to incorporate the DM for the linear viscoelastic characterization. In addition, the Hamburg Wheel Tracking test (HWTT) and Overlay tester (OT) were performed as supplementary screening tests. For the laboratory designed mixtures, the AC was varied for each mixture type; thus, the effect of AC on the EL was evaluated. Figure 31 shows some examples of the S-VECD samples of plant-mixtures and laboratory designed mixtures, respectively.

Table 18. HMA Mixtures Used for S-VECD Testing.

No.	HMA Mixture Type		Variable
1	Plant-produced mixture	Type B	US 82 plant-mix
2		Type C	SH 7 Plant-mix
3		Type C	SH 304 Plant-mix
4		SP-C	20% RAP/RAS
5		SP-D	14% RAP/RAS
6	Lab-designed mixture	SP-C	AC 4.8%
7			AC 5.3%
8			AC 5.8%
9		Type C	AC 4.7 %
10			AC 5.2%
11			AC 5.7%
12		SP-D	AC 5.9%
13			AC 6.4%

Legend: RAP = reclaimed asphalt pavement; RAS = reclaimed asphalt shingles

(a) Plant-Produced Mixtures.



(b) Lab-Designed Mixtures.

Figure 31. HMA Samples Used for S-VECD Testing.

Laboratory Test Results

The ALPHA-Fatigue software (v 3.1.5) developed by Underwood was used to analyze the S-VECD test data and determine the fatigue parameters and ELs (*20*). With the DM and fatigue test results, the ALPHA-Fatigue software produces two outputs, namely: the damage characteristic curve and the energy-based failure criterion. Figure 32 shows an example of using the Alpha-Fatigue software to analyze the S-VECD raw test data. It can be seen that the cycle to failure for the first replicate is 26,476 cycles at a strain level of 300 micro-strains; 5,815 cycles for the second replicate at a strain level of 350 micro-strains, and 895 cycles for the third replicate at a strain level of 400 micro-strains. The determined EL values at different temperatures are shown in Figure 33, which are 51, 54, 61, 66, and 78 micro-strains at 5, 10, 15, 20, and 25°C, respectively.

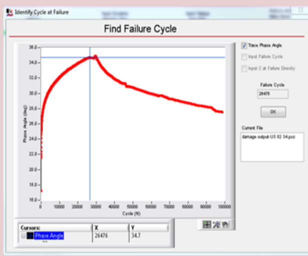
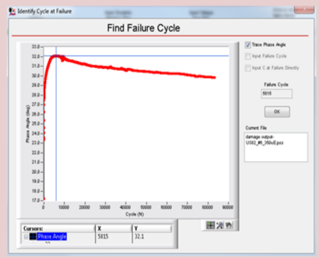
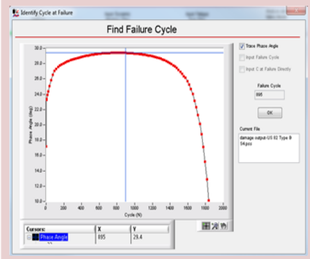
	Sample 1	Sample 2	Sample3
			
Test Temp.	21°C	21°C	21°C
Load Strain Level	300 με	350 με	400 με
Failure Cycles (N_f)	26,476	5,815	895

Figure 32. S-VECD Test Results.

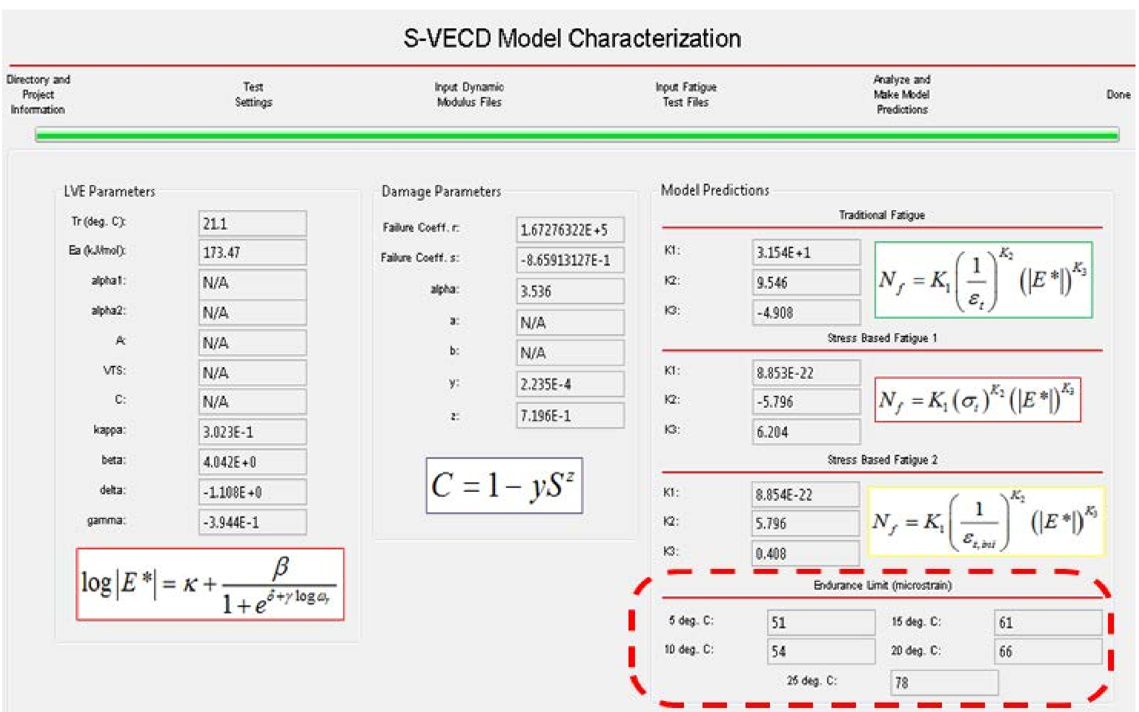


Figure 33. EL Values at Different Temperatures (Alpha-Fatigue Software).

ELs of Different HMA Mixtures

To provide default EL criteria for the enhanced M-E design for PP, S-VECD, DM, and OT tests were conducted using at least three replicates in this study. Table 19 summarizes the EL values for the selected mixtures. Note that no tangible results were obtained for the lab-designed mixture SP-C AC 5.8 percent due to accidental breakage of the two samples during installation.

Table 19. EL Values for Different Mixtures at Different Temperatures.

No.	HMA Mixture Type		Variable	EL at Different Temperature (με)				
				5°C	10°C	15°C	20°C	25°C
1	Plant-produced mixture	Type B	US 82	34	35	38	40	46
2		Type C	SH 7	54	58	67	74	90
3		Type C	SH 304	33	37	45	52	67
4		SP-C	20% RAP	39	40	43	47	58
5		SP-D	14% RAP	29	28	29	30	36
6	Lab-designed mixture	SP-C	AC 4.8%	30	33	38	45	62
7			AC 5.3%	44	49	56	66	89
8		Type C	AC 4.7 %	39	42	47	56	78
9			AC 5.2%	37	42	50	61	85
10			AC 5.7%	37	44	53	69	102
11		SP-D	AC 5.9%	36	40	46	54	73
12			AC 6.4%	57	64	75	90	123

For the convenience of comparison, Figure 34 plots the EL values of all the plant-produced mixtures together while Figure 35 compares the EL values at different ACs for the lab-designed mixtures.

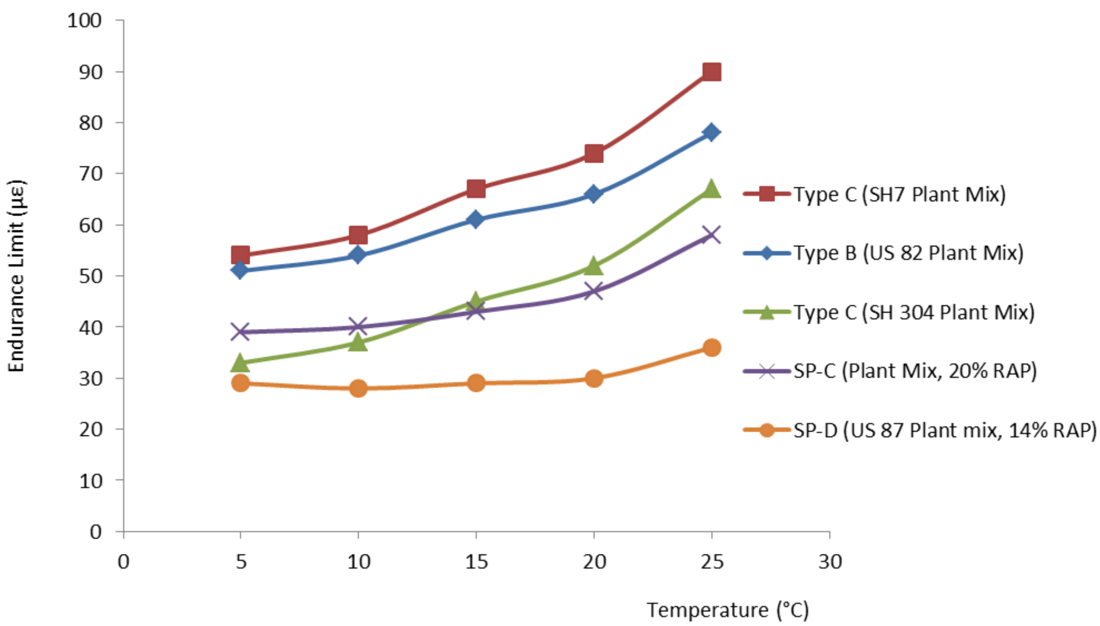


Figure 34. EL Comparison among Plant-Produced Mixtures.

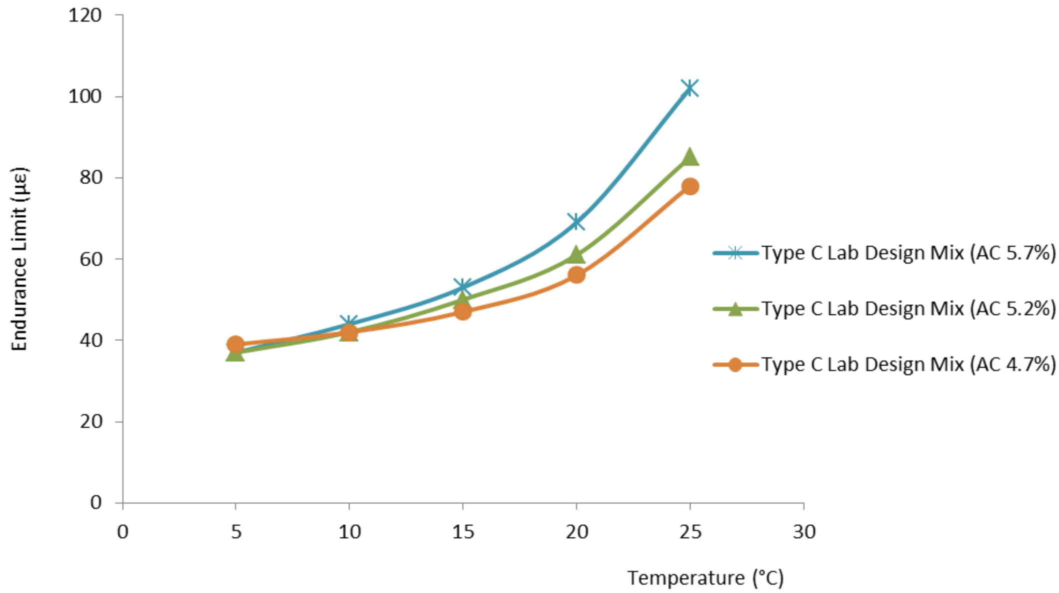


Figure 35. EL Comparison among the Lab-Designed Mixtures at Different AC Levels.

Default of EL Criteria for Different HMA Mixtures

From Figure 34 and Figure 35, it is evident that temperature, gradation, AC, and RAP content have a significant influence on the EL parameter. Since the EL value is related to not only the mixture itself but also the temperature and other mix-design variables, it is not appropriate to assume one EL value for one typical Texas mixture. Thus, to develop the default EL values for the typical Texas mixtures, the relationship between EL and the corresponding OT cycles were developed, as illustrated in Figure 36.

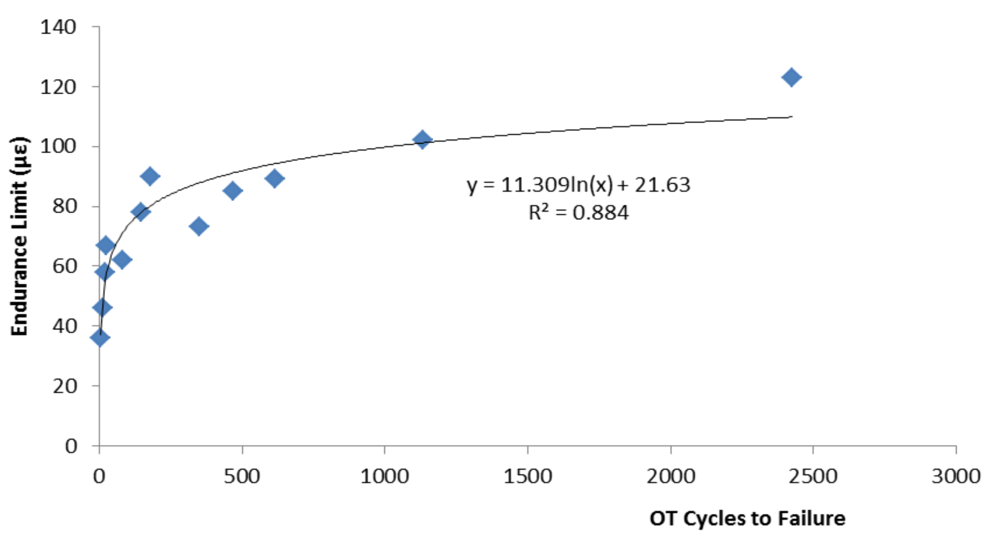
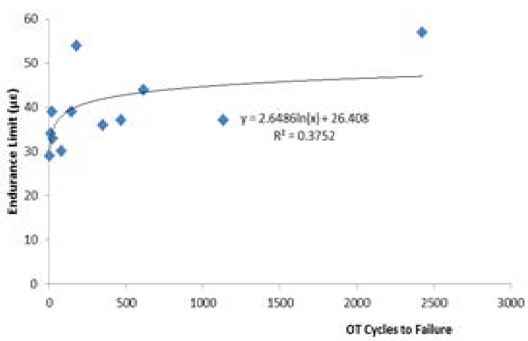
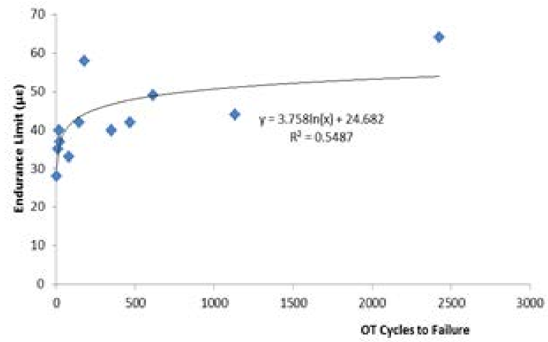


Figure 36. EL at 25°C versus OT Cycles.

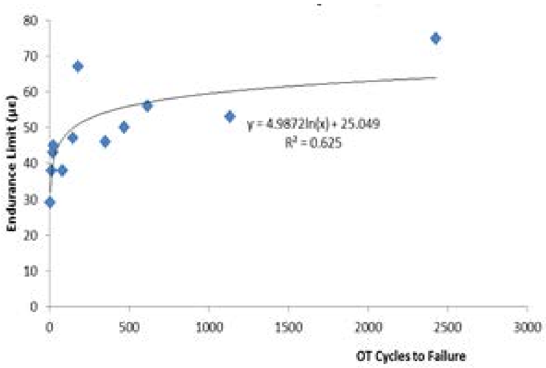
It is seen from Figure 36 that the EL value has a pretty good relationship with the OT cycles. Similarly, the relationships between the EL at other temperatures and the corresponding OT cycles were satisfactorily established, too. Figure 37 presents the EL results at 25°C, 20°C, and 15°C with corresponding coefficients of determination (R^2).



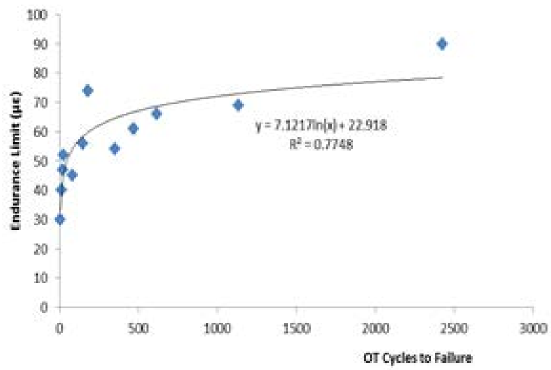
(a) 5°C



(b) 10°C



(c) 15°C



(d) 20°C

Figure 37. EL at Different Temperatures versus OT Cycles.

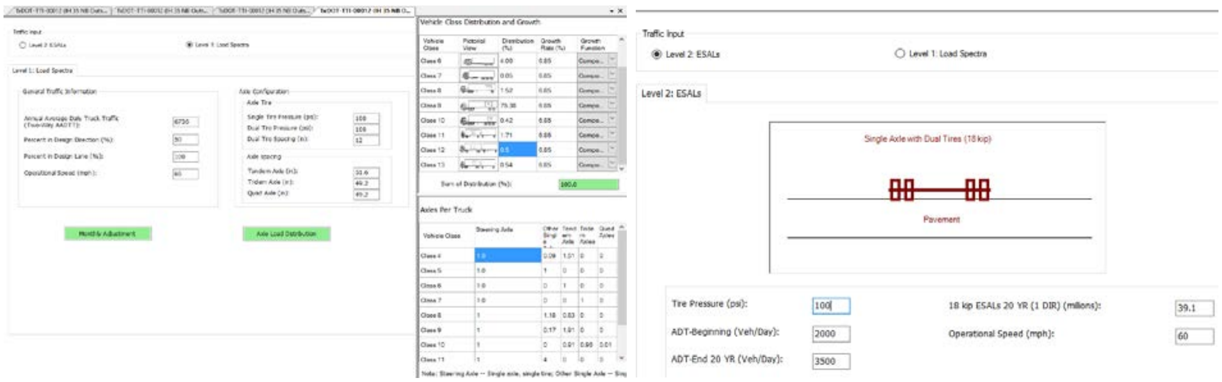
According to the above established relationships, the preliminarily suggested default EL values according to the OT cycles were determined and listed in Table 20. With this table, given the OT cycles, the EL value at any temperature can be easily interpolated and determined. Note that it is suggested herein to estimate the EL values based on the OT test (i.e., OT cycles) because it is much simpler, practical, easy sample preparation/setup, and more cost-effective than the corresponding S-VECD, RDT, and bending beam fatigue tests.

Table 20. Suggested Default EL Values as a Function of Temperature and OT Cycles.

OT Cycles	EL (με) at Different Temperatures				
	25°C	20°C	15°C	10°C	5°C
3	34	31	31	29	29
5	40	34	33	31	31
15	52	42	39	35	34
20	56	44	40	36	34
50	66	51	45	39	37
100	74	56	48	42	39
200	82	61	51	45	40
300	86	64	53	46	42
400	89	66	55	47	42
500	92	67	56	48	43
600	94	68	57	49	43
700	96	70	58	49	44
800	97	71	58	50	44
900	99	71	59	50	44
1000	100	72	59	51	45
1500	104	75	62	52	46
3000	112	80	65	55	48

INCORPORATION OF THE M-E PP DESIGN METHOD INTO TxME

As discussed previously, a two-level of M-E PP design method was incorporated into the TxME. Depending on the traffic input level, the TxME computationally decides whether to calculate a single maximum strain value or strain distribution and compare the result with the corresponding criteria. Figure 38 show the TxME traffic input screen for Level 1 and Level 2, respectively. Note that for Level 2, users only need to input some simple information such as ESALs, while Level 1 requires much more detailed information such as annual average daily traffic (AADT), vehicle class distribution, axles per truck, axle load distribution, monthly adjustment, etc. All this detailed information can be obtained from traffic WIM data.



(a) Level 1 **(b) Level 2**

Figure 38. TxME Traffic Input Screen.

Figure 39 shows the flow chart that illustrates the M-E PP design approach. Note that when traffic input is Level 2 (ESALs), the design criterion is a single EL value; however, when the traffic input is Level 1 (load spectra), the design criteria is a pre-defined strain distribution. In this study, the single EL criterion default is tied to the default OT cycles and the strain distribution criteria adopts the suggested data in Table 16. These criteria can be changed by the user.

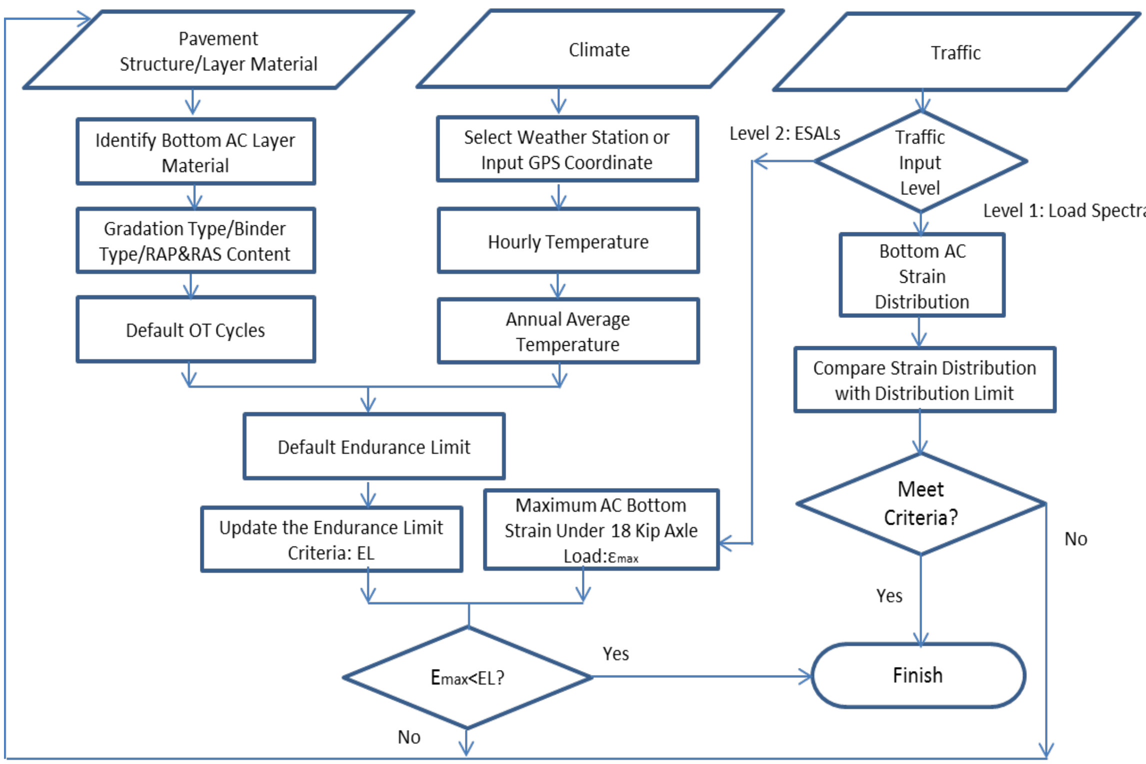
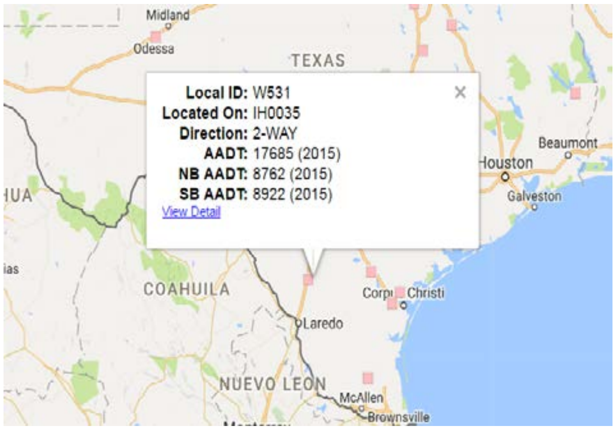
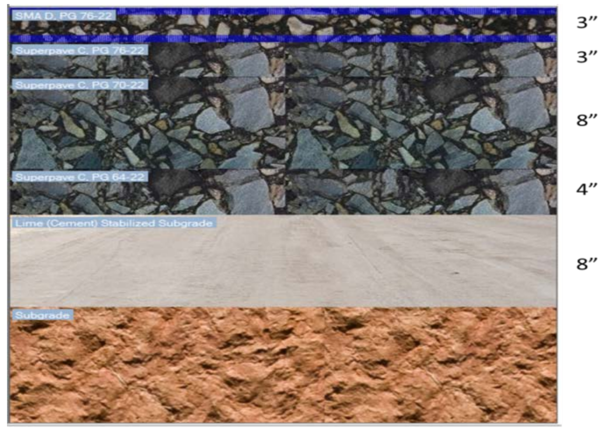


Figure 39. Flow Chart of the ME PP Design Approach.

DEMONSTRATION CASE STUDY

To assess the enhanced TxME, one case study was conducted using an existing Texas PP section on IH 35 in La Salle County, Laredo District. Since there is a permanent WIM station in this location, the detailed load spectra data could be obtained. Figure 40 shows the location and traffic and the pavement structure, respectively. The material properties of each layer were obtained from the Project 0-6658 database to be entered into the TxME.





(a) Location and Traffic Data **(b) Pavement Structure**

Figure 40. IH 35 Section in La Salle County, Laredo District.

If users select Level 2 (ESALs) for the traffic input, the TxME calculates the hourly temperature, annual average temperature, and other parameters based on the section location or weather station information. Next, the TxME calculates the EL value (52 micro-strains) based on the bottom HMA layer (SP-C, PG 64-22) properties and average annual temperature (71.1°F) as shown in Figure 41. Note that at any time when users change the pavement structure or climatic information, the EL value will be automatically re-calculated. Additionally, users can manually change the EL criterion input if they have conducted the EL test.

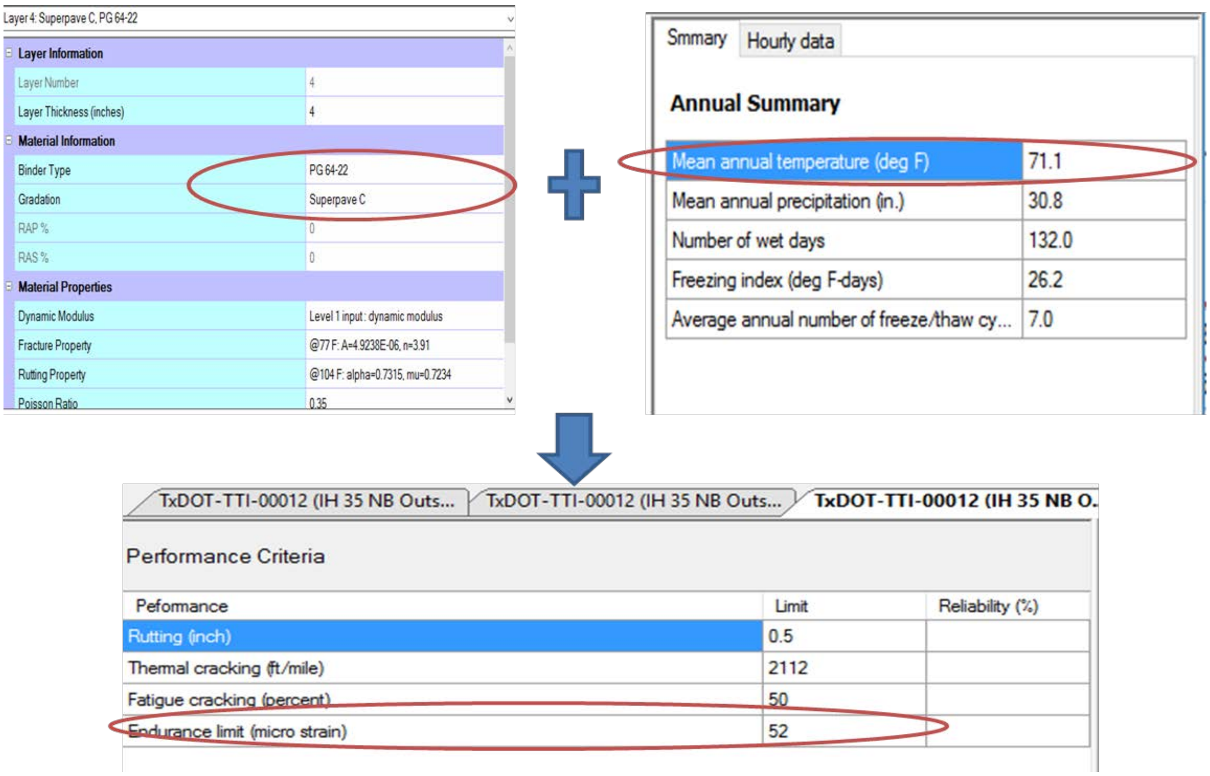
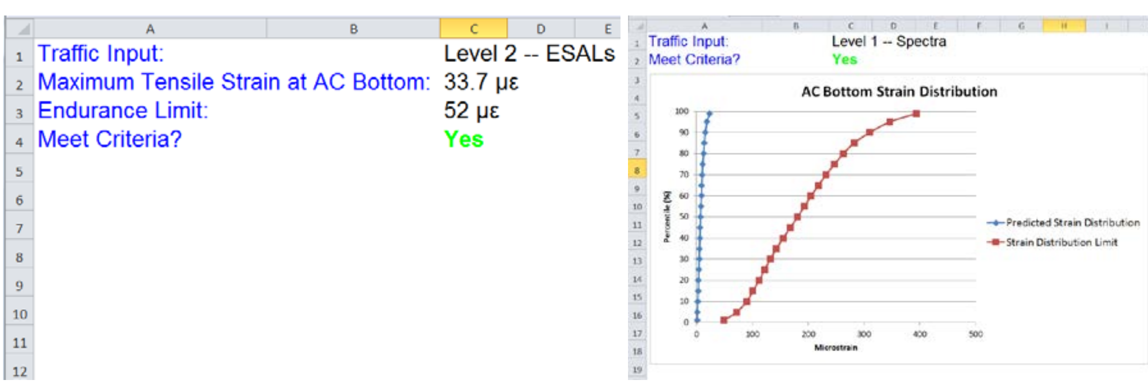


Figure 41. TxME EL Criteria Determined from User Input Data (Level 2).

If users select Level 1 (load spectra) for the traffic input, the TxME calculates the strain distribution according to the load spectra and compares with the strain distribution limit. Figure 42 shows the analysis results for both Level 1 and Level 2 traffic inputs. Figure 42(a) shows that the maximum strain at the AC bottom was determined to be 33.7 micro-strains, which is less than the EL criterion of 52 micro-strains with the 18-kip ESALs input. Figure 42(b) shows that with load spectra input, the predicted strain distribution curve is on the left side of the strain distribution limit curve, which means that for a given percentile, the determined strain is less than the corresponding strain limit. Thus, both cases indicate that the IH 35 PP section meets the perpetual criteria, which is consistent with the actual measured performance on the in-service IH 35 that no fatigue cracking was observed on this section since construction in 2004.



(a) Traffic Level 2: ESALs **(b) Traffic Level 1: Load Spectra**

Figure 42. TxME PP Design Output.

SUMMARY

This chapter described a methodology for incorporating EL into the M-E PP design. The ELs of 12 HMA mixtures were determined in the laboratory using the S-VECD test. Based on these test results, default EL criteria were developed and incorporated into the TxME system. One Texas PP test section (IH 35, La Salle County, Laredo District) with the traffic data obtained from a permanent WIM was simulated using the enhanced TxME design system. The corresponding TxME inputs/outputs in terms of the PP structure, material properties, traffic loading, environmental conditions, and EL was successfully demonstrated with the modeling results matching the actual in-service field performance of the PP structure. However, additional validation and calibration of the enhanced TxME PP design system should continue with other in-service PP test sections as field performance and traffic data are progressively collected.

CHAPTER 6. BEST PRACTICE OF TEXAS PP DESIGN AND CONSTRUCTION

Since the PP consists of different functional asphalt layers, it is important to select proper material and conduct structure design based on the function of each layer. Also, efficient and cost-effective construction methods should be applied to minimize the problems associated with poor construction and reduce the cost required in HMA material production for different pavement layers. This chapter provides the recommendation for the alternative structural design and material selection, improved construction procedure, and innovative quality control/quality assurance (QC/QA) tools for Texas PP.

EVALUATION OF TEXAS PP MATERIAL SELECTION

It is critical to select proper materials based on the function of each layer because the PP structure is composed of different functional HMA layers. The selection of structurally strong, stable foundation material is also important to support the traffic loading during the service period and compaction loadings during construction process. Table 21 summarizes the current Texas PP material and structure by each layer type.

Table 21. Current Materials and Thickness of Texas PP.

Layer No.	Mixture/Material	Thickness (in.)	Function
1	SMA	2.0–3.0	Renewable HMA surface
2	¾" SFHMA	2.0–3.0	LTL
3	1" SFHMA	≥ 8.0	- RRL - Main structural load-carrying layer
4	½" SFHMA	3.0–4.0	- RBL - Fatigue resistant - Impermeable layer
5	Lime treated base	≥ 6.0	Providing stable foundation at the stage of construction
6	Subgrade		

During the construction of PP sections in Texas, SFHMA mixes used for the main structural load-carrying RRL had exhibited undesirable constructability problems with high potential for moisture damage and other forensic defects including density variation, localized voiding, vertical segregation, and poor layer bonding (*4*). It is thus imperative to change the materials and improve the construction methods for the RRL. As a preliminary proposal, the Type B mix was found to be more workable with better constructability and compactability properties, attaining more uniform density with lower potential for moisture induced problems or forensic defects. Researchers investigated and evaluated two PP sections at SH 114 (Fort Worth), which consists of one with SFHMA mix and another with conventional dense-graded Type B mix for the RRL, as illustrated in Figure 43. For the field performance evaluation, the section with Type B was superior and comparable to one with SFHMA mixes in terms of rutting and roughness performance, respectively, as shown in Figure 44, which presents the surface rutting measurements and the roughness (IRI) of two PP sections on SH 114. From the SH 114 performance evaluation, it has been found imperative to change the SFHMA mix used for each layers to traditional dense-graded or SP mixes for improving constructability and compactability.

(a) SH 114 with SFHMA **(b) SH 114 with Dense-Graded HMA**

Figure 43. Section Pictures and Pavement Layer Materials of SH 114.

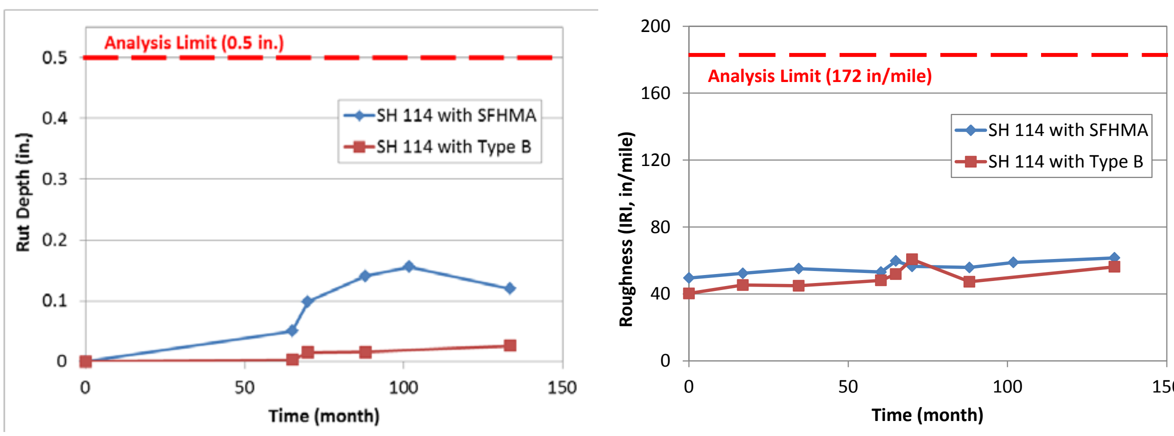


Figure 44. Surface Rutting and Roughness History of SH 114 PPs.

ALTERNATIVE TEXAS PP DESIGN

It is found that the Texas PP needs possible significant improvement in material and thickness reduction for cost-effectiveness from the field performance evaluations of the in-service Texas PP sections and the extensive literature reviews on PP practices.

Recommended Structural and Mix Design

Based on the evaluation of global data related to PP design including 16 states and 21 countries in Chapter 2, the Texas PPs have required the thickest asphalt layers among them. Since those PP sections using thinner asphalt layers than Texas show good field performance, the current Texas PP structural design of 22 in. total HMA layer thickness would be conservative and not cost-effective. Also, it is imperative to change the SFHMA mix currently used for the HMA layers due to undesirable constructability problem. Thus, the Texas PP needs possibly significant improvement in material quality and thickness reduction in terms of cost-effectiveness. For this, researchers propose a Texas PP design having thinner thickness of asphalt layers than current pavement structures with asphalt materials having better compactability and constructability. The total structural HMA is cost-effectively and satisfactorily reducible from the current average of 22 in. to an optimal of about 16 in. without compromising the structural performance (i.e., 36 percent reduction in total HMA layer thickness), as provided in Table 22. A 27 percent reduction in HMA layer thickness may also potentially translate into up to 27 percent cost-savings.

Table 22. Proposed PP Structure and Material Design.

Layer No.	Function	Mixture/Material		Thickness (in.)	
		Current	Recommend	Current*	Recommend
1	Surface	SMA	SMA	2	2.0–3.0
2	LTL	¾" SFHMA	SP-C or Type C	3	2.0–3.0
3	RRL	1" SFHMA	SP-B or Type B	13	6.0–8.0
4	RBL	¾" SFHMA	SP-C or Type C	4	2.0–4.0
5	Base	LTB or CTB	LTB or CTB	8	6.0–12.0

*Average thickness of current Texas PP sections
Legend: CTB = Cement treated base; LTB = Lime treated base

Based on the new Texas PP design in Table 22, alternative structural designs also were recommended as a function of three traffic levels, namely; (a) traffic ESALs ≤ 30 million, (b) 30 million < Traffic ESALs ≤ 50 million, and (c) traffic ESALs > 50 million, as listed in Table 23. These alternative perpetual designs are to use dense-graded mixes such as the SP-B or Type B mix for the main structural load-carrying RRL as opposed to the coarse-graded SFHMA mixes in the current Texas PP design concept. However, the use of higher PG of asphalt-binder such as PG 70-22 for RRL is recommended, especially if the mixtures are placed within 6 in. of the surface (*4*).

Table 23 Alternative Texas PP Designs by Traffic Level.

Layer #	Thickness (in.)	Mix Type	Designation	2014 TxDOT Spec. Item	Asphalt-Binder
(a) Traffic ESALs ≤ 30 million					
1	2	SMA	Surfacing	Item 346	PG 70-28 or better
2	2	SP-C or Type C	LTL	Item 344 or 341	PG 70-22 or better
3	≥ 6	SP-B or Type B	Main structural load carrying RRL	Item 344 or 341	PG 64-22 or better
4	2	SP-C or Type C	Rich bottom fatigue-resistant layer (durability & impermeability)	Item 344 or 341	PG 64-22
5	≥ 6	Base	Lime or cement treatment	Item 260, 263, 275, & 276	
6	Subgrade (in-situ soil material)				
Minimum PP structure thickness = 18 in. (12 in. HMA and 6 in. base)					
(b) 30 million < Traffic ESALs ≤ 50 million					
1	2	SMA	Surfacing	Item 346	PG 70-28 or better
2	3	SP-C or Type C	LTL	Item 344 or 341	PG 70-22 or better
3	≥ 8	SP-B or Type B	Main structural load carrying RRL	Item 344 or 341	PG 64-22 or better
4	2	SP-C or Type C	Rich bottom fatigue-resistant layer (durability & impermeability)	Item 344 or 341	PG 64-22
5	≥ 6	Base	Lime or cement treatment	Item 260, 263, 275, & 276	
6	Subgrade (in-situ soil material)				
Minimum PP structure thickness = 21 in. (15 in. HMA and 6 in. base)					
(c) Traffic ESALs > 50 million					
1	2-3	SMA	Surfacing	Item 346	PG 70-28 or better
2	≥ 3	SP-C or Type C	LTL	Item 344 or 341	PG 70-22 or better
3	≥ 8	SP-B or Type B	Main structural load carrying RRL	Item 344 or 341	PG 64-22 or better
4	2-4	SP-C or Type C	Rich bottom fatigue-resistant layer (durability & impermeability)	Item 344 or 341	PG 64-22
5	≥ 8	Base	Lime or cement treatment	Item 260, 263, 275, & 276	
6	Subgrade (in-situ soil material)				
Minimum PP structure thickness = 23 in. (15 in. HMA and 8 in. base)					

Design Criteria and Computational Validation

Considering the fact that the field performance of existing PP sections has generally been satisfactory with no structural defects to date, the recommendation is that the 70 and 200 micro-strains maximum thresholds should keep being used as the M-E response (strain) design criteria in the future Texas PP designs:

- Horizontal tensile strain at the bottom of the lowest HMA layer (ε_t): ≤ 70 micro-strains (for limiting bottom-up fatigue cracking).
- Vertical compressive strain on the top of subgrade (ε_v): ≤ 200 micro-strains (for limiting rutting).

However, 70 micro-strain of horizontal tensile strain, referred to as EL, should be used for initial thickness design and strain check in the FPS 21. In the TxME, more specific EL values determined based on HMA mix types and climatic condition would be used to check the maximum tensile strain at the HMA bottom and verify the PP designs from FPS as described at Chapter 5.

To assess the validity of the alternative PP structure designs proposed in Table 23 using the above design criteria, FPS and TxME analyses were conducted at 95 percent reliability level. As shown in Table 24, the alternative PP designs were verified to meet all criterions of FPS and show lower rutting depth and fatigue cracking than the performance limits of TxME, which last for the 50 years of design life without significant structural failures. Appendix VIII and IX present all pavement design results and mechanistic analysis from the FPS and the comparative performance predictions from the TxME, respectively.

Table 24. Computational Validation of the Alternative PP Structural Designs.

Traffic Criteria	ESALs ≤ 30M	30M< ESAL ≤ 50M	ESAL > 50M
Traffic loading	30 million	40 million	70 million
Design life (FPS/TxME)*	40 yrs/50 yrs	40 yrs/50 yrs	40 yrs/50 yrs
Environment	Fort Worth	Fort Worth	Fort Worth
PP Structure	2 in. SMA + 2 in. SP-C + 6 in. Type B + 2 in. SP-C + 6 in. CBT + subgrade	2 in. SMA + 3 in. SP-C +8 in. Type B + 2 in. SP-C + 6 in. CBT + subgrade	2 in. SMA + 3 in. SP-C + 8 in. Type B + 2 in. SP-C + 8 in. CBT + subgrade
FPS tensile strain at bottom of lowest HMA layers (≤ 70με)	13.8	14.3	13.7
FPS compressive strain at top of subgrade (≤ 200με)	105	81.2	76.0
TxME rut at 50 yrs (≤ 0.5 in.)	0.55 in.	0.55 in.	0.68 in.
TxME AC fatigue cracking (≤ 50%)	2.36%	0.04%	0.19%
TxME EL	Satisfied	Satisfied	Satisfied

Cost Benefits of Alternatives

To provide economic justification for the alternative Texas PP designs in Table 24, the LCCA was conducted to compare the total agency costs between the alternative and current PP designs. Table 25 presents the pavement structures of current and alternative PP designs for the comparison. The pavement structure of current PP was determined by averaging layer thicknesses of in-service Texas PP sections. It was assumed that all PPs were overlaid every 12 years with 2 in. thickness, and the material cost of each layer was obtained from the 2017 TxDOT average low bid unit prices collected in Chapter 4.

Table 25. PP Structures of Current and Alternative PP Designs for LCC Comparison.

Layer	Current PP design		Alternative PP Designs			
	Mix type	Thickness (in.)	Mix type	Thickness (in.)		
				ESAL<30M	30M<ESAL<50M	ESAL>50M
Surface	SMA	3	SMA	2	2	2
LTL	¾" SFHMA	3	SP-C	2	3	3
RRL	1" SFHMA	12	Type B	6	8	8
RBL	SP-C	4	SP-C	2	2	2
Base	CBT	8	CBT	6	6	8
Subgrade	In-situ soil	-	In-situ soil	-	-	-

As shown in Figure 25, the LCCA results indicated that the alternative PP designs have lower initial construction and agency costs due to their thinner HAM layers while the M/R costs are the same. Also, the percentage-wise comparison using the current PP design as a reference identified that the use of the alternative designs allow agencies to save the total agency cost from 26 percent to 8 percent, as presented in Figure 46.

Total Cost	Agency Cost ($1000)			
	Current PP	ESAL < 30M	30M < ESAL < 50M	ESAL > 50M
Undiscounted sum	*$ 2,819.00*	*$ 2,308.00*	*$ 2,587.00*	*$ 2,668.00*
Present Value	**$ 2,272.63**	**$ 1,761.63**	**$ 2,040.63**	**$ 2,121.63**

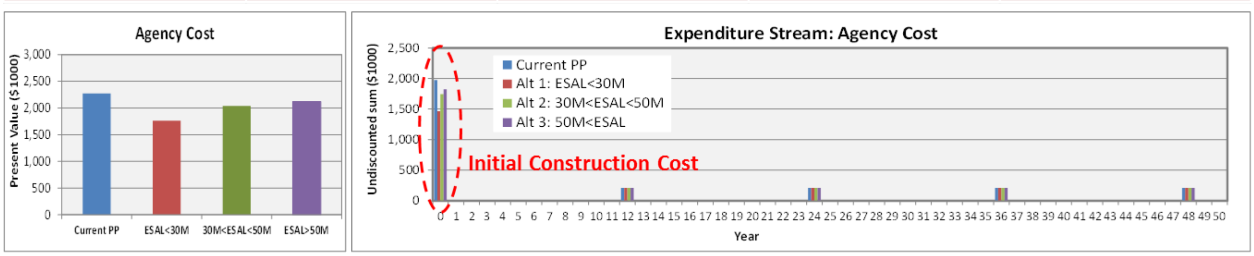


Figure 45. LCC Comparison between Current and Alternative PP Designs.

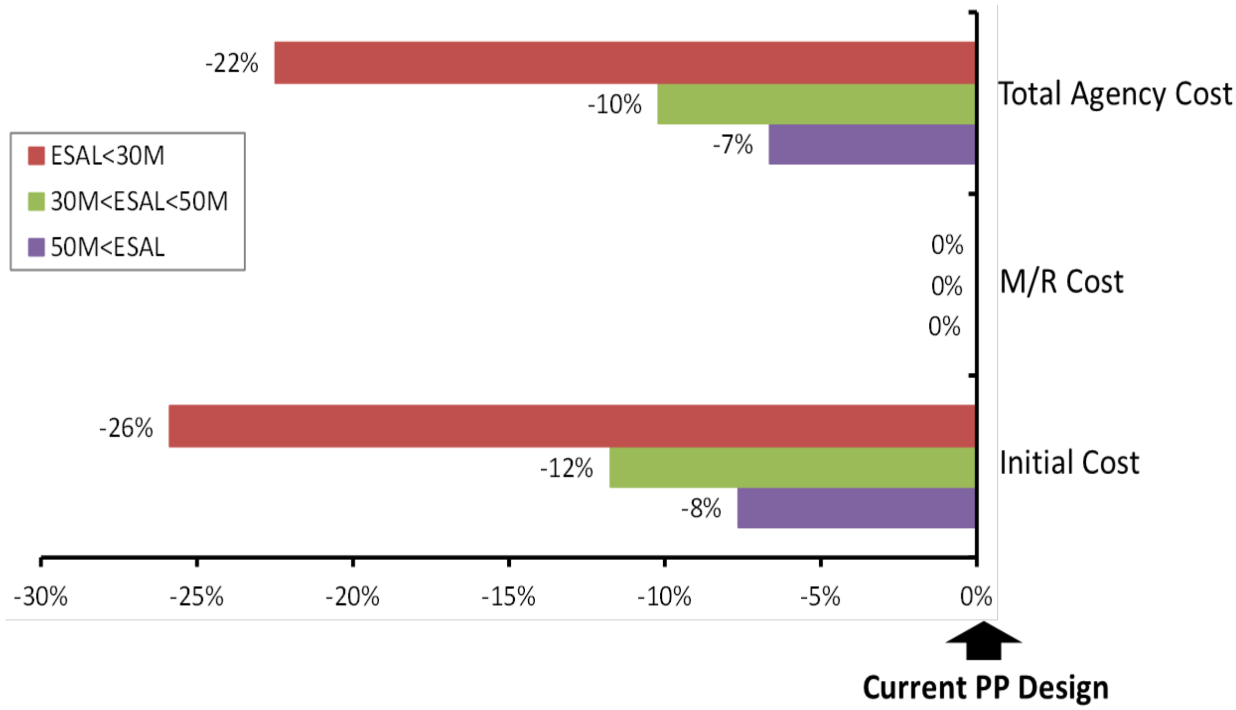


Figure 46. Comparison of Agency Costs on Current PP Design as Reference Base.

Laboratory Test Protocol for Texas PP

To ensure PP structural integrity and adequate performance, a proper testing method should be applied to characterize HMA mix properties required to meet the functional requirements of each layer. Also, as the PP design method moves forward to the M-E design, additional laboratory testing should be performed to obtain typical inputs required to run the M-E design software. Currently, although the HMA mix design seeks to address a number of performance concerns, a laboratory test protocol applicable to the PP design is lacking to readily characterize the material properties and generate the required M-E pavement design inputs. Instead, the indirect tensile (IDT) strength test and HWTT are routinely performed to determine the cracking- and rutting-resistance properties, respectively. Accordingly, researchers proposed and recommended a testing protocol for PP design to characterize the material properties required to meet the function of HMA layers, as listed in Table 26. The testing protocols are tailored to provide typical material inputs required to run the M-E software such as the TxME.

Table 26. Laboratory Testing Protocol for PP Design.

Test	Material Properties	Test Parameter/Output	Test Method/Specification
M-E Design Input	DM	DM: - Temp.: 14–130°F - Freq.: 0.1–25 Hz	AASHTO TP 62-03
	Fracture Property	A and n at 77°F	OT Fracture Test
	Rutting Property	α and μ at 104°F and 122°F	Repeated loading permanent deformation (RLPD) test
	EL	Strains at different temperature	AASHTO TP107-14
Screening Test	HWTT	Rut depth at 20,000 wheel load passing at 122°F	Tex-242-F
	IDT Strength Test	Tensile strength	Tex-226-F

Legend: Temp.= temperature; Freq.= frequency

IMPROVED CONSTRUCTION PROCEDURE

Since the PP uses premium mixtures to resist rutting or bottom-up fatigue cracking that is the most critical distress at each HMA layer, it is important to select proper materials that provide properties required to the function of each layer. Moreover, the PPs have thicker HMA layers so that more workable materials should be used for better constructability and compactability. From the literature reviewed and field observations of the workability aspects of the HMA layers for the Texas PP constructed, the current construction and quality issues can be summarized being related to:

1) Compactability of the SFHMA mixes.
2) HMA material placement of materials transfer device (MTD).
3) Compaction lift thickness of the HMA layers.

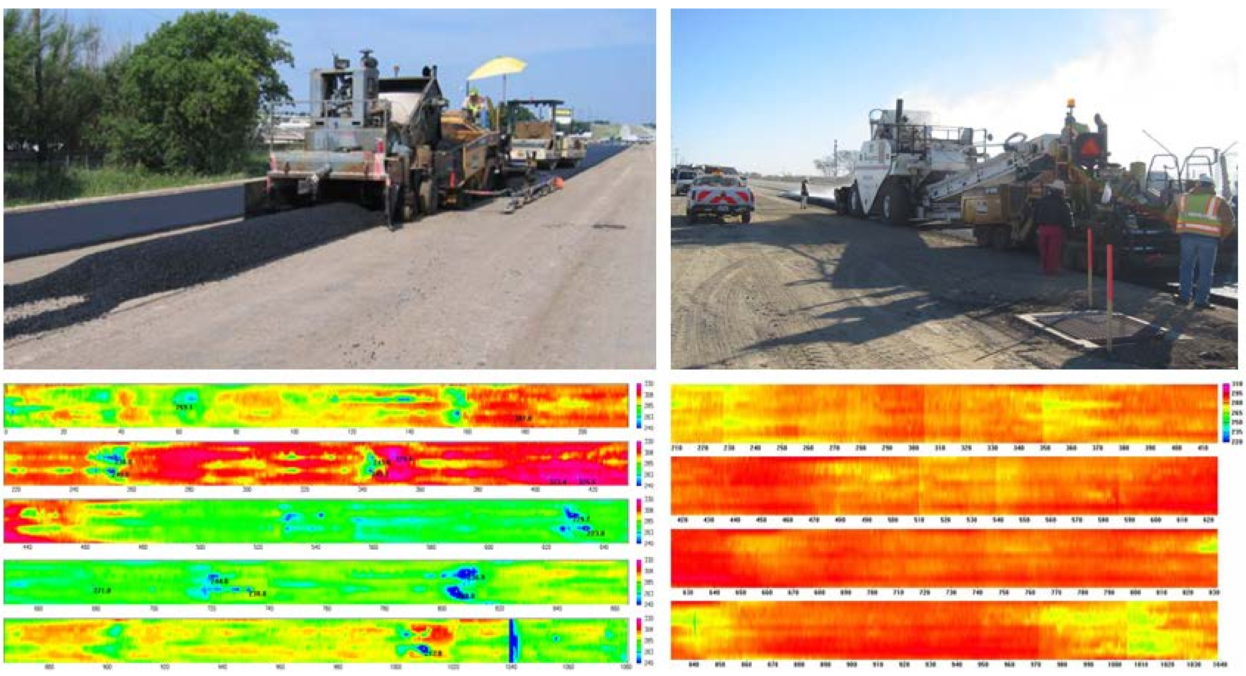
Since it is necessary to optimize construction quality of the PPs, researchers proposed the following recommendations to minimize the aforementioned construct-related issues.

Compactability of the SFHMA Mixes

While no major problems were experienced with the other HMA mixes, constructability and quality issues were experienced with the SFHMA mixes, including workability and compactability. These issues were due to the coarseness (low fines) and moderately low AC (compounded by absorptive limestone aggregates in some instances). Due to the poor constructability and compactability properties, the SFHMA mixes were found to be highly susceptible to forensic defects including low density/in-place density variations, vertical segregation, debonding, and permeability problems (*2*). These defects pose a great risk for moisture damage and compromising the structural integrity of the whole pavement. The constructability and compactability properties must be improved to minimize the occurrence of forensic defects evident in the placement and construction of the SFHMA mixes (e.g., by increasing the AC, adjusting the gradation, using less absorptive quality aggregates). Alternatively, more workable HMA materials such as dense-graded Type B or conventional SP mix as proposed previously in Table 22 are recommended.

HMA Material Placement

For the MTD, the combination of belly-dump trucks and windrow elevator (windrow pick-up system) was observed to be less effective and caused more thermal segregation in the HMA mat during either the cold or hot weather placement. From Figure 47(a), which shows the comparative infrared thermal profiles for a target HMA mat placement temperature of 300°F, the placement temperature of the windrow pick-up MTD system was hardly attained nor was it uniform (*4*). Instead, use of the Roadtec® MTD with its internal remixing capability was observed to yield a more consistent HMA mix/mat with greater temperature uniformity, as shown in Figure 47(b). The thermal segregation caused by lower HMA mat placement temperature observed in the infrared thermal profiles coincided with the end of HMA delivery truck loads and paver stoppages. Thus, it is important to ensure pavers are supplied with sufficient HMA mix material at uniform temperatures to allow continuous, uninterrupted operations.



(a) Windrow pick-up MTD **(b) Roadtec MTD**

Figure 47. Comparison of MTDs and HMA Mat Temperature Profiles.

Compaction Lift Thickness of the HMA Layers

Compacting at a higher lift thickness tended to cause the HMA mixes to segregate vertically, creating highly voided areas capable of detrimentally trapping moisture. For the 1-in. SFHMA material, the content of air void was measured to be high; 12.6 percent, for the 5-in. layer lift-thickness while it was around 7 percent for the 3- and 4-in. layer lift-thickness (*4*). As illustrated in Figure 48, compacting at the lower lift thickness range was observed to yield a more constructible HMA mix and to attain the target in-place density and layer interface bonding than using thicker lifts. Figure 48 shows better construction quality for 3 and 4 in. layer lift thicknesses with no visual evidence of vertical segregation or debonding. Based on these observations, a maximum 4-in. layer lift-thickness would be considered reasonable for PP construction, particularly the SFHMA mix.

(a) 3 in. lift thickness **(b) 4 in. lift thickness** **(c) 5 in. lift thickness**

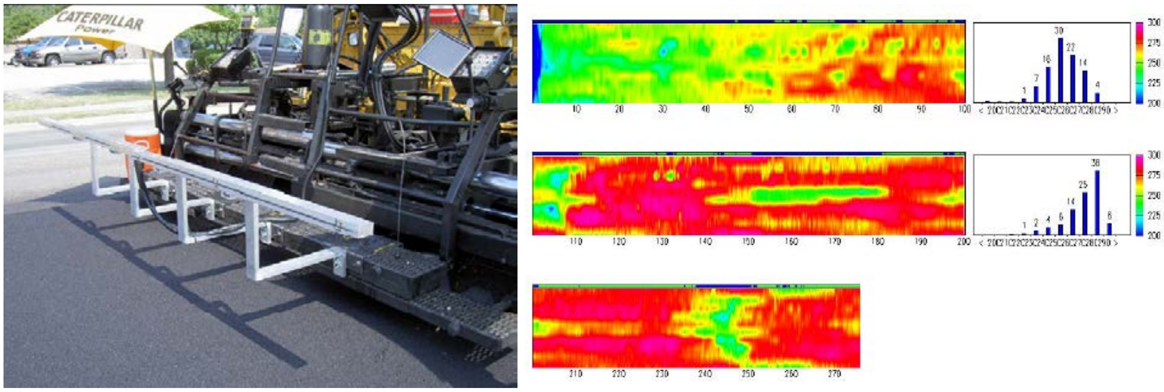
Figure 48. Cores from SFHMA Compacted at Different Lift Thicknesses.

INNOVATIVE QC/QA TOOLS

It is required to formulate QC/QA test protocol consisting of effective tools and equipment that can assist with checking uniformity during the PP construction to improve the construction method. The following tools and testing methods are possibly available for the QC/QA monitoring of PP construction.

Infrared Thermal Imaging System

An infrared temperature monitoring system was developed to detect the temperature segregation in HMA and evaluate the uniformity and the overall quality of paving construction (*21*, *22*, *23*). This system employs a bar with an array of infrared sensors that are mounted onto the rear end of a paver, as shown in Figure 49(a). As the paver moves forward, the sensors measure the surface temperature of uncompacted HMA mixture. Figure 49(b) displays an example of thermal infrared data collected in real time.

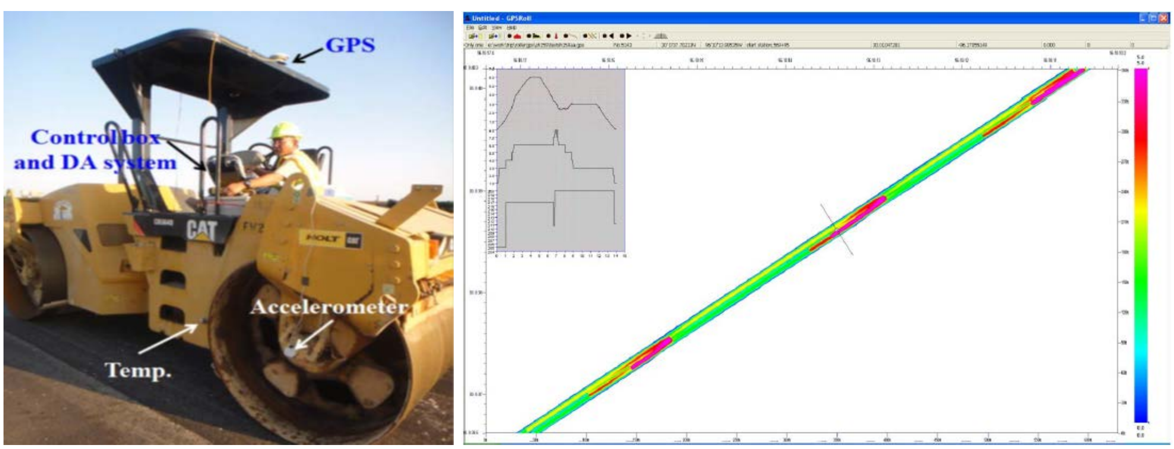


(a) Infrared System Installed on Paver **(b) Data Displayed in Real Time**

Figure 49. Infrared System and Display.

Compaction Monitoring System

To monitor the quality of compaction in real time, the compaction monitoring system developed by TTI funded by TxDOT can be used to check the PP construction quality and layer uniformity. The system consists of a GPS unit for tracking the location, temperature sensors for recording the mat surface temperature, and accelerometer sensor for determining the mode of operation (static or vibratory) on the roller as shown in Figure 50(a). The system monitors the location of the roller on the HMA mat and the number of passes across the mat. Each pass is multiplied by the effectiveness factors across the roller's width to produce the compaction index distribution. Since the distribution is converted to colored maps in real time as displayed in Figure 50(b), the roller operator can use it to adjust the compaction patterns (by changing the number of passes, overlapping, and overhanging) needed to achieve the required density uniformly across the HMA mat. These maps can provide the transverse distribution of compaction and temperature data across the mat at a user-selected location, as shown in Figure 50(b) (*24*, *25*).



(a) Compaction Monitoring System **(b) Real-time Compaction Effort Map**

Figure 50. Compaction Monitoring System and Display.

Ground Penetrating Radar

The ground penetrating radar (GPR) is widely used to characterize and evaluate pavement layer densities (air void), pavement layer thickness, and presence of free moisture both during and after construction. The GPR is non-destructive testing method and can capture the pavement data up to a depth of 2 ft at a maximum operable speed of 70 mph. Thus, this unit can be effectively used for both construction quality monitoring (density, layer thickness uniformity, segregation, etc.) and performance evaluation (i.e., forensic defects such as localized voiding, moisture presence) of PP structures.

Coring Pavement Samples

Cored samples extracted from the field after construction are routinely used to assess the construction quality by measuring the thickness and air void (density) and to characterize the material properties by performing laboratory tests. While this method provides the most accurate detection of forensic defects and construction quality, it is a destructive test method damaging the pavement surface. Nonetheless, this is one of the cheapest, oldest, and simplest conventional methods for construction quality control assessment of HMA including PP structures; and is also an invaluable method for forensic evaluation during performance monitoring/evaluation of in-service pavement structures including PPs.

FIELD TESTING AND PERFORMANCE EVALUATION

To evaluate the in-situ material behaviors and performance, the field performance data should be collected on Texas PP sections. The performance data such as rutting and cracking histories are the main source for calibrating the empirical component of the M-E models in comparing the predicted and actual field pavement performance. Thus, researchers recommended selecting a single or multiple 500- or 1,000-ft test sections on each Texas PP projects and conducting field performance monitoring/evaluation sequentially as follows:

1) During construction to aid in selecting homogeneous PP test sections (and collect pavement materials for each layer to be used for laboratory testing).
2) During and just after construction to monitor the construction process and the pavement condition just after construction.
3) Periodic in-service test section visits for performance evaluation and documentation of the historical performance.

After construction, periodic summer and winter performance monitoring is recommended to evaluate hot and cold weather related distresses. Table 27 lists the recommended field performance testing and data characteristics to be collected from Texas PP sections.

Table 27. List of Field Performance Testing and Data Characteristics.

No.	Test	Test Procedure	Frequency	Output Data
1	Cracking	Visual walking surveys • Alligator cracking • Block cracking • Transverse cracking • Longitudinal cracking	• Just after construction • Periodically at in-service phase (i.e., twice per year—just after winter and summer, respectively)	• Crack length/width • # of cracks • % of cracking • Severity
2	Surface Rutting	Straightedge at 100-ft interval in both wheel paths		Rut depth (in.)
3	Other Distress	Visual walking surveys • Raveling, • Bleeding, • Patching, etc.		• # of distresses • Severity • % coverage
4	Surface Profiles	High-speed profiler in both wheel paths		• IRI (in./mile) • PSI
5	FWD	9 kips drop every 25 ft in outside wheel path		• Surface deflections • Back-calculated modulus
6	GPR	Outside wheel path		• Layer thickness • Forensic defects

In addition to the routine performance monitoring listed in Table 27, traffic and climatic data should be collected periodically. For collecting the traffic data, the following methods will be available:

- Pneumatic traffic tubes and traffic counters/classifiers.
- High-speed portable WIM systems, if available and where applicable.
- Existing permanent WIM stations, where available and if close to the PP test section.

The field data collected from the test sections can be used as indicators or thresholds for maintenance requirement of PPs. Since the PP has a different HMA structure and superior performance compared to conventional flexible pavements, its own indicators and thresholds should be established for metrics of rehabilitation and maintenance requirement. Researchers recommended the performance thresholds for Texas PP listed in Table 28.

Table 28. Recommended Performance Thresholds for Texas PP.

Item		Thresholds for Good Performance
Surface roughness	QC/QA IRI	65 in./mile
	IRI after 20 years	172 in./mile
Surface rutting after 20 years		0.5 in.
Fatigue cracking after 20 years		25%

TEXAS PP DATABASE SYSTEM

To support future analysis and research studies as well as serving as a reference data source and diagnostic tool for engineers, researchers have developed a prototype database to store all data collected from 10 in-service Texas PP sections. Some of the material properties of each layer, traffic, climatic, and field performance data collected from the existing in-service Texas PP sections were obtained from the Project 0-6658 database and stored in the Texas PP database. For processing and storing data, the Microsoft Access was selected as the database platform due to its commercial availability, familiarity, friendliness, and easy access to TxDOT engineers. Figure 51 and Table 29 shows a screenshot of the prototype Texas PP database and data contents in the database, respectively.

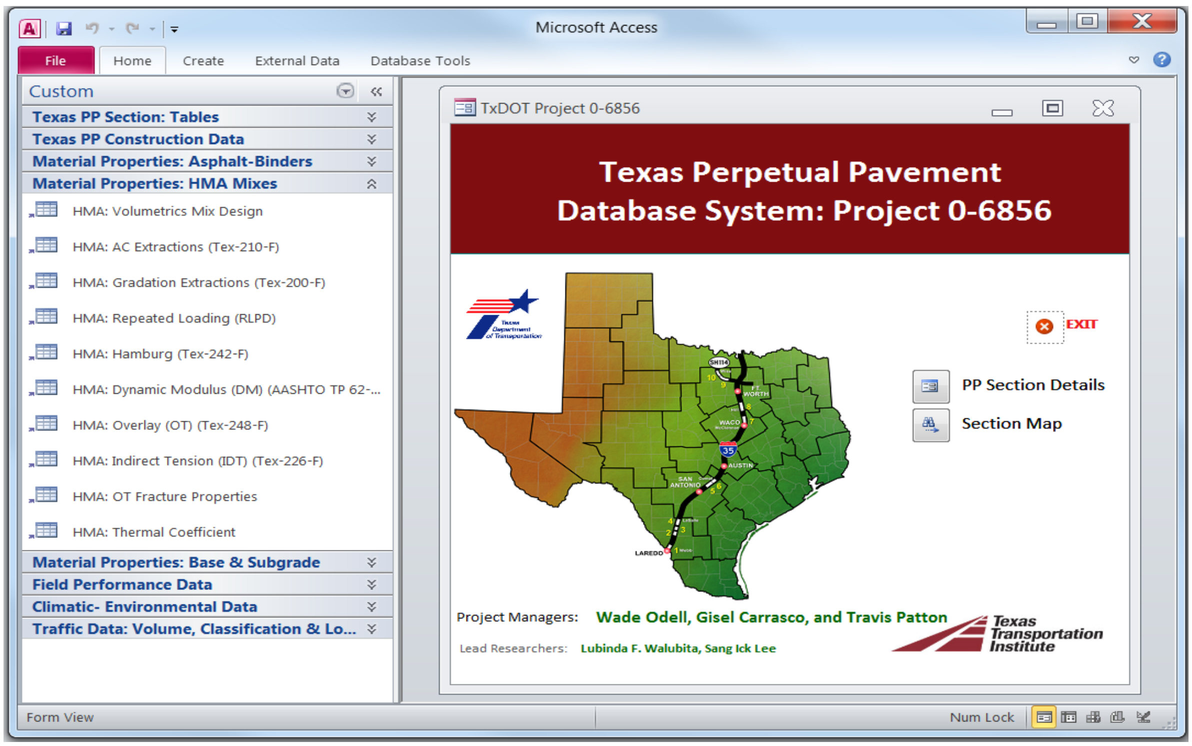


Figure 51. Screenshot of Prototype Texas PP Database.

Table 29. Types of Data Collected in Texas PP Database.

No.	Item		Type of Data
1	Section details		Control-section-job (CSJ), district/county, construction date, mile marker, coordinate, etc.
2	Material property	Asphalt binder	Specific gravity, viscosity, dynamic shear rheometer, multi-stress creep and recovery, bending beam rheometer, elastic recovery, PG, etc.
3		HMA	RLPD, HWTT, DM, OT, IDT test, thermal coefficient, etc.
4		Base/subgrade soil	Gradation, Atterberg limit, Specific gravity, moisture-density curve, Texas triaxial, shear strength, unconfined compressive strength, etc.
5	Field performance		Surface rutting and cracking survey, profiling, FWD, dynamic cone penetrometer, GPR, etc.
6	Climate		Avg. temperature, precipitation, ground water table, etc.
7	Traffic		Volume and classification, load spectra by axle types, truck distribution and growth factor, etc.

SUMMARY

For best practices of Texas PP design and construction, researchers recommend:

- Dense-graded mixes such as the SP and/or Type B mix should be used for the main structural load-carrying RRL as opposed to the coarse-graded SFHMA mixes used in the current Texas PP design concept for better compactability and constructability.
- The total HMA thickness of alternative Texas PP structure is reducible to around 14 in. from the current average 22 in. This 36 percent reduction in HMA layer thickness may also potentially translate into up to 36 percent cost savings.
- Three PP structural design alternatives were proposed based on three traffic levels, namely: (a) ESALs ≤ 30 million, (b) 30 million < ESALs ≤ 50 million, and (c) ESALs > 50 million. Computational modeling using FPS and TxME based on actual measured traffic data and material properties indicated that the proposed 14 in. total HMA thickness was structurally sufficient for an expected traffic level of up to 70 million 18-kip ESALs.
- The material properties of PP layers should be characterized in the laboratory to meet the functional requirements of each layer and obtain the typical data inputs required for M-E designs and analysis.
- The PP constructability should be improved by using more workable material (dense-graded or SP mixes), proper material transfer device (e.g., the Roadtec MTD), and optimized compaction lift thickness of HMA layers (i.e., ≤ 4 in.).
- To assist with checking pavement construction quality and uniformity, the IR thermal imaging system, GPR, and CMS should be considered as effective tools for the PP construction QA/QC program.

- During and after construction, field data should be collected on new Texas PP sections to evaluate the material properties and in-situ pavement performance and also to generate the required data inputs for calibrating the M-E models.
- Prototype Texas PP database system was developed and managed to store all the data collected from the field monitoring of the in-service PP test sections, design phase, and construction phase. The PP database will ultimately serve as a vital reference data source and diagnostic tool for researchers and engineers, respectively.

CHAPTER 7. SUMMARY AND CONCLUSIONS

This final report documents and provides the work performed, results achieved, and alternatives recommended in Project 0-6856, *Sustainable Perpetual Asphalt Pavements and Comparative Analysis of Life-Cycle Cost Comparison with Conventional Pavements*. This study reviewed the existing PP design and construction practices with a view of enhancing the design procedures and recommending the best construction practices to meet the current traffic demands. This final chapter summarizes the overall work, conclusions, and recommendations drawn from this study, as follows:

- It was reviewed that the Texas PP design theory based on the Asphalt Institute design philosophy is for heavily trafficked highways without major structural rehabilitation and/or reconstruction activities up to 50-year service life. This long-lasting pavement system is realized with enough structural strength from multiple functional asphalt layers to mitigate bottom-up fatigue cracking and rutting by minimizing horizontal tensile strain at the bottom of HMA layer and compressive strain at the top of subgrade, respectively.
- The global PP data collected from a total of 16 states and 21 countries showed that Texas PP design requires the thickest asphalt layers (22 in.) using premier mixtures among the states and countries reviewed while those PP sections using thinner layers showed good field performance. Thus, it is imperative that current Texas PP design procedures need significant improvement in material quality and thickness reduction for cost-effectiveness.
- In Texas, 10 PP sections had been constructed since 2001 in four TxDOT districts, including Fort Worth, Laredo, San Antonio, and Waco. The majority of the PP structures are conservatively thicker with a total HMA layer and base thicknesses averaging 22 and 8 in., respectively.
- The field performance of each Texas in-service PP section was evaluated using the Texas flexible pavement database from Project 0-6658. The evaluation was performed based on the field performance data collected from the 500-ft test sections, including rutting, surface cracks, and IRI/PSI are under analysis limit and still in good condition. While the in-service PPs were more than 10 years old, it is likely that they are still in good condition without major structural M/R activities.
- Using the enhanced TxME for perpetual and flexible pavements and AASHTOWare Pavement ME Design for rigid pavement, the comparative performance prediction between the in-service PP and conventional pavements were conducted for defensible performance-effectiveness justifications of PP. From the comparative evaluation, the PPs last mainly for 50-year design life without significant structural failures while the conventional flexible and rigid pavements were predicted to fail earlier with shorter service life under the same traffic loading and climatic conditions.
- As a key objective of this study, LCCA was conducted to compare the total agency (expenditures) and user costs of PPs against competing project alternatives such as conventional flexible and/or rigid pavements under the same traffic and climatic condition. For the comparisons, the activity plans of conventional flexible pavement were set with two scenarios of an overlay ever 4 and 8 years, respectively, while the conventional rigid pavement (CRCP) was set to full-depth repair at 30 years after construction.

- The LCCA of conventional flexible pavements indicated that the agency cost was higher in Scenario 1 (overlay after 4 years) and comparable in Scenario 2 (overlay after 8 years) to the PP. However, the user cost was the highest due to frequent M/R activities for both scenarios. On the other hand, the LCCA of conventional rigid pavement indicated the highest agency cost during the analysis period due to expensive initial construction and repair while it has the lowest user cost. The LCCA shows higher cost-effectiveness of PP compared to the conventional flexible and rigid pavements during their life cycle (50 years) due to lower and/or comparable agency and user costs as follows:
  - Initial construction cost: conventional rigid > PP > conventional flexible.
  - Total agency cost: conventional rigid > conventional flexible ≥ PP.
  - User cost: conventional flexible > PP > conventional rigid.
- Researchers compared the S-VECD and RDT tests to pick one for determining the EL of typical TxDOT mixtures. From the comparison, researchers found that using S-VECD can obtain the EL values at different temperatures by testing at only one temperature while the RDT needs more sets of tests for certain temperatures. In addition, only the S-VECD has a standard test procedure and corresponding data analyzing software. Thus, the S-VECD test was identified and recommended to be the suitable EL test method for the typical Texas mixtures in this project.
- To determine the default EL values for typical Texas mixtures, five plant-mixtures and eight laboratory designed mixtures were tested with the S-VECD testing. Also, three replicates were made for DM test since the S-VECD model needs to incorporate the DM for the linear viscoelastic characterization. For the laboratory designed mixtures, the AC was varied for each mixture type; thus, the effect of AC on the EL was also evaluated.
- Based on laboratory testing of 12 typical Texas mixtures, the following mix-design variables were found to have a significant influence on the EL: mixture type, aggregate gradation, binder grade/content, and RAP/RAS content.
- In this study, default EL values for the typical Texas mixtures were proposed with the relationship between EL obtained from the S-VECD testing and the corresponding OT cycles. Also, the correlative relationships can be used for the arithmetical determination of EL value at any temperature for a given OT cycles.
- Two traffic input levels of M-E PP design methods were proposed by incorporating the EL into the enhanced TxME, namely:
  - When the traffic input is ESALs (Level 2), the tensile strain at the bottom of asphalt layer is determined based on the 18-kip ESALs at the average annual temperature and compared to a single-value EL criterion. This single-value EL criterion should be either a default value based on mixture type, binder type, and climatic condition or directly determined from laboratory testing.
  - When the traffic input is load spectra (Level 1), the tensile strain distribution under different load levels and temperature conditions is determined and compared with the user defined strain distribution criteria.
- The field performance evaluations of the in-service PP sections and the extensive literature reviews indicated that the Texas PP needs possible significant improvement in material and thickness reduction for cost-effectiveness. In view of HMA materials, it is

recommended that the SP and/or Type B mix should be used for the main structural load-carrying RRL, as opposed to the SFHMA mixes for better compactability and constructability. Also, the total structural HMA is satisfactorily reducible from the current average of 22 in. to an optimal of about 14 to 16 inches without compromising the structural performance.

- Three PP structural design alternatives were proposed based on three traffic levels, namely: (a) ESALs ≤ 30 million, (b) 30 million < ESALs ≤ 50 million, and (c) ESALs > 50 million. Computational modeling using FPS and TxME based on actual measured traffic data and material properties indicated that the proposed 14 in. total HMA thickness was structurally sufficient for an expected traffic level of up to 70 million 18-kip ESALs.
- The PP constructability should be improved by using more workable materials such as dense-graded or SP mixes instead of SFHMA, proper material transfer device using the Roadtec MTD, and optimized compaction lift thickness of HMA layers (i.e., ≤ 4 in.). Also, the IR thermal imaging system, GPR, and CMS should be considered as effective tools for the PP construction QA/QC program.
- As a vital reference data source and diagnostic tool for researchers and engineers, prototype Texas PP database system was developed and managed to store all the data collected from the field monitoring of the in-service PP test sections and the design and construction phase. The PP data including field performance are also contained in the DSS – Database for Texas Flexible Pavements and Overlays.

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APPENDIX I. GLOBAL DATA ON PPS: UNITED STATES AND INTERNATIONAL COUNTRIES

Table I-1. Global Data on PPs: U.S.

Item		Texas	New Mexico	California	Illinois
Nomenclature		PPs	PPs	Full depth asphalt	PPs
Design Specification		Asphalt Inst. Design philosophy	AMEC 2007	-	IDOT Bureau of Design and Environment Manual
Design Software		FPS 21W	MEPDG Version 1.0	CIRCY, CA-4PRS	ILLIPAVE
Design Traffic		ADT > 100,000	32 million ESALs	10,000 AADTT	11,760 AADTT
Highway Class		Heavy trafficked (IH, SH)	Heavy trafficked	IH	IH
Strain Criteria	**Tensile**	$\varepsilon_t \le 70$ με	$\varepsilon_t \le 60$ με	$\varepsilon_t \le 70$ με	$\varepsilon_t \le 70$ με
	Compress.	$\varepsilon_v \le 200$ με	-	$\varepsilon_v \le 200$ με	$\varepsilon_v \le 200$ με
Design Life		50 years	30 years	40 years	30 years
PVMNT Thick.	**HMA**	≈ 22 in.	15 in.	13 in.	17.5 in.
	Sublayer	At least 8-in. base	6 in.	6-in. granular base	8-in. granular base
# of Layer	**HMA**	4 layers	3 layers	3 layers	2 layers
	Total	6 layers	5 layers	4 layers	-
Construction Specifications		TxDOT Standard Specifications	NMDOT Standard Specification	Caltrans Construction Manual (Chapter 4)	IDOT Standard Specifications
HMA Mixes	**Surface**	PG 70-22	PG 76-22	PG 46-40	PG 76-28
	Lower lift	PG 64-22	PG 70-22	PG 64-16	PG 70-22
Performance History		Minor surface renewal after 20 years	Minor surface renewal after 8 years	Minor surface renewal after 19 years	Minor surface renewal after 19 years
No. of PP Sections		10	1	1	1

Legend: AADTT = Annual Average Daily Truck Traffic

Table I-1. Global Data on PPs: U.S. (Continued).

Item		Ohio	Wisconsin	Iowa	Kansas	Kentucky
Nomenclature		PPs	PPs	PPs	PPs	PPs
Design Specification		-	AASHTO 1972	AASHTO 1993	AASHTO 1993	-
Design Software		Kenlayer, PerRoadExpress	WisPave	PerRoad 3.5, PerRoadXpress 1.0	EVERSTRESS	-
Design Traffic		-	5500 AADTT	5000 AADT	5000 AADTT	10,000 AADTT
Highway Class		IH, SH	SH	Heavy trafficked (IH, SH)	IH	IH
Strain Criteria	**Tensile**	$\varepsilon_t \leq 70\ \mu\varepsilon$	$\varepsilon_t \leq 70\ \mu\varepsilon$	$\varepsilon_t \leq 65\ \mu\varepsilon$	$\varepsilon_t \leq 70\ \mu\varepsilon$	$\varepsilon_t \leq 70\ \mu\varepsilon$
	Compress.	$\varepsilon_v \leq 200\ \mu\varepsilon$	-	$\varepsilon_v \leq 200\ \mu\varepsilon$	-	-
Design Life		50 years	50 years	50 years	20 years	40 years
PVMNT Thick.	**HMA**	17.5 in.	12 in.	18 in.	16 in.	11 in.
	Sublayer	6-in. base	4-in. base	9-in. base	6 in.	-
# of Layer	**HMA**	4 layers	3 layers	3 layers	3 layers	2 layers
	Total	-	5 layers	-	3 layers	-
Construction Specifications		-	-	-	Standard specifications for state road and bridge construction – 2007	Standard specifications for state road and bridge construction – 2012
HMA Mixes	**Surface**	PG 76-22	PG 76-28	PG 64-34	PG 70-28	PG 76-22
	Lower lift	PG 58-28	PG 70-28	PG 58-38	PG 64-22	PG 76-22
Performance History		Minor surface renewal after 20 years	-	-	-	Minor surface renewal after 20 years
No. of PP Sections		2	5	1	4	2

Table I-1. Global Data on PPs: U.S. (Continued).

Item		Maryland	Minnesota	New Jersey	Oklahoma	Oregon
Nomenclature		PPs	PPs	PPs	PPs	PPs
Design Specification		-	Von Quintus Catalog 2001	Asphalt Institute Design Philosophy	-	AASHTO 1993
Design Software		-	ELSYM 5, WESLEA	JULEA	PerRoad	WESLEA
Design Traffic		15,750 AADTT	864 AADTT	12,000 AADTT	2000 AADTT	12,240 AADTT
Highway Class		IH	IH, TH	IH	SH	IH
Strain Criteria	**Tensile**	$\varepsilon_t \leq 65$ με	$\varepsilon_t \leq 65$ με	$\varepsilon_t \leq 70$ με	$\varepsilon_t \leq 60$ με	$\varepsilon_t \leq 70$ με
	Compress.	-	-	-	-	-
Design Life		30 years	30 years	20 years	50 years	30 years
PVMNT Thick.	**HMA**	15.5 in.	6–16 in.	16 in.	14 in.	12 in.
	Sublayer	6 in.	3–6-in. base	3-in. base	-	2-in. base
# of Layer	**HMA**	2 layers	2–4 layers	3 layers	5 layers	2 layers
	Total	-	-	-	6 layers	3 layers
Construction Specifications		-	-	-	-	-
HMA Mixes	**Surface**	PG 70-20	PG 58-28	PG 76-22	PG 76-28	PG 64-22
	Lower lift	PG 70-20	PG 64-28	PG 64-22	PG 64-22	PG 64-22
Performance History		Minor surface renewal after 12.5 years	Minor surface renewal after 16 years	Minor surface renewal after 12 years	Minor surface renewal after 20 years	Minor surface renewal after 15 years
No. of PP Sections		1	5	1	1	1

Table I-1. Global Data on PP: U.S. (Continued).

Item		Washington	Michigan	Virginia
Nomenclature		Full depth/PPs	PPs	PPs
Design Specification		AASHTO 1993	-	-
Design Software		EVERSERIES	-	-
Design Traffic		5400 AADTT	-	-
Highway Class		IH	-	-
Strain Criteria	**Tensile**	-	$\varepsilon_t \leq 65\ \mu\varepsilon$	-
	Compress.	-	-	-
Design Life		50 years	40 years	-
PVMNT Thick.	**HMA**	16 in.	6.5–14 in.	20.5 in.
	Sublayer	12-in. base	12–16-in. base	3-in. base
# of Layer	**HMA**	2 layers	3 layers	2 layers
	Total	-	-	-
Construction Specifications		-	-	-
HMA Mixes	**Surface**	PG 64-22	PG 76-22	PG 70-22
	Lower lift	PG 64-22	PG 70-22	PG 64-22
Performance History		Minor surface renewal after 18.5 years	Minor surface renewal after 20 years	Minor surface renewal after 18.5 years
No. of PP Sections		1	3	1

Table I-2. Global Data on PP: International (America, Australia, and Africa).

Item		America			Australia & NZ	South Africa
		Canada	Mexico	Brazil		
Nomenclature		PPs	PPs	Flexible pavements	Full depth asphalt pavements	Long life pavements
Design Specification		AASHTO ’93	AASHTO ’93	AASHTO ’02	Pavement design supplement	AASHTO ’02
Design Software		DARWin/PerRoad 2.4	-	-	PermPave, Lockpave, Everstress	Illipave
Design Traffic		20,400 AADTT	7022 AADT	-	-	30 million ESAL
Highway Class		Highways	-	Highways	-	Highways
Strain Criteria	Tensile	$\varepsilon_t \leq 70$ με	$\varepsilon_t \leq 120$ με	$\varepsilon_t \leq 70$ με	$\varepsilon_t \leq 50$ με	-
	Compress.	$\varepsilon_v \leq 200$ με	$\varepsilon_v \leq 250$ με	$\varepsilon_v \leq 200$ με	$\varepsilon_v \leq 200$ με	-
Design Life		50 years	-	20 years	40 years	50 years
PVMNT Thick.	HMA	13.5 in.	12.5 in.	6 in.	13 in.	2–3.5 in.
	Sublayer	21.6 in.	8 in.	10 in.	Min. 6 in.	12–20 in.
# of Layer	HMA	3 layers	3 layers	3 layers	2 layers	1 layer
	Total	5 layers	4 layers	4 layers	4 layers	3 layers
Construction Specifications		-	-	-	-	-
HMA Mixes	Surface	SP 12.5 SP 19 SP 25	PG 70-28	-	-	-
	Lower lift		PG 64-22	-	-	-
Performance History		Minor surface renewal after 18 years	Minor surface renewal after 18.5 years	-	-	-

Table I-3. Global Data on PP: International (Asia).

Item		India	China	Pakistan	Korea	Iran	Israel
Nomenclature		PPs	PPs	PPs	Long life asphalt pavements	PPs	PPs
Design Specification		IRC 37-2012	-	AASHTO 1986	Korean pavement design guide	-	Asphalt Inst. Design ’82
Design Software		IITPave, Kenpave	PerRoad	MICHPAVE	Illipave	MEPDG, Kenlayer	-
Design Traffic		>200 AADT	127,000 ADT	28.7 Millions ESALs	15 million ESAL	2000 AADTT	15 Millions ESALs
Highway Class		Expressways, National Highways	Freeways	-	-	Rural & Urban	-
Strain Criteria	**Tensile**	$\varepsilon_t \leq 70$ με	$\varepsilon_t \leq 120$ με	$\varepsilon_t \leq 117$ με	$\varepsilon_t \leq 60$ με	-	$\varepsilon_t \leq 70$ με
	Compress.	$\varepsilon_v \leq 200$ με	$\varepsilon_v \leq 280$ με	$\varepsilon_v \leq 277$ με	$\varepsilon_v \leq 200$ με	$\varepsilon_v \leq 200$ με	$\varepsilon_v \leq 200$ με
Design Life		50 years	20 years	25 years	40 years	50 years	30 years
PVMNT Thick.	**HMA**	15 in.	13 in.	10 in.	10 in.	9–15 in.	12 in.
	Sublayer	11.5 in.	16 in.	12 in.	16 in.	21 in.	6–25.5 in.
# of Layer	**HMA**	3 layers	3 layers	-	3 layers	3 layers	4 layers
	Total	6 layers	6 layers	3 layers	5 layers	4 layers	6 layers
Construction Specifications		-	-	-	-	-	-
HMA Mixes	**Surface**	-	PG 76-22	-	-	PG 58-22	PG 76-10
	Lower lift	-	PG 76-22	-	-	PG 58-22	PG 76-10
Performance History		Minor surface renewal after 20 years	Minor surface renewal after 12 years	-	-	Only 1% bottom-up fatigue cracking after 40 years	Minor surface renewal after 20 years

Table I-4. Global Data on PP: International (Europe*).

Item		Austria	Belgium	Denmark	France
Nomenclature		Long life pavements	Long life pavements	Long life pavements	Long life pavements
Design Specification		-	-	-	-
Design software		-	DimMET, EvalMET	-	-
Design Traffic		10–25 MSA	>18,000 ADT	-	14 Millions ESALs
Highway Class		-	-	-	-
Strain Criteria	**Tensile**	-	-	-	-
	Compress.	-	-	-	-
Design life			20 years	20 years	30 years
PVMNT Thick.	**HMA**	5.6 in.	7.4 in.	(3.4–6.8 in.)	3 in.
	Sublayer	16.6 in.	13.4 in.	(5–8.5 in.)	13 in.
# of layer	**HMA**	1 layer	2 layers	1 layer	2 layers
	Total	3 layers	5 layers	3 layers	4 layers
Construction Specifications		-	-	-	-
HMA Mixes	**Surface**	-	-	-	-
	Lower lift	-	-	-	-
Performance History		Minor surface renewal after 20 years	Minor surface renewal after 12 years	-	-

*European Long-Life Pavement Group—an organization formed in 1999 with the member countries listed is currently reviewing and developing technical specifications for long life pavements through its four phases of report. (This current information is listed from the Phase I and Phase II reports).

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APPENDIX II. AXLE LOAD SPECTRA DATA FOR IH 35 (COTULLA, LAREDO DISTRICT) AND SH 114 (FORT WORTH DISTRICT)

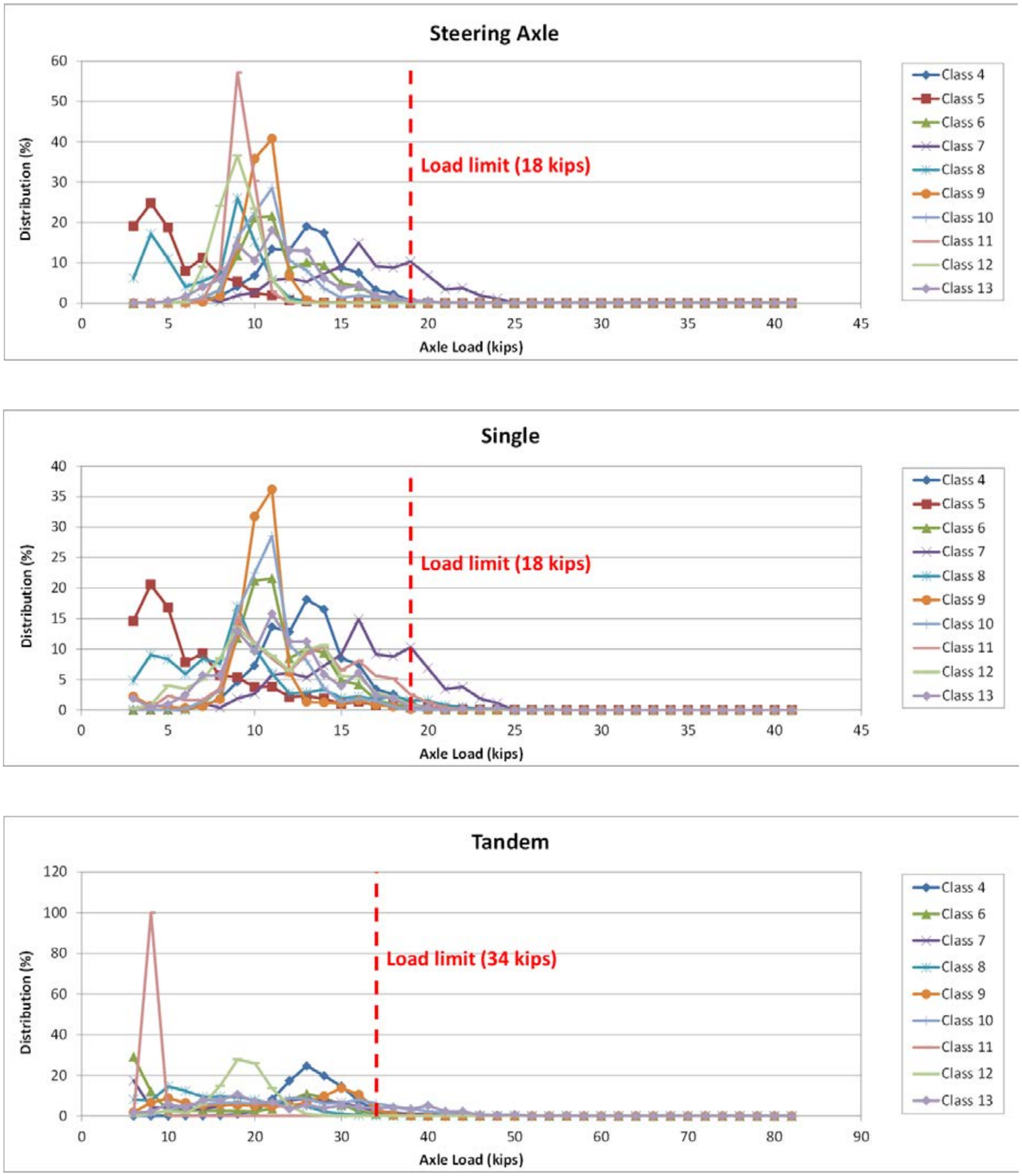
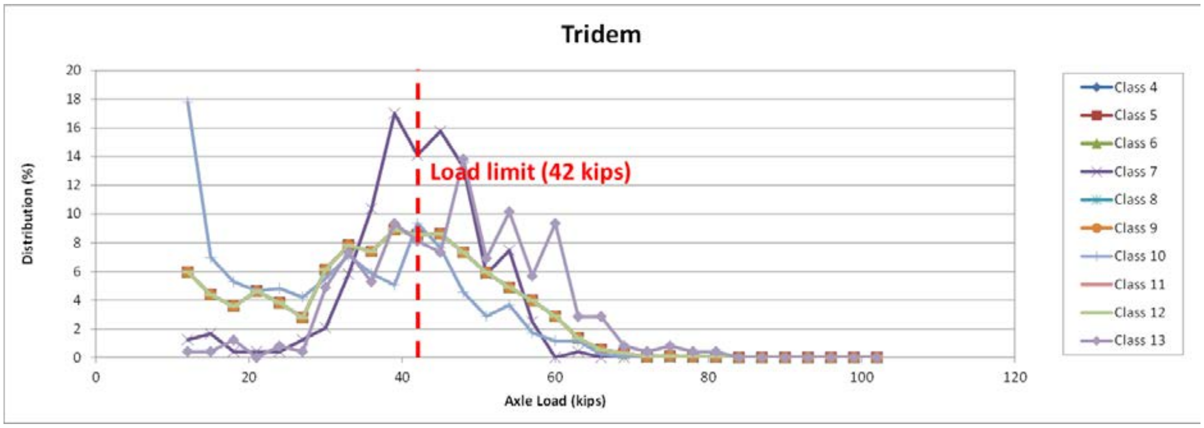


Figure II-1. Axle Load Spectra from WIM Station (IH 35 Cotulla, 2015).



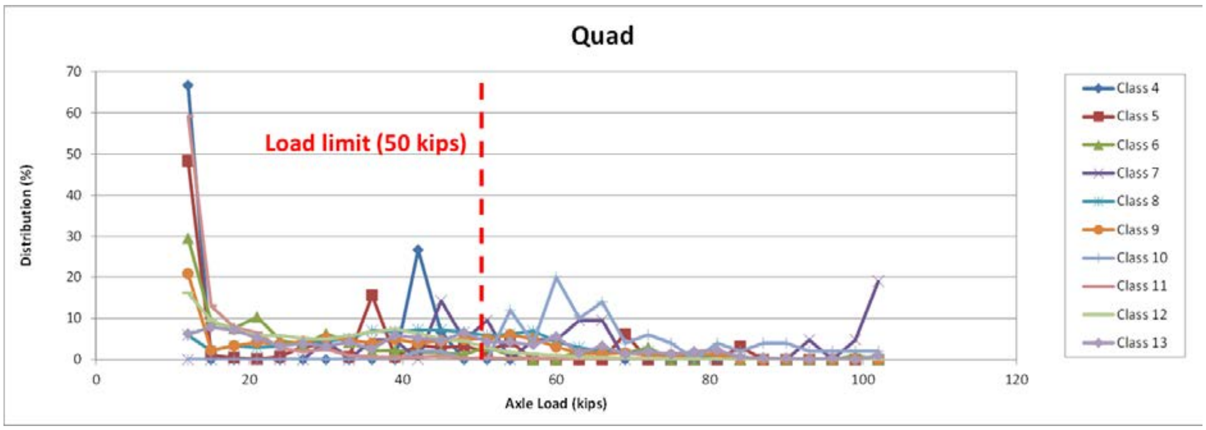


Figure II-1. Axle Load Spectra from WIM Station (IH 35 Cotulla, 2015) (Continued).

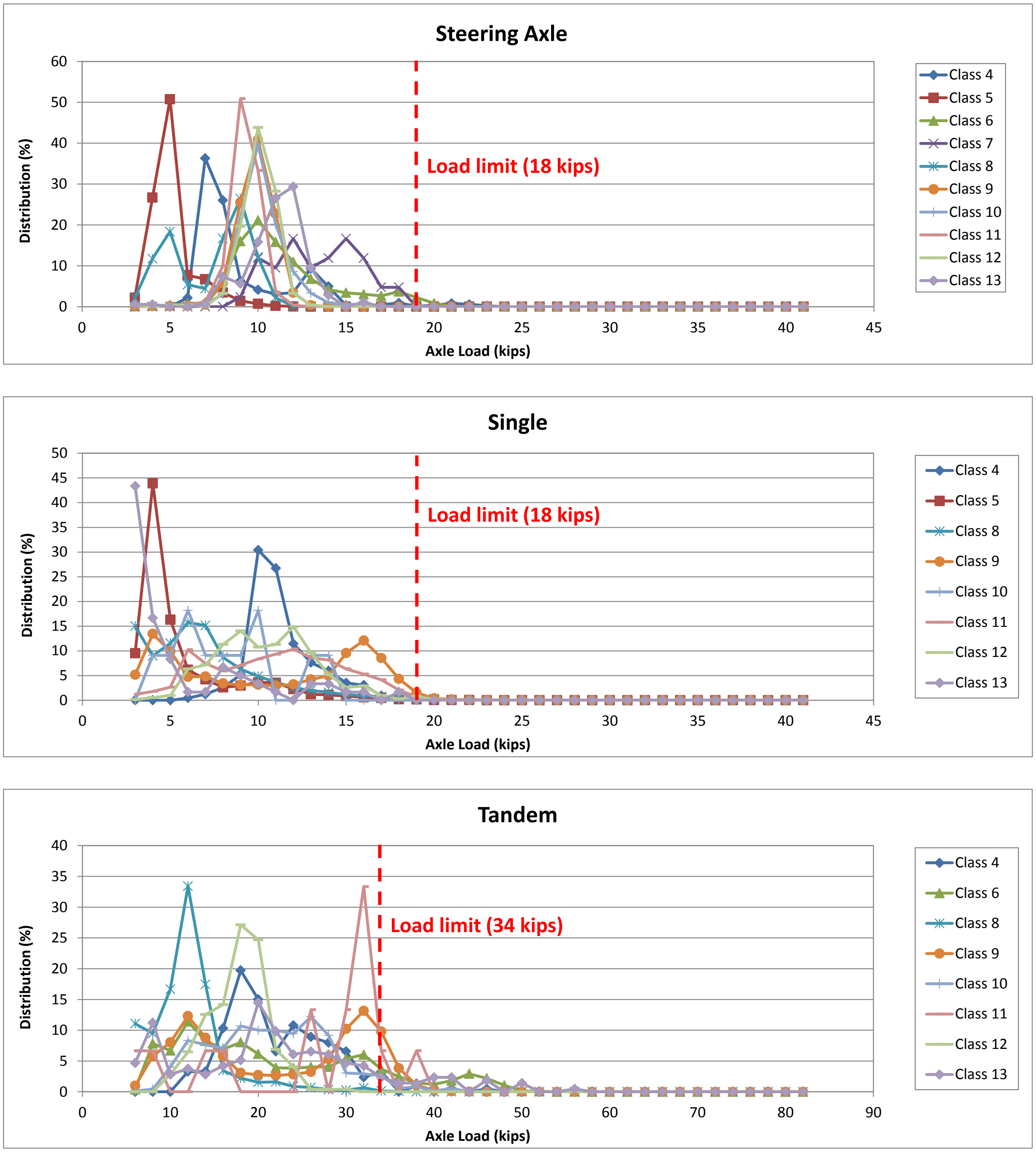


Figure II-2. Axle Load Spectra from WIM Station (SH 114 FTW, 2011).

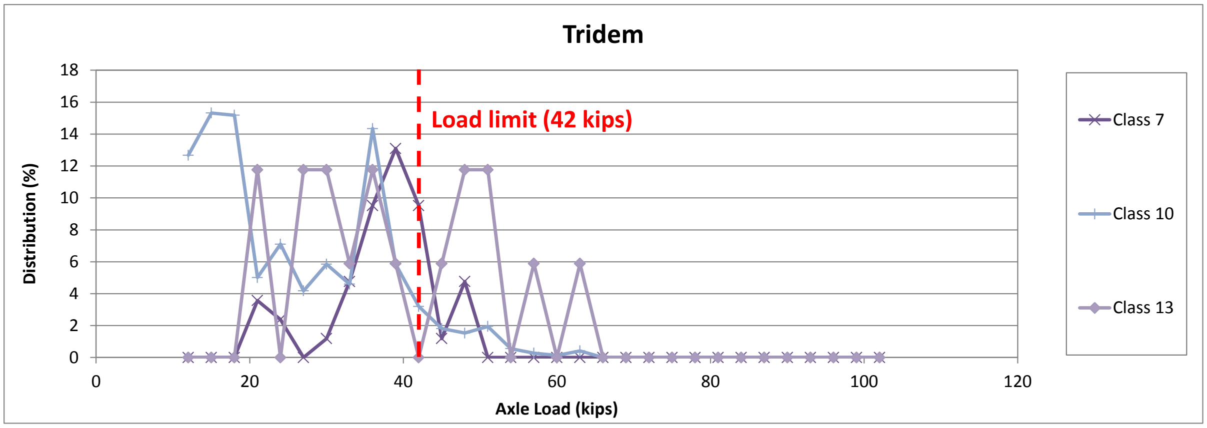


Figure II-2. Axle Load Spectra from WIM Station (SH 114 FTW, 2011) (Continued).

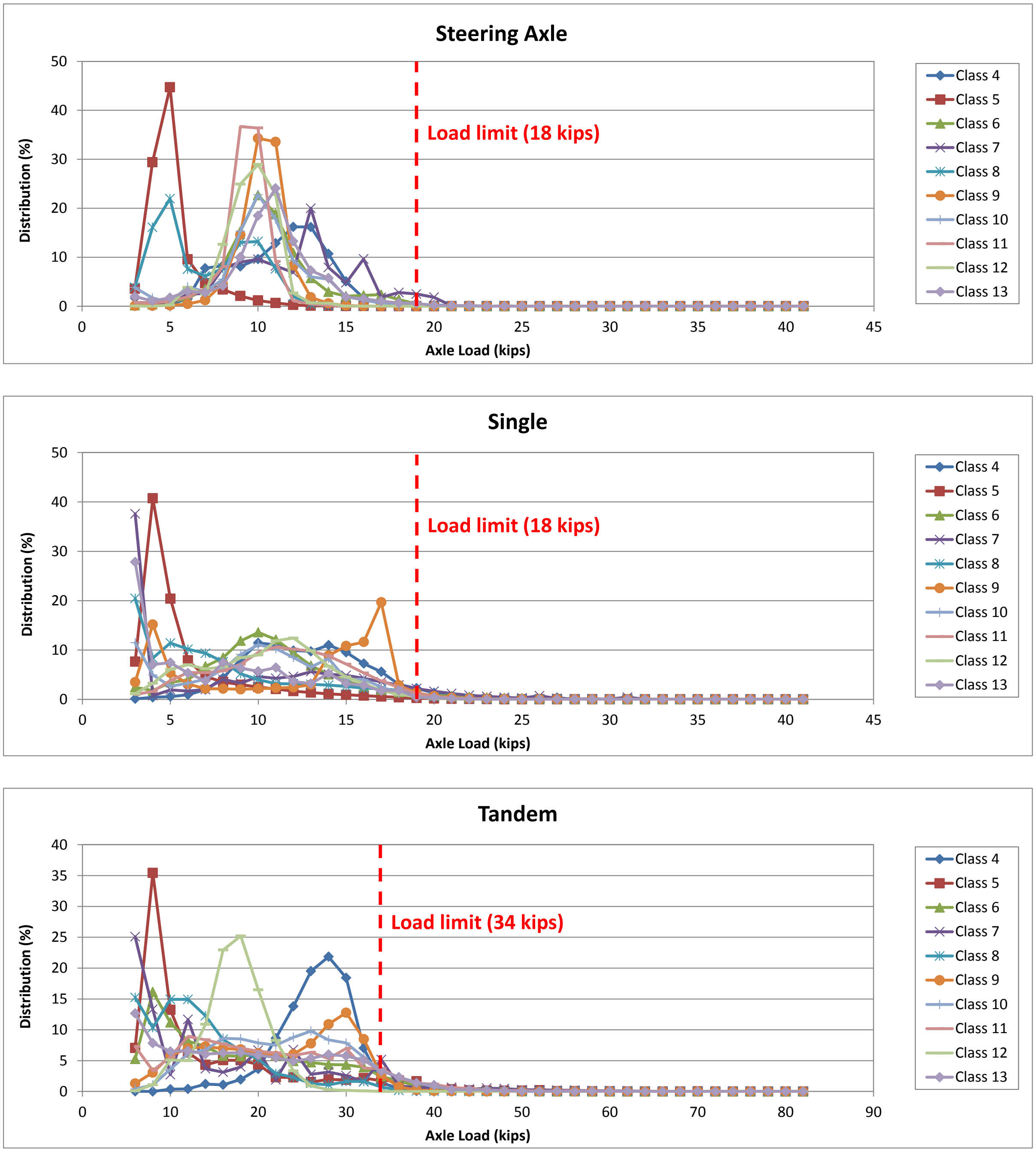


Figure II-3. Axle Load Spectra from Traffic Tube (SH 114, Fort Worth, 2014).

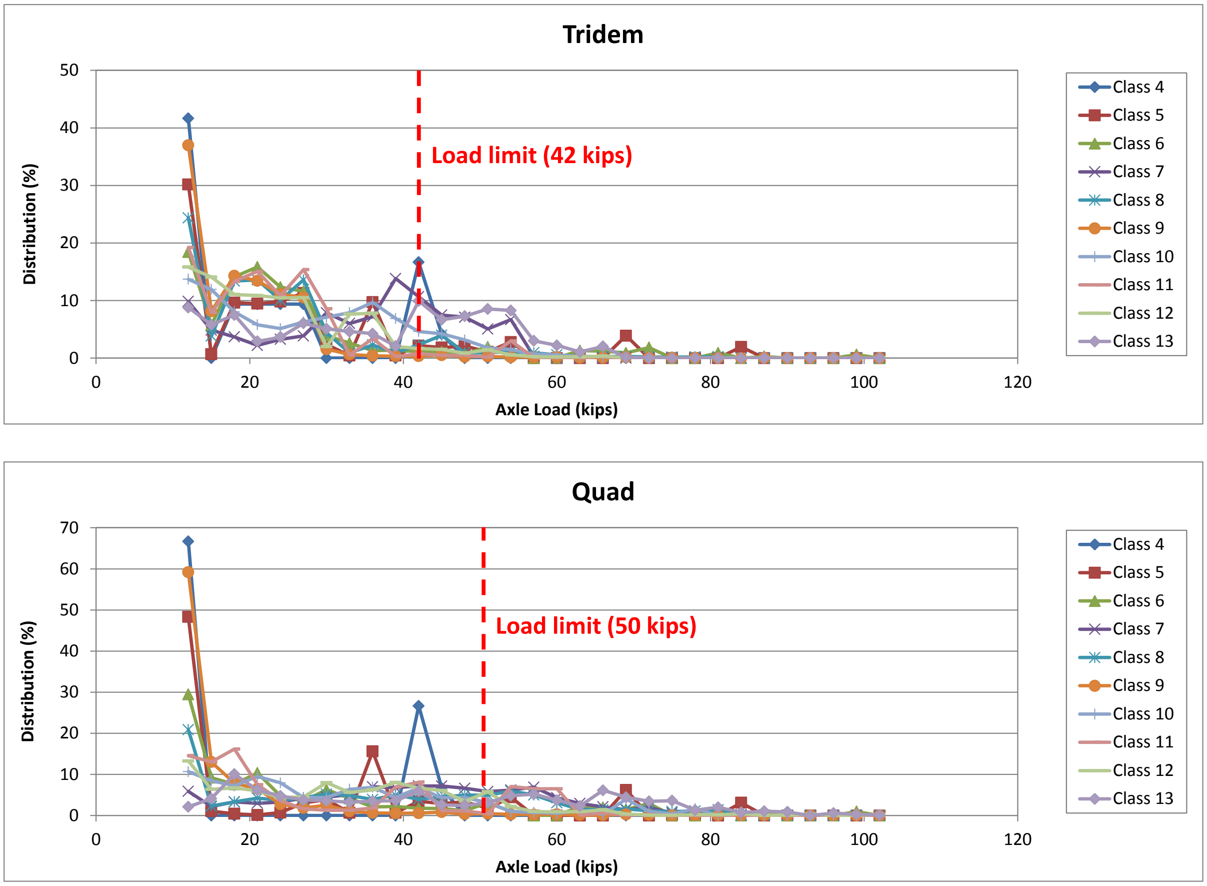


Figure II-3. Axle Load Spectra from Traffic Tube (SH 114, Fort Worth, 2014) (Continued).

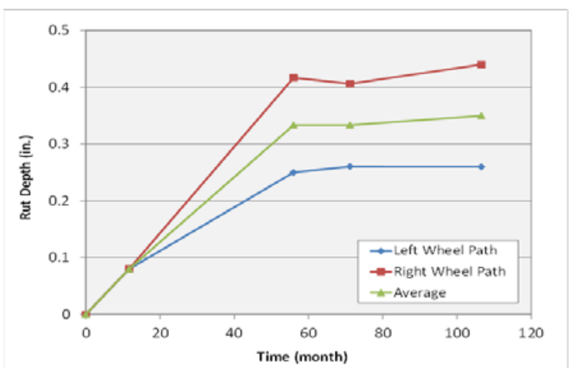
APPENDIX III. FIELD PERFORMANCE OF TEXAS PP SECTIONS

Layer No.	Thickness (in.)	Layer Material	Year Constructed
5	3.0	SMA	2007–2008
4	3.0	¾" SFHMA	
3	8.0	1" SFHMA (RRL)	
2	2.0	½" SP (RBL)	
1	8.0	3% Lime Treated Subgrade Soil	
0	∞	In-situ Subgrade Soil	

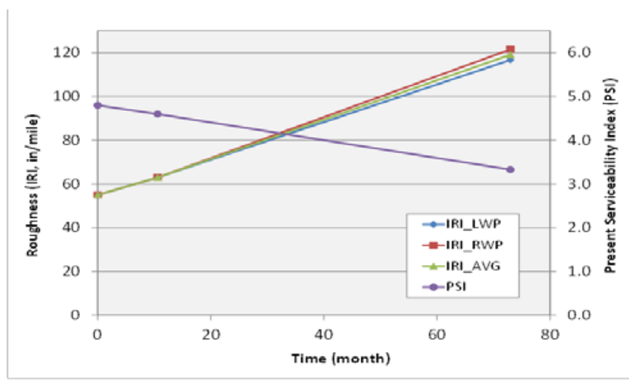
(a) Pavement Structure and Layer Materials

(b) Section Location Map

(c) Section Picture



(d) Rut Depth



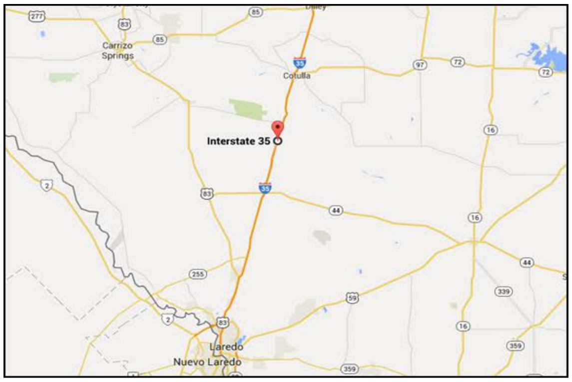
(e) IRI and PSI

(f) Surface Picture

Figure III-1. Field Performance of Sec#01 IH 35 in LRD, Webb County.

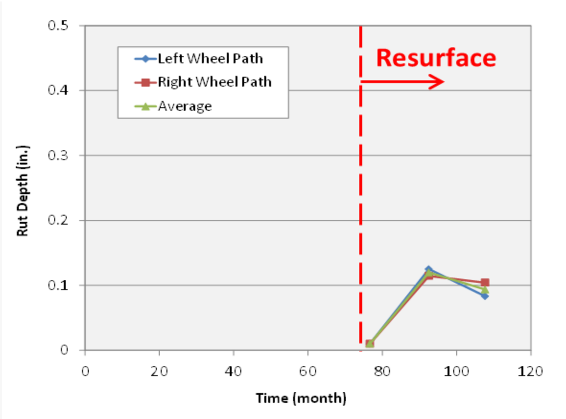
Layer No.	Thickness (in.)	Layer Material	Year Constructed
6	1.5	Type D	2011
5	3.0	SMA	2005
4	3.0	¾" SFHMA	
3	8.0	1" SFHMA (RRL)	
2	3.0	RBL (½" SP)	
1	8.0	2% CTB & Precrack Material	
0	∞	In-situ Subgrade Soil	

(a) Pavement Structure and Layer Materials

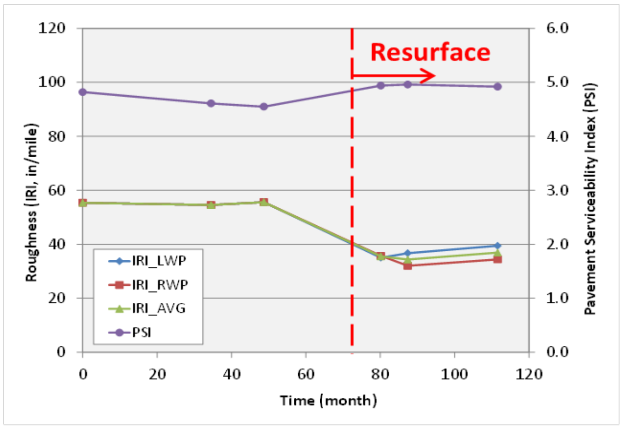


(b) Section Location Map

(c) Section Picture



(d) Rut Depth



(e) IRI and PSI

(f) Surface Picture

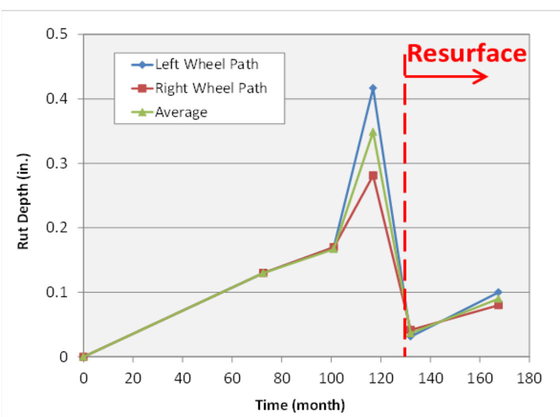
Figure III-2. Field Performance of Sec#02 IH 35 in LRD, La Salle County (Cotulla).

Layer No.	Thickness (in.)	Layer Material	Year Constructed
6	1.5–2.0	Resurfaced/Overlayed	2013/2014
5	3.0	SMA	2003
4	3.0	¾" SFHMA	
3	12.0	1" SFHMA (RRL)	
2	2.0	RBL (½" SP)	
1	8.0	3% Lime Treated Subgrade Soil	
0	∞	In-situ Subgrade Soil	

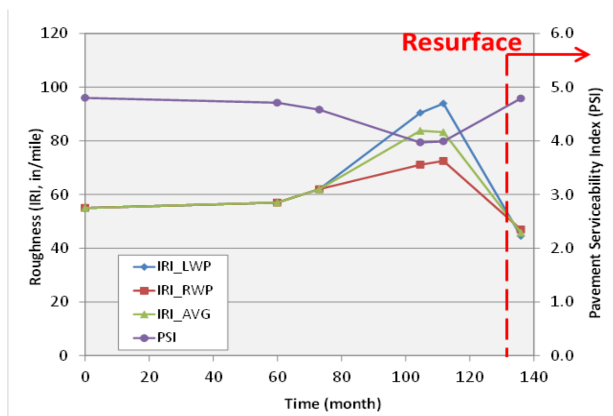
(a) Pavement Structure and Layer Materials

(b) Section Location Map

(c) Section Picture



(d) Rut Depth



(e) IRI and PSI

(f) Surface Picture

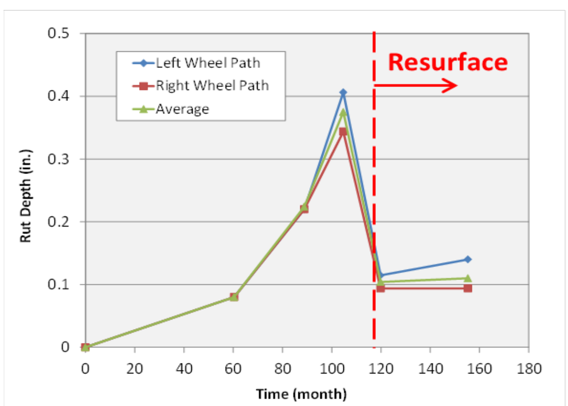
Figure III-3. Field Performance of Sec#03 IH 35 in LRD, La Salle County (Cotulla).

Layer No.	Thickness (in.)	Layer Material	Year Constructed
6	1.5–2.0	Resurfaced/Overlayed	2013/2014
5	3.0	SMA	2004
4	3.0	¾" SFHMA	
3	8.0	1" SFHMA (RRL)	
2	4.0	RBL (½" SP)	
1	8.0	3% Lime Treated Subgrade Soil	
0	∞	In-situ Subgrade Soil	

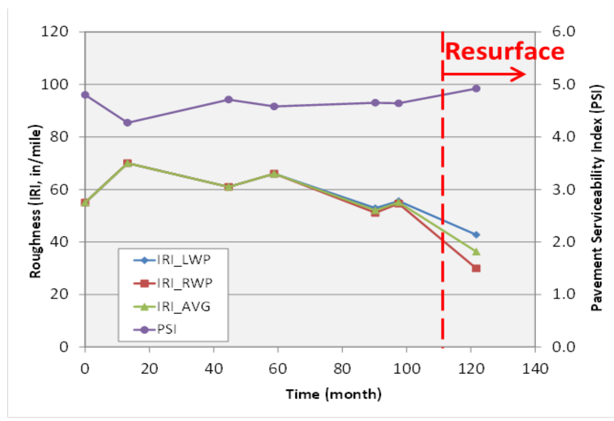
(a) Pavement Structure and Layer Materials

(b) Section Location Map

(c) Section Picture



(d) Rut Depth



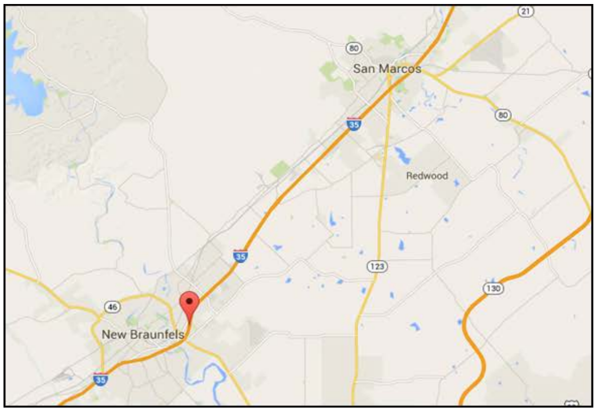
(e) IRI and PSI

(f) Surface Picture

Figure III-4. Field Performance of Sec#04 IH 35 in LRD, La Salle County (Cotulla).

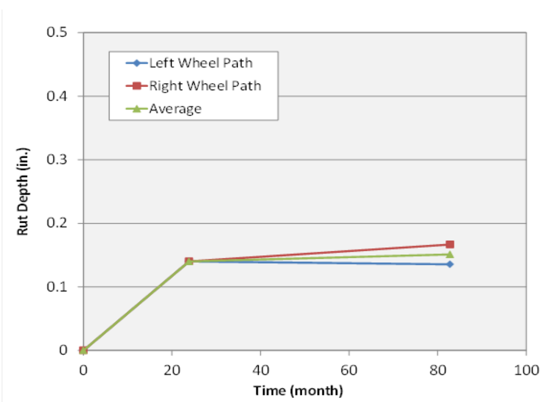
Layer No.	Thickness (in.)	Layer Material	Year Constructed
6	1.5	PFC	2007
5	2.0	SMA	2005
4	2.0	½" SFHMA	
3	12.0	1" SFHMA (RRL)	
2	4.0	RBL (½" SP)	
1	6.0	3% Lime Treated Subgrade Material	
0	∞	In-situ Subgrade Soil	

(a) Pavement Structure and Layer Materials

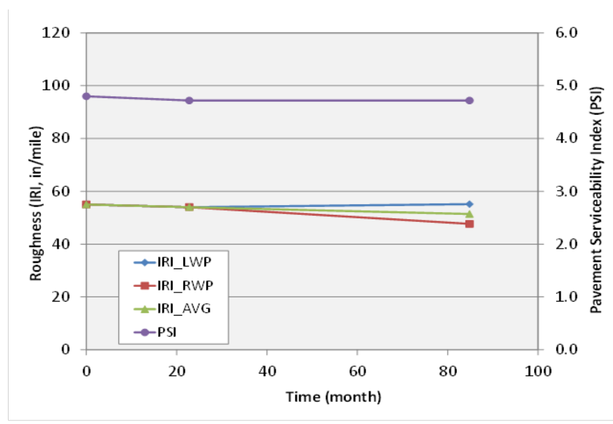


(b) Section Location Map

(c) Section Picture



(d) Rut Depth



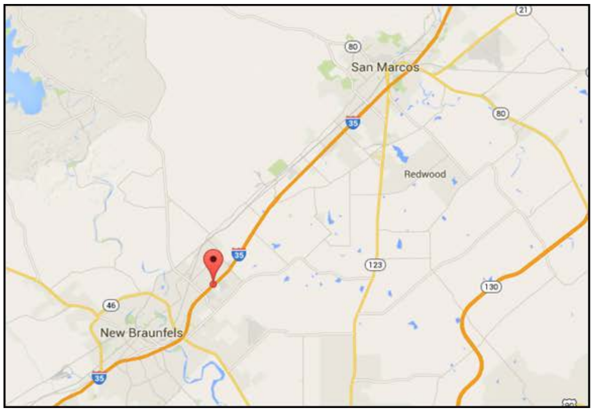
(e) IRI and PSI

(f) Surface Picture

Figure III-5. Field Performance of Sec#05 IH 35 in SAT, Comal County.

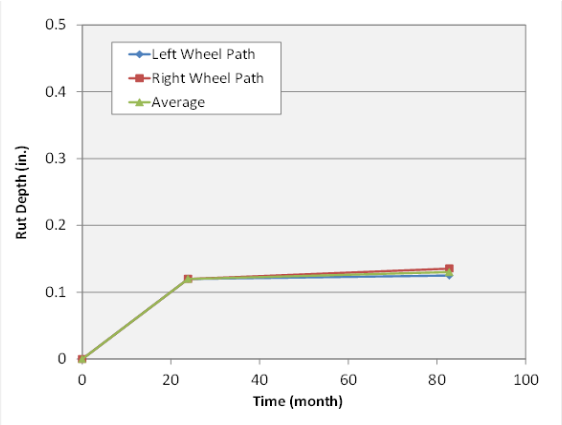
Layer No.	Thickness (in.)	Layer Material	Year Constructed
6	1.5	PFC	2007
5	2.0	SMA	2006
4	2.0	½" SFHMA	
3	12.0	1" SFHMA (RRL)	
2	4.0	RBL (½" SP)	
1	6.0	3% Lime Treated Subgrade Material	
0	∞	In-situ Subgrade Soil	

(a) Pavement Structure and Layer Materials

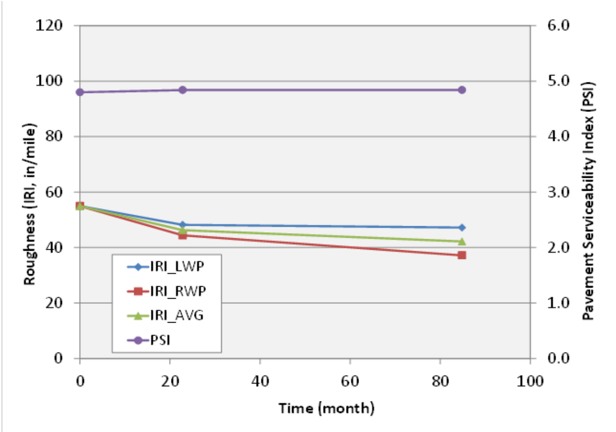


(b) Section Location Map

(c) Section Picture



(d) Rut Depth



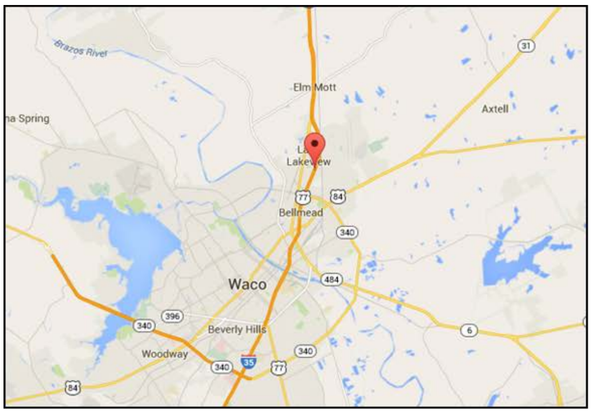
(e) IRI and PSI

(f) Surface Picture

Figure III-6. Field Performance of Sec#06 IH 35 in SAT, Comal County.

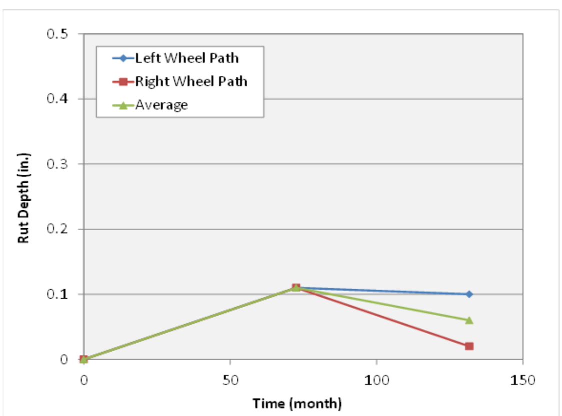
Layer No.	Thickness (in.)	Layer Material	Year Constructed
6	2.0	PFC	2003
5	3.0	SMA	
4	3.0	½" SFHMA	
3	10.0	1" SFHMA (RRL)	
2	4.0	RBL (½" SP)	
1	14.0	6% Lime Treated Subgrade Material	
0	∞	In-situ Subgrade Soil	

(a) Pavement Structure and Layer Materials

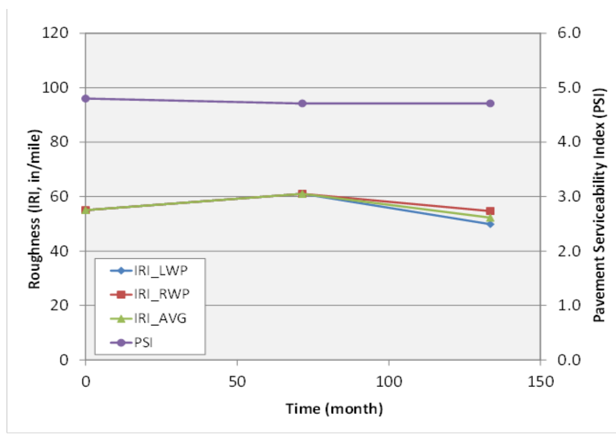


(b) Section Location Map

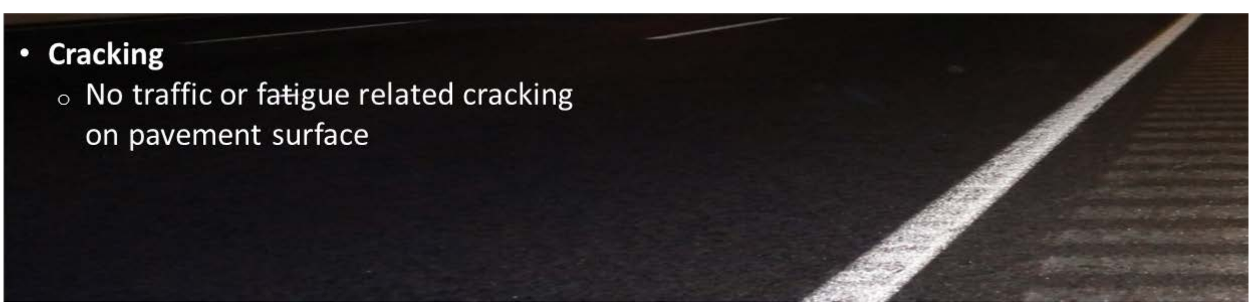
(c) Section Picture



(d) Rut Depth



(e) IRI and PSI

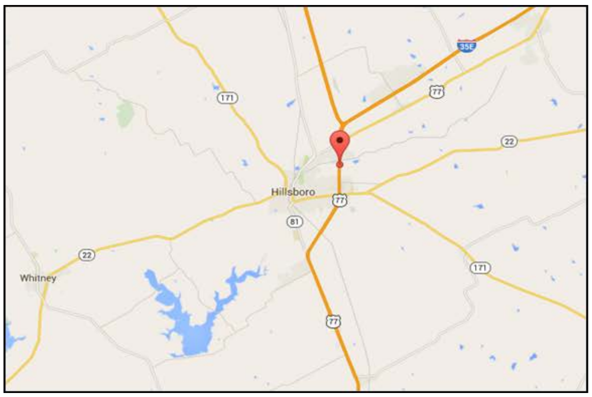


(f) Surface Picture

Figure III-7. Field Performance of Sec#07 IH 35 in WAC, McLennan County.

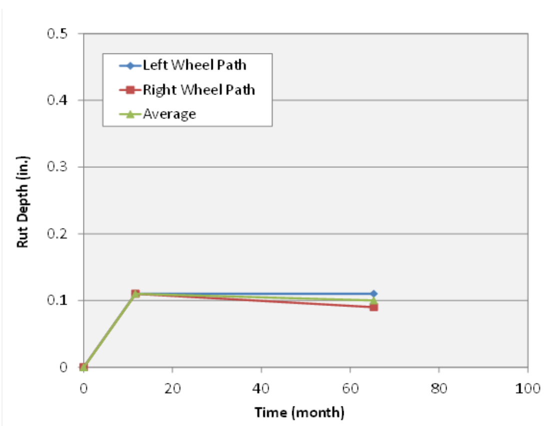
Layer No.	Thickness (in.)	Layer Material	Year Constructed
6	1.5	PFC	2006–2008
5	2.0	SMA	
4	3.0	3/4" SFHMA	
3	12.0	1" SFHMA (RRL)	
2	4.0	RBL (Type B)	
1	8.0	6% Lime Treated Subgrade	
0	∞	In-situ Subgrade Soil	

(a) Pavement Structure and Layer Materials

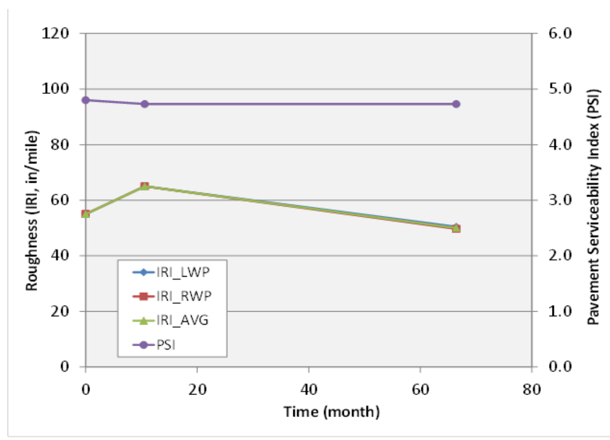


(b) Section Location Map

(c) Section Picture



(d) Rut Depth



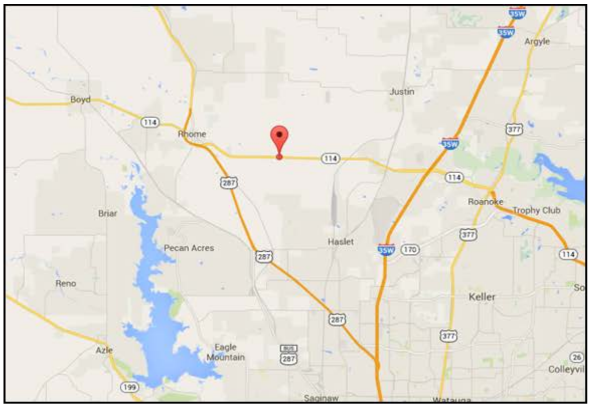
(e) IRI and PSI

(f) Surface Picture

Figure III-8. Field Performance of Sec#08 IH 35 in WAC, Hill County.

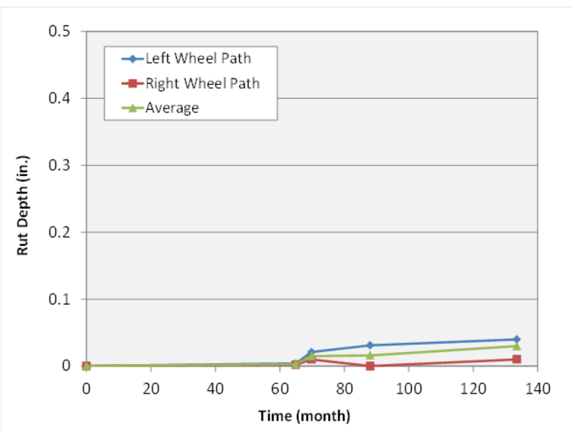
Layer No.	Thickness (in.)	Layer Material	Year Constructed
5	2.0	SMA	2006
4	3.0	Type C	2004–2006
3	13.0	Type C (RRL)	
2	4.0	RBL – Type C	
1	8.0	6% Lime Treated Subgrade	
0	∞	Subgrade (In-situ Soil Material)	

(a) Pavement Structure and Layer Materials

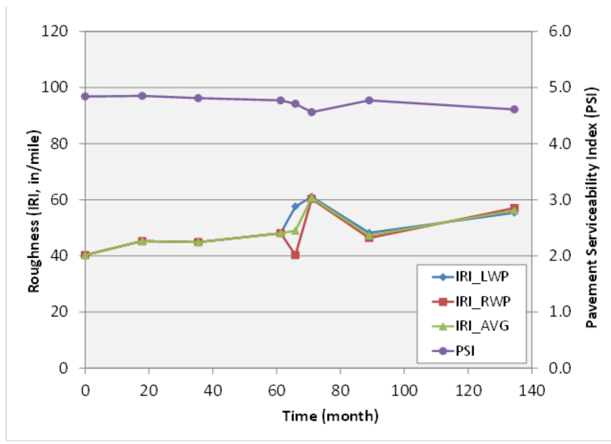


(b) Section Location Map

(c) Section Picture



(d) Rut Depth



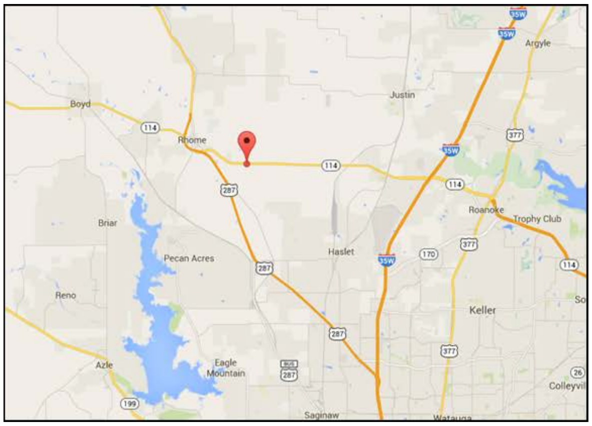
(e) IRI and PSI

(f) Surface Picture

Figure III-9. Field Performance of Sec#09 IH 35 in FTW, Wise County.

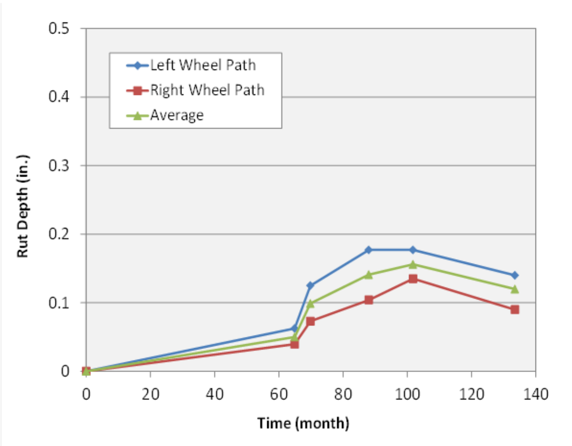
Layer No.	Thickness (in.)	Layer Material	Year Constructed
5	2.0	SMA	2006
4	3.0	3/4" SFHMA	
3	13.0	1" SFHMA (RRL)	
2	4.0	¾" SP (RPL)	2004–2006
1	8.0	6% Lime Treated Subgrade	
0	∞	Subgrade	

(a) Pavement Structure and Layer Materials

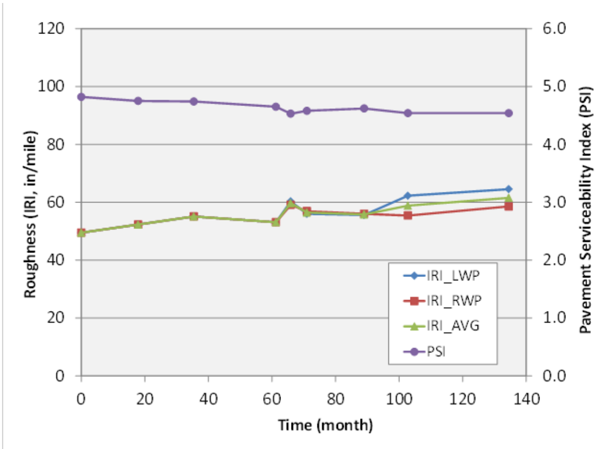


(b) Section Location Map

(c) Section Picture



(d) Rut Depth



(e) IRI and PSI

(f) Surface Picture

Figure III-10. Field Performance of Sec#10 IH 35 in FTW, Wise County.

APPENDIX IV. COMPARATIVE PERFORMANCE PREDICTION: PERPETUAL VS. CONVENTIONAL FLEXIBLE VS. CONVENTIONAL RIGID PAVEMENTS

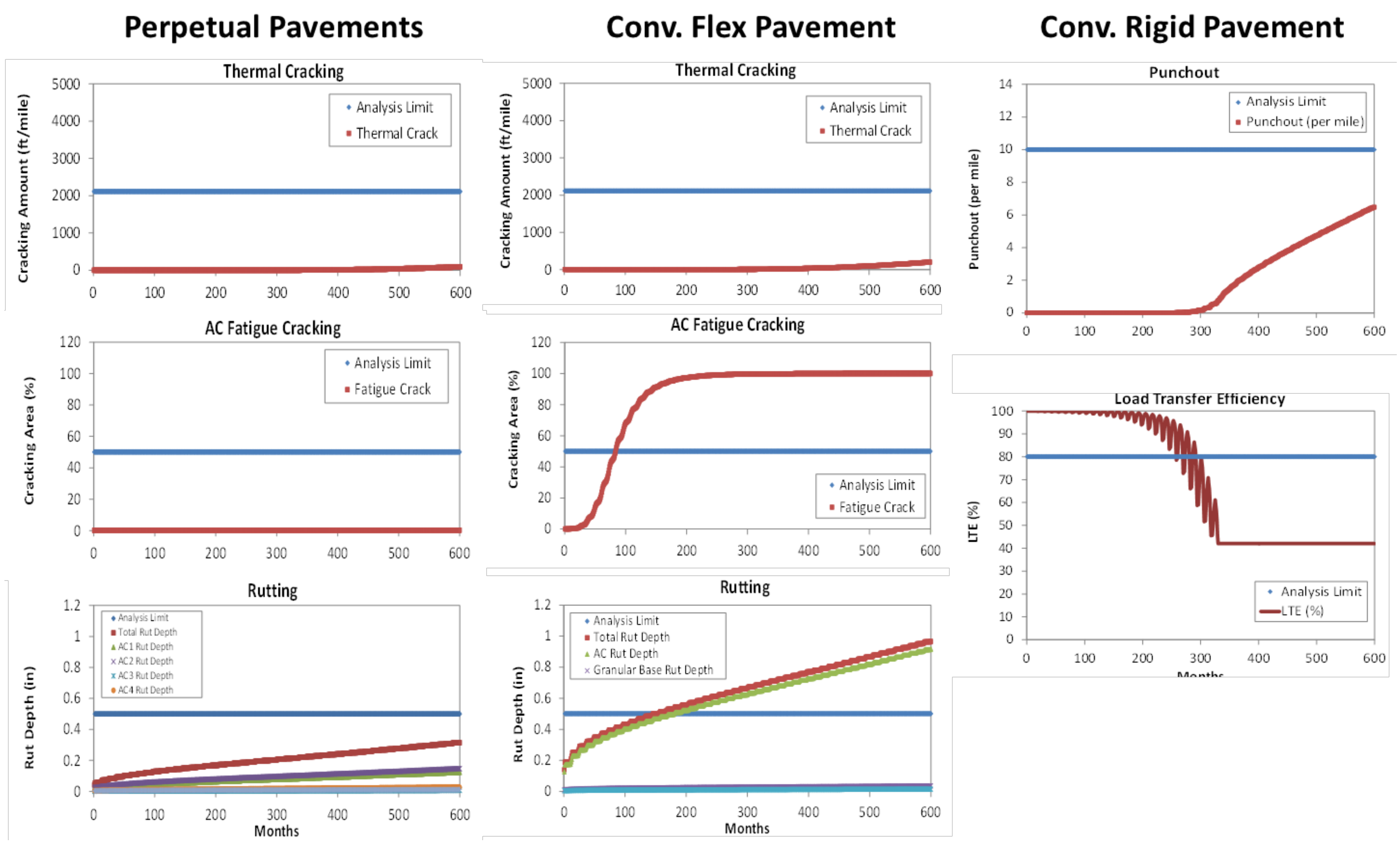


Figure IV-1. Sec #1 IH 35 LRD, Webb County.

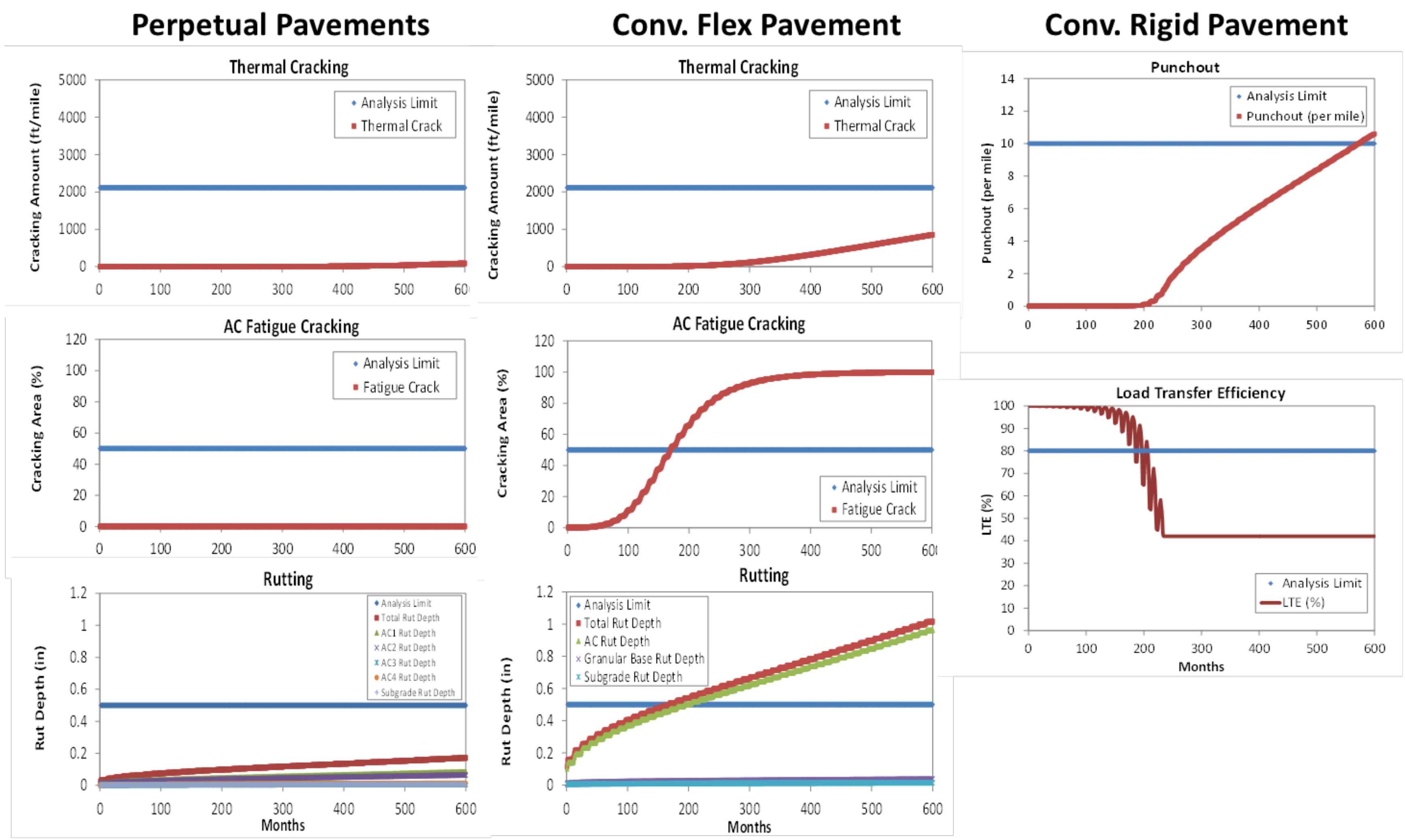


Figure IV-2. Sec #2 IH 35 LRD, La Salle County.

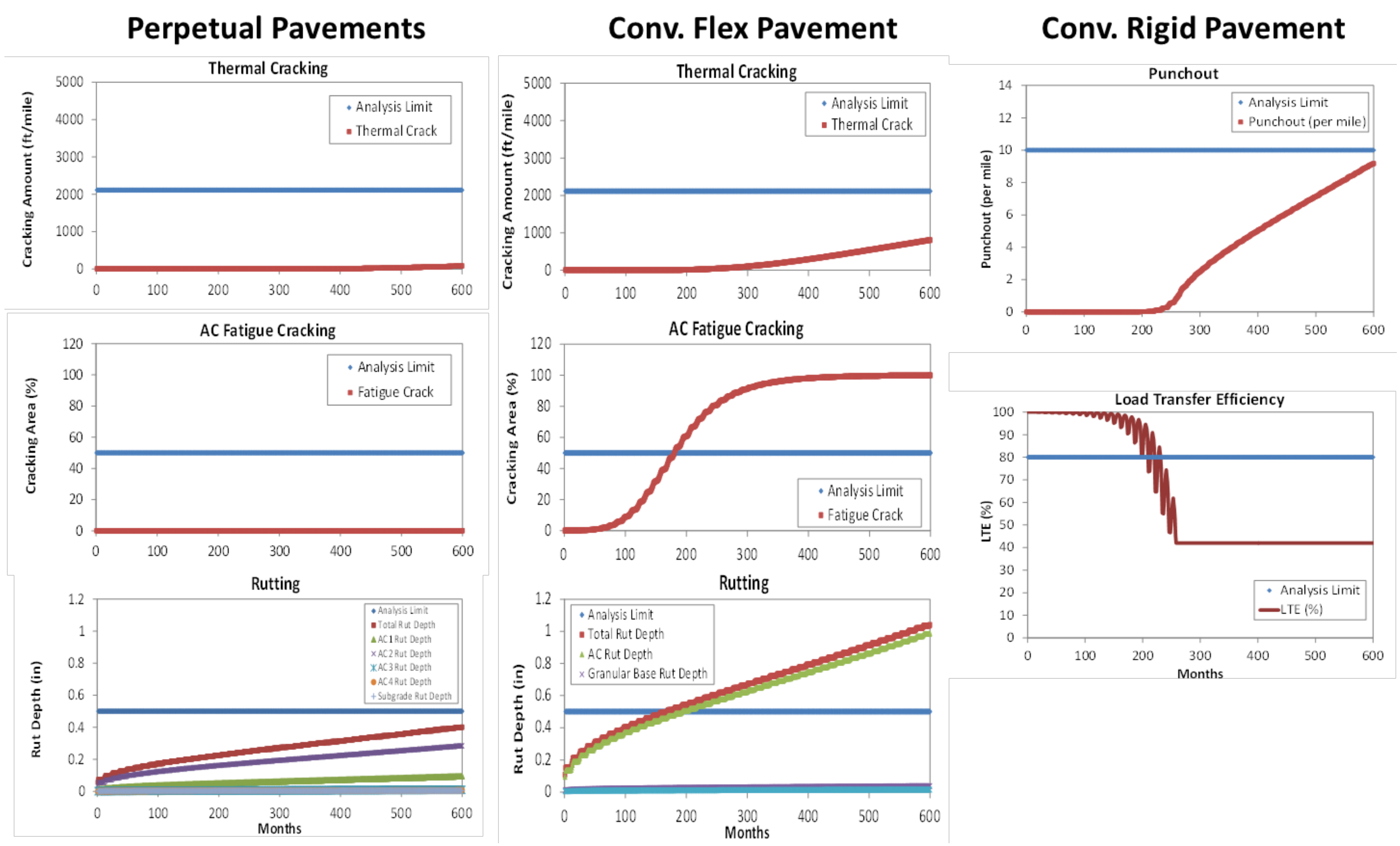


Figure IV-3. Sec #3 IH 35 LRD, La Salle County.

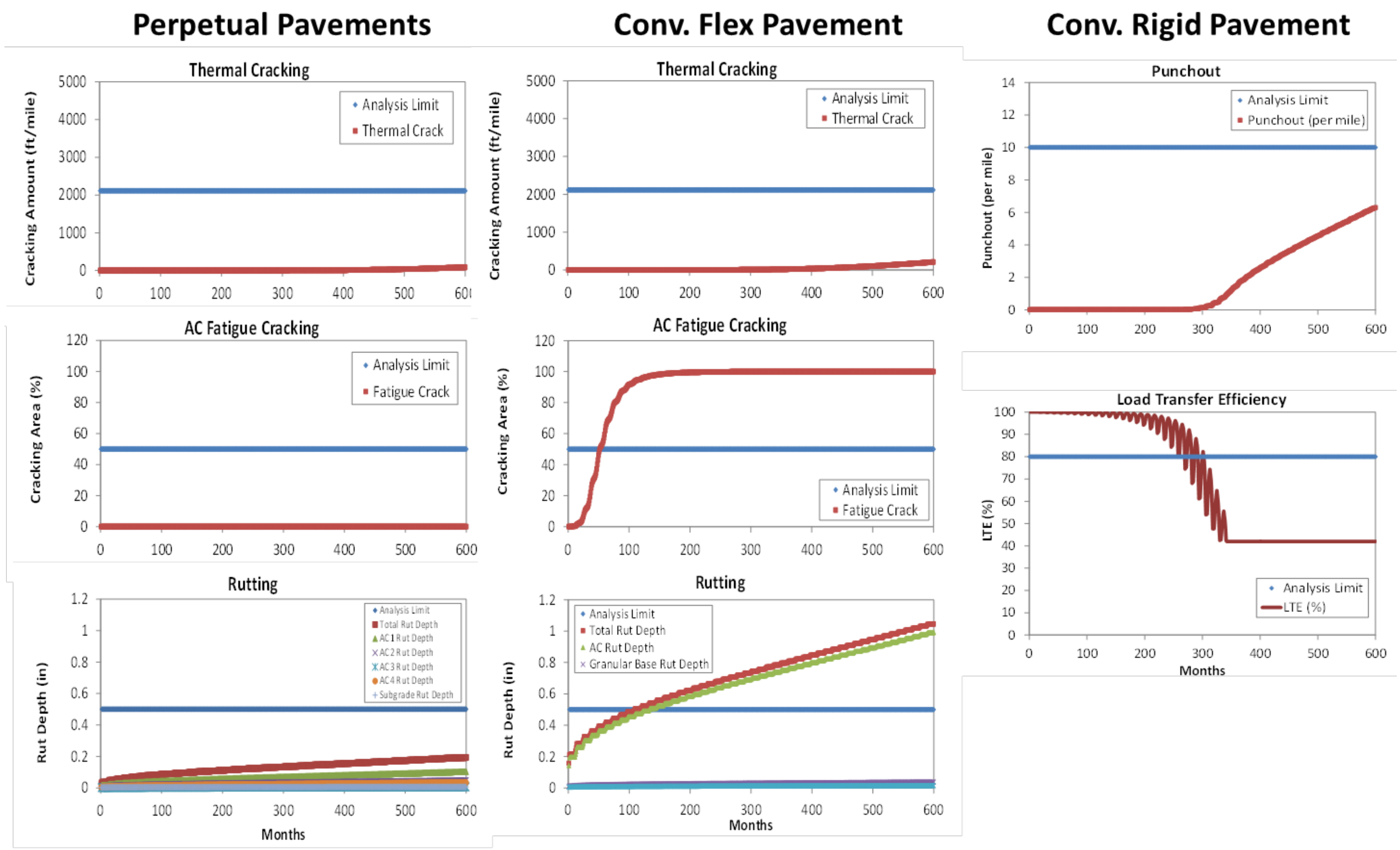


Figure IV-4. Sec #4 IH 35 LRD, La Salle County.

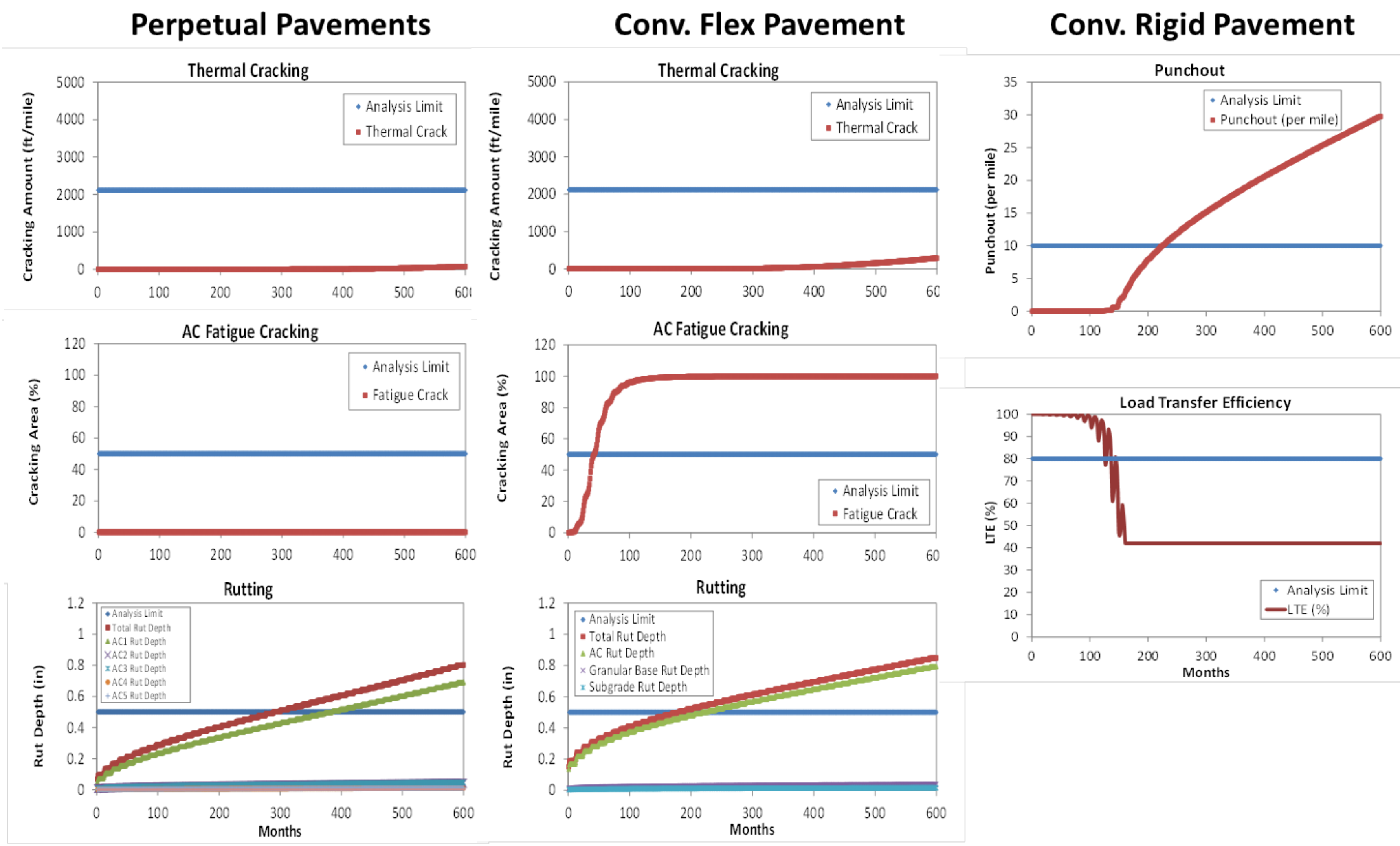


Figure IV-5. Sec #5 IH 35 SAT, Comal County.

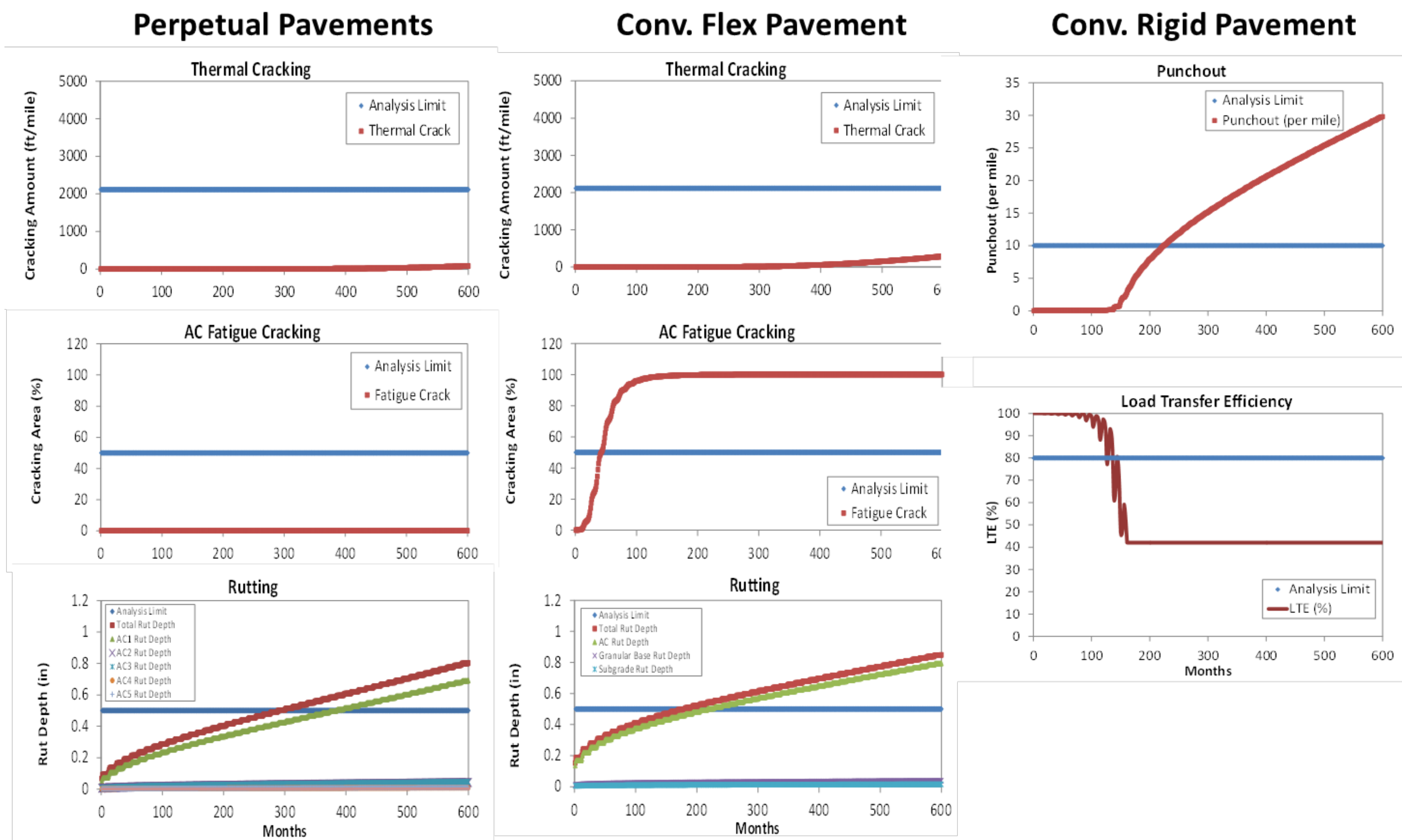


Figure IV-6. Sec #6 IH 35 SAT, Comal County.

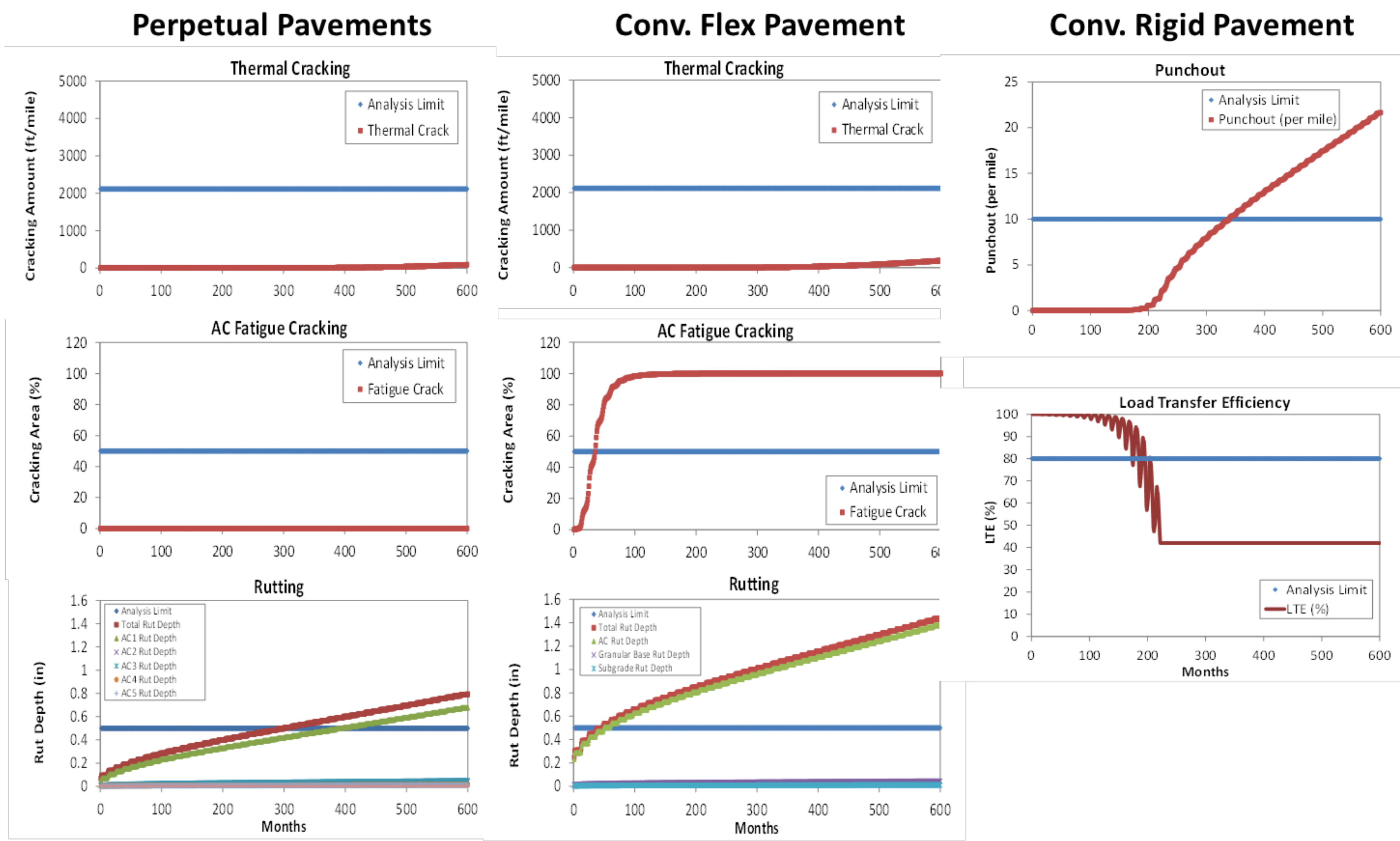


Figure IV-7. Sec #7 IH 35 WAC, McLennan County.

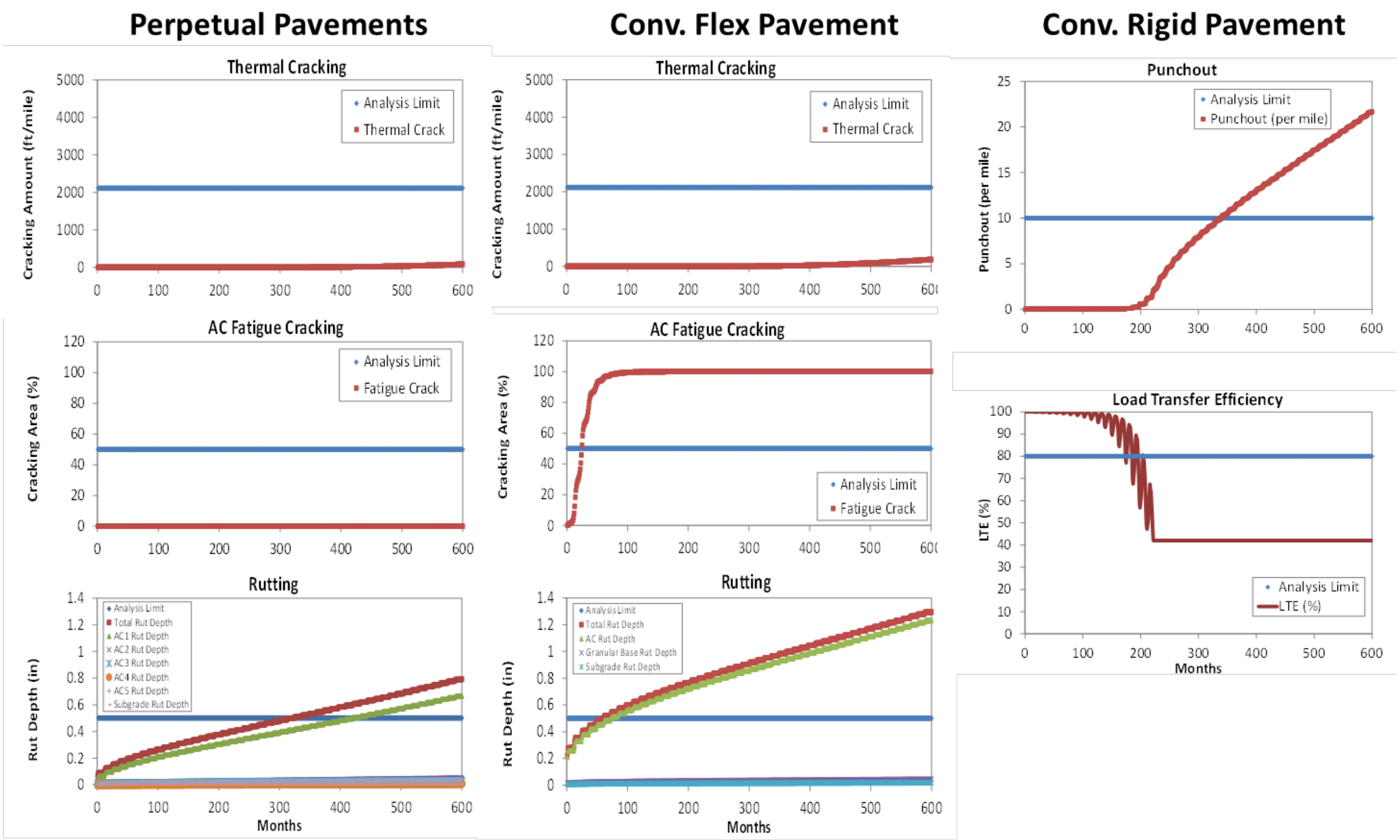


Figure IV-8. Sec #8 IH 35 WAC, Hill County.

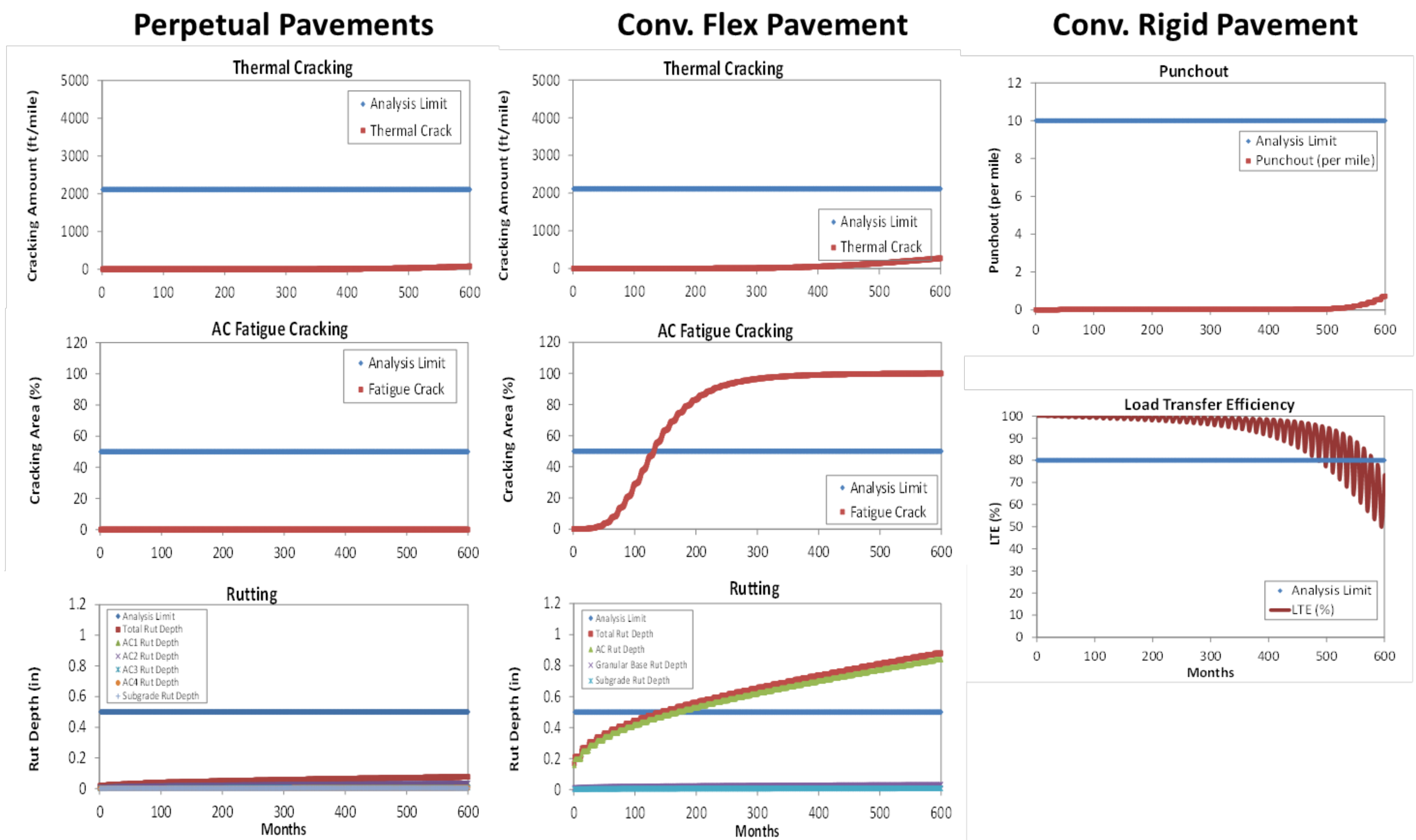


Figure IV-9. Sec #9 SH 114 FTW, Wise County.

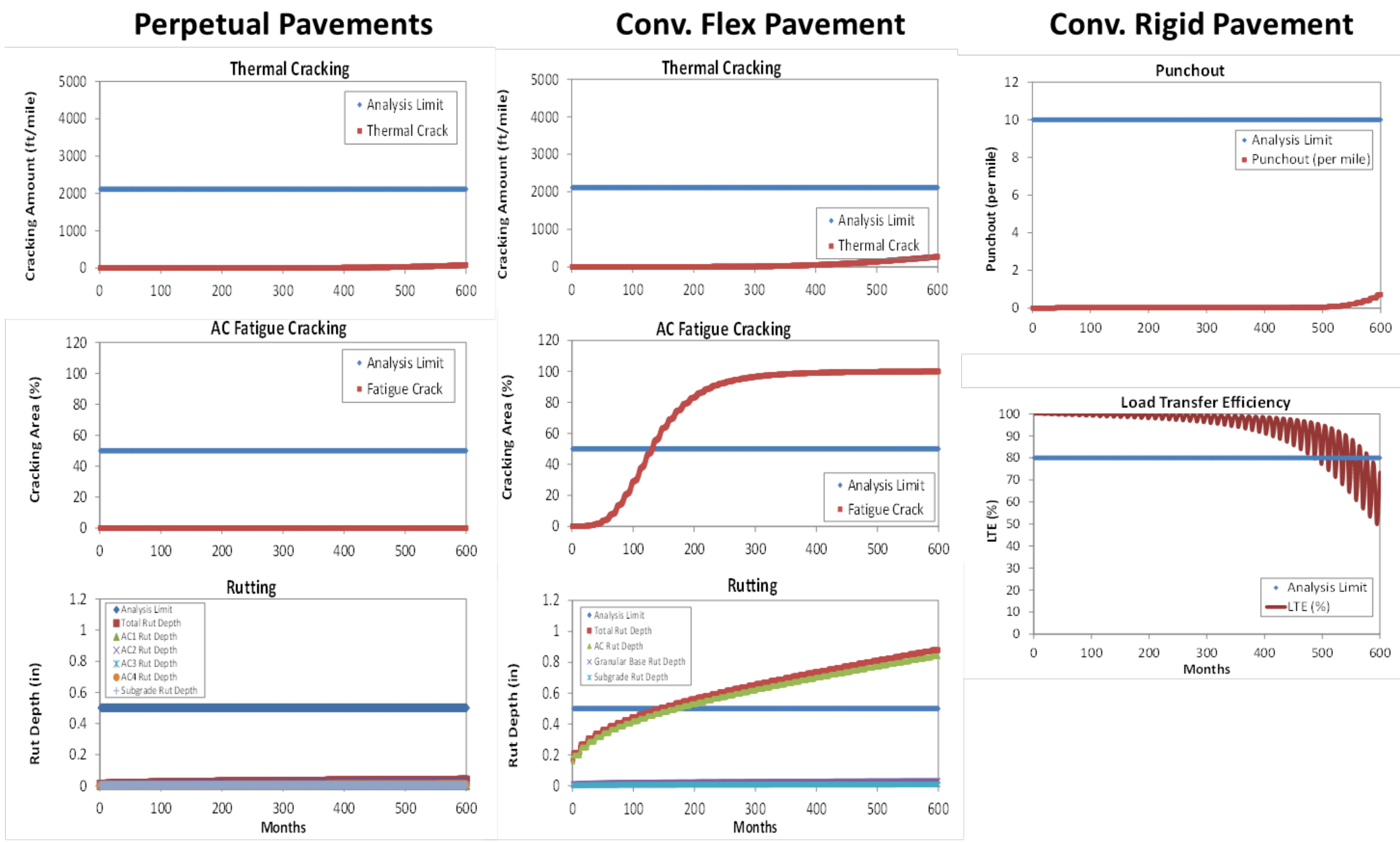


Figure IV-10. Sec #10 SH 114 FTW, Wise County.

APPENDIX V. COMPARATIVE LCCA: IN-SERVICE PPS VERSUS CONVENTIONAL FLEXIBLE (OVERLAID EVERY 4 YEARS) AND RIGID PAVEMENTS

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 3,538.00*	*$ 1,744.29*	*$ 7,748.00*	*$ 6,886.08*	*$ 11,442.00*	*$ 1,745.67*
Present Value	**$ 2,717.15**	**$ 1,487.10**	**$ 3,911.82**	**$ 2,509.29**	**$ 5,896.79**	**$ 1,499.60**

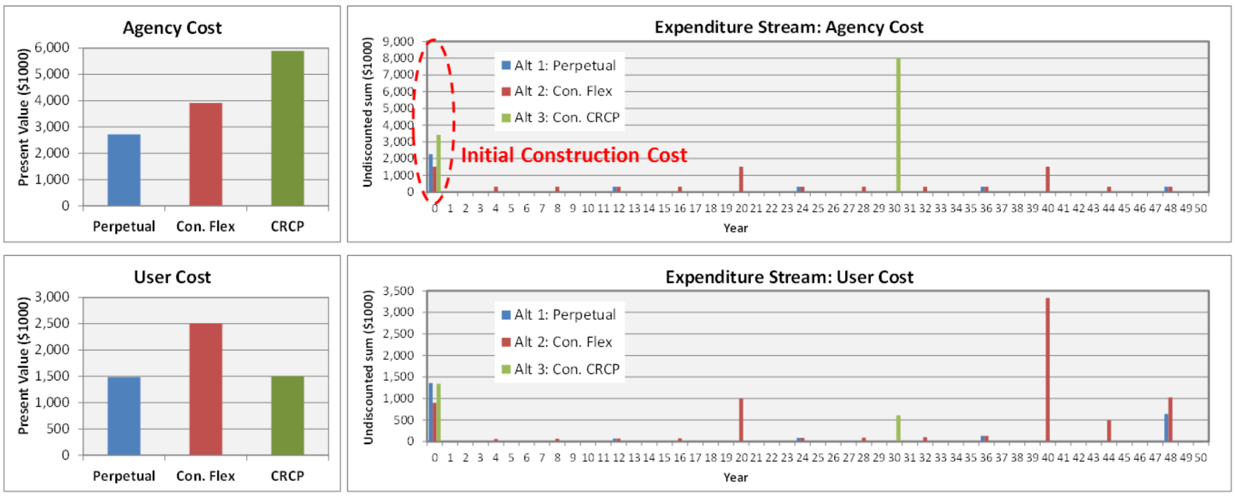


Figure V-1. Sec #1 IH 35 LRD, Webb County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,471.00*	*$ 1,376.13*	*$ 5,161.00*	*$ 4,270.52*	*$ 7,628.00*	*$ 1,402.77*
Present Value	**$ 1,924.63**	**$ 1,160.50**	**$ 2,605.88**	**$ 1,758.71**	**$ 3,930.96**	**$ 1,181.89**

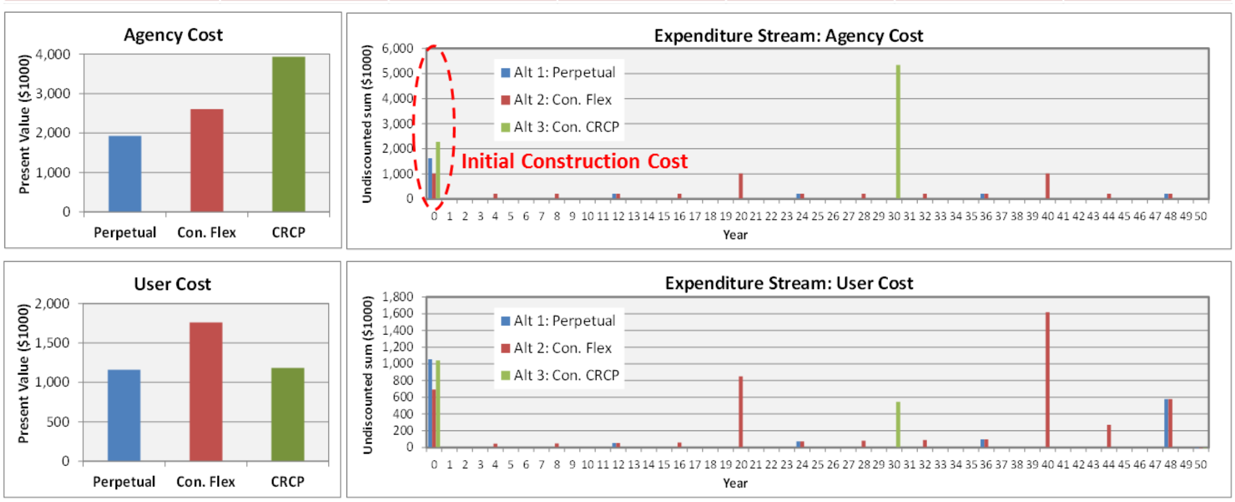


Figure V-2. Sec #2 IH 35 LRD, La Salle County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,583.00*	*$ 1,310.79*	*$ 5,161.00*	*$ 4,617.58*	*$ 7,628.00*	*$ 1,288.91*
Present Value	**$ 2,036.63**	**$ 1,076.02**	**$ 2,605.88**	**$ 1,751.41**	**$ 3,930.96**	**$ 1,085.96**

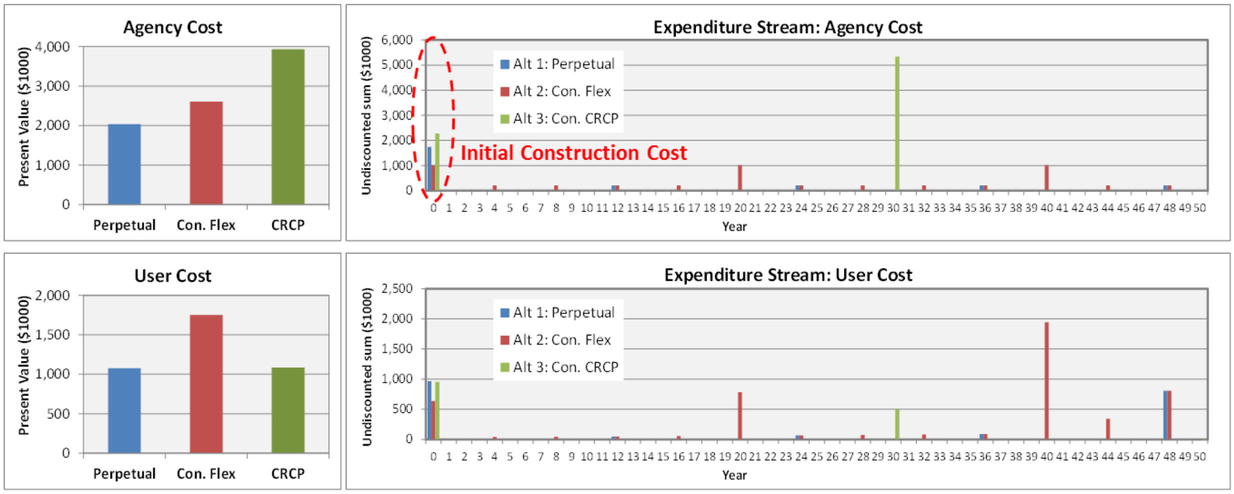


Figure V-3. Sec #3 IH 35 LRD, La Salle County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,566.00*	*$ 1,523.41*	*$ 5,161.00*	*$ 7,035.06*	*$ 7,628.00*	*$ 1,224.64*
Present Value	**$ 2,019.63**	**$ 1,240.17**	**$ 2,605.88**	**$ 2,355.64**	**$ 3,930.96**	**$ 1,146.95**

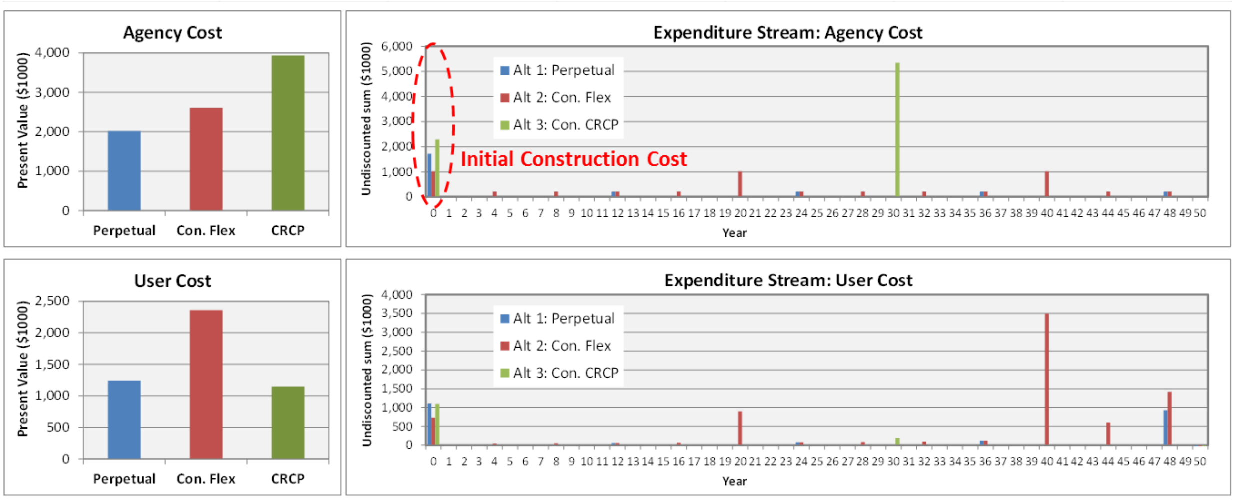


Figure V-4. Sec #4 IH 35 LRD, La Salle County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 4,001.00*	*$ 296,363.00*	*$ 7,748.00*	*$ 962,720.81*	*$ 11,441.00*	*$ 273,633.53*
Present Value	**$ 3,180.15**	**$ 253,063.77**	**$ 3,911.82**	**$ 434,953.44**	**$ 5,895.79**	**$ 211,419.31**

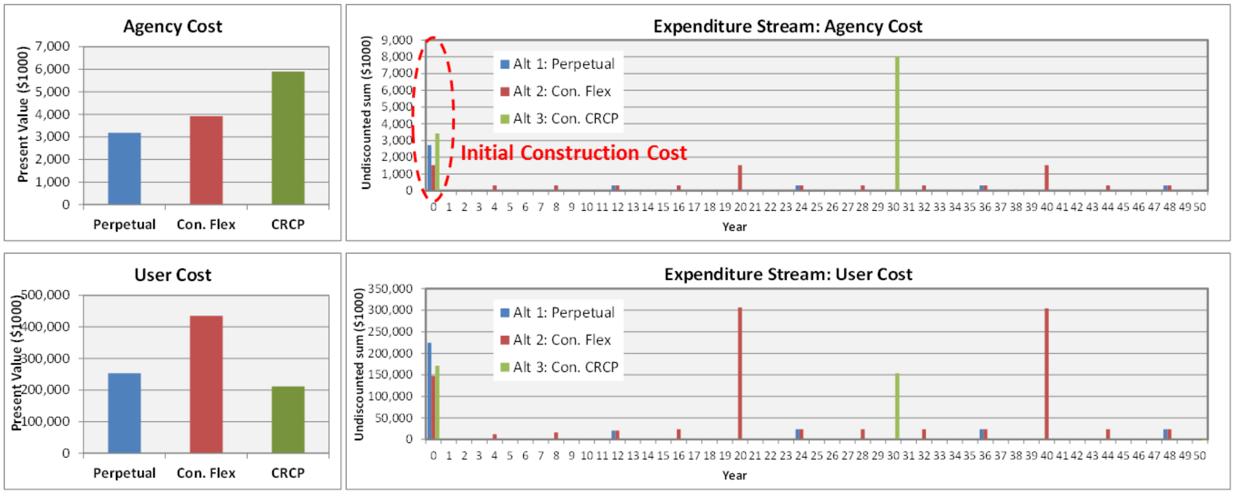


Figure V-5. Sec #5 IH 35 SAT, Comal County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 4,170.00*	*$ 296,363.00*	*$ 7,748.00*	*$ 962,720.81*	*$ 11,441.00*	*$ 273,633.53*
Present Value	**$ 3,349.15**	**$ 253,063.77**	**$ 3,911.82**	**$ 434,953.44**	**$ 5,895.79**	**$ 211,419.31**

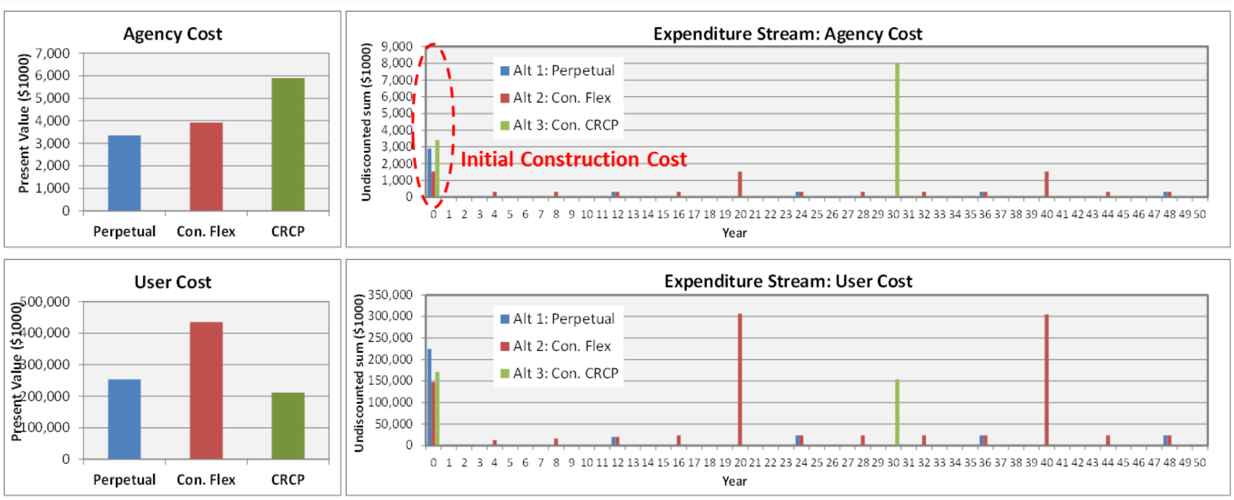


Figure V-6. Sec #6 IH 35 SAT, Comal County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 4,845.00*	*$ 694,320.19*	*$ 7,748.00*	*$ 1,341,188.13*	*$ 11,441.00*	*$ 720,796.75*
Present Value	**$ 4,024.15**	**$ 636,322.81**	**$ 3,911.82**	**$ 646,532.00**	**$ 5,895.79**	**$ 640,142.00**

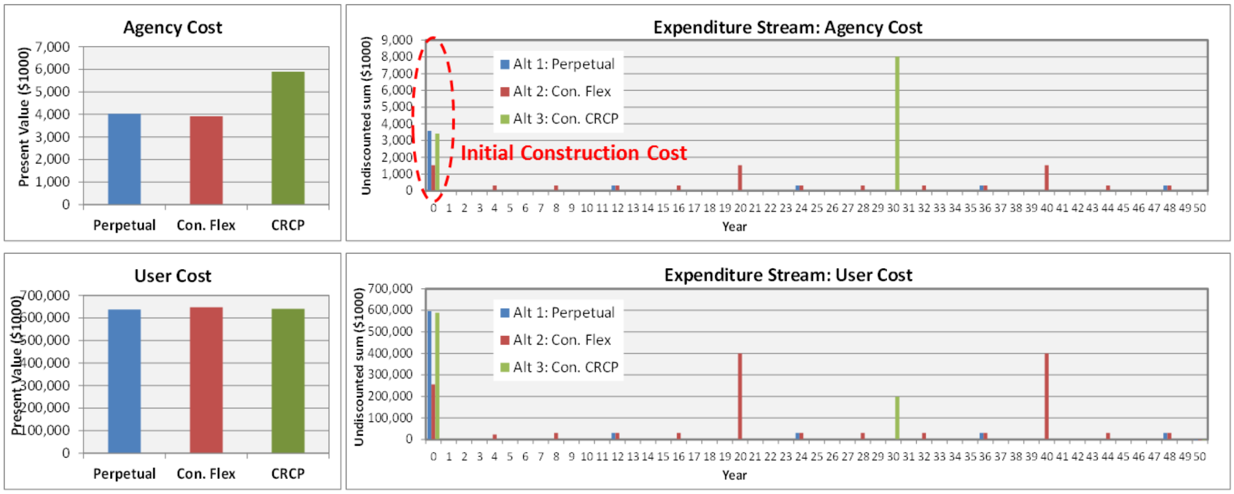


Figure V-7. Sec #7 IH 35 WAC, McLennan County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 5,622.00*	*$ 27,152.41*	*$ 10,332.00*	*$ 109,418.43*	*$ 15,257.00*	*$ 23,334.14*
Present Value	**$ 4,526.68**	**$ 23,203.33**	**$ 5,216.09**	**$ 34,561.46**	**$ 7,862.93**	**$ 22,306.64**

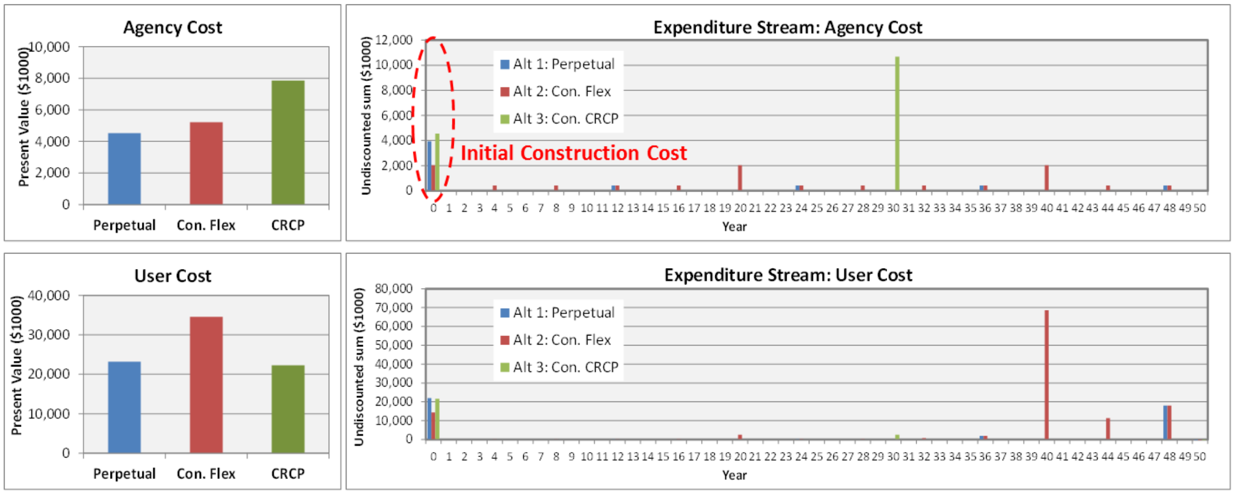


Figure V-8. Sec #8 IH 35 WAC, Hill County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 3,257.00*	*$ 596.12*	*$ 5,161.00*	*$ 1,467.08*	*$ 7,628.00*	*$ 632.28*
Present Value	**$ 2,720.99**	**$ 535.99**	**$ 2,605.88**	**$ 706.50**	**$ 3,930.96**	**$ 547.31**

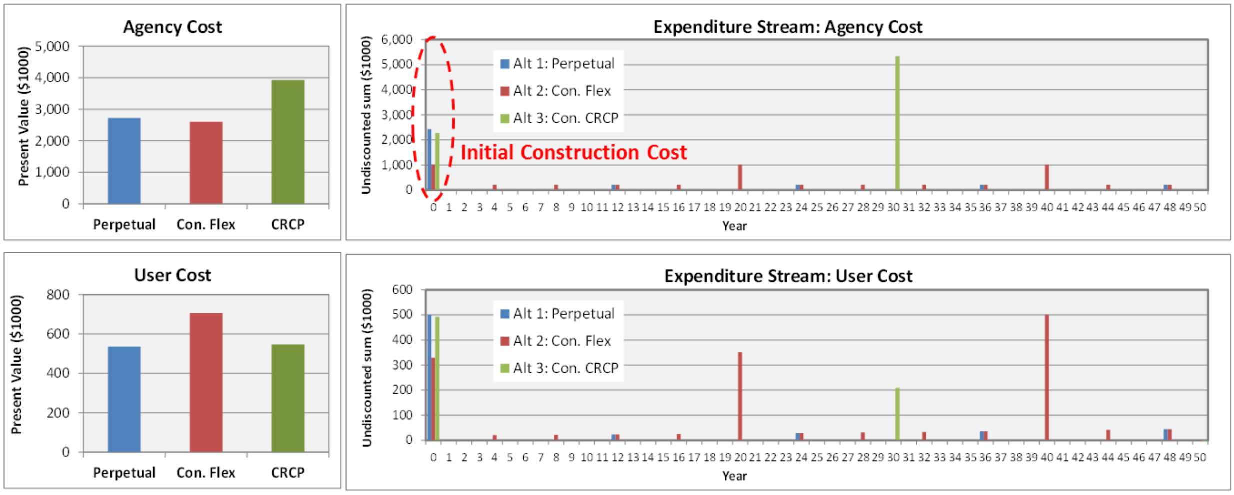


Figure V-9. Sec #9 SH 114 FTW, Wise County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,565.00*	*$ 596.12*	*$ 5,161.00*	*$ 1,467.08*	*$ 7,628.00*	*$ 632.28*
Present Value	**$ 2,018.63**	**$ 535.99**	**$ 2,605.88**	**$ 706.50**	**$ 3,930.96**	**$ 547.31**

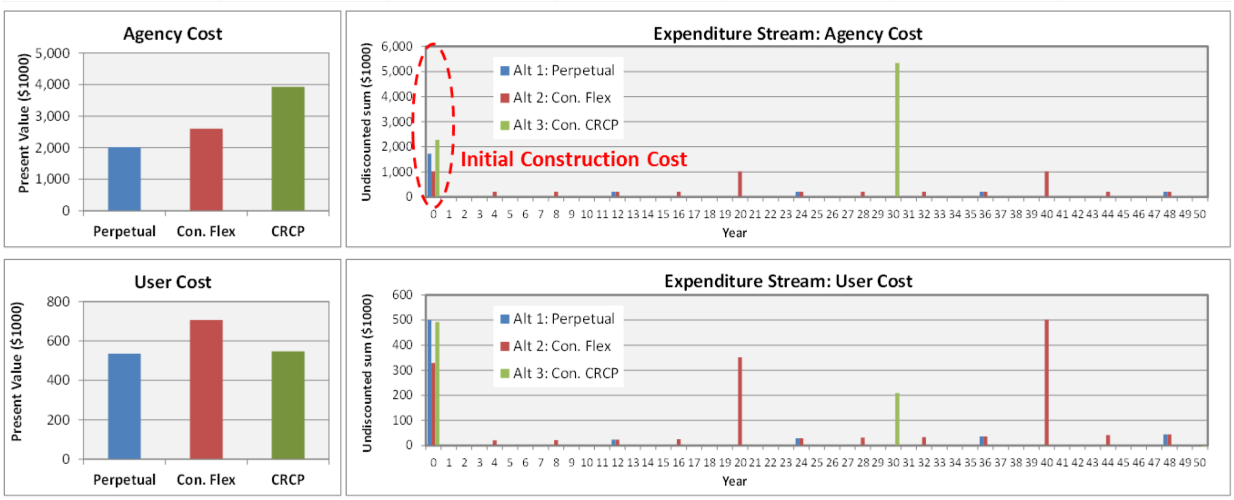


Figure V-10. Sec #10 SH 114 FTW, Wise County.

APPENDIX VI. COMPARATIVE LCCA: IN-SERVICE PPS VERSUS CONVENTIONAL FLEXIBLE (OVERLAID EVERY 8 YEARS) AND RIGID PAVEMENTS

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 3,538.00*	*$ 1,744.29*	*$ 6,163.00*	*$ 5,829.46*	*$ 11,442.00*	*$ 1,745.67*
Present Value	**$ 2,717.15**	**$ 1,487.10**	**$ 3,172.37**	**$ 2,234.69**	**$ 5,896.79**	**$ 1,499.60**

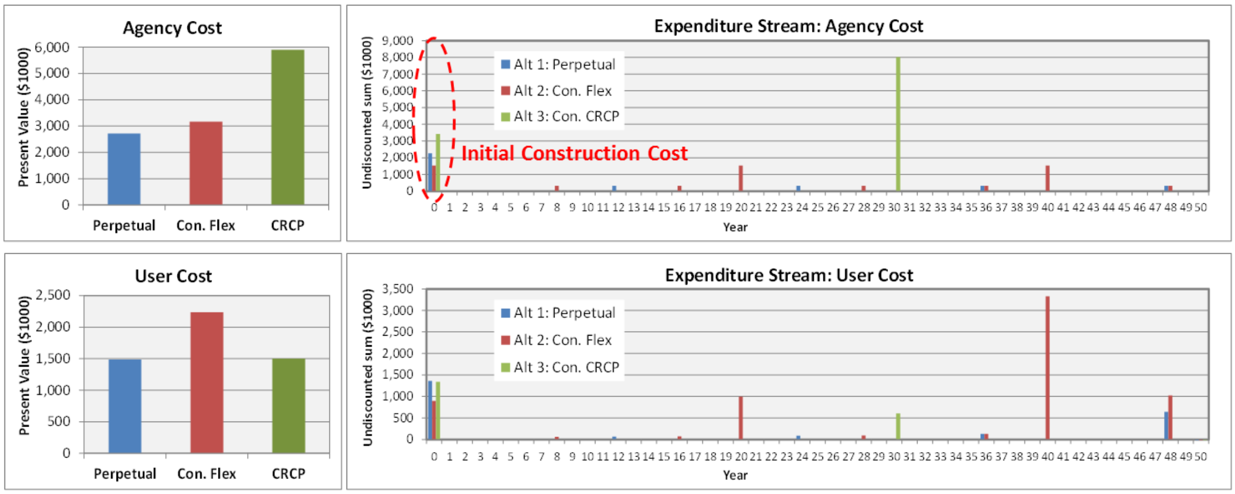


Figure VI-1. Sec #1 IH 35 LRD, Webb County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,471.00*	*$ 1,376.13*	*$ 4,106.00*	*$ 3,592.87*	*$ 7,628.00*	*$ 1,402.77*
Present Value	**$ 1,924.63**	**$ 1,160.50**	**$ 2,113.70**	**$ 1,563.95**	**$ 3,930.96**	**$ 1,181.89**

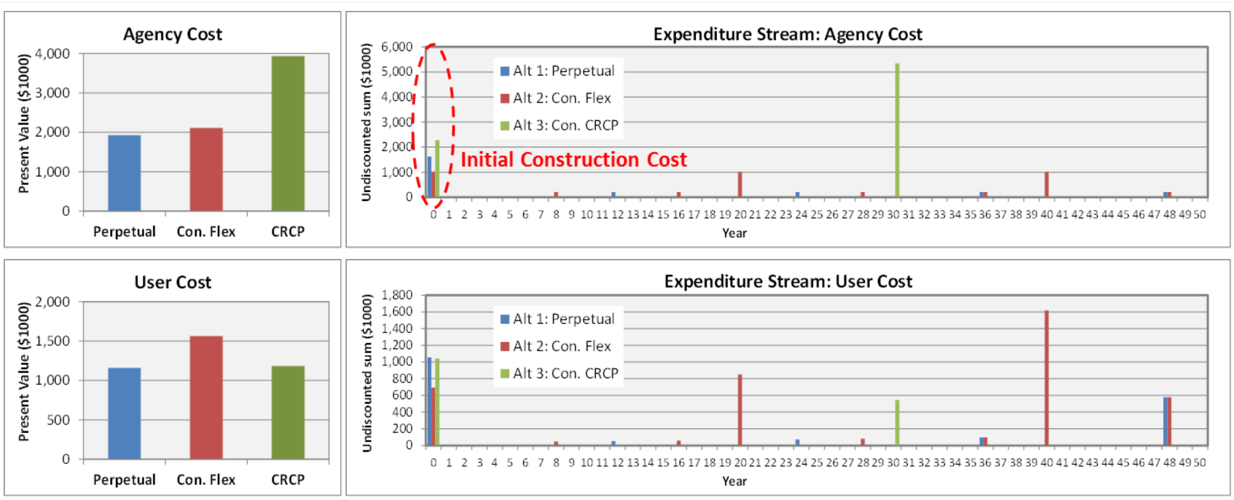


Figure VI-2. Sec #2 IH 35 LRD, La Salle County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,583.00*	*$ 1,310.79*	*$ 4,106.00*	*$ 3,831.50*	*$ 7,628.00*	*$ 1,288.91*
Present Value	**$ 2,036.63**	**$ 1,076.02**	**$ 2,113.70**	**$ 1,545.95**	**$ 3,930.96**	**$ 1,085.96**

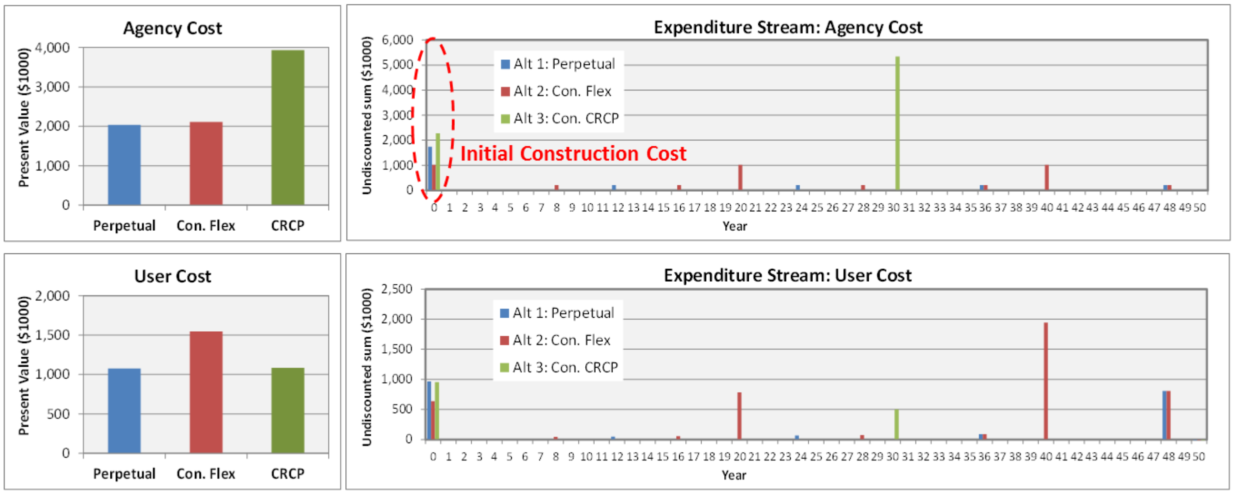


Figure VI-3. Sec #3 IH 35 LRD, La Salle County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,566.00*	*$ 1,523.41*	*$ 4,106.00*	*$ 5,802.70*	*$ 7,628.00*	*$ 1,224.64*
Present Value	**$ 2,019.63**	**$ 1,240.17**	**$ 2,113.70**	**$ 2,065.56**	**$ 3,930.96**	**$ 1,146.95**

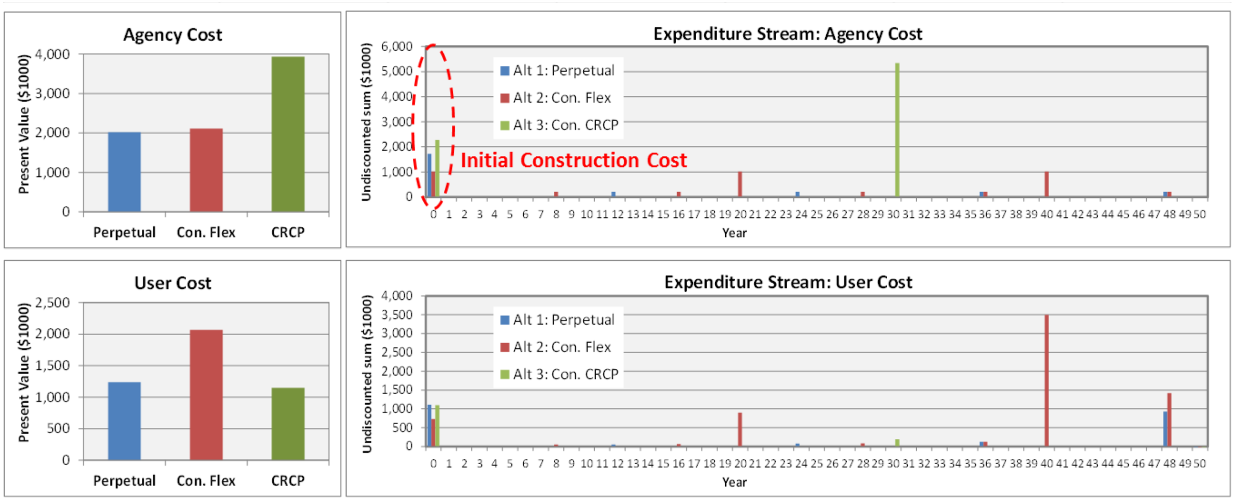


Figure VI-4. Sec #4 IH 35 LRD, La Salle County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 4,001.00*	*$ 296,363.00*	*$ 6,163.00*	*$ 817,021.50*	*$ 11,441.00*	*$ 273,633.53*
Present Value	**$ 3,180.15**	**$ 253,063.77**	**$ 3,172.37**	**$ 354,568.72**	**$ 5,895.79**	**$ 211,419.31**

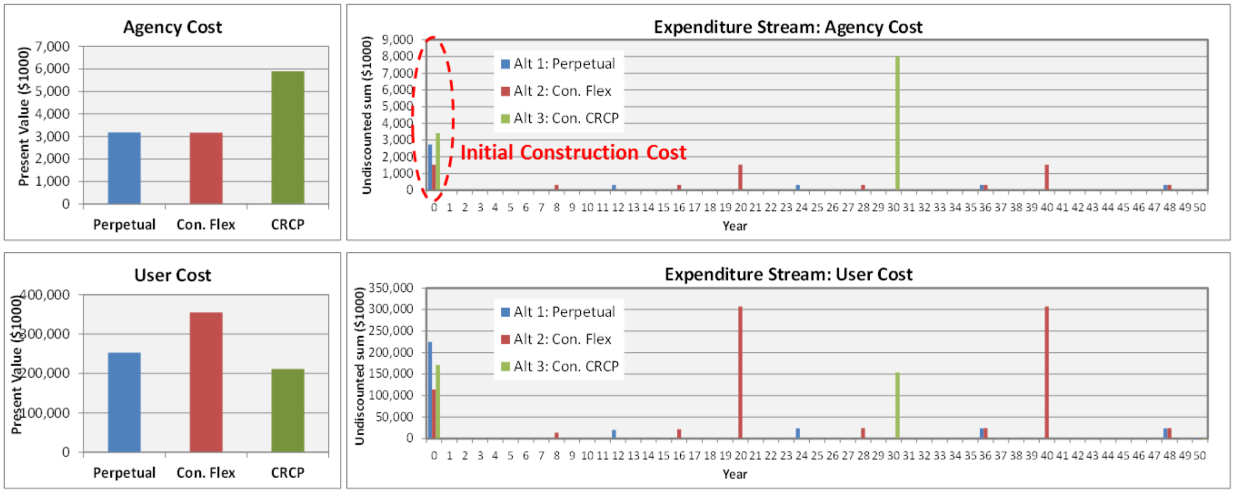


Figure VI-5. Sec #5 IH 35 SAT, Comal County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 4,170.00*	*$ 296,363.00*	*$ 6,163.00*	*$ 817,021.50*	*$ 11,441.00*	*$ 273,633.53*
Present Value	**$ 3,349.15**	**$ 253,063.77**	**$ 3,172.37**	**$ 354,568.72**	**$ 5,895.79**	**$ 211,419.31**

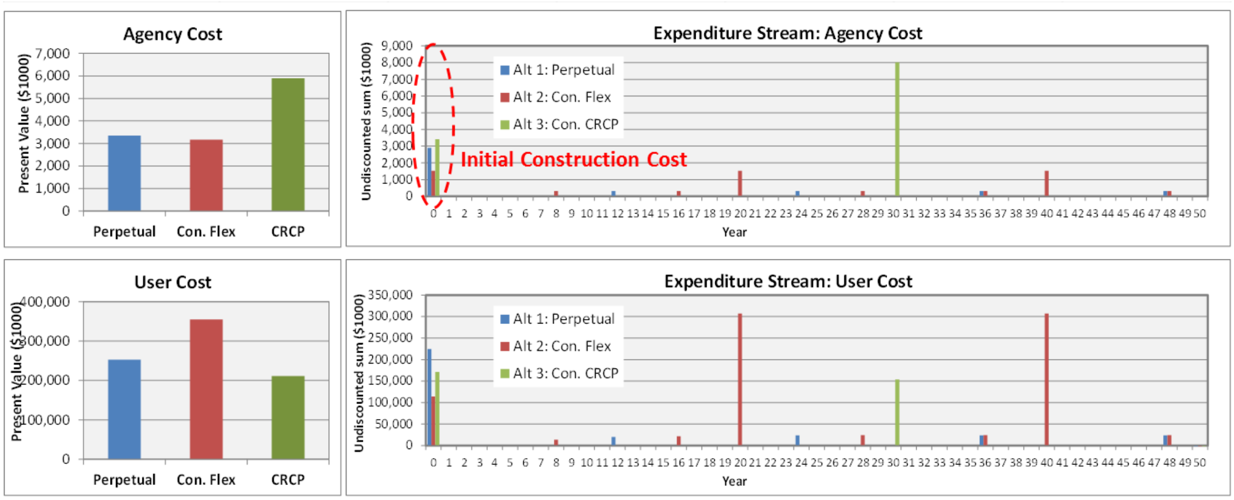


Figure VI-6. Sec #6 IH 35 SAT, Comal County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 4,845.00*	*$ 694,320.19*	*$ 6,163.00*	*$ 1,319,747.88*	*$ 11,441.00*	*$ 720,796.75*
Present Value	**$ 4,024.15**	**$ 636,322.81**	**$ 3,172.37**	**$ 715,112.38**	**$ 5,895.79**	**$ 640,142.00**

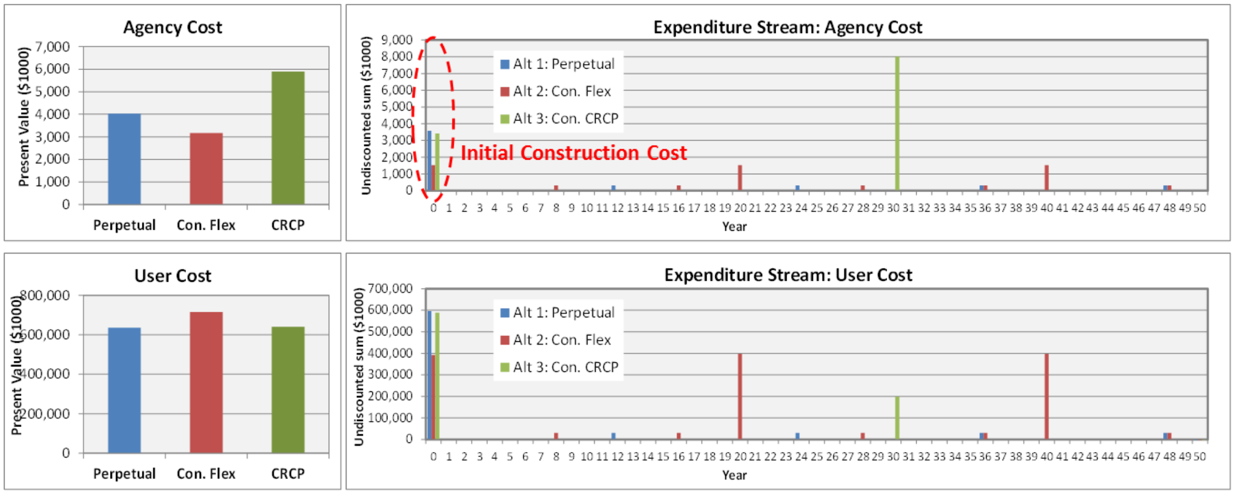


Figure VI-7. Sec #7 IH 35 WAC, McLennan County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 5,622.00*	*$ 27,152.41*	*$ 10,332.00*	*$ 109,418.43*	*$ 15,257.00*	*$ 23,334.14*
Present Value	**$ 4,526.68**	**$ 23,203.33**	**$ 5,216.09**	**$ 34,561.46**	**$ 7,862.93**	**$ 22,306.64**

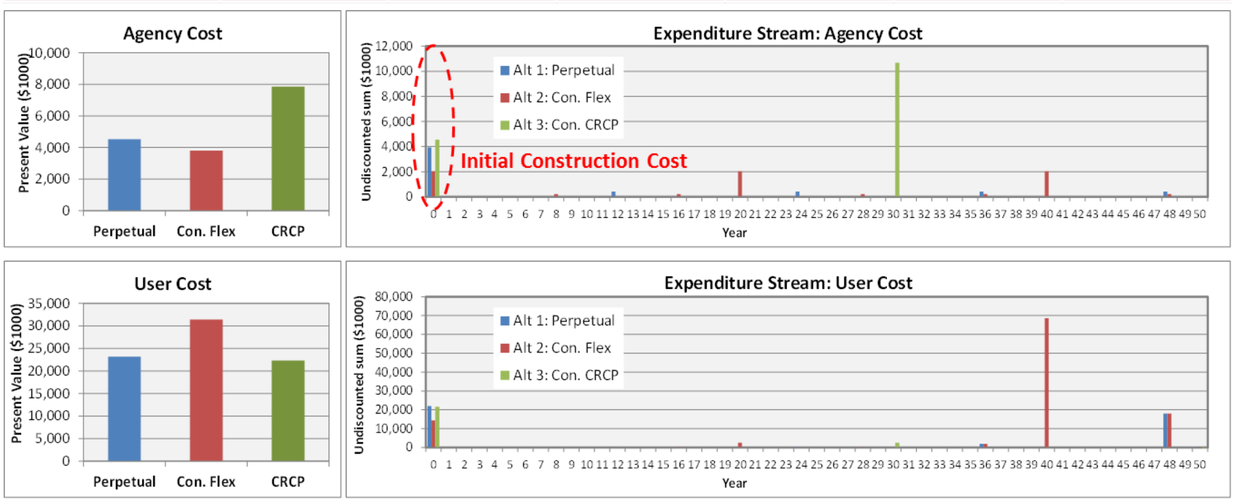


Figure VI-8. Sec #8 IH 35 WAC, Hill County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 3,257.00*	*$ 596.12*	*$ 4,106.00*	*$ 1,306.68*	*$ 7,628.00*	*$ 632.28*
Present Value	**$ 2,720.99**	**$ 535.99**	**$ 2,113.70**	**$ 644.03**	**$ 3,930.96**	**$ 547.31**

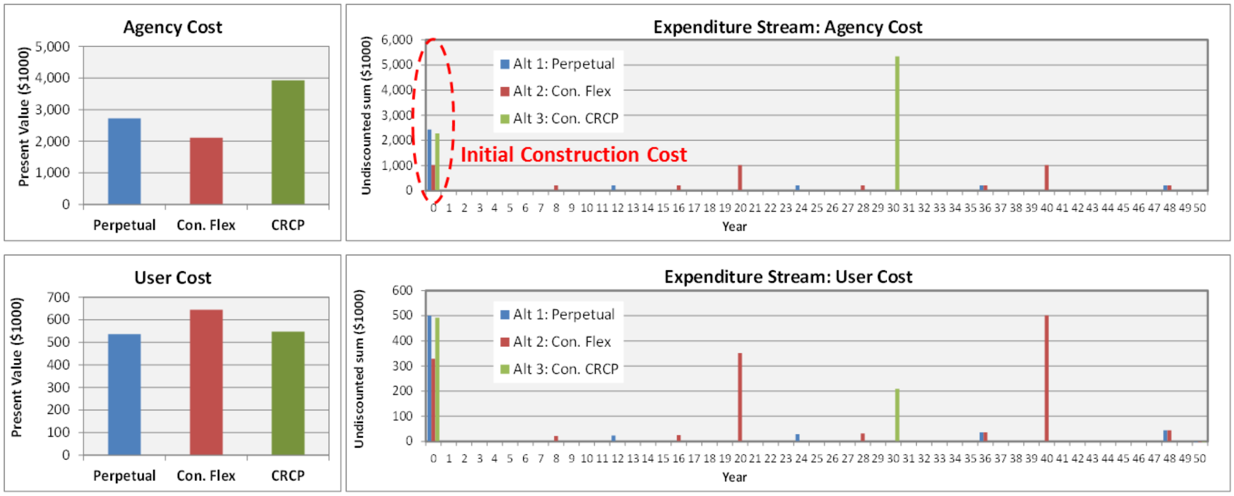


Figure VI-9. Sec #9 SH 114 FTW, Wise County.

Total Cost	Alt 1: Perpetual PVMNT		Alt 2: Con. Flex PVMNT		Alt 3: Con. CRCP PVMNT	
	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)	Agency cost ($1000)	User cost ($1000)
Undiscounted sum	*$ 2,565.00*	*$ 596.12*	*$ 4,106.00*	*$ 1,306.68*	*$ 7,628.00*	*$ 632.28*
Present Value	**$ 2,018.63**	**$ 535.99**	**$ 2,113.70**	**$ 644.03**	**$ 3,930.96**	**$ 547.31**

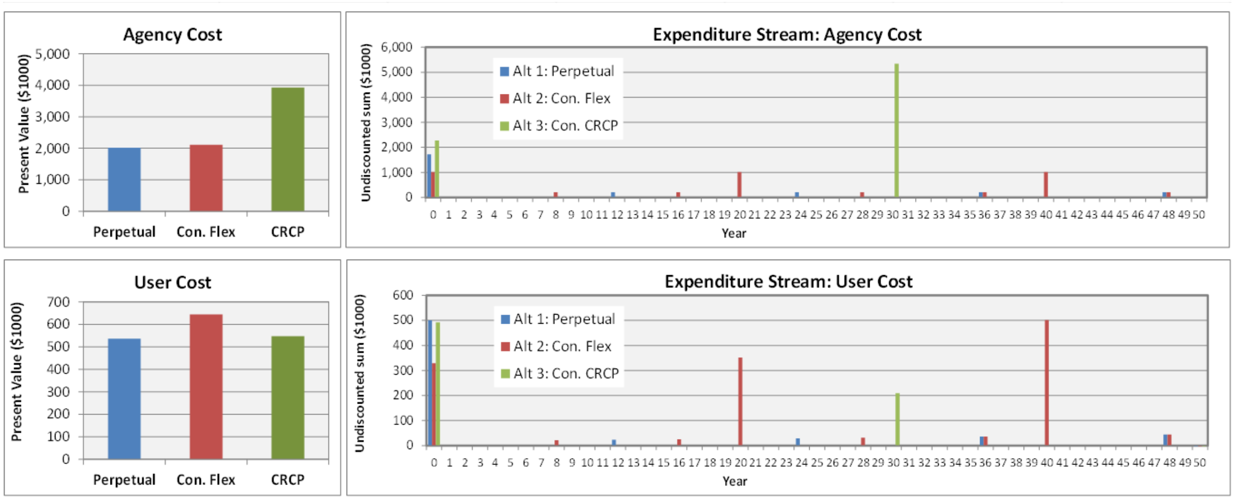


Figure VI-10. Sec #10 SH 114 FTW, Wise County.

APPENDIX VII. EL DETERMINATION APPROACH/METHOD AND TEST PROTOCOL

THE 4-POINT LOADING BENDING BEAM FATIGUE TEST

The 4-point bending beam fatigue test can be conducted according to AASHTO T 321, "Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending" (AASHTO, 2014). In this procedure, beam specimens (380 mm length, 63 mm width, 50 mm height) are loaded under strain-controlled conditions using sinusoidal loading at 10 Hz at a temperature of 20°C, as shown in Figure VII-1. The literature has indicated that beam fatigue tests were historically the most commonly used form of fatigue test in the United States.

Figure VII-1. The Bending Beam Fatigue Test Apparatus and HMA Specimen.

AASHTO T 321 states that typical test strain levels range between 250 and 750 micro-strains. The literature suggests that the EL in the laboratory is on the order of 70 micro-strains and possibly up to 200 micro-strains in the field. The air void contents for the optimum asphalt content samples are typically targeted at 7 ± 0.5 percent (Prowell et al., 2010).

Since the test can be very time consuming, up to 50 days in some instances, the NCHRP 9-38 researchers explored four techniques to extrapolate the stiffness versus loading cycle data, such as AASHTO T321 exponential function, the single- and three-stage Weibull functions, and the ratio of dissipated energy change method (AASHTO, 2014; Prowell et al., 2010). According to the conclusions of NCHRP 9-38, the single-stage Weibull model produced fairly accurate extrapolations that appear to be conservative. Therefore, the single-stage Weibull model was recommended for extrapolating low strain fatigue test results to confirm the existence of the EL. This can cost-effectively reduce the test time.

THE S-VECD TEST

The viscoelastic continuum damage (VECD) theory resulted from the work of Kim and Little (Kim and Little, 1990), which applies Schapery's viscoelastic constitutive theory (Schapery, 1987) for materials with distributed damage to describe the behavior of asphalt under controlled-strain cyclic loading. A S-VECD form can maintain mathematical rigor and can be quickly characterized with cyclic test results (Underwood et al., 2010).

The key function in the S-VECD model is the damage characteristic curve, as seen in Figure VII-2. This function relates the overall amount of damage, S, in an HMA specimen to the pseudo secant modulus, or material integrity, which is denoted as C. The pseudo secant modulus quantifies the relationship between stress, σ, and pseudo strain, ε_R, whereas the secant modulus relates stress and strain, ε. The pseudo secant modulus is used instead of the secant modulus because the latter is affected by material time dependence, whereas the former is not. A detailed description of pseudo modulus and pseudo strain concepts, as well as a detailed derivation of the S-VECD model and discussions of the ways it differs from other similar models, can be found elsewhere (Underwood, 2010).

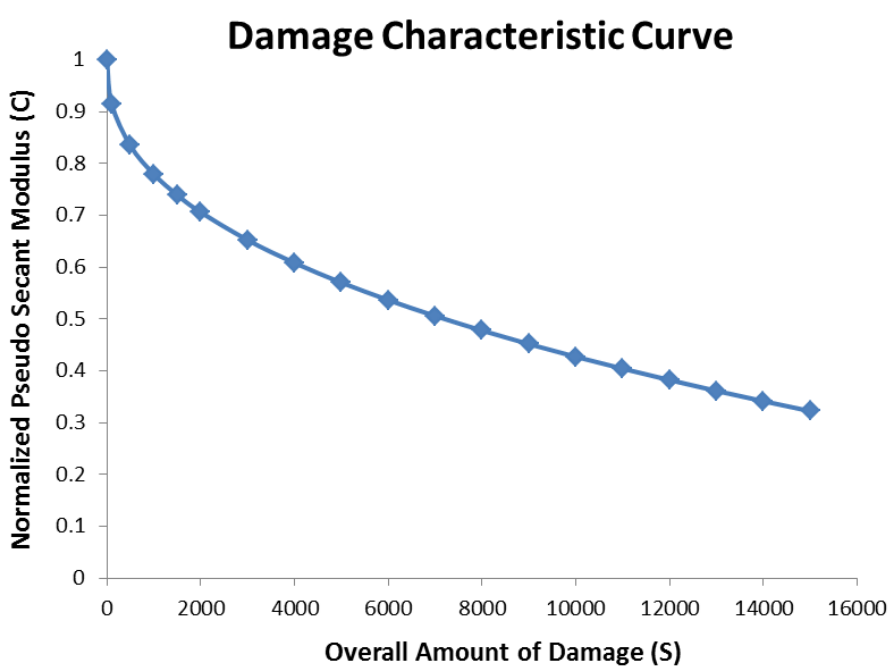


Figure VII-2. Damage Characteristic Curve for the S-VECD Model.

The LVE characterization procedure found in both AASHTO TP 62 and AASHTO TP 79/PP 61 can be used to characterize the DM, which needs to be incorporated into the S-VECD model. The two most recent developments with the S-VECD model form are particularly important because they allow for complete model characterization with the use of the AMPT (see Figure VII-3). It is important that the complete S-VECD protocol is compatible with the AMPT's capabilities because this device is likely to become the standard asphalt mixture test equipment that agencies use in their laboratories.

Figure VII-3. The AMPT Setup and S-VECD HMA Sample.

According to the test procedure AASHTO TP107/S-VECD, "Standard Method of Test For Determining The Damage Characteristic Curve Of Asphalt Mixtures From Direct Tension Cyclic Fatigue Tests," the specimen size (dia × h) is 100 × 130 mm; the test temperature in degrees Celsius should be determined as the average of the high- and low-temperature climatic PG temperatures minus 3°C (AASHTO, 2014). Minimum three replicates are needed for one S-VECD test. The first specimen can be tested with a peak-to-peak on-specimen strain amplitude of 300 micro-strains (ε_{os1}). The peak-to-peak on-specimen strain levels of the second specimen (ε_{os2}) and the third specimen (ε_{os3}) in micro-strains can be found in Table VII-1 based on the resultant number of cycles (N_{f1}) to failure of the first specimen. Normally the tests for all the specimens can be completed within 1 or 2 days.

Table VII-1. On-Specimen Strain Levels for the Second and Third S-VECD Specimens.

Case	Strain, ε_{os2}	Strain, ε_{os3}
$500<N_{f1}<1000$	ε_{os1}-100	ε_{os1}-150
$1,000<N_{f1}<5,000$	ε_{os1}-50	ε_{os1}-100
$5,000<N_{f1}<20,000$	ε_{os1}+50	ε_{os1}-50
$20,000<N_{f1}<100,000$	ε_{os1}+100	ε_{os1}+50
$100,000<N_{f1}$	ε_{os1}+150	ε_{os1}+100

Since the data analysis for S-VECD is complicated, the ALPHA-Fatigue software was developed to determine the fatigue parameters and EL. Figure VII -4 shows the main user interface of the ALPHA-Fatigue software.

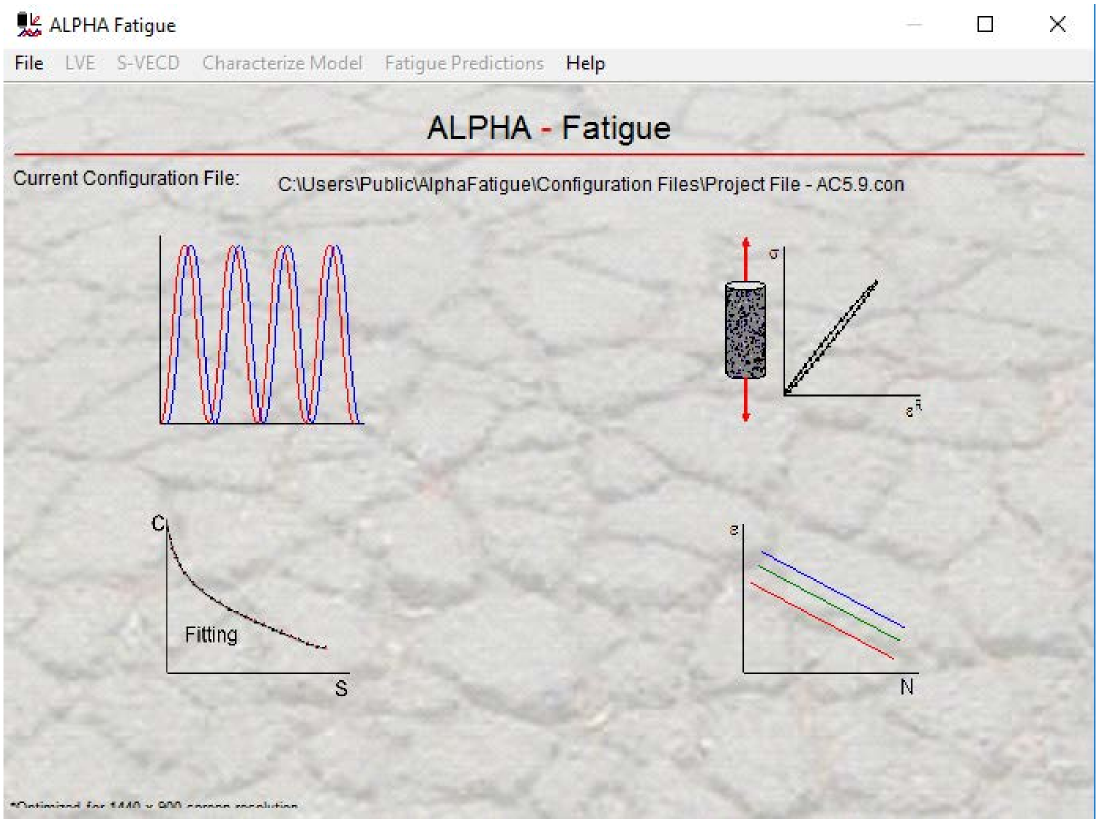


Figure VII-4. The S-VECD: Alpha-Fatigue Software User Interface.

THE RDT TEST

Researchers have recently developed a new approach, the Energy-based Mechanics (EBM) approach (Luo et al., 2013; 2014), to determine the EL of asphalt mixtures through RDT testing, which is another potential candidate test evaluated in this study. The EBM approach studies the damage history of asphalt mixtures, as shown in Figure VII-5. The threshold between the undamaged and damaged states (i.e., critical nonlinear viscoelastic point) is the EL. The testing method used to obtain the damage history contains 3 to 4 simple fatigue tests (3 min of each), which measures the material properties of asphalt mixtures at different strain levels. The MTS apparatus can be used to conduct the RDT test (Figure VII-6). The sample size (dia × h) is 100 × 150 mm.

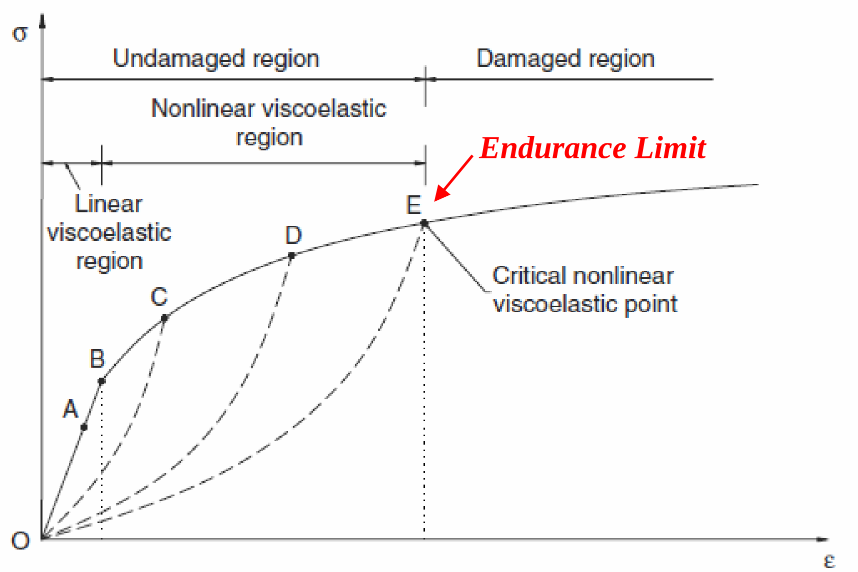


Figure VII-5. Damage History of Typical Asphalt Mixtures at Different Loading Levels.

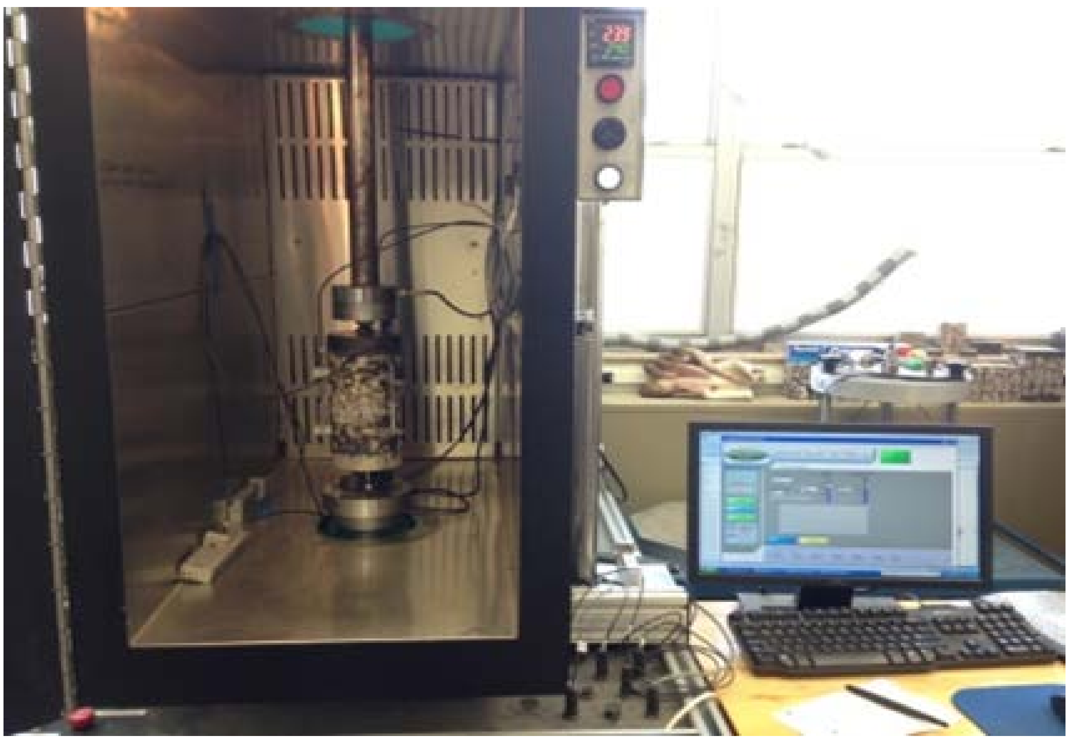


Figure VII-6. MTS Apparatus and HMA Sample Setup for RDT Testing.

At least four samples are needed to run the RDT test and the recommended strain levels are at 40, 50, 60, and 70 $\mu\varepsilon$, respectively, as shown in Figure VII-7. The test data are analyzed by statistical techniques to decide whether the material is damaged or not. It has been proven that the EL from the EBM approach is sensitive to the asphalt-binder type, air void content, and aging (Luo et al., 2013; 2014).

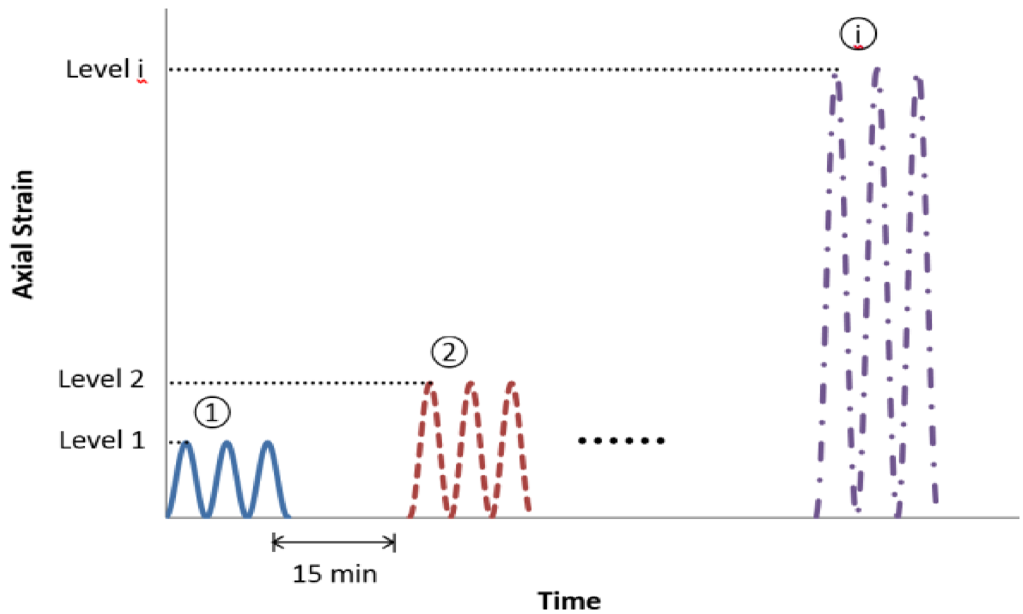


Figure VII-7. Example of RDT Test Strain Levels.

REFERENCES RELATED TO ENDURANCE LIMIT DETERMINATION APPROACH/METHOD

AASHTO. AASHTO T 321: Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending. Washington DC, USA. 2014.

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Kim, Y. R., and D. N. Little. One-Dimensional Constitutive Modeling of Asphalt Concrete. *Journal of Engineering Mechanics,* ASCE, Vol. 116, No. 4, 1990, pp. 751–772.

Luo, X., R. Luo, and R. L. Lytton. Energy-Based Mechanistic Approach to Characterize Crack Growth of Asphalt Mixtures. *Journal of Materials in Civil Engineering*, ASCE, 25(9), 2013, pp. 1198-1208.

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Schapery, R. A. Deformation and Fracture Characterization of Inelastic Composite Materials using Potentials. Polymer Engineering and Science, Vol. 27, No. 1, 1987, pp. 63–76.

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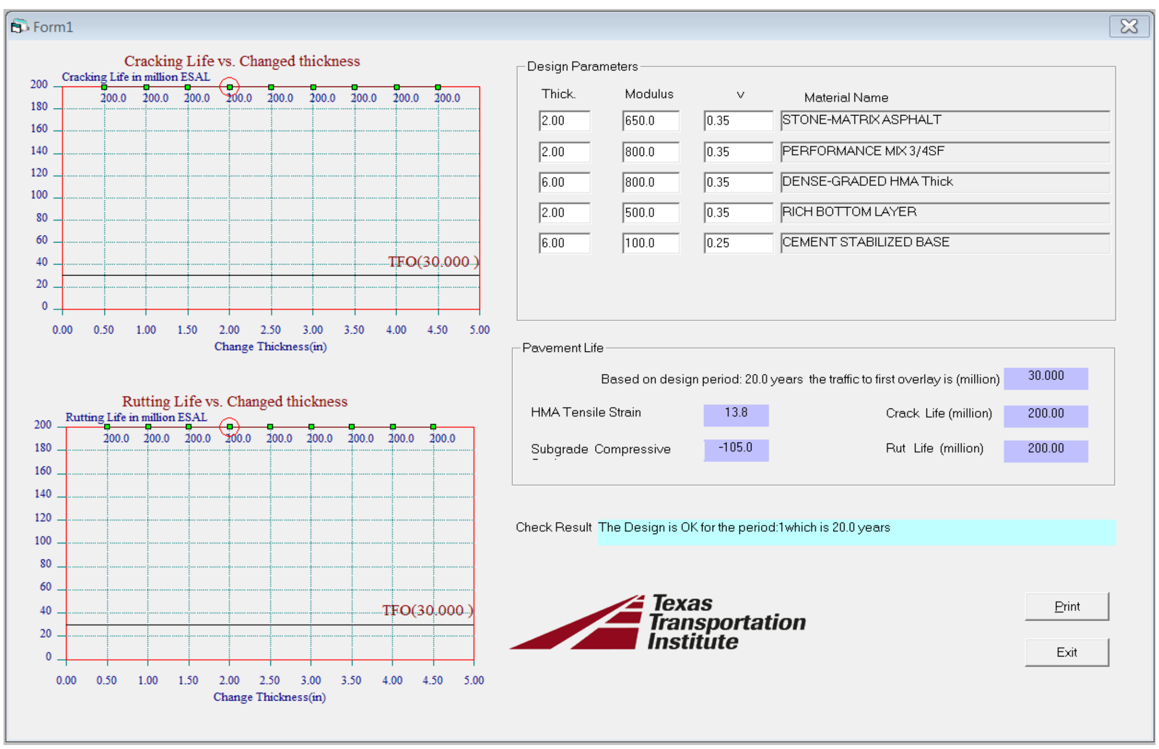
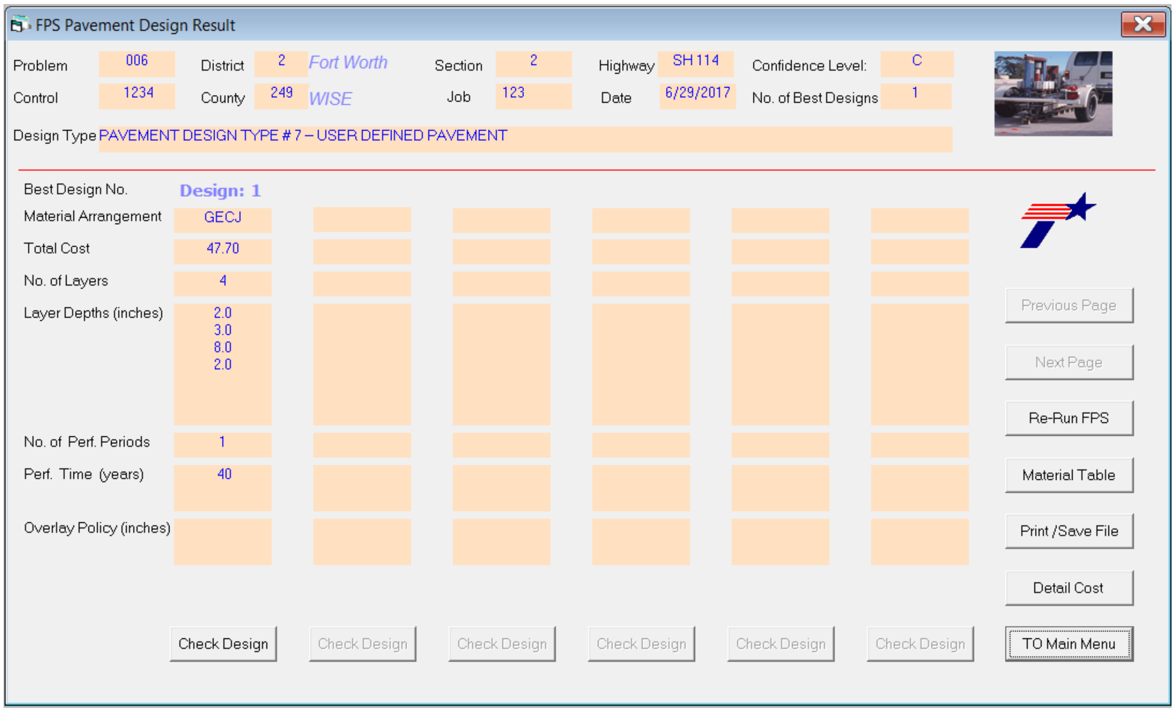
APPENDIX VIII. FPS PAVEMENT DESIGN RESULTS AND MECHANISTIC ANALYSIS OF ALTERNATIVE PP STRUCTURAL DESIGNS

(a) FPS Pavement Design Result.

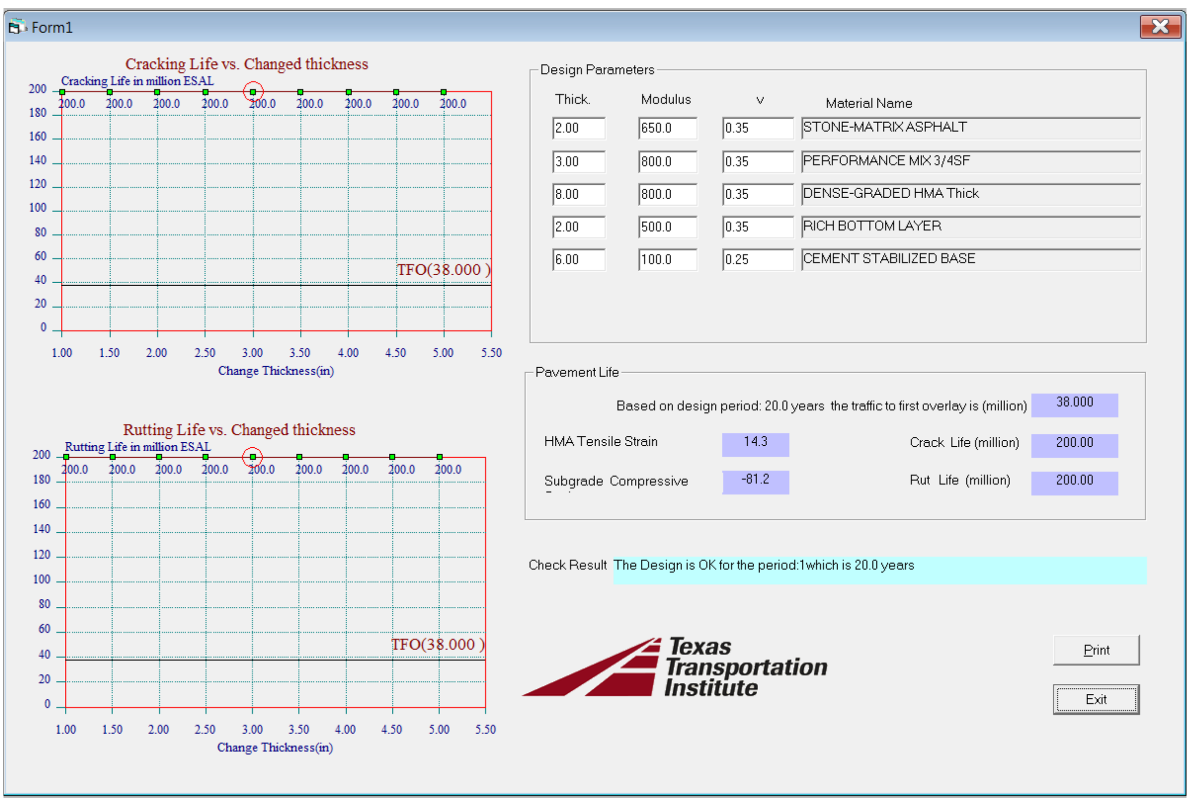


(b) FPS Mechanistic Analysis.

Figure VIII-1. Alternative Structural Design for ESALs ≤ 30 Million.

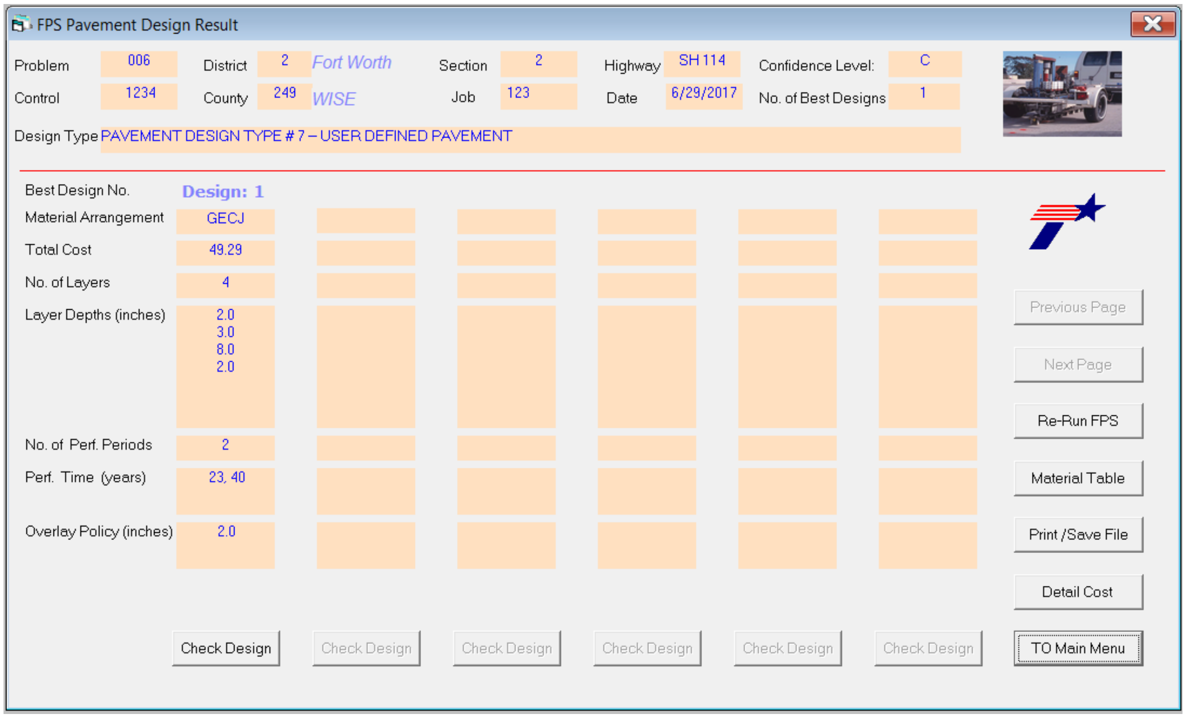


(a) FPS Pavement Design Result.

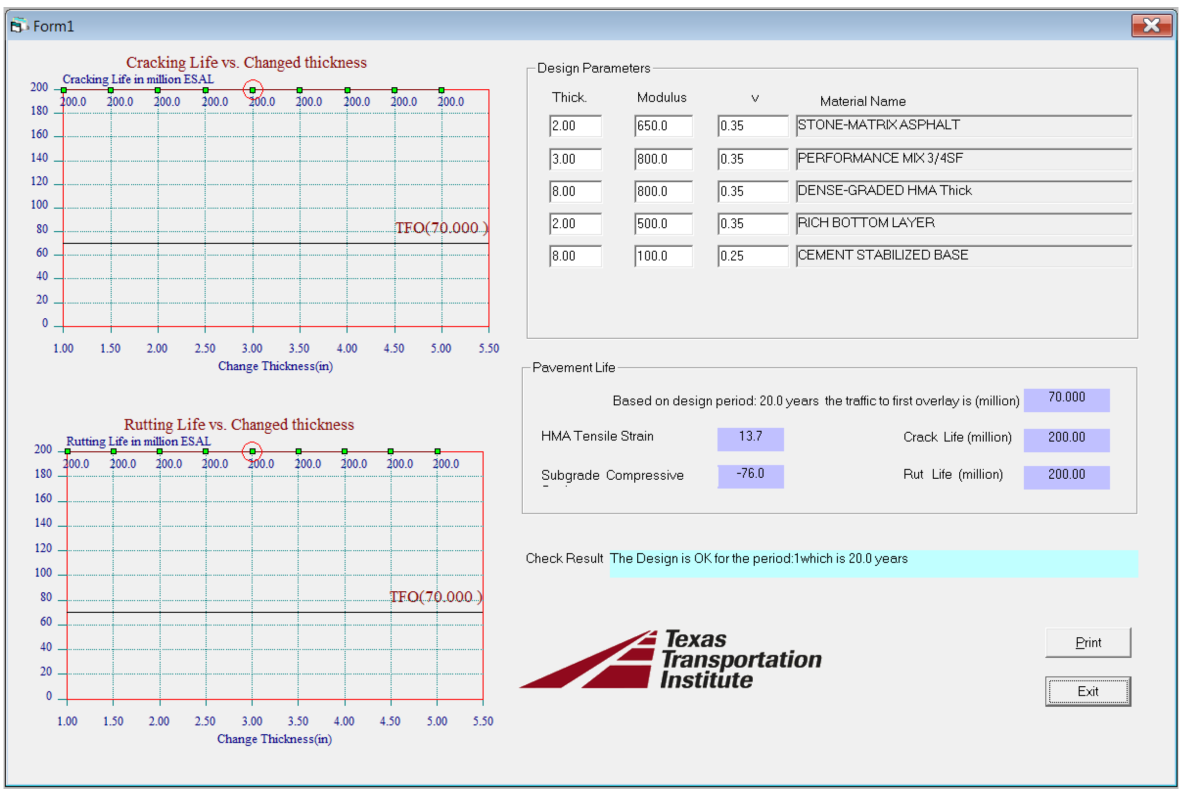


(b) FPS Mechanistic Analysis.

Figure VIII-2. Alternative Structural Design for 30 Million < ESALs ≤ 50 Million.



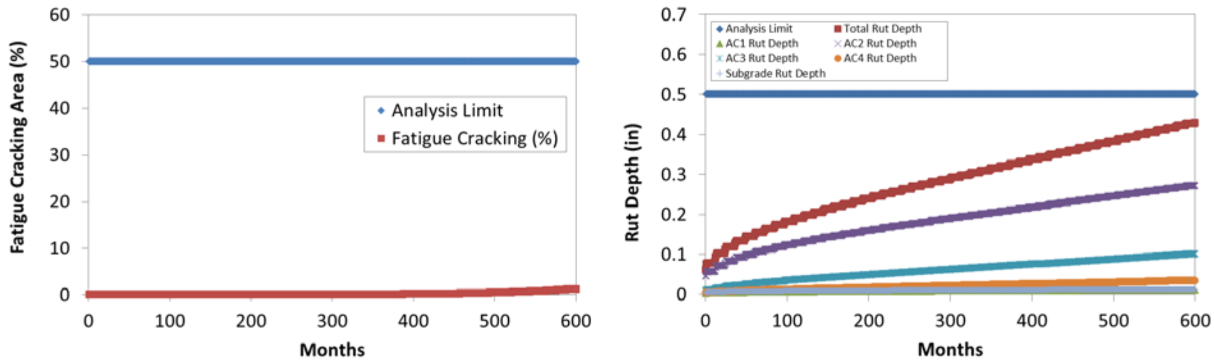
(a) FPS Pavement Design Result.



(b) FPS Mechanistic Analysis.

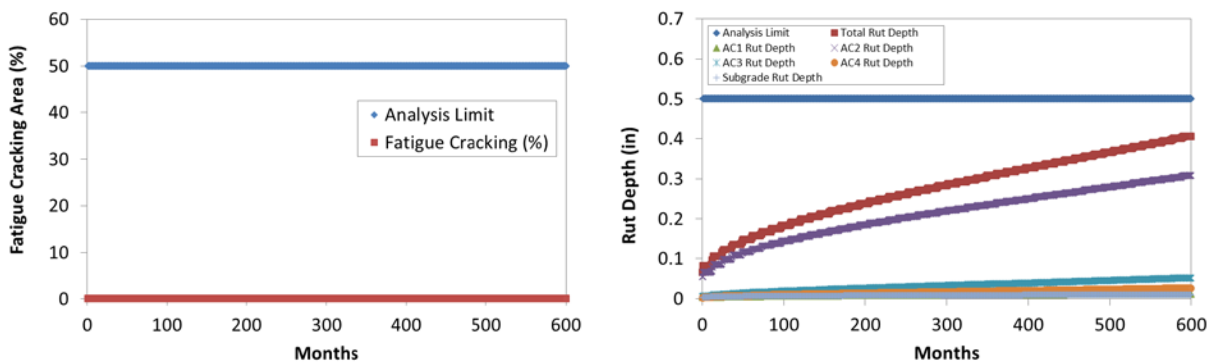
Figure VIII-3. Alternative Structural Design for ESALs > 50 Million.

APPENDIX IX. TXME COMPARATIVE PERFORMANCE PREDICTION OF ALTERNATIVE PP STRUCTURAL DESIGNS



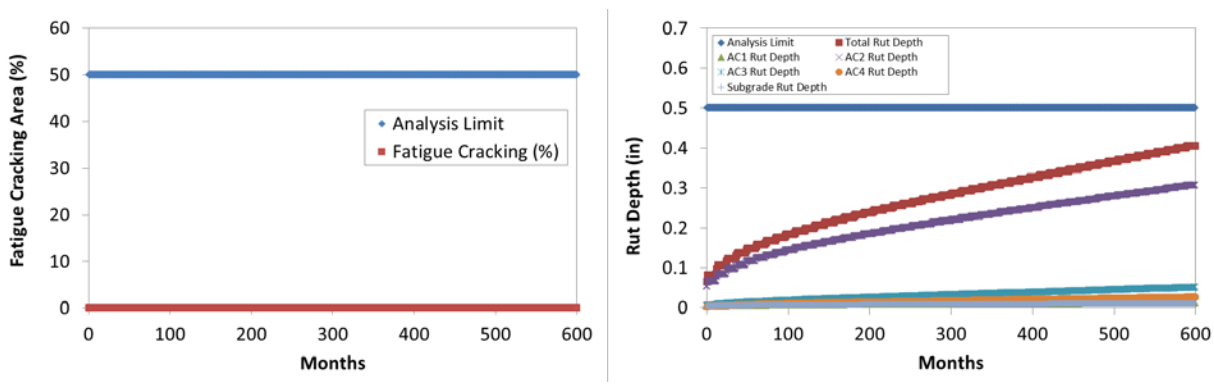
(a) Fatigue Cracking (b) Rutting

Figure IX-1. Alternative Structural Design for ESALs ≤ 30 Million.



(a) Fatigue Cracking (b) Rutting

Figure IX-2. Alternative Structural Design for 30 Million < ESALs ≤ 50 Million.



(a) Fatigue Cracking (b) Rutting

Figure IX-3. Alternative Structural Design for ESALs > 50 Million.