Pavement Performance Measures and Forecasting and the Effects of Maintenance and Rehabilitation Strategy on Treatment Effectiveness (Revised)

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Research, Development, and Technology
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LTPP™

FOREWORD

“How effective will a particular treatment be on this roadway?” Data from the Long-Term Pavement Performance (LTPP) program have supported efforts to answer this question. This report documents work to advance the development of pavement performance measures to classify pavement sections, estimate future pavement conditions, evaluate the effectiveness of pavement maintenance and rehabilitation treatments, and analyze the role of pavement treatment strategies in extending pavement life.

Data from the various LTPP experiments were analyzed to define pavement performance in a way that supports the selection of cost-effective pavement treatment strategies. The research approach and results are presented in this report, including 1) the newly developed dual (functional and structural) pavement rating systems and their impact on pavement management, 2) the impacts of pavement preservation actions on pavement performance and longevity, 3) the effects of various pavement design parameters and climatic factors on pavement performance, 4) the aspects of a newly developed global model to accurately adjust the measured pavement deflections, and 5) the elements of a new statistical model to predict future pavement condition and distress based on a single data point.

This report can be used by pavement researchers, teachers in academic and lifelong (continuing) education settings, practicing engineers and contractors involved in pavement preservation, and road owners to develop cost-effective pavement preservation strategies.

Cheryl Allen Richter
Director, Office of Infrastructure
Research, Development, and Technology

TECHNICAL REPORT DOCUMENTATION PAGE

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16. Abstract

This is a revised version of the following report: *Pavement Performance Measures and Forecasting and the Effects of Maintenance and Rehabilitation Strategy on Treatment Effectiveness* (FHWA-HRT-16-047), available at https://www.fhwa.dot.gov/publications/research/infrastructure/pavements/ltpp/16047/index.cfm.

New and innovative pavement performance measures and rating systems were developed and successfully implemented using pavement condition and distress data obtained from the Long-Term Pavement Performance (LTPP) database and the databases of three State transportation departments—Colorado Department of Transportation, Louisiana Department of Transportation and Development, and Washington State Department of Transportation. The new systems were used to calculate the benefits of various pavement treatments, determine the impacts of the pavement conditions in climatic regions on pavement performance, and assess the effects of design variables on pavement longevity. All analyses were conducted using MATLAB®-based computer programs that were specifically developed during this study. A preliminary method to estimate pavement performance based on a single data point was developed. Finally, the LTPP measured pavement deflection data were analyzed to determine whether the data could be used as pavement distress indicators. During the analyses, existing temperature-adjustment procedures were evaluated, and a global algorithm was developed that was applied to all deflection sensors in all climatic regions.

Due to editorial errors, this report is a revised version of a report with the same name issued in January 2017 under publication number FHWA-HRT-16-047. Revisions were made to tables 109–111 and figures 83–90. An errata document is available at https://www.fhwa.dot.gov/publications/research/infrastructure/pavements/ltpp/16047/.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
AI	Asphalt Institute
CDOT	Colorado Department of Transportation
CFP	change in functional period
CPI	Consumer Price Index
CRCP	continuously reinforced concrete pavement
CS	condition state
CSP	change in structural period
DF	dry-freeze
DNF	dry-no-freeze
DRR	deterioration rate reduction
DSL	design service life
ESAL	equivalent single-axle load
EUAC	equivalent uniform annual cost
FCROP	functional condition reoccurrence period
FHWA	Federal Highway Administration
FWD	falling weight deflectometer
GPS	General Pavement Studies
HMA	hot-mix asphalt
IRI	International Roughness Index
JPCC	jointed portland cement concrete
JPCP	jointed plain concrete pavement
JRCP	jointed reinforced concrete pavement
LADOTD	Louisiana Department of Transportation and Development
LCCA	lifecycle cost analysis
LTE	load transfer efficiency
LTPP	Long-Term Pavement Performance
MEPDG	*Mechanistic-Empirical Pavement Design Guide*
NPW	net present worth
ORCSE	One Record Condition State Estimate
PCC	portland cement concrete
PCI	Pavement Condition Index
PCR	pavement condition rating
PDF	probability density function
PJ	performance jump
PMS	pavement management system
PQI	Pavement Quality Index
PSI	Present Serviceability Index
PSR	present serviceability rating
RFP	remaining functional period
RMS	root mean square
RSI	remaining service interval
RSL	remaining service life

RSP	remaining structural period
RWD	rolling wheel deflectometer
SA	surface age
SCROP	structural condition reoccurrence period
SHRP	Strategic Highway Research Program
SLE	service life extension
SMP	Seasonal Monitoring Program
SPS	Specific Pavement Studies
SR	surface rating
T^2M	treatment transition matrix
TAF	temperature adjustment factor
TB	total benefits
TL	treatment life
VOC	vehicle operating costs
WF	wet-freeze
WMA	warm-mix asphalt
WNF	wet-no-freeze
WSDOT	Washington State Department of Transportation

EXECUTIVE SUMMARY

The efforts of the research team in this study focused on the following two objectives of the Federal Highway Administration (FHWA) *Long-Term Pavement Performance (LTPP) Data Analysis Program—Expanded Strategic Plan*:[1]

- **Objective 7:** Quantification of the performance impact of specific design features and its problem statement "Common Characteristics of Good and Poorly Performing AC and PCC Pavements."
- **Objective 9**: Comprehensive use of LTPP to improve the management of pavement assets.

These two objectives were addressed and accomplished through the development of the following products:

- Two sets of pavement performance measures (good, fair, and poor) and (very good, good, fair, poor, and very poor) that were used to establish dual pavement condition rating systems based on pavement functional and structural performance over time.
- Efficient pavement performance quantification and prediction methodologies for pavement maintenance and rehabilitation treatment options.
- Tools for the evaluation of the roles of pavement preservation/maintenance and rehabilitation in the design of long-life pavement.

This study had the following specific objectives:

- Define pavement performance in a way that supports the selection of cost-effective pavement treatment strategy.
- Provide better estimates of pavement treatment effectiveness and the role of pavement treatments in the pavement's service lifecycle.
- Develop pavement performance prediction methodologies that are applicable to the pavement condition and distress data collected before and after the application of treatments or series of treatments.
- Analyze whether falling weight deflectometer (FWD) data can be used to indicate impending surface defects.
- Make recommendations for subsequent studies regarding the impacts and/or selection of pavement maintenance, preservation, and rehabilitation treatment options and strategies and their impacts on the pavement service life.

The dual pavement condition rating systems (described in chapter 3) are based on proposed ranges of the remaining functional period (RFP) and the remaining structural period (RSP). The research team used RFP, RSP, and LTPP-measured time-series pavement condition and distresses to develop efficient pavement performance quantification and prediction methodologies (see chapters 5 and 6). The methodologies were then used to study the benefits of various pavement treatments. The team calculated the pavement treatment benefits using various approaches, the LTPP data, and databases obtained from three State transportation departments—the Colorado Department of Transportation (CDOT), Louisiana Department of Transportation and Development (LADOTD, and the Washington Department of Transportation (WSDOT). The resulting benefits were used to analyze the impacts of pavement design factors (such as asphalt and concrete thickness and base drainage) on pavement performance. Further, the research team calculated and scrutinized the weighted average benefits of treatments applied to all pavement types in each climatic region.

To define the pavement deterioration curve with a reasonable level of certainty, all functional and structural data analyses were based on three or more time-series data points (all available data from the LTPP or State databases). The team found that the LTPP database contains fewer than three International Roughness Index (IRI) and/or distress data points over time for some of the LTPP test sections (see chapter 4 for details). Indeed, in several cases, only one data point had been collected between the applications of consecutive treatments. To increase the number of test sections that could be analyzed, the research team developed the following two procedures (see chapters 5 and 6):

- For asphalt concrete (AC) overlay or mill-and-fill treatments, one data point of 0.0039 inches (0.1 mm) at 0.01 years after construction was added to the rut depth and cracking data. This addition facilitated the analyses of most test sections having only two time-series data points.

- A novel probabilistic approach was developed using the LTPP data for the estimation of RFP or RSP of pavement sections where only one IRI and/or one distress data point was available in the database. The method was referred to as the One Record Condition State Estimate (ORCSE) method (see chapter 5).

During the study, the research team conducted two sets of analyses. In one set, the LTPP inventory and pavement condition and distress data from LTPP Standard Data Release 28.0 (2014) were used. Results of the analyses included RFP and RSP and the treatment benefits expressed in various terms. In the second set, the inventory and pavement condition and distress data that were measured along various pavement projects by three State transportation departments (CDOT, LADOTD, and WSDOT) were requested, received, reviewed for compatibility with the LTPP data, and analyzed. For each treated pavement project, the treatment benefits were calculated using the same parameters as those used for the LTPP data analyses. The team then compared the results of the analyses of the LTPP and State data. Chapter 8 details the objectives of the comparison, which included the following:

- Determine whether the LTPP data are representative of the State data.

- Assess whether the dual pavement condition rating systems developed in this study are also applicable to the State data.

- Evaluate whether the developed treatment benefit measures are applicable to the State data.

In this study, the measured pavement deflection data along flexible and rigid pavement test sections were also studied to determine whether pavement deflection could be used as an indicator of future conditions or surface distresses. Because the deflection data were measured at different times and temperatures, the data for flexible pavement test sections were adjusted to a standard temperature of 70 ºF (21 ºC) using the Asphalt Institute (AI) and other procedures. It was determined that existing procedures were not accurate. Therefore, a new global temperature correction procedure applicable to all deflections measured by the FWD sensors in the four climatic regions was developed (see chapter 7). The impact of this global procedure on the backcalculated layer moduli in flexible pavement was also assessed.

Chapter 9 summarizes the numerous conclusions the research team reached based on the results of the analyses. The following milestone conclusions were reached:

- The dual pavement condition rating systems developed in this study and based on ranges in RFP and RSP were applicable to both the LTPP and State data.

- For any data collection cycle, the pavement condition and distress data varied from one test section to the next. Hence, the measured pavement condition and distress data along some control sections were not representative of the data along the corresponding test sections.

- The flexible pavement treatment benefits calculated from the LTPP data indicated that drainable bases had substantial positive impact on pavement performance in the wet-freeze (WF) region, and drainable bases did not add benefits in the other three climatic regions (wet-no-freeze (WNF), dry-freeze (DF), and dry-no-freeze (DNF)).

- Comparable test sections located in the four climatic regions did not perform the same with regard to IRI; rut depth; or alligator, longitudinal, or transverse cracking. Sections located in the WF region performed the worst compared with test sections located in the other three climatic regions.

- The impact on pavement performance in the WF region could be lessened by increasing the thickness of the AC and/or improving drainage.

- Thin overlay treatment had little to no impact on pavement performance in terms of alligator, longitudinal, and transverse cracking. Reflective cracking appeared in a short time period after the application of thin overlay.

- The condition and distress of the pavement sections before treatment affected the treatment longevity. The worse the pavement condition and distress was before treatment, the shorter the expected service period of the treatment was.

- The LTPP Seasonal Monitoring Program (SMP) deflection data did not support the AI temperature correction procedure.

- A new and innovative algorithm was developed to adjust the measured deflection data to a standard temperature of 70 ºF (21 ºC). The new algorithm applied to all deflection sensors and in all climatic regions.

- No consistent trends in the pavement deflection were observed over time. Hence, inclusion of deflection data in the algorithm of the dual pavement condition rating systems was not appropriate.

- Deflection data measured using the FWD could be used neither as an indicator of future pavement condition or distress nor to develop threshold values for the analysis of the pavement RFP and RSP.

- The methodologies used in the analyses of the LTPP data were also successfully applied to the State data. These methodologies were computerized using MATLAB® computer programs and Microsoft® Excel spreadsheets for formatting and organizing the results.

- The major difference between the LTPP and the State data was that, for each pavement condition and distress type, the LTTP database contained one data point per test section for every data collection cycle. In contrast, for a given pavement project, the State databases contained as many data points as the number of 0.1-mi (0.16-km)-long pavement segments along the project.

- The LTPP-measured IRI and distress data were similar and representative of the State-measured IRI and distress data in terms of magnitude and variability.

- The treatment benefits calculated using the LTPP data were parallel to the benefits of similar treatments calculated using the State data.

- The treatment benefits calculated using the LTPP data could be used as benchmarks for State transportation departments to check the performance of their pavement treatments and to assist them in conducting lifecycle cost analyses.

Based on the results of these analyses and the conclusions, the research team makes the following recommendations:

- Adopt the new dual pavement condition rating systems based on ranges in RFP and RSP as national standard measures to classify pavement condition and performance in a way that supports the selection of cost-effective pavement treatment strategy.

- Embrace the pavement performance prediction methodologies as national standard methodologies to unify and standardize the assessment of pavement performance and pavement treatment effectiveness.

- Conduct further research studies to incorporate the new dual pavement condition rating systems and the pavement performance prediction methodologies in the lifecycle cost analyses to optimize short- and long-term pavement treatment strategies.

- Conduct studies to determine the factors causing variability in the measured pavement condition and distress data and develop procedures to minimize their effects.

- Measure, at minimum, three sets of pavement condition and distress data over time before treatment.

- Initiate studies to establish automated quality control and assurance procedures for data collection and storage that minimize the impact of subjective factors. These studies may include the newly developed self-powered wireless macro-sensors that can be embedded in pavement and transportation infrastructures)

- Fund a study to determine the most efficient data collection frequency based on treatment types (such as thin and thick overlay, chip seal, and so forth) and their expected service periods.

CHAPTER 1. INTRODUCTION

This final report of the study entitled *Pavement Performance Measures and Forecasting and the Effects of Maintenance and Rehabilitation Strategy on Treatment Effectiveness* presents the findings and recommendations of the research team. The study was based on the following three groups of objectives:

- **Group A—Condition State (CS)**: Address the classification of the CS (good, fair, or poor) of the various pavement segments and types based on their identified and measured functional condition (ride quality and safety) and structural integrity (rut depth and cracking) and their rates of deterioration

- **Group B—Treatment Effectiveness**: Address the developed methodology used to estimate the effectiveness of each pavement treatment type and combinations of treatments based on the time-series pavement condition and distress data. The series of pavement maintenance and rehabilitation treatments required to improve the longevity of the pavement structures and hence the pavement lifecycle costs were evaluated.

- **Group C—Roles of Pavement Treatments**: Document the roles of pavement maintenance and rehabilitation treatments in the design of long-life pavements through quantification of the pavement RFP and RSP.

Analyses of the pavement conditions, distresses, and other data were conducted to support each objective group. The data used in the analyses were obtained from Standard Data Release 28.0 of the LTPP database published in January 2014 and from three State transportation departments—CDOT, LADOTD, and WSDOT. The data for all LTPP treatment types performed on test sections within the Specific Pavement Studies (SPS) (SPS-1 through -7) and General Pavement Studies (GPS) (GPS-6, -7, and -9) experiments were downloaded and organized for analyses. For some treatments, the before-treatment and the after-treatment pavement condition and distress data of each test section were modeled and analyzed to do the following:

- Determine the treatment benefits.
- Assess the impacts of the applied treatment strategies on the pavement performance.

The study objectives were formulated to perform analyses and make recommendations in support of objectives 7 and 9 of the *Long-Term Pavement Performance (LTTP) Data Analysis Program—Expanded Strategic Plan*.(1) The following specific LTPP goals and objectives were addressed and accomplished in this study:

- Performance measures of various distress indicators for different pavement types were established. The measures were used to develop pavement condition classification systems based on good, fair, and poor ratings.

- Efficient pavement performance prediction methodologies for pavement maintenance and rehabilitation options were developed to characterize series of pavement maintenance and rehabilitation activities applied over the life of a pavement system. Recommendations for incorporating the methodologies into subsequent studies on maintenance and rehabilitation timing and strategy selection were also made.

- The roles of pavement preservation/maintenance and rehabilitation in the design of long-life pavement and quantification of the remaining service life (RSL) of cracked or damaged pavements were addressed.

During the study, pavement condition and distress data and other inventory and treatment data from the LTPP experiments and from three State transportation departments (CDOT, LADOTD, and WSDOT) were obtained, organized, and analyzed to do the following:

- Define the pavement performance in a way that supports the selection of cost-effective pavement treatment strategy.

- Provide better estimates of pavement treatment effectiveness and the role of pavement treatments in the pavement's service lifecycle.

- Develop pavement performance prediction methodologies applicable to the pavement condition and distress data collected before and after the application of treatments or series of treatments.

- Analyze whether FWD data could be used to indicate impending surface defects.

- Make appropriate recommendations for subsequent studies regarding the impacts and/or selection of pavement maintenance, preservation, and rehabilitation options and strategies and their impacts on the pavement service life.

Consequently, the contents of this final report are organized in the following nine chapters:

- **Chapter 1. Introduction**: This chapter describes the objectives of the study and outlines the contents of the report.

- **Chapter 2. Literature Review**: This chapter includes a detailed literature review conducted in support of phases I and II of the study. The literature review addresses the state of the practice of various State transportation departments and the methodologies used in their analyses and interpretations of the pavement performance data. This chapter also provides a detailed review of previous analyses of the LTPP data and their consequent findings and recommendations as they relate to this study.

- **Chapter 3. Pavement Condition Classification**: This chapter presents the newly developed dual pavement condition rating systems.

- **Chapter 4. Data Mining and Synthesis**: This chapter summarizes the number of LTPP test sections and the number of applied treatments for which the LTPP database has sufficient time-series data to conduct the analyses in phase II.

- **Chapter 5. LTPP Data Analyses of Flexible Pavements**: This chapter summarizes the results of the analyses of the LTPP flexible pavement data.

- **Chapter 6. LTPP Data Analyses of Rigid Pavements**: This chapter summarizes the results of the analyses of the LTPP rigid pavement data.

- **Chapter 7. FWD Deflection Data Analyses**: This chapter summarizes the results of the analyses of the LTPP deflection data

- **Chapter 8. State Data Analyses**: This chapter details and discusses the methodologies used in the analyses and the results of the analyses of the pavement performance data obtained from three State transportation departments—CDOT, LADOTD, and WSDOT.

- **Chapter 9. Summary, Conclusions, and Recommendations**: This chapter summarizes the report, presents the conclusions, and makes recommendations regarding practices going forward as well as suggestions for future studies.

Readers should note detailed data and results from this study are available through LTPP Customer Support Services by telephone at (202) 493–3035 or by email at ltppinfo@fhwa.dot.gov.

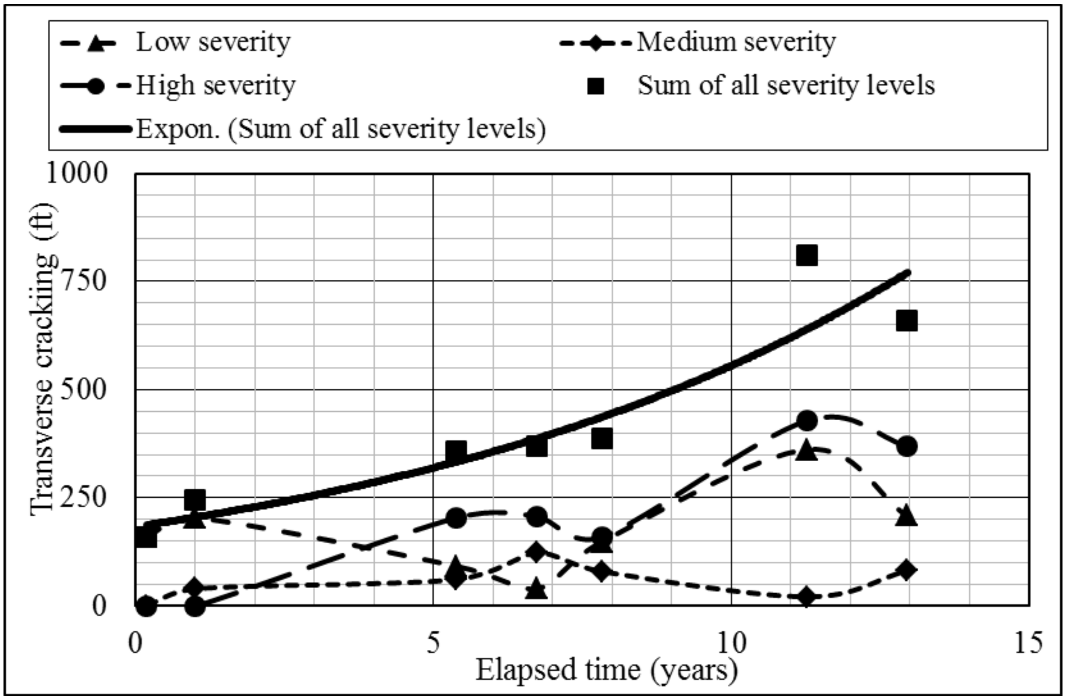
CHAPTER 2. LITERATURE REVIEW

The research team conducted an extensive literature review in support of this study. The review focused on various topics, including the following:

- Pavement distress severity levels.
- Threshold values.
- Definitions and methodologies used to determine good, fair, and poor pavement conditions.
- Effectiveness of various pavement maintenance and rehabilitation treatments and their predicted and measured performance.
- Selection of pavement preservation and rehabilitation strategies and their impacts on the pavement service life.
- Advantages and shortcomings of pavement treatment benefits, including the RSL concept.
- LTPP experimental design and the in-place pavement sections.
- Research findings from previous studies of the LTPP data.

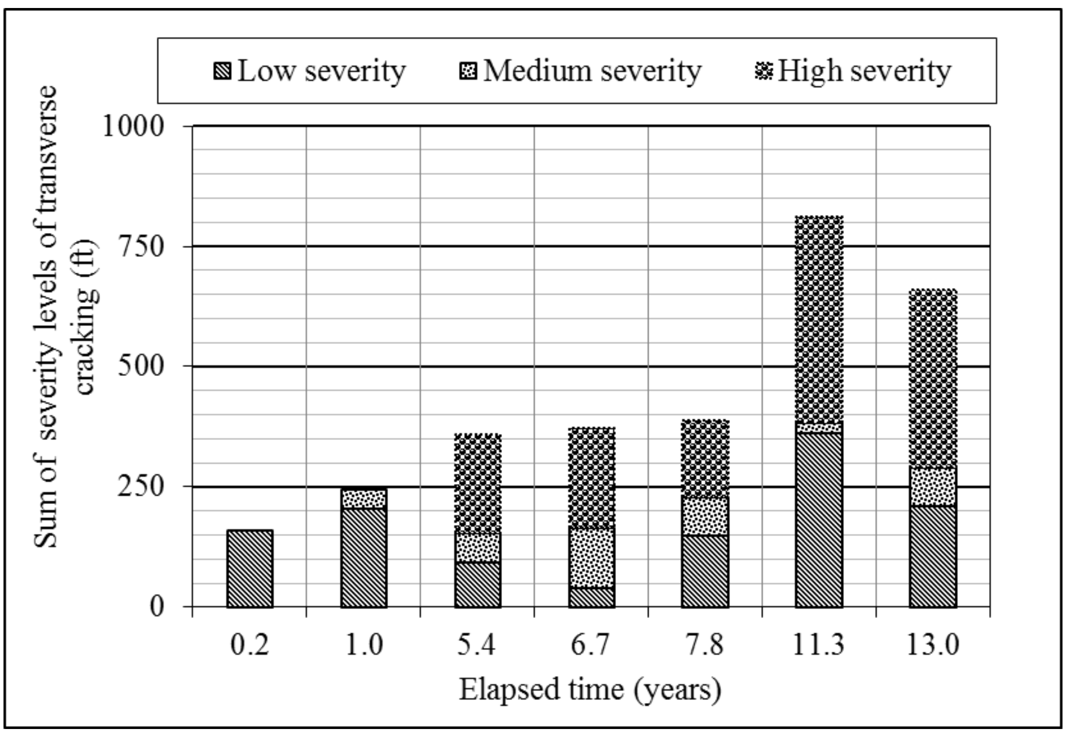
PAVEMENT DISTRESS SEVERITY LEVELS

The LTPP and the majority of State transportation department pavement distress data are collected based on three severity levels: low, medium, and high. The distress severity rating can be problematic because it is a function of the judgment of the surveyor who is observing the pavement or, in the case of many State transportation departments, the surveyor who is reviewing and digitizing the electronic pavement surface images. Such judgment is a function of the degree of training and experience of the surveyors. Further, the same pavement segment may not be reviewed by the same surveyor each year or each data collection cycle. In addition, the crack severity level is a function of the crack opening, which is a function of the pavement temperature at the time of data collection. Thus, a crack may be labeled “high severity” in one year and medium severity the next year or vice versa. Figure 1 and figure 2 depict an example of the time-series data for each transverse crack severity level for LTPP test section A330 of the SPS-3 experiment in California.



1 ft = 0.305 m.

Figure 1. Graph. Time-series transverse cracking data for each severity level and the sum of all severity levels for LTTP SPS-3 test section A330 in California.



1 ft = 0.305 m.

Figure 2. Graph. Cumulative time-series transverse cracking data showing individual transverse crack severity level and the sum of all severity levels for LTTP SPS-3 test section A330 in California.

The data in the two figures indicate the following:

- The length of transverse cracks for any given severity level changes from one year to the next without the application of any pavement treatment. To illustrate, the length of the low severity transverse crack in figure 1 is about 197 ft (60 m) in year 1, 82 ft (25 m) in year 5, 49 ft (15 m) in year 7, more than 328 ft (100 m) in year 11, and 197 ft (60 m) in year 13. The medium severity crack length is approximately 33 ft (10 m) in year 1,131 ft (40 m) in year 7, only about 16 ft (5 m) in year 11, and about 82 ft (25 m) in year 13. Finally, the length of the high severity transverse cracks is about 230 ft (70 m) in year 6, 164 ft (50 m) in year 8, 427 ft (130 m) in year 11, and 377 ft (115 m) in year 13.

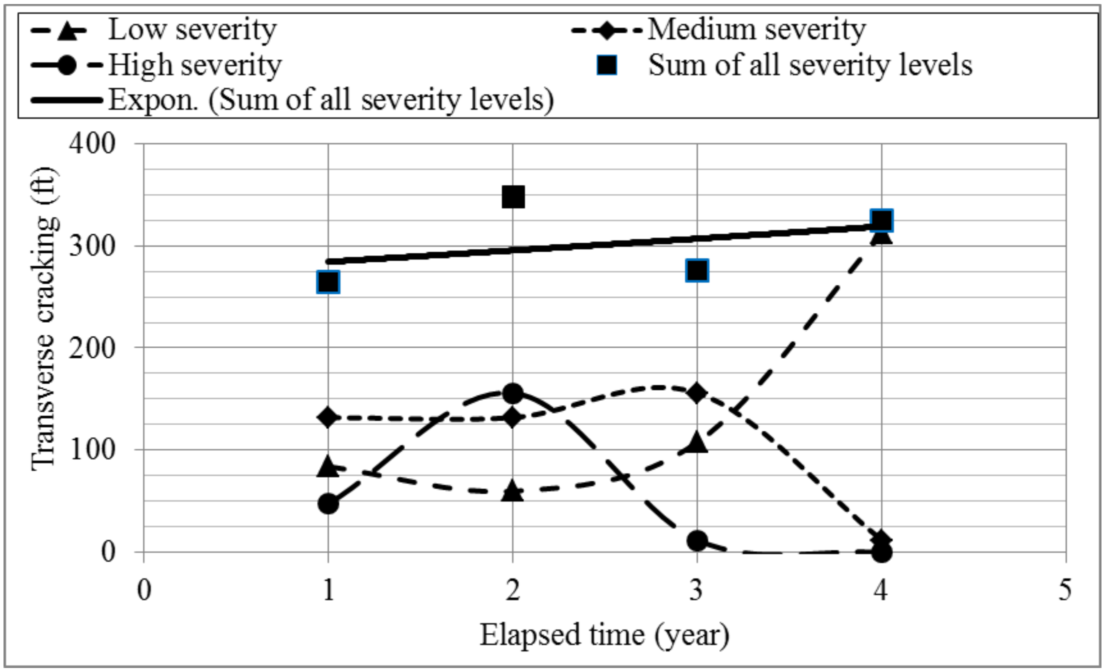
- The variability of the crack lengths of the three severity levels could be attributed to several reasons, the three most important of which are the following:

  - The pavement temperature at the time of data collection influences the data. Higher temperature causes the crack width to decrease, resulting in an observed change in severity level. This problem cannot be addressed unless the pavement temperature is measured during the survey, and an accurate temperature-dependent crack width model is developed. Note that the LTPP surveyors do collect pavement temperature data during a survey while most State transportation departments collect temperature data only on a limited basis.

  - The pavement surveyor judges and labels some cracks as low severity in one survey year and medium or high severity in subsequent survey year. This inconsistency could be addressed through computerized crack-rating quality control and/or enhanced observer training.

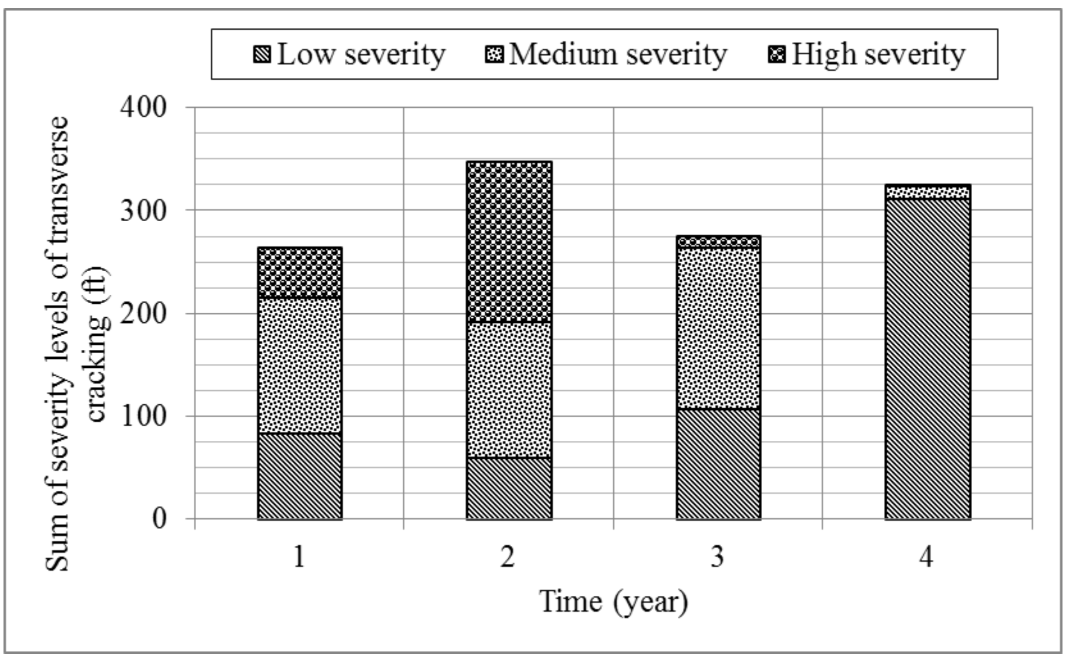
  - The high variability of the individual severity levels does not allow accurate modeling of the crack propagation over time. In fact, the data indicate that the medium and high-severity transverse crack lengths decrease and then increase over time without any pavement treatment. A previous study sponsored by the FHWA expressed the pavement cracking data as the sum of the three severity levels.(2) This yielded much less data variability, as evidenced by the exponential model of the total transverse crack length shown in figure 1.

The crack severity level data could be used to roughly estimate the amount of work needed to preserve the pavement section. For example, cracks in the medium- and high-severity levels need to be sealed or patched. Low-severity cracks are typically not sealed or patched. For rigid pavements, low-severity transverse cracks may be subjected to dowel bar retrofit, while medium-and high-severity cracks typically are not.(3) Similar patterns can be found in the State transportation departments' cracking data as shown in figure 3 and figure 4 for locations along a portion of Highway 24 in Colorado.



1 ft = 0.305 m.

Figure 3. Graph. Time-series transverse cracking data for each severity level and the sum of all severity levels, Highway 24, direction 2, BMP 329.9, in Colorado.



1 ft = 0.305 m.

Figure 4. Graph. Cumulative time-series transverse cracking data showing individual transverse crack severity level and the sum of all severity levels, Highway 24, direction 2, BMP 329.9, in Colorado.

ENGINEERING THRESHOLDS (CRITERIA)

Some State transportation departments express pavement conditions and distresses using one or more of the following methods (see figure 5 and figure 6):[4,5]

- A descriptive scale, such as very good, good, fair, poor, and very poor.
- A distress index based on a continuous rating scale (i.e., 0 to 10 or 0 to 100). One end of the scale defines failed pavement, and the other end defines excellent pavement condition and/or no distress, such as in the new pavement shown in figure 5. Some State transportation departments use the rating scale to calculate one distress index for each type of distress (i.e., individual distress indices), while others use a composite pavement index. A composite index is typically based on several types of distress and/or condition. Examples of composite pavement indices include Pavement Condition Index (PCI), Pavement Quality Index (PQI), overall pavement index, and so forth.
- Along the rating scale, one or more threshold values are typically established to flag pavement sections for possible treatment actions. One threshold value could be based on the need for maintenance, another on the need of preservation action, and a third could be based on rehabilitation. Depending on the functionality of the threshold value (maintenance, preservation, or rehabilitation), a distress index value below the established threshold value indicates the need to maintain, preserve, or rehabilitate the pavement section in question. The rehabilitation threshold value typically separates acceptable from nonacceptable pavement conditions.

Deduct distress points rating scale with a threshold value of 60 points (100 = Excellent pavement)										
0	10	20	30	40	50	60	70	80	90	100
Very poor			Poor			Fair		Good	Very good	

Descriptive scale

Engineering criterion = maximum acceptable number of cracks = 50 transverse cracks per 0.1 mile long pavement segment, which is equivalent to 1 crack every 10.5 feet and corresponds to 60 points on the rating scale

Deduct value method

Distress points per transverse crack = DP/TC

$$\mathrm{DP/TC} = \frac{(\text{maximum rating scale value} - \text{threshold value})}{\text{Maximum acceptable number of transverse cracks}} = \frac{(100-60)}{50} = 0.8$$

Distress points = DP = 100 - (0.8)(the number of TC)

Cumulative distress points rating scale with a threshold value of 40 points (0.0 = Excellent pavement)										
0	10	20	30	40	50	60	70	80	90	100
Very good	Good		Fair	Poor			Very poor			

Descriptive scale

Cumulative distress points

Distress points per transverse crack = DP/TC

$$\mathrm{DP/TC} = \frac{(\text{Threshold value})}{\text{Maximum acceptable number of transverse cracks}} = \frac{40}{50} = 0.8$$

Distress points = DP = (0.8)(the number of TC)

1 mi = 1.61 km.
1 ft = 0.305 m.

Figure 5. Chart. Rating and descriptive scales and distress points.

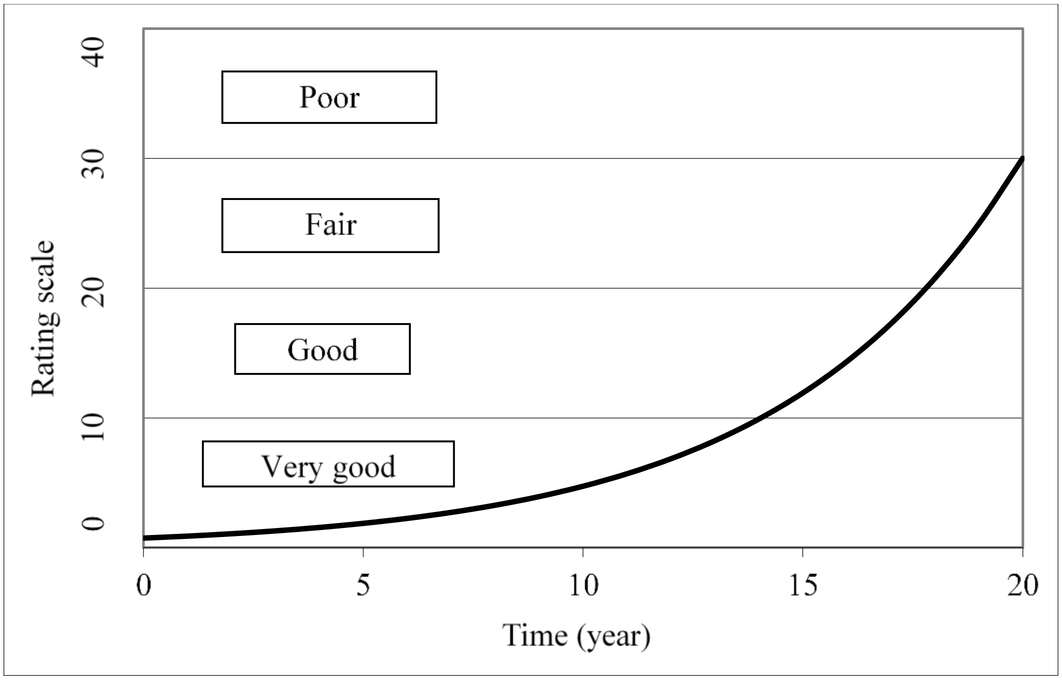


Figure 6. Graph. Descriptive rating scale for pavement condition.

It is important to note that if the threshold value is established based on engineering criteria, the pavement condition rating will be such that the relative condition of the pavement segment is constant for a given condition. The engineering criterion should be selected based on the experience of the transportation department and should address the extent of the condition or distress at which the pavement section in question is deemed in need of repair (maintenance, preservation, or rehabilitation) within the constraints of the department. An example of engineering criterion for transverse cracking could be 600 ft (183 m) of cracking or 50 transverse cracks (crack spacing of about 10.5 ft (3.2 m) along a 0.1-mi (0.16-km)-long asphalt pavement segment). Based on the engineering criterion, distress points can be assigned to each occurrence of the distress (each transverse crack) and the rating scale threshold value. To illustrate, consider the continuous rating scale of 0 to 100 (100 indicates no transverse cracks) and its threshold value of 60 points as shown in figure 5. An engineering criterion of 50 transverse cracks per 0.1 mi (0.16 km) implies that the asphalt pavement score is 60 (it loses 40 distress points) when the pavement segment accumulates 50 transverse cracks. Based on a linear accumulation of distress points, each transverse crack is worth 0.8 distress points.(4) If the agency decided to change the threshold value from 40 to 50 but to maintain the engineering criterion of 50 transverse cracks, then 50 cracks would cause the pavement section to lose 50 distress points and each crack would be worth 1 distress point. Stated differently, the engineering criteria for establishing the threshold value should be based on the extent of the distress rather than a number on the rating scale.

Finally, the engineering criteria express the conditions of the pavement and could be based on the user or the agency. Examples of roadway user-based criteria are ride quality (IRI) and rut depth. Examples of agency-based criteria are cracking and faulting. One other factor to note is that the engineering criteria for certain distress or condition types could be global or could be established based on pavement class, traffic volume, regional needs, and so forth. Nevertheless, the methods used to develop the engineering criteria should be well documented, and the criteria should be studied and calibrated as more pavement condition and distress data become available.

Many State transportation departments, such as the LADOTD, the CDOT, the Michigan Department of Transportation, and the WSDOT, have developed engineering criteria for each distress type and severity level. Examples of such criteria for alligator cracking and the associated deduct points are listed in table 1.(6)

Table 1. Engineering criteria and deduct points for alligator cracking.(6)

Severity	Alligator Cracking Deduct Points Extent (ft^2)					
	0–51	**51–701**	**701–1,301**	**1,301–2,401**	**2,401–3,168**	**> 3,168**
Low	0	1–16	16–21	21–25	25–28	28
Medium	0	1–21	21–29	29–36	36–49	48
High	0	1–29	29–43	43–50	43–61	61

1 ft^2 = 0.093 m^2.

PAVEMENT DISTRESS AND CONDITION INDICES

Pavement distress or condition indices are often based on one or more condition or distress types. For example, the Alligator Cracking Index (an individual index) is based on the severity levels (low, medium, and high) and the extent of the alligator cracks, whereas a combined pavement distress or condition index (such as PCI) consists of two or more condition or distress types. The combined index expresses the sum of the distress points assigned to each distress or condition type and severity level divided by the number of pavement segments (see figure 7). Hence, a combined pavement distress index expresses the average pavement condition and not the actual condition based on individual distress types.(4,7)

$$DI = \frac{\sum DP}{N} = \left(\frac{0.1}{L}\right)\left(\sum DP\right)$$

Figure 7. Equation. *DI*.

Where:

DI = Distress index.
ΣDP = Sum of the distress points along the project.
N = Number of 0.1-mi (0.16-km)-long segments along the project ($N = L/0.1$).
L = Project length in mi (km).

Finally, the distress points or the pavement condition indices do not express the true nature of the pavement conditions. For example, immediately after construction, the cumulative distress points of a pavement project subjected to a 2-inch (51-mm) asphalt overlay are exactly the same as the cumulative distress points for another project subjected to a 6-inch (152-mm) asphalt overlay. The pavement surface conditions of both projects are free of distresses. Stated differently, neither the distress points nor the condition indices express the design life of the overlay or the impact of the type of pavement preservation or rehabilitation on the pavement service life. Further, the differences between the distress points and the pavement distress index before and after treatment cannot and should not be used to express the benefits of the applied pavement maintenance or rehabilitation treatments. Consider three pavement sections having the same distress points and distress index that were subjected to 2-, 4-, and 6-inch (51-, 102-, and 152-mm) asphalt overlays, respectively. The differences in the distress points and distress index before and after treatment were exactly the same although the costs of the overlays were substantially different and so were their design service lives (DSLs) and future pavement performance. The DSL of the treatment and the pavement rate of deterioration must be accounted for in the calculation of the true benefits of the treatments.(2)

DESCRIPTIVE PAVEMENT CONDITIONS

Descriptive terms (such as good, fair, and poor) are also used to express the various categories of the pavement conditions. Although the terms hide important details, they are universal and easily communicated to legislators and the general public. The three terms are typically based on the pavement appearance and/or ride quality at the time of rating. Descriptive classifications of good, normal or fair, and poorly performing AC and portland cement concrete (PCC) pavements were previously addressed in four FHWA reports published in 1998, 1999, 2011, and 2012. (See references 8–11.) The shortcomings of the first two reports are that the descriptive terms are based on the last collected pavement condition data as shown in figure 8. Figure 9 depicts the actual time-series IRI data for three in-service pavement sections located in the State of Washington. Over the 10-year period, these sections were not subjected to any pavement treatment. Figure 9 clearly shows the following:

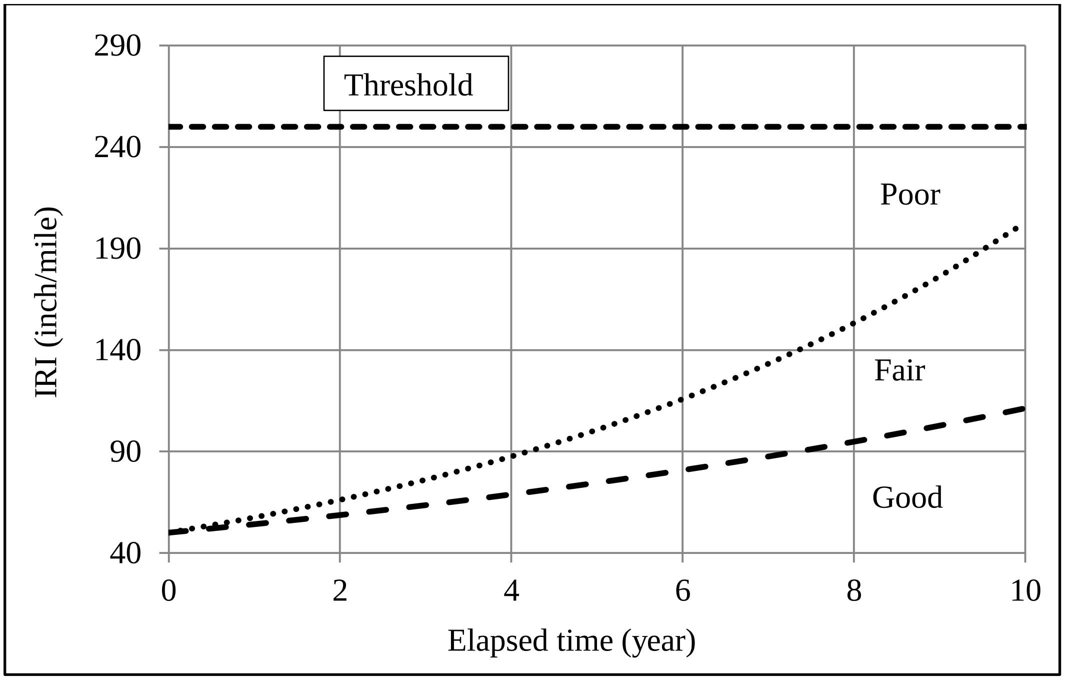
- Data along a 2.4-mi (3.9-km)-long pavement segment of road 2 indicate that the descriptive term changed from good to fair to poor in only 3 years. Thus, the descriptive terms do not accurately reflect the true pavement performance.

- Data along a 3.6-mi (5.8-km)-long pavement segment of road 3 changed performance descriptions over time from poor to fair (labeled normal in the reports) and then to good.

- Data along a 4.8-mi (7.7-km)-long pavement segment of road 1 show that the pavement description fluctuated between good and fair and then between fair and poor. Once again, such descriptive terms do not reflect the true in-service pavement performance over time.

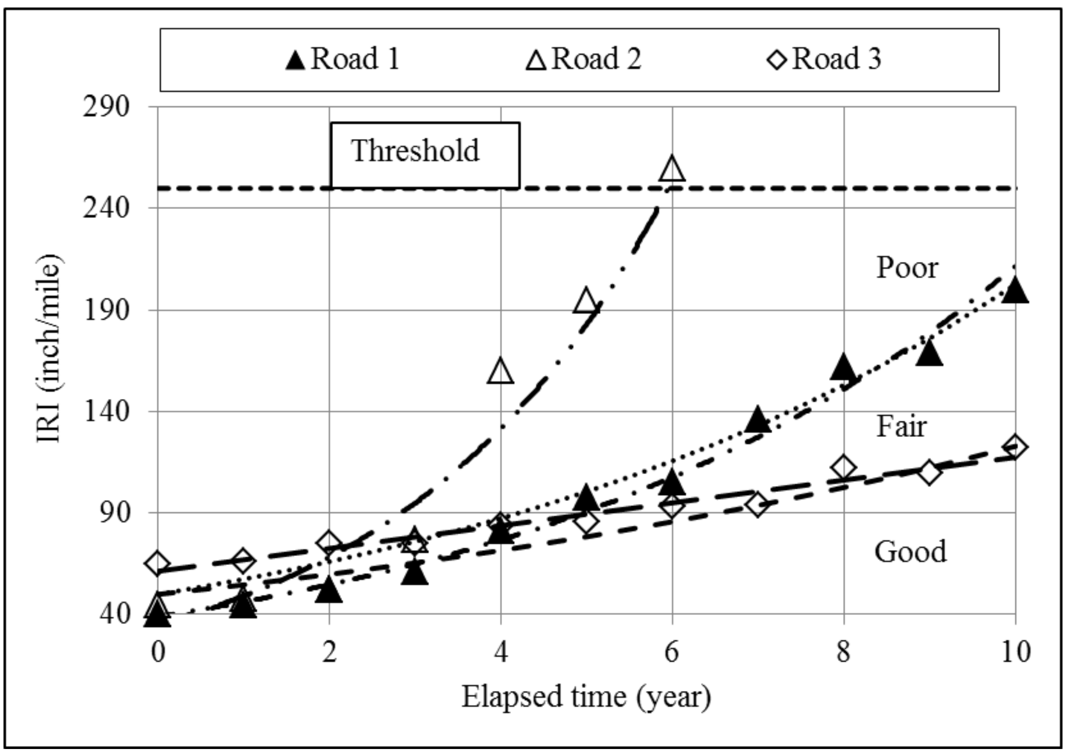
- After construction, Roads 1 and 2 were considered good, and then at the elapsed time of 4 years, the description of road 1 was fair while road 2 was poor. The pavement rate of deterioration was not reflected in the characterization.

The three descriptive terms could be improved to better express the pavement conditions if they were based on the pavement's rates of deterioration.



1 inch/mi = 0.0158 m/km.

Figure 8. Graph. Pavement condition classification system.



1 inch/mi = 0.0158 m/km.

Figure 9. Graph. Shortcomings of the recommended classification system when dealing with real but good data (not the worst-case scenario).

On the other hand, the latter studies describe the terms good, fair, and poor and their potentially associated treatment categories as follows:(11)

- **Good**: Pavement infrastructure that is free of significant defects and has a condition that does not adversely affect its performance. This level of condition typically requires only preventive maintenance activities.

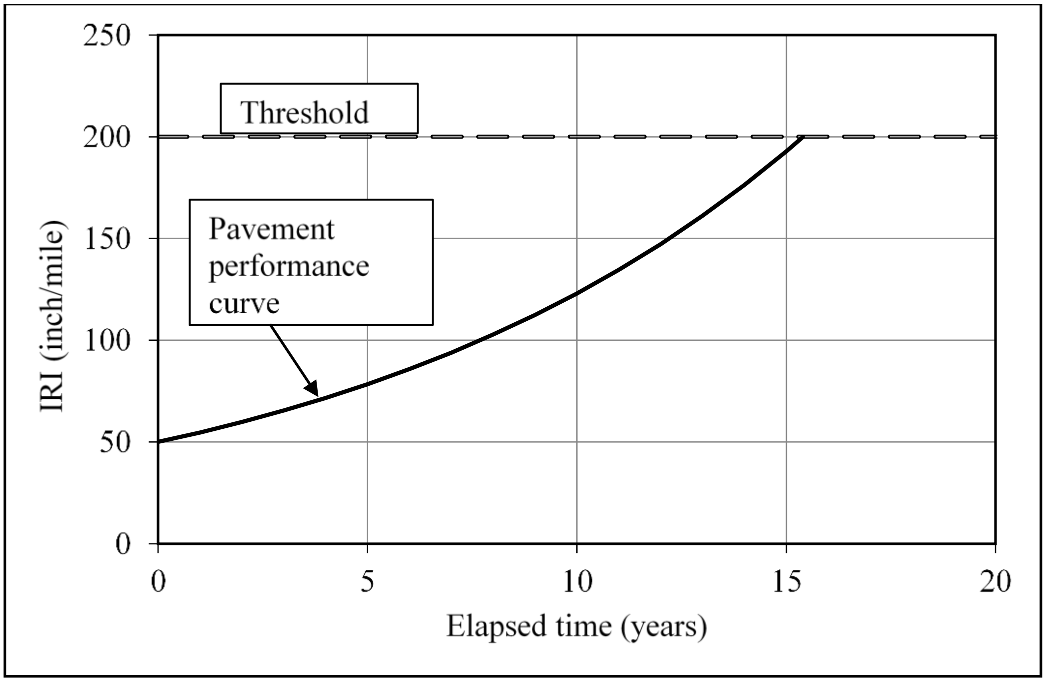
- **Fair**: Pavement infrastructure that has isolated surface defects or functional deficiencies. This level of condition typically could be addressed through minor rehabilitation, such as overlays and patching of pavements that do not require full-depth structural improvements.

- **Poor**: Pavement infrastructure that is exhibiting advanced deterioration and conditions that affect structural capacity. This level of condition typically requires structural repair, rehabilitation, reconstruction, or replacement.

Once again, the significant issue with these types of definitions is that they do not consider the changes in conditions and distresses over time. The terms do convey the current conditions of a pavement well, but pavement health is not best demonstrated by a snapshot in time. The pavement conditions and distresses generally deteriorate with time, and the pavement health depends on the current conditions and the rates of deterioration over time. The specific terms could still serve their purpose, but the criteria used to assign the rating should be modified to account for the effects of time. Consideration of condition and rates of deterioration facilitates planning and pavement management, while condition alone has limited use as a tool for managing pavement unless the deterioration rate (curve) is estimated based on the available data.

PAVEMENT PERFORMANCE MODELING AND PREDICTION

The performance of a pavement segment is often illustrated by the progression of pavement condition or distress over time, as shown in figure 10. The level of performance at any given time is equivalent to the level of pavement condition or distress at that time compared with the threshold value. Therefore, the performance of a pavement segment over its service life is defined by the level of service over time or by the accumulation of damage over time.(12)



1 inch/mi = 0.0158 m/km.

Figure 10. Graph. A typical pavement performance curve.

Most State transportation departments collect pavement condition and distress data. Some use the data to observe the condition of the pavement, while others use the time-series pavement condition and distress data to predict future pavement conditions. The combination of both practices allows the development of current and future strategies for management of the pavement network.

Many State transportation departments have studied the effectiveness of various pavement treatments using historical pavement performance data. Based on the various studies, the minimum and maximum treatment service lives listed in table 2 were published in the various sources listed for each treatment type. These estimated averages are adequate to be used in the analysis at the network level. For project-level analysis, more accurate estimates are required. Such estimates could be based on predictions of past and future pavement conditions through the modeling of pavement condition and distress data before and after treatment to create pavement performance curves.

Table 2. Estimated and reported pavement treatment service life.

Treatment Type	Reference	Estimated Treatment Service Life Expectancy (Years)		
		Minimum	Average	Maximum
Thin (< 2.5-inch) hot-mix asphalt (HMA) overlay	16–22	2	8	12
Thick (≥ 2.5-inch) HMA overlay	23	6	10	17
Single chip seal	16–18, 24, and 25	1	6	12
Double chip seal	17, 18, 21, 24, and 26	4	9	15
Thin (< 2.5-inch) mill and fill	21 and 23	4	8	20
Thick (≥ 2.5-inch) mill and fill	23	6	10	17
Cold-in-place HMA recycling	17, 26, and 27	5	10	20
Crack sealing	16 and 17	2	3	10
Microsurfacing	16, 18, 20, 22, 28, and 29	4	6	10

1 inch = 25.4 mm.

Several predictive pavement performance models have been developed to estimate the pavement performance curve based on parameters such as traffic, weather, and pavement type. These models include straight-line extrapolation, regression, polynomial constrained least squares, application of Shaped curves, use of probability distributions, and Markov chain models.(13) One such example, for thin HMA overlays, is presented in figure 11. The β parameters were determined for different performance indicators (IRI, pavement condition rating (PCR), and rut depth) as well as different road types (interstate, noninterstate highway, and nonhighway).(14)

$$PI = e^{\beta_1 + \beta_2 \times CAATT + \beta_3 \times CAFDX}$$

Figure 11. Equation. *PI*.

Where:

PI = Performance indicator for a pavement segment in a given year.
CAATT = Cumulative average annual daily truck traffic (in millions) predicted for the pavement segment from the time of treatment to the given year.
CAFDX = Cumulative annual freeze index (in thousands of degree-days) predicted from the time of treatment to the given year.
β_1, β_2, and β_3 = Statistical parameters.

The most common method for modeling pavement condition and distress data as a function of time is by ordinary least squares regression. It should be noted that a minimum of three time-series data points are required to model the nonlinear data. The method used to determine the parameters of the selected mathematical function (see figure 12 through figure 14) consists of minimizing the sum of squared errors. This method works when the data of the particular pavement segment indicate deterioration over time. If the method does not capture the progression of condition or distress over time, other models may be required.(15)

$$RD = \gamma t^{\omega}$$

Figure 12. Equation. *RD*.

$$IRI = \alpha e^{\beta t}$$

Figure 13. Equation. *IRI*.

$$Crack = \frac{k}{1 + exp[-\theta(t - \mu)]}$$

Figure 14. Equation. *Crack*.

Where:

RD = Rut depth.
α, β, γ, ω, k, θ, and μ = Regression parameters.
Crack = Crack length, area, or percent.
t = Elapsed time in years.

Such models can be used to estimate the time until a certain threshold value is reached. *Threshold* is the prespecified condition or distress level indicating unacceptable pavement condition or distress has been reached and that the pavement segment is in need of maintenance, preservation, or rehabilitation depending on the level at which the prespecified threshold value is set.

Another method of modeling pavement condition and distress data is the clusterwise regression procedure, which was introduced by Spath and later modified by others. (See references 15 and 30–35.) Clusterwise regression involves splitting the data into subgroups based on their characteristics and fitting separate models to each subgroup. The resulting pavement performance models could be more accurate because they model small subsets of data with similar trends. However, the resulting models are discrete (each model represents a certain time period).

PAVEMENT PRESERVATION BENEFITS

Most procedures for estimating pavement preservation benefits are based on the prediction of future pavement performance, comparison of the pavement performance before and after treatment, and immediate changes in the pavement conditions resulting from treatment. Although the commonality among all procedures is a prespecified threshold value, the actual value of the threshold varies from one procedure to another. Some procedures set one threshold value for each type of pavement distress and condition, while others use the combined distress index and an overall threshold value. Still others use one threshold value for maintenance, one for preservation, and one for rehabilitation and/or reconstruction. Further, some procedures use the term *life* to express the benefits. Such a term should not be taken separately from service life. For example, the term *pavement design life* used by the *American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures*, in reality, expresses the pavement DSL.[36] The pavement design itself is based on the following threshold values:

- The 1993 *AASHTO Design Guide* specifies a threshold value based on the Present Serviceability Index (PSI) of 2.5 (which is equivalent to an IRI of about 200 inches/mi

(3.16 m/km)).[36] The threshold value expresses the minimum ride quality standard. A pavement at or below a PSI of 2.5 is providing the user with substandard ride quality. The ride quality could be restored by many preservation and/or rehabilitation actions (such as thin or thick overlays). Stated differently, when the prespecified threshold is reached, it implies that the minimum standard is reached while the pavement is not in need of reconstruction. If the threshold value, on the other hand, is set at the reconstruction limit, then reconstruction is needed.

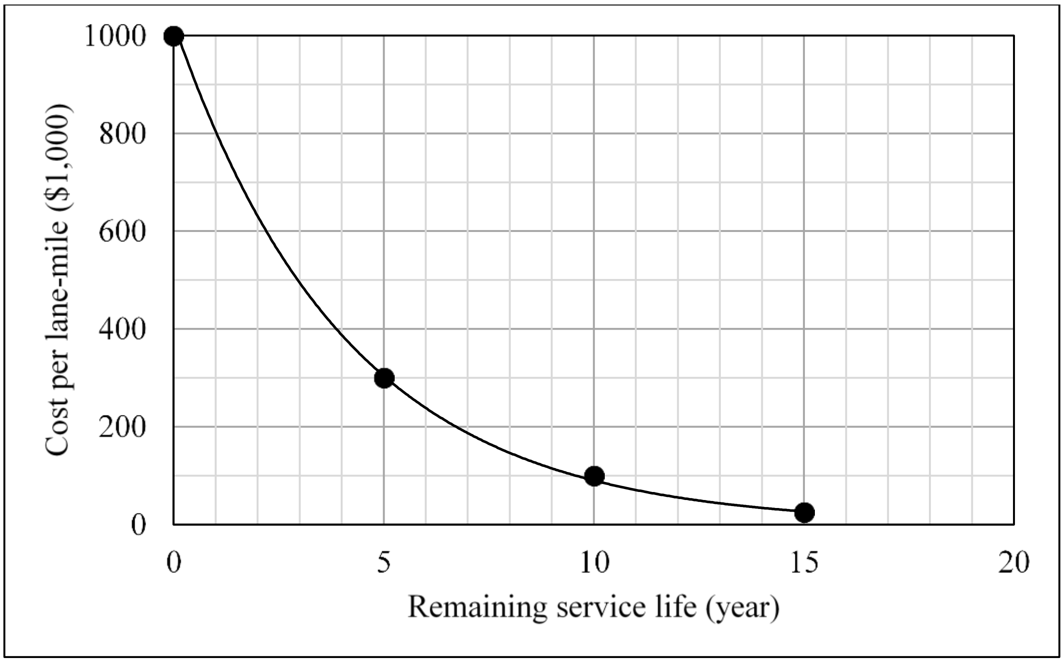
- The new AASHTO *Guide for Mechanistic-Empirical Pavement Design of New and Rehabilitated Pavement Structures* (referred to as the MEPDG) is based on one threshold value for ride quality (IRI), one for rut depth, one for faulting, and one for each type of cracking, similar to that listed in table 3.[37]

Table 3. Threshold values that could be used in the AASHTO MEPDG.

Pavement Condition or Distress Type	Threshold Value	Explanation
Alligator cracking	1,267 ft^2/0.1 mi (73 m^2/0.1 km)	Twenty percent of lane area cracked (assuming 12-ft (3.66-m) lane width)
Longitudinal cracking	1,056 ft/0.1 mi (200 m/0.1 km)	Two cracks along the entire section length
Transverse cracking (Jointed concrete pavements)	396 ft/0.1 mi (50 m/0.1 km)	Two thirds of the slabs are cracked (assuming 16-ft (4.88-m)-long slab)
Transverse cracking (HMA)	350 ft/0.1 mi (67 m/0.1 km)	Crack spacing = 12 ft (3.66 m)
Faulting	0.25 inches, average over 0.1 mi (6.35 mm, average over 100 m)	Dowel bars have likely sheared or concrete around dowels has deteriorated and may be spalled

Nevertheless, various procedures were developed for estimating treatment benefits. The following are some of these procedures:

- **RSL**: RSL is the estimated number of years from any given year (usually from the last condition survey year) to the date when the conditions or distresses of the pavement section reach a prespecified threshold value. It is very important to note that the prespecified threshold values could be set at the level when the pavement is in need of maintenance, at the pavement preservation level, or at the major rehabilitation or reconstruction level. It is strongly recommended that the prespecified threshold value be set by the roadway owner to support its pavement preservation system, such as those recommended by the AASHTO MEPDG or listed in table 3.[37] At these levels, a given pavement section requires major rehabilitation, and the RSL is zero. State transportation departments can also establish the RSLs or the ranges of RSL at which pavement preservation and pavement maintenance can be applied. The reason for this recommendation is that the cost of pavement maintenance, preservation, or rehabilitation at the specified ranges of RSLs could be included in the analyses. This recommendation can be illustrated as follows. Suppose the threshold value for longitudinal cracking is set at 1,056 ft/0.1 mi (200 m/0.1 km) as listed in table 3. When a pavement section reaches the threshold value, the RSL is 0 and the cost of preservation is much higher than the cost of an earlier preservation when the RSL is 5 years. Further, when the RSL is 10 years or more, the cost of preservation is much lower yet and it may be limited to the cost of the required maintenance treatment only. Thus, the transportation department could establish a continuous scale of costs versus RSLs such as that shown in figure 15.



1 lane-mile = 1.61 lane-km.

Figure 15. Graph. Relationship between *RSL* and cost of managing pavements.

For each pavement section, the following steps are required for calculating the RSL:

- Download from the database the pavement surface age and all consecutive pavement condition and distress data points collected over a time period where no treatment was applied.
- Use the condition and distress data points and the corresponding data collection times to obtain the equation of the best fit curve using the proper mathematical function.
- Input to the best fit equation the threshold value of the condition or distress in question and calculate the time in years between construction and the time when the pavement condition will reach the preestablished threshold value. The RSL is the difference between the calculated time and the pavement surface age.

In the case of a new pavement structure or a newly rehabilitated pavement, the estimated RSL must be positive and restricted to be less than or equal to the DSL of the pavement or the DSL of the treatment, as stated in figure 16.[4] The reason is that for a few years after treatment, the pavement may or may not show any distress, and hence, the estimated

RSL is very large and meaningless. The restriction could be dropped when a significant number of data points indicating pavement deterioration are available. At that time, the RSL could be greater or less than the DSL, and the information could be used as feedback to the pavement design and construction processes.

$$0 \leq RSL = \{t(PC = Th) - SA\} \leq (DSL - SA)$$

Figure 16. Equation. *RSL*.

Where:

$t\ (PC = Th)$ = Time (the number of years) at which the pavement condition reaches the threshold value (Th).
SA = Pavement surface age in years.
DSL = Pavement design service life in years.

The important point is that the RSL does not advocate worst-first as perceived by a few people. It is a management tool that allows State transportation departments to manage their pavement asset based on engineered criteria and a long-term preservation program. It should be noted that the accuracy of RSL is a function of the accuracy and variability of the pavement condition and distress data. In addition, the accuracy of the estimated RSL decreases as the value of that RSL increases (i.e., predicts much further out in time).

The *RSL* of a given pavement network can be calculated as the weighted average *RSL* of the total number of pavement sections, *n*, within the network using figure 17:

$$RSL_{network} = \frac{\sum_{i=1}^{n}(RSL_i)(SL_i)}{\sum_{i=1}^{n}(RSL_i)(SL_i)}$$

Figure 17. Equation. $RSL_{network}$.

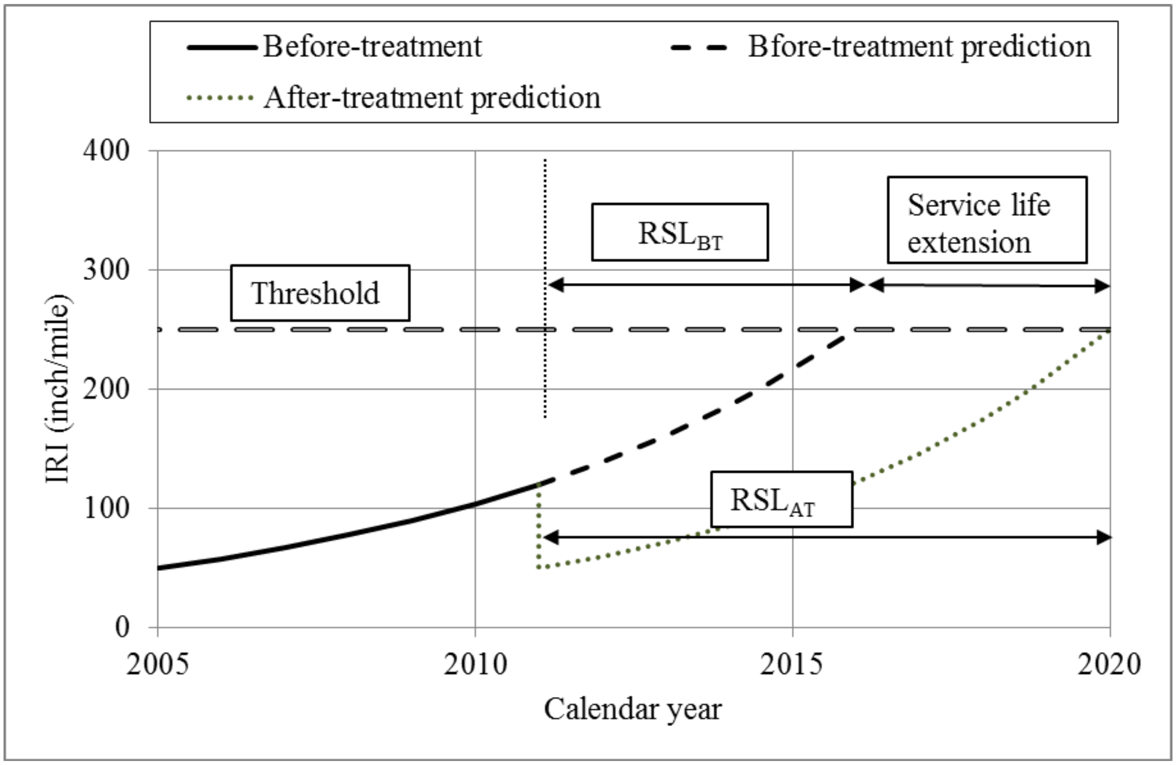
Where:

i = ith pavement segment.
n = Total number of pavement segments or sections in the network.
RSL = Remaining service life.
SL = Segment length.

It should be noted that any pavement segment that falls below the threshold value has an *RSL* of zero. In general, no negative *RSL* should be assigned to any pavement regardless of its condition. For a newly designed and constructed or rehabilitated pavement segment, its *RSL* is equal to the design life.[(4)]

- **Remaining service interval (RSI):** RSI is similar to RSL but with some differences. RSI is a new pavement performance measure, which, at the time of this report, was being analyzed on another research study sponsored by FHWA. The final algorithm of the RSI was not used in the current study because it was not available during the study.

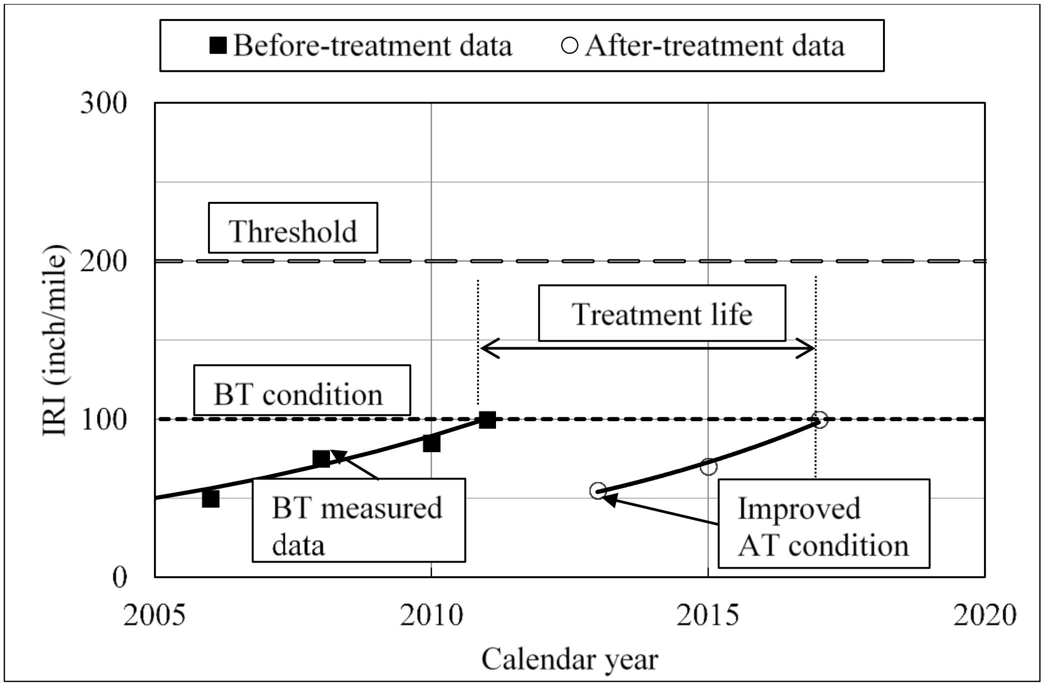
- **Service life extension (SLE):** SLE is the gain in service life resulting from a pavement treatment, as shown in figure 18.(5) The accuracy of SLE is a function of the accuracy of the two estimated RSLs before and after treatment. SLE is a useful tool for determining the time benefit resulting from a pavement treatment.



1 inch/mi = 0.0158 m/km.

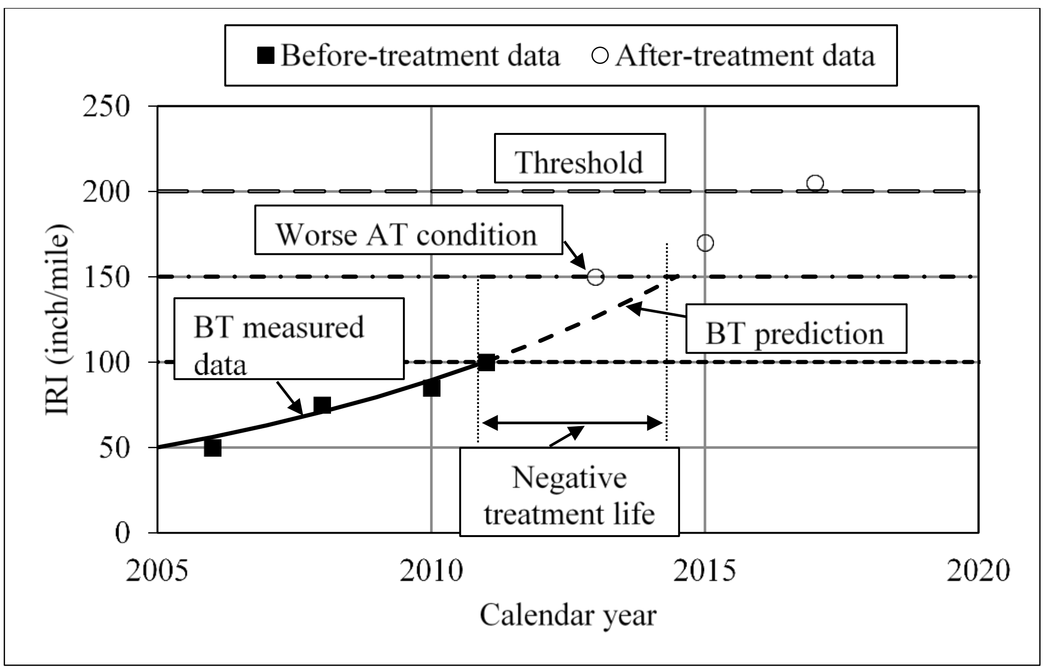
Figure 18. Graph. Schematic of the definition of SLE.

- **Treatment life (TL):** TL is the time between the treatment date and the date when the pavement conditions or distresses reach the lesser of the threshold value or the before treatment pavement condition or distress, as shown in figure 19.(5) TL involves the same limitations as the predicted RSL after treatment (note that the TL does not require any RSL prediction before treatment), except with a shorter prediction in time. TL is a good tool to determine the time until the before treatment conditions return. In the case of worse pavement condition or distress after treatment, the TL is taken as the negative of the time for the before-treatment conditions or distresses to reach the after-treatment conditions, as shown in figure 20.(5)



1 inch/mi = 0.0158 m/km.

Figure 19. Graph. Schematic of the definition of TL.



1 inch/mi = 0.0158 m/km.

Figure 20. Graph. Schematic of the definition of negative TL.

- **Total benefits (TB)**: TB is the ratio of the benefit area to the do-nothing area, as depicted in figure 21.(22) TB accounts for the improved condition over a given time, the area bound by the performance curve and a threshold value; however, it has some significant flaws. TB can be misunderstood because two pavement sections can have the same area ratio but completely different performance. Stated differently, any ratio can be obtained by the division of an infinite set of two numbers such as 1 and 3, 2 and 6, 6 and 18, and so forth. The different perspective of the TB is that the benefit area is normalized relative to the do-nothing area. The do-nothing area, on the other hand, is a function of the pavement conditions and rate of deterioration before treatment.

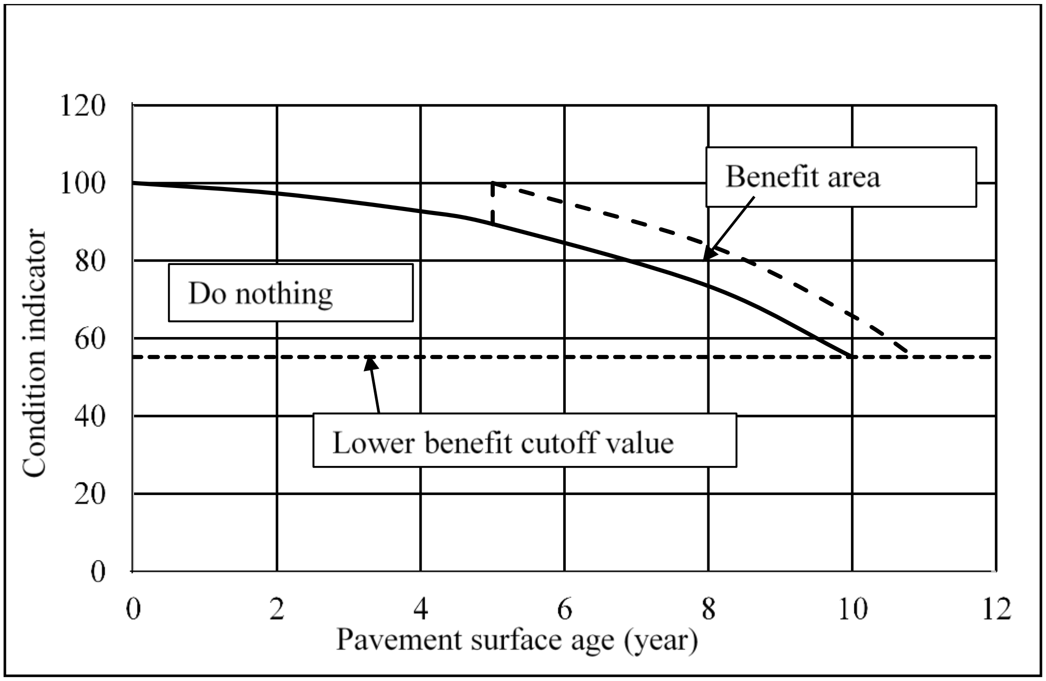
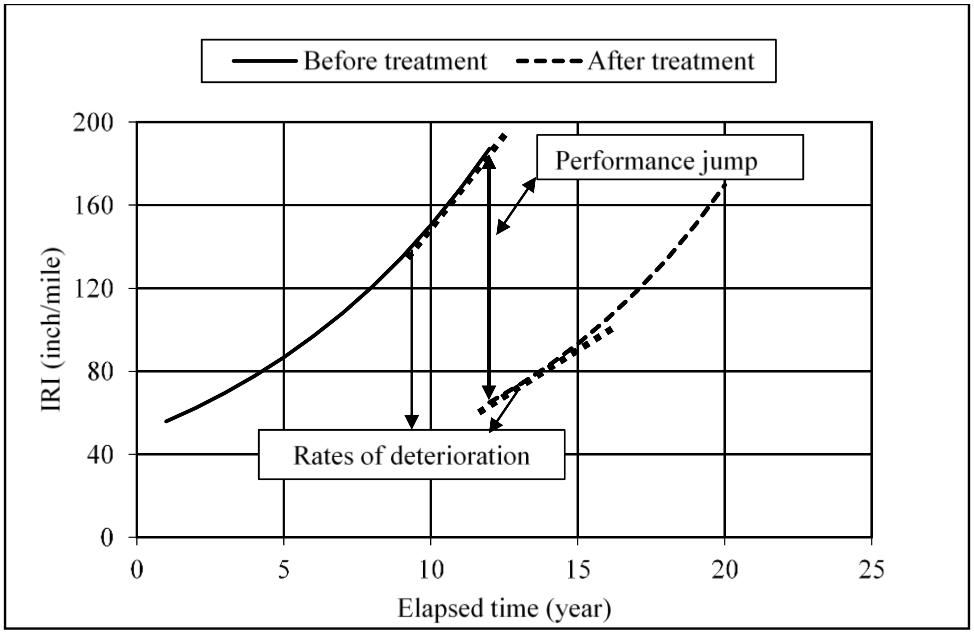


Figure 21. Graph. Schematic of the definition of TB.[22]

- **Performance jump (PJ)**: PJ is the immediate improvement in the pavement condition resulting from treatment.[38] PJ indicates the temporary improvement resulting from treatment but has no way to predict the future conditions or how long the improvement will last. An example of PJ is depicted in figure 22.

- **Deterioration rate reduction (DRR):** DRR (see figure 22) is the change in deterioration rate from immediately before to immediately after treatment.[38] The measure is short term and therefore is not a true measure of performance.



1 inch/mi = 0.0158 m/km.

Figure 22. Graph. PJ and DRR.

PAVEMENT TREATMENT TYPES

The number of available pavement treatment types is large and ever growing as new techniques and materials are developed. Each State transportation department has a group of treatments it chooses to apply based on its experience and the results achieved over time. The selection of a particular pavement treatment from this pool of options is often specific to each State transportation department. The next section provides a discussion of the selection process.

PAVEMENT TREATMENT SELECTION

Many State transportation departments have developed plans and methodologies for selecting pavement treatments. The most common are decision trees and matrices. These are often developed from past experience and tend to focus on one or two options. An example of a decision tree is shown in figure 23 and its reference table 4, and an example of a matrix is shown in table 5. The values in table 4 indicate the trigger values corresponding to roadway functional classifications for use in figure 23. The table contains trigger values based on PQI, present serviceability rating (PSR), and surface rating (SR). These trees/matrices are rarely updated and often neglect new technology. Nonetheless, they are typically based on the following data:[17]

- Pavement surface type and/or construction history and environmental conditions.
- An indication of the functional classification and/or traffic level.
- At least one type of PCI, including distress and/or roughness. More specific information about the type of deterioration present, either in terms of an amount of load-related deterioration or the presence of a particular condition or distress type.
- Geometric data indicating whether pavement widening or shoulder repair are required.

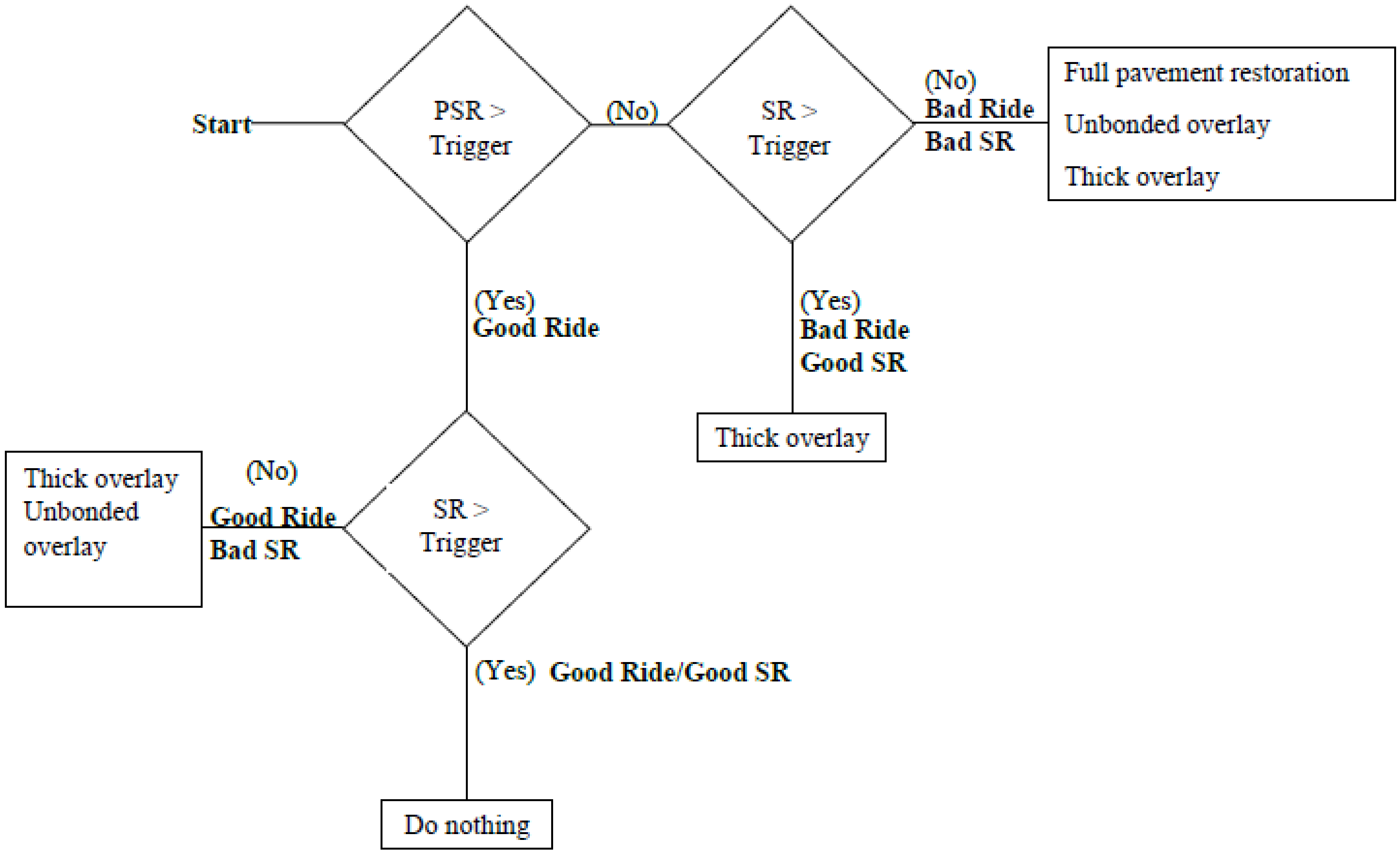


Figure 23. Illustration. Example of decision tree for continuously reinforced concrete pavement (CRCP).

Table 4. Trigger values for functional classifications.

Functional Classification	Trigger Value		
	PSR	SR	PQI
Rural principal interstate	3.0	2.7	3.0
Rural principal arterial	3.0	2.7	2.9
Rural minor arterial	2.8	2.5	2.8
Rural major collector	2.8	2.5	2.6
Rural minor collector	2.8	2.5	26
Rural local	2.7	2.4	2.6
Urban interstate	3.1	2.7	3.0
Urban principal arterial freeway	3.1	2.7	2.9
Urban principal arterial	2.8	2.5	2.9
Urban minor arterial	2.7	2.4	2.8
Urban collector	2.6	2.4	2.6
Urban local	2.5	2.4	2.6

Table 5. Example of decision matrix.

Problem	Possible Cause				Maintenance				Rehabilitation					
	Structural failure	Mix composition	Temperature or moisture changes	Construction	Patching & routine maintenance	Fog seal	Surface treatment	Slurry seal	Surface recycling	Thin overlay	Open-graded surface	Structural overlay	Structural recycling	Reconstruction
Alligator cracking	X	—	—	—	X	—	X	X	—	—	—	X	X	X
Edge joint cracks	X	—	X	X	X	—	—	—	—	—	—	—	—	—
Reflection cracks	—	—	—	—	X	—	X	X	—	—	X	X	X	—
Shrinkage cracking	—	X	X	—	—	—	X	X	X	—	X	X	X	—
Slippage cracks	—	—	—	X	X	—	—	—	—	—	—	—	—	—
Rut depth	X	X	—	X	—	—	—	—	X	X	—	X	X	X
Corrugation	X	X	—	X	—	—	—	—	X	X	—	X	X	X
Depressions	X	—	—	X	X	—	—	—	—	—	—	—	X	X
Upheaval	—	—	X	—	X	—	—	—	—	—	—	—	X	X
Potholes	X	—	X	X	X	—	—	—	—	—	—	X	—	—
Raveling	—	X	—	X	—	X	X	X	X	X	—	—	—	—
Flushing asphalt	—	X	—	X	—	—	X	—	X	—	X	—	—	—
Polished aggregate	—	X	X	—	—	—	X	—	X	X	X	—	—	—
Loss of cover aggregate	—	X	—	X	—	—	X	—	—	—	—	—	—	—

— Indicates not applicable.

Pavement Preservation Costs

This subsection reviews pavement preservation costs and lifecycle cost analyses (LCCAs). Owing to a lack of available cost data required for detailed analyses, no such analyses are included in this report. However, the information is provided to assist in such analyses if more data become available.

The costs of any pavement treatment can be divided into two categories: agency costs and user costs. The agency cost is the physical cost of the pavement project, including design and construction less the residual value of the pavement section at reconstruction. This is often referred to as direct costs.(39) User costs are much more difficult to estimate than agency costs because they are not based on specific monetary value but on vehicle operating costs (VOC), delay costs, and accident costs. The three types of user costs and how they relate to normal and work zone conditions are listed in table 6 and discussed in the next few subsections.(40)

Table 6. Review of user cost components.

Component	Normal Operation	Work Zone Conditions
VOC	Based on total delay-hours and driving cost caused by accidents	Based on total delay-hours caused by work zone and accidents in the work zone
Delay	Total delay-hours caused by accidents	Total delay-hours (caused by work zone and accidents in work zone)
Accidents	Number and severity of accidents	Number and severity of work zone accidents

One problem that arises when estimating user costs is the transformation of delay, accidents, etc., to a monetary value.(41) Some believe that user costs should be defined as "user benefit" and expressed qualitatively as improvements in performance or safety.(42,43) The user benefits of one treatment compared with another or with the do-nothing alternative could be used to choose between treatments with similar agency costs. In other words, if two treatment options satisfy the pavement needs and have similar agency costs, then the deciding factor would be the user costs. This would greatly simplify the process, which is often considered complicated and deficient, especially when applicable data are not available for the various detailed user cost models.(44) However, LCCA should be completed to evaluate both the agency and user costs over the pavement lifecycle and to select the most cost-effective treatment strategy. For completion, these costs and LCCA are discussed in the next few subsections.

LCCA

Recently, State transportation departments have faced many constraints, including public demand for quality pavement and budget shortfalls. Hence, the considerations become the following:(45)

- What pavement preservation alternatives should be used, and how often should a given pavement section be preserved?
- How many miles of each pavement class should be preserved annually?

- What is the optimum time or the optimum pavement conditions and distresses at which pavement preservation actions should be taken?

- What are the associated agency and user costs and benefits of each pavement treatment?

- What are the optimum and most cost-effective short- and long-term pavement preservation strategies that can be applied to keep the pavement network healthy in a cost-effective manner?

These questions cannot be properly and accurately answered unless LCCA is conducted. Such analysis should address the agency and the user costs and must be based on accurate and up-to-date data so that the costs and benefits of various pavement preservation alternatives can be compared.

The Need for LCCA

In general, highway pavements are designed and constructed to provide services for a limited time called the "service life." Over time, the combined effects of traffic loads and environmental factors accelerate the pavement deterioration and reduce its level of serviceability. Maintenance, preservation, and rehabilitation treatments are designed and applied to pavement sections to slow their rates of deterioration and to extend their service lives. The application of most pavement treatments requires traffic control (lane closures and/or detours), which significantly affects traffic flow, increases travel time, and increases VOC. The costs and benefits of pavement treatments are composed of many elements, including the following:

- Agency costs of the pavement treatment, which consist of many attributes, including the following:

    - Material and contractual costs.

    - The cost of traffic control in the *work zone*, which is defined as an area along the highway systems where maintenance and construction operations adversely affect the number of lanes open to traffic or affect the operational characteristics of traffic flow through the work zone.[46]

    - Quality assurance and quality control costs.

    - Costs of future treatments.

- Agency benefits, which could be measured by the service life of the treatment or the SLE of the treated pavement sections.

- User costs, which are composed of many attributes, including the following:[47,48]

    - Time delay user costs or *work zone user costs*, which are defined as the associated costs of time delays due to lane closures because of roadway construction, rehabilitation, and maintenance activities.[49]

- Costs incurred by those highway users who cannot use the facility because of either agency or self-imposed detour requirements.[50]

- VOC in terms of fuel, wear and tear, and depreciation over the delay periods.

- Accident costs.

- Environmental costs resulting from air pollution caused by excessive use of gasoline or diesel fuel owing to lower speed and time delay, including noise pollution.

- User benefits, which are composed of improved serviceability and ride quality that would lower the VOC and improve traffic flow.

Methods of LCCA

Because LCCA considers all planned pavement treatments of a given analysis period, the service life and the value of money over time should be considered. One hundred dollars in 2014 likely bought much more than $100 will in 2024. Hence, the following two common methods are incorporated in LCCA to account for this:

- **Net present worth (NPW)**: NPW or net present value is a common economic indicator. NPW is the monetary value of an action accounting for the transformation of the value of money over time using the discount rate (see figure 24). The use of NPW allows fair comparison of actions taken at different times by converting to a common unit of measure.[50]

$$NPW = initial\ cost + \sum_{k=1}^{N} Preservation\ Cost_k \left[\frac{1}{(1+i)^{n_k}}\right]$$

Figure 24. Equation. *NPW*.

Where:

NPW = Net present worth.
N = Total number of preservation treatments.
i = Discount rate.
n = Number of years into the future.
k = Preservation action number.

The discount rate reflects the rate of inflation adjusted to the opportunity cost to the public. The opportunity cost is often indicated by a comparison with the discount rate of the conservative U.S. Treasury bill. The historical inflation rate trend from 1999 to 2014 indicates a range of –0.35 to 3.58 percent with an average of 2.39 percent. Table 7 lists common discount rates used by State transportation departments in the 1990s. The discount rate should reflect historical trends in the nation or region where the analysis is conducted.[50] Alternatively, the discount rate could be determined from the Consumer

Price Index (CPI). The average CPI discount rate from 2001 to 2010 was about 2.54 percent.(45,51)

Table 7. Historical discount rates.[1]

Year	Analysis Period (Years)				
	3	5	7	10	30
1992	2.7	3.1	3.3	3.6	3.8
1993	3.1	3.6	4.0	4.3	4.5
1994	2.1	2.3	2.5	2.7	2.8
1995	4.2	4.5	4.6	4.8	4.9
1996	2.7	2.7	2.8	2.8	3.0
1997	3.2	3.3	3.4	3.5	3.6
1998	3.4	3.5	3.5	3.6	3.8
Average	3.1	3.3	3.4	3.6	3.8

[1]Effects of discount rates on $100 from 2014 to 2024 using the average CPI of 2.54 percent. One hundred dollars in 2014 has the same purchasing power as $128 in 2024.

- **Equivalent uniform annual cost (EUAC):** The EUAC method is also widely used as a common economic indicator in LCCA and is typically derived from NPW as calculated in figure 25. The use of either value allows fair comparison of actions taken at different times by converting to a common unit of measure.(50)

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

Figure 25. Equation. *EUAC*.

Where:

EUAC = Equivalent uniform annual cost.
NPW = Net present worth
i = Discount rate.
n = Number of years into the future.

PAVEMENT PRESERVATION EFFECTIVENESS

In the past, some State transportation departments and almost all local road agencies allowed their pavement assets to deteriorate to levels requiring major rehabilitation or reconstruction. For many years, their treatment policies were based on a worst-first policy in which severely deteriorated pavement sections were subjected to preservation treatments while the condition of the rest of the pavement network continued to deteriorate. Recently, many State transportation departments have initiated and implemented comprehensive pavement preservation programs at the entire road network level.

The programs are based on cost-effective treatment of sections of the pavement network in relatively good condition to restore their conditions, decrease their rates of deterioration, and enhance the safety of the motorists. Over time, the preservation program becomes a part of the annual pavement treatment strategy of the State transportation department.[52] The pavement preservation program typically consists of light pavement treatments, such as crack sealing, nonstructural overlay, light rehabilitation actions, mill and fill, and so forth. The alternative to pavement preservation is the old practice of letting the pavement network deteriorate until expensive rehabilitation or reconstruction actions are necessary. Several studies have been conducted on the effectiveness of pavement preservation and are summarized in the following subsections.

The effectiveness of pavement treatments can be measured in the short term and/or the long term. Short-term benefits are defined by the immediate improvement to the pavement conditions and rates of deterioration, while long-term benefits are defined over the service life of the pavement section by the performance and extension in service life. The costs can also be short term (individual treatment) or long term (LCCA).

Pavement Preservation Cost Effectiveness at the Project Level

Pavement preservation can be applied through a series of pavement treatments over the pavement lifecycle (a treatment strategy). The alternative to pavement preservation is allowing the pavement to deteriorate until reconstruction is required, the worst-first or do-nothing scenario. The cost effectiveness of pavement preservation at the project level can be quantified using LCCA. The analysis could be conducted on the various alternative pavement preservation treatments that could be applied to a pavement section over time and on the do-nothing scenario followed by reconstruction. Comparison of the results from the various strategy analyses indicates the cost savings or extra expenditures of performing preservation over the life of the pavement segment.

The cost effectiveness of pavement preservation at the project level has been well documented. Most literature agrees that pavement preservation can be conducted at minimal cost and create major savings over the life of the pavement. One study found that the cost savings of pavement preservation was as high as $5 saved for every $1 spent on preservation.[53] Another reports savings of $4 to $10 for every $1 spent on preservation.[54] Other benefits include improving ride quality and creating a pavement network with consistent needs from year to year.[55]

Pavement Preservation Effectiveness at the Network Level

The effectiveness of pavement preservation at the network level is more difficult to quantify than at the project level. Funds designated for preservation reduce the funds available for rehabilitation and reconstruction. In other words, pavement preservation is thought to decrease the lifecycle cost of a given pavement project, but the effect on the network is often unknown. In addition, the public and legislators may not understand why pavements in seemingly good condition are being treated, while others in poor conditions are not. State transportation departments should document and communicate the effects of preservation maintenance on the health of the pavement network and on the lifecycle cost in a clear and consistent manner. Educating the public and the legislature is necessary to establish and maintain a successful pavement preservation program.[55]

The short- and long-term benefits and effectiveness of pavement preservation at the project and network levels should be quantified. Short-term benefits include improving ride quality and addressing safety issues, while long-term benefits (cost savings) are not realized until years or decades into the future. Therefore, pavement preservation strategies must be optimized through projection of needs and funds into the future. In this way, the effects of performing or deferring various pavement projects can be evaluated.[55,56]

TREATMENT TRANSITION MATRIX (T^2M)

Pavement treatment effectiveness is often described with a single value or a range of values, such as 5 to 10 years gained or an average 7-year service life. The probabilities of the various results are not typically reported. The probabilistic effectiveness of treatments can be quantified and communicated using an innovative matrix format called T^2M.[3] T^2M shows the following:

- Distribution of the pavement CSs before and after treatment.
- Transitions of the pavement CSs from before treatment to after treatment resulting from the treatment.
- List of the treatment benefits.
- Long-term results of the pavement treatment.

Table 8 shows an example of a T^2M that lists the results of single chip seal applications in Colorado. The cells display this information in the following convenient way:

- Columns A through D list the following before treatment information: pavement CSs (RSL bracket numbers), RSL ranges, and the number and percent of 0.1-mi (0.16-km)-long pavement segments in each before treatment CS.
- Columns E through I list the following after treatment information: CSs (RSL bracket numbers), RSL ranges, the number of 0.1-mi (0.16-km)-long pavement segments transitioned from the given before treatment CS to each after treatment CS, and the total number of 0.1-mi (0.16-km)-long pavement segments transitioned to each after treatment CS.
- Columns J through L list the following pavement treatment benefits: average TL, SLE, and after treatment RSL of all 0.1-mi (0.16-km)-long pavement segments transitioned from a given before treatment CS to all after treatment CSs; and the overall average TL, SLE, and after treatment RSL.

Table 8. T²M for single chip seal in Colorado.

A	B	C	D	E	F	G	H	I	J	K	L
Condition/Distress Type: Condition/Distress Causing the Minimum RSL Before and After Treatment											
Before Treatment Data				**After Treatment Data**							
				CS or RSL Bracket Number and Range in Years and Number of the 0.1-Mi (0.16-Km)-Long Pavement Segments Transitioned From Each Before Treatment RSL Bracket to the Indicated After Treatment RSL Brackets					**Treatment Benefits in Terms of TL, SLE, and RSL of the Treatment (Years)**		
CS or RSL Bracket Number	**RSL Bracket Range (Years)**	**0.1-Mi (0.16-Km)-Long Pavement Segments**		**1**	**2**	**3**	**4**	**5**			
		Number	**Percent**	**0 to 2**	**3 to 5**	**6 to 10**	**11 to 15**	**16 to 25**	**TL**	**SLE**	**RSL**
1	0 to 2	2,329	58	**125**	453	1,230	267	254	4	8	9
2	3 to 5	746	18	3	**88**	379	121	155	3	7	11
3	6 to 10	365	9	1	52	**157**	55	100	2	4	12
4	11 to 15	141	3	0	8	52	**27**	54	2	1	14
5	16 to 25	452	11	1	24	128	55	**244**	1	–5	15
Total		4,033	100	130	625	1,946	525	807	—	—	—
Average		—	—	—	—	—	—	—	3	6	10

— Not applicable.
Bold indicates the number of pavement segments where no change in the CS resulted from the treatment.

PRESERVATION TIME SELECTION

The effectiveness of pavement treatments is often determined simply based on the benefits gained, as mentioned previously. The benefits, however, do not indicate effectiveness relative to cost, which is the main constraint for all State transportation departments. Most literature agreed that treatments applied to pavements in better condition produce more benefits, and the cost of the treatment was a function of the conditions.[38,57,58] Further, the time-value relationship of money affects the cost of the treatment and its cost effectiveness. Therefore, benefits must be compared relative to costs to determine the cost effectiveness of the treatment and to select the treatment timing.[41] Performing pavement treatments at the optimum time provides the greatest benefit-to-cost ratio. The idea of optimum timing is not new; in fact, the concept was built into the AASHTO 1993 design guide.[36] Few methodologies for the determination of optimum treatment timing for preventive maintenance and rehabilitation actions were developed.[22] This methodology is designed to optimize treatment timing based on the treatment benefit (calculated by the area under the performance curve). However, the most cost-effective treatments should consider the pavement longevity and should be based on the ratio of dollars to years of service.[3] To improve lifecycle costs, State transportation departments should base their preservation strategies on maximizing the longevity of the pavement network rather than maximizing the condition of the network.

THE LTPP PROGRAM

The LTPP Program was established under the Strategic Highway Research Program (SHRP) in 1987. Since 1991, FHWA has managed and funded the LTPP Program. The program houses two fundamental groups of experiments: SPS and GPS. The LTPP Program has addressed myriad studies of pavement-related issues ranging from validation of pavement design procedures to traffic and material variability, and pavement maintenance, preservation, and rehabilitation actions. The conclusions of these studies are documented in countless publications in the forms of research reports, product briefs, and techbriefs, which have substantially contributed to the development of advanced pavement technology and highlighted the importance of the LTPP Program and its associated database.

OBJECTIVES AND SCOPE OF THE LTPP PROGRAM

The overall objective of the LTPP Program is to collect, store, and make various data elements relating to pavement performance available to researchers, scientists, and the general public. These include the pavement structures and conditions, traffic volume and load, and environmental conditions for various pavement sections located along the existing North American highway systems. Over a more than 20-year period, the data have been used by researchers, practitioners, and other stakeholders to assess the long-term performance of pavements under various loading and environmental conditions and with different structural and material compositions. The specific established objectives of the LTPP Program include the following:[59]

- Evaluate the existing pavement design methodologies.

- Develop improved design methodologies and strategies for the rehabilitation of existing pavements.

- Develop improved design equations for new and reconstructed pavements.

- Determine the effects of loading, environment, material properties and variability, construction quality, and maintenance levels on pavement condition, distress, and performance over time.

- Determine the effects of specific design features on pavement performance.

- Establish a national long-term pavement database with detailed information suitable for various assessments and studies.

LTPP TEST SECTIONS

The LTPP Program consists of about 2,500 500-ft (152.4-m)-long, mostly in-service test sections located in all 50 U.S. States, Puerto Rico, and 10 Canadian provinces. The test sections are divided between the two main studies, SPS and GPS. Some of the SPS test sections were reconstructed to investigate certain pavement engineering factors, while others were specially preserved to study the impacts of some preservation treatments. In contrast, the GPS test sections consist of sections of existing roads that were subjected to various typical maintenance and preservation treatments. Thus, eight types of existing in-service pavements make up the GPS and are monitored throughout North America. More details on the SPS and GPS test sections can be found throughout the remainder of this report.

SPS

SPS is a long-term program designed to study specifically constructed, maintained, or rehabilitated pavement sections incorporating controlled sets of experimental design and construction features. The main objective of the SPS experiments is to provide more detailed and complete sets of data to extend and refine the results obtained from the GPS experiments. There are nine SPS experiments grouped by the five categories listed in table 9.[59]

Table 9. The SPS categories and experiments.

Category	Experiment	Title
Pavement structural factors	SPS-1	Strategic Study of Structural Factors for Flexible Pavements
	SPS-2	Strategic Study of Structural Factors for Rigid Pavements
Pavement maintenance	SPS-3	Preventive Maintenance Effectiveness of Flexible Pavements
	SPS-4	Preventive Maintenance Effectiveness of Rigid Pavements
Pavement rehabilitation	SPS-5	Rehabilitation of Asphalt Concrete (AC) Pavements
	SPS-6	Rehabilitation of Jointed Portland Cement Concrete (JPCC) Pavements
	SPS-7	Bonded Portland Cement Concrete (PCC) Overlays of Concrete Pavements
Environmental effects	SPS-8	Study of Environmental Effects in the Absence of Heavy Loads
Asphalt aggregate mixture specifications	SPS-9P	Validation and Refinements of Superpave Asphalt Specifications and Mix Design Process
	SPS-9A	Superpave Asphalt Binder Study
	SPS-10	Warm-Mix Asphalt (WMA)—in design stage

The SPS experiments involve monitoring the newly constructed pavement sections and the existing pavement sections that were subjected to maintenance or rehabilitation treatments after assignment to the SPS. The SPS is divided into various SPS experiments numbered SPS-1 through -9. Each experiment includes multiple test sites, and each test site contains between 2 and 12 pavement test sections depending on the experiment. Following the original assignment of test sections in 1992, numerous supplemental test sections were constructed by different State transportation departments to study aspects of particular interest to them.(59) FHWA is initiating new sites for the study of WMA (SPS-10) and is currently considering new pavement preservation experiments in addition to the existing SPS experiments.

GPS

GPS is also a long-term program designed to study a series of experiments on selected in-service pavement structures with the objective of establishing a national pavement performance database. Pavement sections believed to be built with proper materials and good engineering design were selected as part of the GPS program.(59)

The pavement structures included in the GPS were constructed or reconstructed up to 15 years before the start of the LTPP Program. Unfortunately, detailed data were often not available for the period between the original construction time and the time when they were selected for the LTPP Program. However, it was believed that some beneficial insights might be drawn without this data. Finally, some SPS test sections have been reclassified as GPS test sections upon the application of rehabilitation treatments. Table 10 lists the titles of each of the GPS experiments.(59)

Table 10. The GPS experiments.

Experiment	Title
GPS-1	Asphalt Concrete (AC) Pavement on Granular Base
GPS-2	AC Pavement on Bound Base
GPS-3	Jointed Plain Concrete Pavement (JPCP)
GPS-4	Jointed Reinforced Concrete Pavement (JRCP)
GPS-5	Continuously Reinforced Concrete Pavement (CRCP)
GPS-6A	Existing AC Overlay of AC Pavement (existing at the start of the program)
GPS-6B	AC Overlay Using Conventional Asphalt of AC Pavement—No Milling
GPS-6C	AC Overlay Using Modified Asphalt of AC Pavement—No Milling
GPS-6D	AC Overlay on Previously Overlaid AC Pavement Using Conventional Asphalt
GPS-6S	AC Overlay of Milled AC Pavement Using Conventional or Modified Asphalt
GPS-7A	Existing AC Overlay on Portland Cement Concrete (PCC) Pavement
GPS-7B	AC Overlay Using Conventional Asphalt on PCC Pavement
GPS-7C	AC Overlay Using Modified Asphalt on PCC Pavement
GPS-7D	AC Overlay on Previously Overlaid PCC Pavement Using Conventional Asphalt
GPS-7F	AC Overlay Using Conventional or Modified Asphalt on Fractured PCC Pavement
GPS-7R	Concrete Pavement Restoration Treatments With No Overlay
GPS-7S	Second AC Overlay, Which Includes Milling Or Geotextile Application, on PCC Pavement With Previous AC Overlay
GPS-9	Unbonded PCC Overlay on PCC Pavement

Summary of Previous Findings

In this study, previously published reports regarding the LTPP Program and data analyses were scrutinized. The topics of these reports include the effects of design factors on pavement performance measures and the selection of appropriate and cost-effective treatment type. However, the findings of these reports did not adequately address the relationships between the maintenance and rehabilitation actions and the performance of the various pavement sections or the optimal timing for treatment application. Nevertheless, some of the relevant reported findings are enumerated and summarized in the following subsections.

Impacts of Pavement Treatment on Pavement Performance

This subsection summarizes reported findings related to the impacts of pavement treatments on the collected pavement condition and distress data of various LTPP experiments.

For the SPS-3 experiment, the following findings were reported:

- Thin asphalt overlay was found to be the most effective maintenance treatment followed by chip seal and slurry seal treatments in terms of roughness, rut depth, and fatigue cracking.[60]

- Crack sealing had no significant effect on long-term roughness, rut depth, or fatigue cracking.(60)

- Crack sealing had only marginal impact on longitudinal and transverse cracking. This is mainly because sealed cracks are counted as separate distresses in the LTPP distress survey procedures.(61)

For the SPS-4 experiment, the findings were inconsistent. Some researchers reported the following findings:

- Sealed joints performed better than unsealed joints, while other researchers reported that there were no significant differences between sealed and unsealed joints.(61–63)

- Silicone joint sealant materials performed better than compression seals and hot pours in terms of the overall failure (adhesion loss and joint spalling).(28)

- The lack of a significant quantity of data is a drawback in the analyses. Survey measurements of sealed joints/cracks were collected for 34 test sites, while undersealed test section data were available for only 10 sites.(63)

For the SPS-5 experiment, the following findings were reported:

- Thick overlays performed better than thin overlays with respect to transverse and fatigue cracking.(63)

- Inconsistent results were reported for longitudinal cracking, rut depth, and IRI. Some researchers reported that there was no apparent effect of thick and thin overlays on rut depth or IRI, while others reported that thicker overlays provided better IRI.(63–65)

- Virgin and recycled HMA used in overlays were found to have no significant impact on transverse, longitudinal, or fatigue cracking, rut depth, or IRI.(60,64) On the other hand, it was reported that recycled HMA performed better than virgin HMA with respect to fatigue and transverse cracking in dry climates and/or low traffic roadways.(63) Further, virgin HMA performed better than recycled HMA with respect to rut depth.(63)

- The type of pavement surface preparation performed before overlay had no significant effect on rut depth or IRI.(60,64)

- Inconsistent results were reported regarding the effects of pavement surface preparation before overlay on long-term cracking performance. It was reported that intense and minimal pavement surface preparations made no significant difference in long-term cracking performance, whereas others stated that intensely prepared pavement sections performed better than minimally prepared sections with respect to fatigue and longitudinal cracking.(61,63)

For the SPS-6 experiment, the following findings were reported:

- The 8-inch (203-mm) AC overlay was the most effective rehabilitation option followed by the 4-inch (102-mm) AC overlay and concrete pavement restoration with and without diamond grinding.(60) However, on the contrary, it was reported that rehabilitation strategies without AC overlays were best to mitigate the crack initiation and propagation.(63)
- Pavement rut depth on composite pavement was independent of overlay thickness, pre-overlay repairs, and mixture type.(60)
- No significant difference in long-term cracking performance was detected among the following:(60)
    - Test sections subjected to minimal versus intensive pre-overlay preparation.
    - Test sections with and without sawed and sealed joints.
    - Test sections with 4-inch (102-mm) overlays, with sawed-and-sealed joints, and cracked and seated sections.
    - Test sections with 4- and 8-inch (102- and 203-mm) overlays.
- Fractured PCC test sections with an AC overlay performed better in roughness than those nonfractured test PCC sections subjected to the same AC overlay. Further, the nonfractured sections that were subjected to AC overlay performed better than nonfractured PCC sections that were subjected to diamond grinding and patching without AC overlay.(66)
- Pavement roughness was independent of whether the pavement sections were subjected to sawing and sealing before the AC overlay.(60)

Impacts of Design Variables on Pavement Performance

The findings of various studies regarding the impacts of various design factors on pavement performance are summarized in the next six subsections.

Climatic Variables

One study suggested that dowel bars should be used in JPCP to reduce joint faulting in WF climates.(8) In dryer climates, the joint spacing should be reduced to decrease transverse cracking potential due to high thermal gradients. This is because the precipitation has the following two effects on the pavement material temperatures:

- Precipitation cools or heats the pavement surface relative to the subsurface temperature, thereby reducing the difference in temperature with depth.

- Water generally requires much more energy to change temperature than air, binder, and aggregate materials. Therefore, a higher water content or more frequent saturation reduces the magnitude and rate of heating and cooling of the pavement layers.

In addition, IRI was found to be higher for similar pavements located in colder and wetter climates than those in other climates. Further, higher initial roughness led to higher rates of deterioration. The researchers stated that the results should be reviewed with caution because the PCC durability was not included in the analysis, which might have affected the results.(8)

One study indicated that faulting in undoweled JPCP test sections in DF regions was similar to those sections in DNF regions.(67) The mean faulting values were 0.126, 0.079, 0.063, and 0.039 inches (3.2, 2.0, 1.6, and 1.0 mm) in the WF, WNF, DF, and DNF regions, respectively. On the other hand, the doweled JPCP test sections showed no significant differences in joint faulting between the WF and the WNF regions. Doweled joint faulting occurred mostly in the DF regions, followed by the DNF and the WF regions. Test sections in the WNF regions showed the lowest faulting values.

On the other hand, an initial evaluation of SPS-2 test sections indicated that for doweled joints in rigid pavements, faulting was most prevalent in the DF region, followed by the DNF, and the WF regions.(68) In addition, the total longitudinal crack length was found to be longer in the DNF region, followed by the DF and the WF regions.

A strong relationship was reported between IRI and climatic conditions for flexible pavements.(69) Higher roughness was measured in pavement sections located in areas with higher precipitation, higher freezing index, and/or higher number of freeze-thaw cycles. The researchers also stated that adequate frost protection was an important factor for good pavement performance. In hot climates, roughness values were strongly related to the number of days with temperatures above 90 °F (32 °C). In addition, the roughness was lower for pavement sections in hot regions with higher precipitation than for those with less precipitation. The researchers related this finding to the cooling effect that precipitation may have on asphalt pavements, thereby reducing deformations resulting from high temperatures. On the other hand, rigid, jointed pavements were found to have higher roughness in climates with higher precipitation.

Roadbed Soils

One study indicated that better subgrade support (higher backcalculated modulus of subgrade reaction, *k*-value) resulted in fewer transverse cracks with deteriorated edges and in lower roughness (IRI) for JPCP, JRCP, and CRCP.(70)

Another study concluded that PCC pavements constructed over fine-grained roadbed soils had higher joint faulting than those constructed on coarse-grained roadbed soils.(8) This was likely due to increased soil erosion and reduced water permeability. Likewise, JPCP sections constructed on coarse-grained roadbed soils had lower IRI than those constructed on fine-grained roadbed soils. The researchers recommended that a thick layer of granular material be placed and compacted beneath the aggregate base course to improve drainage and reduce faulting, especially for undoweled pavements. The study also concluded that PCC slabs supported on strong foundations, such as stabilized bases or granular roadbed soils, often had a lower initial roughness.

A study based on the SPS-8 experiment data found that the most prevalent early pavement distress was longitudinal cracking outside the wheelpath.(71) Further, this distress was most commonly observed in sections located in the WF region and on an active roadbed soil (frost-susceptible or swelling soils due to freeze-thaw cycles). It was also observed that flexible and rigid pavements constructed on active roadbed soils had the highest mean initial roughness values and the highest rates of deterioration compared with pavements constructed on fine and coarse-grained roadbed soils. This study were in agreement with other studies that a good working platform such as stabilized base and granular subgrade contributed to a smoother pavement after construction.(8,70)

Joint Load Transfer

A common finding from a few studies was that the presence of dowel bars had a significant impact in reducing joint faulting.(8,67,70) In fact, after 10 years in service, JPCP sections with dowel bars showed 50 percent less joint faulting than those without dowel bars. In wet and/or freeze climates, the use of dowel bars appeared to negate the effects of cold temperatures and increased moisture that could often lead to erosion and pumping of fines. It was also found that the use of doweled joints could have more impact on pavement performance than design features such as subdrainage, tied-concrete shoulders, and joint spacing.(67) Further, it was found that properly sized dowel bars could eliminate corner breaks and transverse cracking near the joints as well as minimize joint faulting.(72)

In an FHWA report, the impacts of various parameters on the variability of the load transfer, as quantified by the load transfer efficiency (LTE) measured over time, were documented.(73) The researchers reported the following findings:

- For undoweled JPCP joints, the variability of LTE along a pavement section was inversely correlated to the average LTE. As the average LTE of a pavement section increased, the variability decreased.
- The LTE variability was not affected by joint spacing, base type, or shoulder type (PCC or AC).
- The LTE variability was higher in pavements with subsurface drainage systems than in pavements without subsurface drainage systems.
- The variability of the average LTE for pavements with granular roadbed soils was higher than that of pavements with silty clay roadbed soils.
- The variability of the average LTE was not affected by the amount of annual precipitation, the number of annual freeze-thaw cycles, or the average mean annual temperature.
- The variability of the average LTE decreased as the annual freezing index increased.
- No direct relationship was found between pavement age and the variability of the average LTE measurements over time.

- The variability of the average LTE was higher in pavements with tied concrete shoulders than in pavements with an asphalt shoulder.

Drainage

In a National Cooperative Highway Research Program *Research Results Digest*, it was stated that for properly designed doweled joints in JPCP, joint faulting was fairly low, and permeable bases had relatively small effect on reducing joint faulting.[(74)] Edge drains were found not to have a significant effect on joint faulting when dense-graded bases were used. For undoweled JPCP, joint faulting in general was higher, and permeable bases had a significant effect in reducing joint faulting. However, the permeable bases should be designed and maintained to reduce or eliminate the migration of fines from the lower materials. Similarly, slab cracking was found to be reduced in pavements constructed on asphalt-treated permeable bases. D-cracking was also found to be less prevalent in pavement sections constructed on permeable bases, likely because of reduced base saturation and introduction of water and various compounds into the concrete slab. Note that all of these findings were based on limited data.

Base Type

In separate studies sponsored by FHWA, it was reported that JPCP constructed over a stabilized base had less faulting and smoother surfaces than those constructed with an untreated aggregate base, especially in undoweled JPCP.[(8,75)] JPCP with an asphalt-stabilized base or lean concrete base had significantly lower initial roughness when compared with other base materials. In addition, JPCP sections constructed with granular and asphalt-stabilized bases had significantly lower percentages of cracked sections than JPCP with cement-treated or lean concrete bases. The cracking was not associated with an increased roughness.

In an LTPP Program sponsored study using the SPS-2 experiment data, it was found that pavement sections with permeable asphalt-treated bases developed the fewest transverse and longitudinal cracks.[(76)] On the other hand, pavement sections with lean concrete base developed the most transverse and longitudinal cracks during the first 10 years of pavement service life. This confirmed earlier findings.[(8,75)]

Slab Width

Two studies concluded that increasing slab width by 2 ft (0.6 m) reduced faulting of concrete pavements by reducing the critical deflections at the corner of the slab from heavy truck axles.[(8,67)] The mean faulting data for undoweled sections (10 years old or less) indicated about 50 percent less faulting with a widened slab. It was stated that this outcome agreed with previous non-LTPP field performance data. No difference was found between the faulting of doweled widened slab sections and doweled conventional-width JPCP. However, JPCP sections with widened lanes did not show any transverse cracking. In addition, the initial evaluation on SPS-2 data revealed that widened slabs have less initial roughness.[(76)]

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This chapter presented a review of the state of the practice of various State transportation departments with regard to several aspects of pavement condition and distress data analyses and treatment selection. A summary of the results of previous studies conducted using the LTPP database was also included. The following topics were covered:

- Pavement distress severity levels.
- Pavement condition and distress descriptions.
- Pavement performance modeling and treatment benefit calculations.
- Treatment types and selection.
- Preservation costs and LCCA.
- Effectiveness of pavement treatments at the project and network levels.
- Pavement treatment time selection.
- LTPP Program and its objectives.
- SPS and GPS test sections of the LTPP Program.
- A summary of previous findings, including the impacts of pavement treatments and various design factors on pavement performance.

The various topics were evaluated, and the research team reached the following conclusions and made the following recommendations regarding execution of the current study:

- The pavement cracking data were typically collected and stored based on three severity levels (low, medium, and high). For most cases, the data could not be analyzed because of the high data variability from one year to the next. Therefore, it was recommended that the sum of crack lengths or crack areas of all severity levels be used in this study for time-series modeling.
- Various methods exist to rate pavement conditions based on threshold values, indices, and descriptive terms. It was recommended to rate pavement based on its conditions and rates of deterioration over time to facilitate planning and pavement management. The dual condition rating systems described in chapter 3 address this issue.
- Pavement performance could be established and estimated over time through modeling the collected condition and distress data. The performance curves could be used to estimate future conditions and the time period to reach certain threshold values. This methodology was recommended for use in this study.
- The benefits of pavement treatments could be quantified in several ways based on changes in conditions, rates of deterioration, and a combination thereof. In this study, the treatment benefits were calculated based on the dual condition rating systems detailed (see chapter 3).

- Numerous pavement treatments could be selected to address the pavement conditions. The selections could be based on the conditions, the rates of deterioration, the causes of distress, or combinations thereof. It was recommended to select treatments that addressed most pavement conditions and their causes.

- Pavement treatment costs consist of agency and user costs. The costs of various pavement treatments and series of treatments (treatment strategy) could be evaluated using LCCA.

- Pavement treatment cost effectiveness could be evaluated at the project and network levels. It was recommended to perform LCCA at the project level and strategy optimization at the network level to improve the overall cost effectiveness of the pavement asset management.

- For pavement projects that received the same treatment type, T^2Ms could be developed to display the distribution of the pavement conditions before and after treatment and to estimate the treatment effectiveness and the selection of the optimum treatment time. In this study, T^2Ms were populated using the State data.

- The SPS and GPS experiments were designed such that pavement treatments were applied to pavement sections representing an assortment of different structures located in different environments and subjected to varying traffic levels. In this study, all treated LTPP test sections that had adequate time-series pavement condition and distress data were analyzed to evaluate the effectiveness and the impacts of the treatments on pavement performance.

- Numerous LTPP studies were evaluated that dealt with the different impacts of treatments on pavement conditions and distresses. Few studies considered pavement conditions and rates of deterioration over time. It was recommended to include the before and after treatment pavement conditions and rates of deterioration in studying the pavement performance.

- Previous LTPP studies focused on the effects of environment, roadbed soil, joint load transfer, drainage, base type, and slab width on pavement performance. Few conclusions regarding the effects of these factors were made. It was recommended to analyze the effects of the environmental regions on the effectiveness of the treatment using the before treatment and after treatment pavement condition and distress and the associated rates of deterioration.

CHAPTER 3. PAVEMENT CONDITION CLASSIFICATION

To address the objectives of this study, as described in chapter 1, a dual pavement rating system was developed that considered both the pavement condition and its rates of deterioration. The rationale for the development of the system is expressed in chapter 2 and further detailed in this chapter. An accurate pavement condition rating system represents pavement behavior best when based on the current and future pavement conditions. The main benefit of including the estimation of future conditions is the ability of pavement managers to plan, budget, and create long-term treatment strategies to preserve the pavement network. Pavement condition ratings based on the current conditions alone only allow decisions to be made for the given data collection cycle. As interpreted by AASHTO *Transportation Asset Management Guide—A Focus on Implementation*, the *Moving Ahead for Progress in the 21st Century Act of 2012* (MAP-21) sets forth the following directive for asset management:[77]

> Asset management is a strategic and systematic process of operating, maintaining, and improving physical assets, with a focus on engineering and economic analysis based upon quality information, to identify a structured sequence of maintenance, preservation, repair, rehabilitation, and replacement actions that will achieve and sustain a desired state of good repair over the lifecycle of the assets at minimum practicable cost. (from section 1103)

The two key statements are the following:

- **"Asset management is a strategic and systematic process of operating."** A strategic and systematic process implies the ability to model and estimate future conditions and times for corresponding actions on a regular basis.

- **"Identify a structured sequence of maintenance, preservation, repair, rehabilitation, and replacement actions that will achieve and sustain a desired state of good repair over the lifecycle of the assets."** The sequence of maintenance, repair, and rehabilitation implies current and future actions. Hence, accurate prediction of future conditions and timing of future actions are essential to the implementation of MAP-21.

The measured pavement distresses and conditions generally increase over time as the pavement deteriorates as a result of traffic loads and environmental conditions. Periodically over the pavement lifecycle, preservation and rehabilitation treatments are applied to reduce the pavement distresses and improve its conditions, as illustrated in figure 26. The costs of these treatments generally increase as the pavement deteriorates and the distresses or conditions worsen. Pavement condition ratings based on current distresses and conditions are most commonly found to be problematic because they do not include the pavement rates of deterioration. The current condition alone does not support LCCA. Two pavement sections in equally good condition this year may or may not be in similar good or fair condition 2 or 3 years later. Consequently, an accurate pavement rating system should include the current distresses and conditions as well as the pavement rates of deterioration.

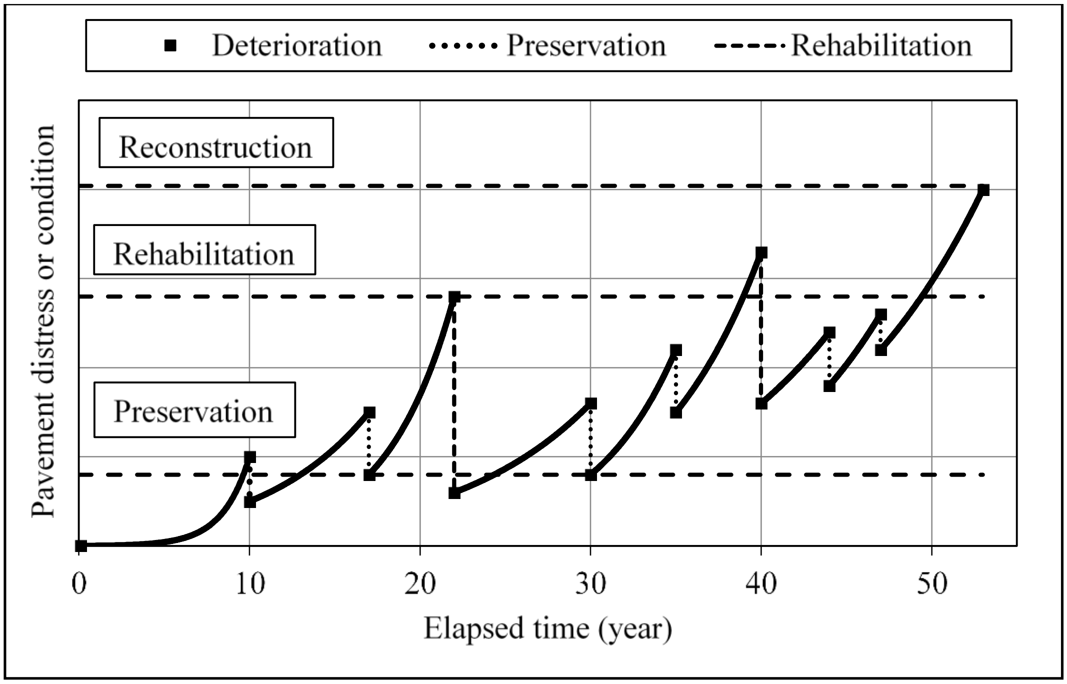
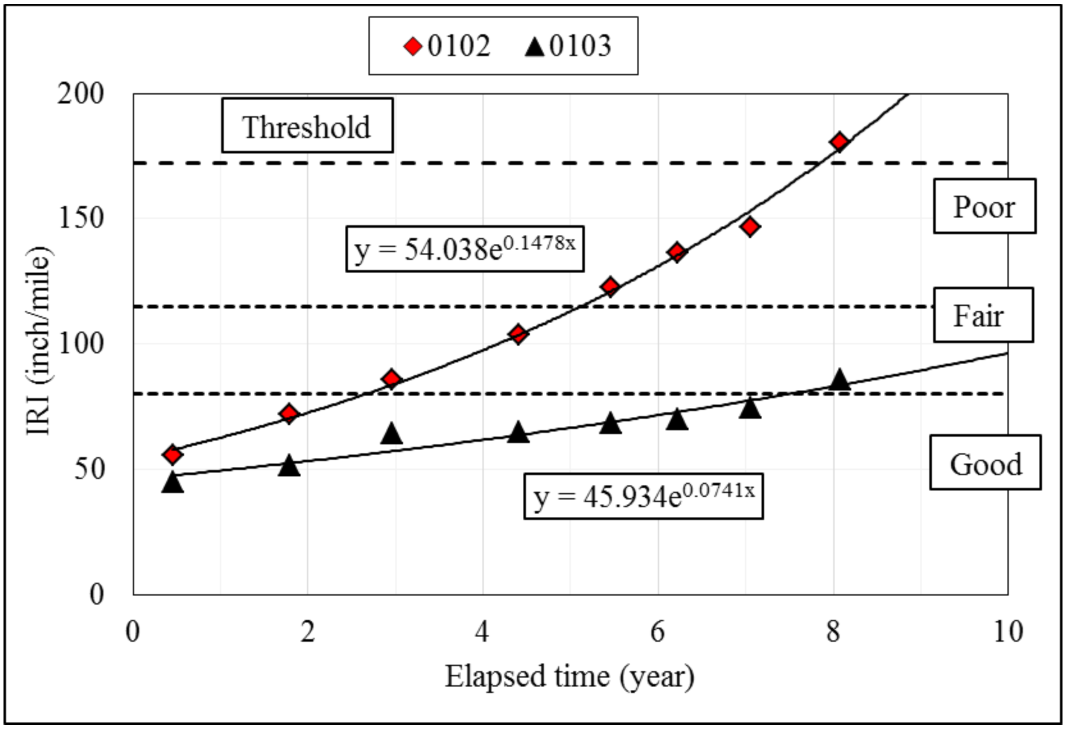


Figure 26. Graph. Typical pavement condition or distress over the pavement lifecycle.

Figure 27 illustrates the progression of roughness, as described by the IRI, of SPS-1 test sections 0102 and 0103 in Iowa. The figure also provides an example rating system based on the current condition (IRI). The data in this figure indicate the following:

- The initial IRIs for test sections 0102 and 0103 were, respectively, about 55 and 45 inches/mi (0.9 and 0.7 m/km) (good condition)).
- The IRIs in the third year were about 63 and 85 inches/mi (1.3 and 1.0 m/km) for test sections 0102 and 0103, indicating good and fair conditions, respectively.
- Approximately 2 years later, the IRI of test section 0102 was more than 120 inches/mi (2 m/km) (poor condition)) whereas the IRI for test section 0103 was only about 65 inches/mi (1 m/km) (good condition)).



1 inch/mi = 0.0158 m/km.

Figure 27. Graph. IRI versus elapsed time for LTTP SPS-1 test sections 0102 and 0103 in Iowa.

These observations indicate that test section 0102 changed from good to fair condition in about 1 year and from fair to poor in about 4 years, whereas 8 years after construction, test section 0103 was in the middle of the fair condition range. This example indicates that the latest measured IRI data of any test section should not be used alone to predict future condition and consequently to plan possible treatment actions. The time-series data must be used. The reason is that almost all pavement sections deteriorate over time, and their rates of deterioration vary substantially from one pavement section to the next. Hence, to effectively and comprehensively plan pavement preservation actions for a pavement network, the rate of deterioration with respect to each condition and distress type of each pavement section must be known. Therefore, an accurate pavement rating system should be based on each measured pavement condition and distress type and the corresponding rates of deterioration.

PAVEMENT CONDITION RATING SYSTEM

A balanced and comprehensive pavement condition rating system should be based on the two types of pavement conditions, functional and structural. In this study, the functional rating was based on ride quality (IRI) and safety (skid resistance and rut depth) and was expressed by the RFP. (A detailed definition of this measurement is provided later in this section.) For a given pavement section and when supported by the available data, three RFPs should be calculated; one based on IRI, one on rut depth, and one on skid resistance. The shortest RFP was assigned to the pavement section in question to flag that section for potential treatment actions.

The structural rating was based on cracking and rut depth or faulting and was expressed by RSP. (A detailed definition of this measurement is provided later in this section.) For any given pavement section, six RSPs should be calculated; one for each of transverse, longitudinal, alligator, edge, and block cracking, and one for either rut depth for flexible pavements or faulting for rigid pavements. The shortest of the six RSPs was assigned to the pavement section in question to flag it for potential treatment actions.

Based on this discussion, *RFP* is defined as the shortest time period in years from the time of the last data collection to the time when a functional condition or distress reaches its corresponding prespecified threshold value. Although in this study two RFP measures (IRI and rut depth) were used to define RFP, the pool of functional measures could be expanded by State transportation departments to include, as an example, skid resistance. Nevertheless, in this study, after calculating two RFPs (one for rut depth and one for IRI), the shortest RFP was assigned to the pavement section in question to flag that section. The other RFP was retained in the database. *RSP*, on the other hand, is defined as the shortest time period in years from the time of the last data collection to the time when a structural distress reaches its corresponding prespecified threshold value. In this study, for each test section, six RSPs were calculated, and the shortest RSP was assigned to the pavement section in question to flag it for potential action Once again, the pool of structural measures used in this study for calculating RSPs could be expanded by the State transportation departments. It is important to note that once a pavement section is flagged for potential action, all available functional and structural data should be downloaded and examined before a treatment strategy is selected. Also note that the recommended threshold values for each pavement condition and distress are presented in a later section in this chapter.

These definitions indicate that RFP and RSP are not combined condition indices. Each condition and distress type is analyzed separately, and the results are retained for further analyses. It is the minimum RFP or RSP assigned to the pavement section that flags that section for potential actions. In addition, RFP and RSP do not indicate the treatment to be applied to a pavement section. Rather, they flag mechanisms for identifying pavement sections that are in need of further attention. Once again, the stakeholder should review all available data for the flagged pavement sections and examine the distress, condition, and other related data. After this examination, the stakeholder can select treatment alternatives that address all or most defects and their causes. Ideally, each treatment alternative should then be subjected to LCCA and its impact on the entire pavement network should be determined before the treatment is selected.

The RFP and RSP concept differs from the condition indicators evaluated in chapter 2 of this report. RFP and RSP account for the pavement rates of deterioration. The RFP and RSP are calculated based on nonlinear mathematical functions that model the progression of the condition or distress over time. These equations are flexible and can be selected by the users or can be replaced with the equations of their choosing.

RFP and RSP is a dual rating system; RFP can be considered a pavement rating for the users, whereas RSP is an agency rating. The rating scale of the dual rating systems and the corresponding descriptive terms are listed in table 11. The scale is divided into three CSs numbered 1, 2, and 3 that correspond to poor (red), fair (yellow), and good (green) conditions, respectively, and to the three RFP and RSP ranges listed in table 11. Note that the main reason for using the same year ranges for RFP and RSP is ease of communication.

Table 11. Pavement condition rating based on three CSs.

CS			RFP Range (Years)	RSP Range (Years)
Code	Color	Descriptor		
1	Red	Poor	< 4	< 4
2	Yellow	Fair	4 to < 8	4 to < 8
3	Green	Good	≥ 8	≥ 8

Note: The dual rating systems could be used to select treatment categories at the network level. For example, preservation treatments should generally be applied to pavement sections with fair or better CSs. Heavy preservation treatment, or more likely rehabilitation, should generally be applied to pavement sections with poor RSP CSs. The treatment selection should be verified at the project level.

The dual rating system shown in table 11 was expanded to the five-level CS dual rating system presented in table 12. The main advantage of this five-level rating system is that the condition of the pavement sections in one CS or within a given RFP or RSP range is more uniform. It is recommended to use the three-level rating system for communication while the five-level rating system should be used for analyses and management. Note that this is possible because the poor and good ratings of the three-level CS system encompass the two additional CSs from the five-level CS system, while the fair CSs are equivalent.

Table 12. Pavement condition rating based on five CSs.

CS			RFP Range (Years)	RSP Range (Years)
Code	Color	Descriptor		
1a	Red	Very poor	< 2	< 2
1b	Pink	Poor	2 to < 4	2 to < 4
2	Yellow	Fair	4 to < 8	4 to < 8
3a	Light Green	Good	8 to < 13	8 to < 13
3b	Green	Very good	≥ 13	≥ 13

The main advantage of using RFP and RSP together is that each should decrease 1 year for every calendar year (i.e., RFP and RSP are linear functions of time although they are modeled as nonlinear functions of the pavement distress and conditions). To illustrate this point, consider the power function (see figure 28) that is typically used to model the rut depth data as a function of time.

$$Rut\ depth = RD = \alpha t^{\beta}; the\ time\ t = exp^{\left(\frac{1}{\beta}\right)ln\left(\frac{RD}{\alpha}\right)}$$

Figure 28. Equation. *Rut depth*.

The time at which the rut depth is equal to the prespecified threshold value (*Th*) can then be calculated as shown in figure 29:

$$t = exp^{\left(\frac{1}{\beta}\right)ln\left(\frac{Th}{\alpha}\right)} = e^{\left(\frac{1}{\beta}\right)ln\left(\frac{Th}{\alpha}\right)}$$

Figure 29. Equation. *t*.

The time *t* in figure 29 is constant and is equal to the time in years between the end of construction of the last treatment action and the time when the RD threshold value is reached. RSP is then calculated as the time *t* minus the surface age of the pavement section in question as stated in figure 30.

$$0 \leq RSP = (t - SA) = \left\{ e^{\frac{1}{\beta} ln\left(\frac{Th}{\alpha}\right)} - SA \right\} \leq (DSL - SA)$$

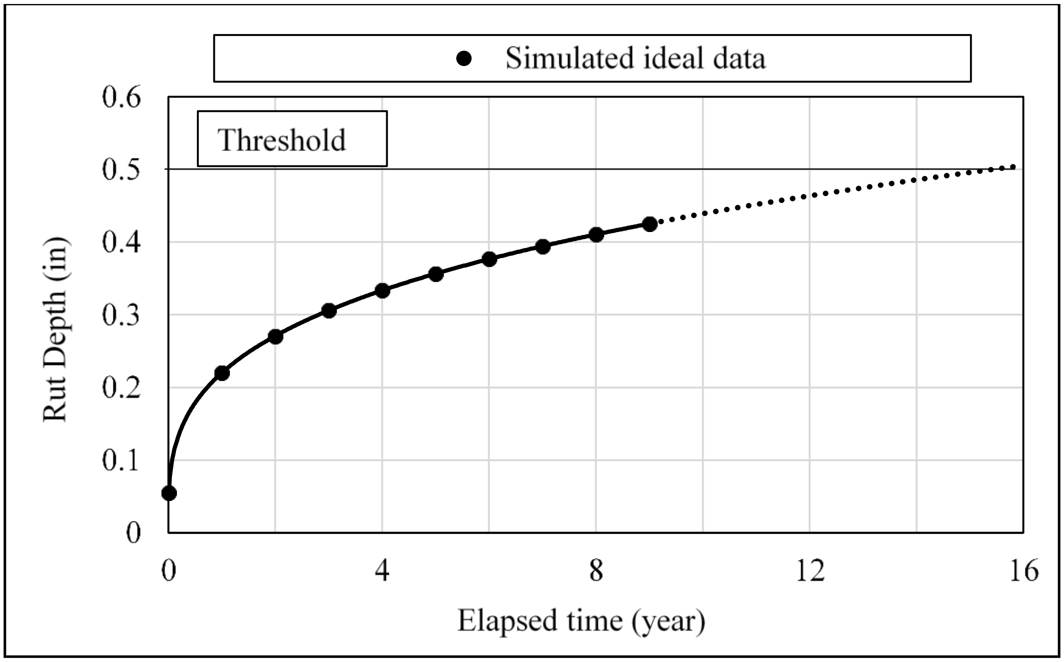
Figure 30. Equation. *RSP*.

Where:

RD = rut depth.
α and β = Statistical parameters of the nonlinear function of figure 28.
Th = Threshold value for rut depth (typical value is 0.5 inch (12.5 mm)).
SA= Surface age (years).
DSL = Design service life of the last treatment (years).
ln = natural logarithm.
e = Exponential function.

Finally, RSP should be positive and is limited to the DSL minus the SA of the pavement section in question. Such limitation is required until at least three time-dependent RD data are measured and available in the database.

Because the time *t* in figure 30 is constant, RSP decreases by 1 year as the surface age of the pavement section increases by 1 year. To illustrate, consider the idealized rut depth power function of figure 28 and the corresponding idealized data shown in figure 31. The solid circles and curve in figure 31 simulate the idealized measured data whereas the dotted curve simulates the predicted rut depth data. At time 0 (end of construction), no rut depth data were available, and RSP of the pavement section was equal to the DSL of 15 years minus the pavement SA of 0 years. Similarly, 1 and 2 years after construction, RSP was equal to the DSL minus the pavement SA. In the future, when the third measured rut depth data point becomes available, the data could be modeled using a power function, and the time at which the rut depth reaches the prespecified threshold value of 0.5 inch (12.5 mm) can be estimated using figure 29. RSP at that time was equal to the calculated time to threshold minus the pavement surface age. This procedure was repeated when a new data point became available, and a new RSP was calculated as displayed in table 13. RSPs and SAs listed in table 13 were plotted in figure 32. It can be seen that RSP decreased by 1 year as SA increased by 1 year. Similarly, the IRI data measured as a function of time along LTPP test section 0102 in Iowa and depicted in figure 27 (repeated herein for easy reference as figure 33), were modeled with an exponential function, and six RFPs were calculated based on sets of three, four, five, six, seven, and eight time-series data points. The results are presented in table 14 and depicted in figure 34. Once again, the actual measured IRI data when modeled using an exponential function yielded RSPs that decreased by 1 year as the pavement SA increased by 1 year. Once again, similar results were obtained from the analyses of other LTPP test sections using various cracking data. They are included in chapters 6 and 7.



1 inch = 25.4 mm.

Figure 31. Graph. Example of idealized rut depth data and function versus elapsed time.

Table 13. Progressive calculation of RSP of the idealized rut depth shown in figure 31.

Number of Available Data Points	Surface Age (Years)	Values from Figure 28		RSP (Years)	Calculation Equation
		α	β		
1	0.01	N/A	N/A	14.99	See figure 30
2	1	N/A	N/A	14	
3	2	N/A	N/A	13	
3	2	0.2217	0.3	13	See figure 30
4	3	0.2217	0.3	12	
5	4	0.2217	0.3	11	
6	5	0.2217	0.3	10	
7	6	0.2217	0.3	9	
8	7	0.2217	0.3	8	
9	8	0.2217	0.3	7	
10	9	0.2217	0.3	6	

N/A = Not applicable.

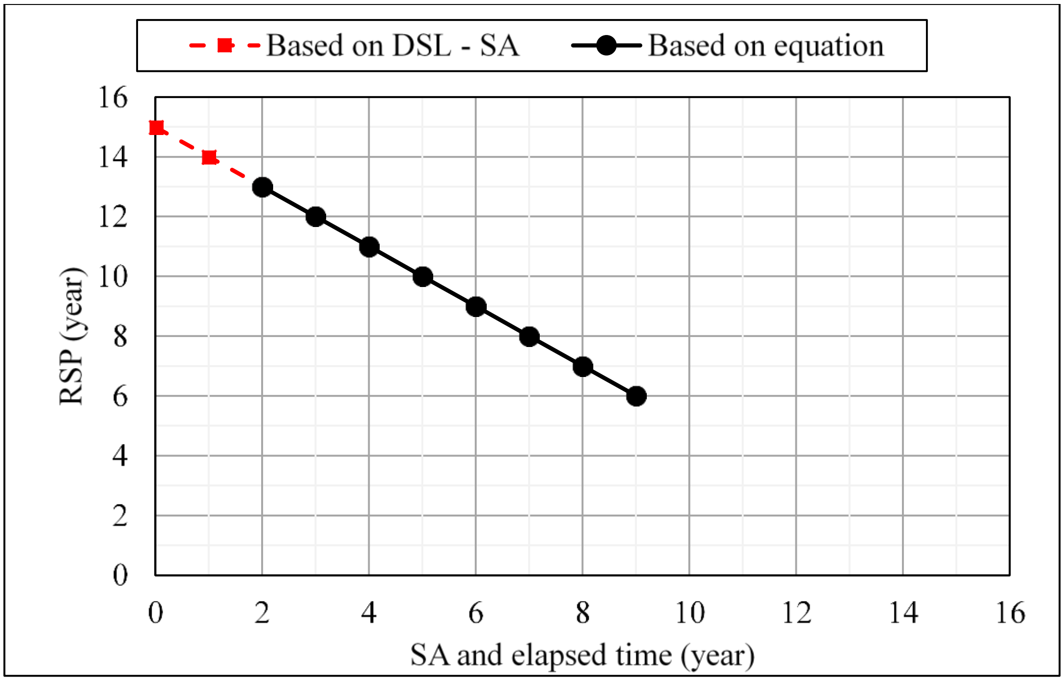
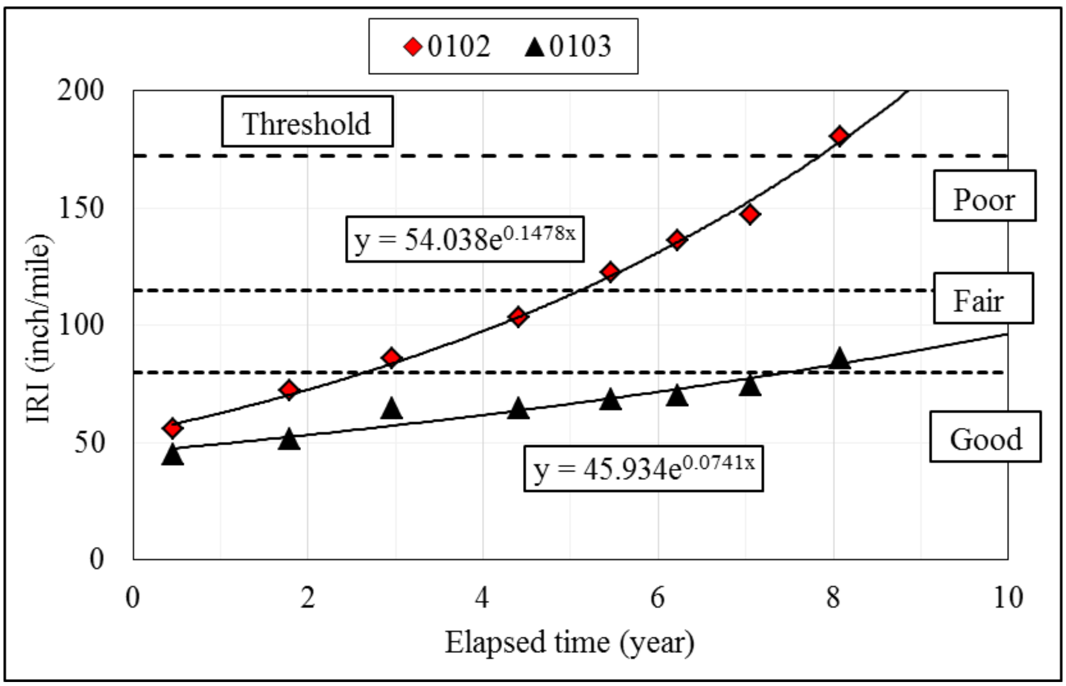


Figure 32. Graph. RSP versus the pavement surface age for an idealized power function.



1 inch/mi = 0.0158 m/km.

Figure 33. Graph. IRI versus elapsed time for LTPP SPS-1 test sections 0102 and 0103 in Iowa.

Table 14. RFP of LTTP test section 0102 in Iowa based on the IRI data shown and depicted in figure 34.

Number of Data Points	Pavement Surface Age (Years)	IRI (Inches/Mi)	IRI See Figure 28 α	IRI See Figure 28 β	RFP (Years) (See Figure 30)
3	2.88	85.75	52.771	0.1721	3.99
4	4.32	103.62	53.864	0.1559	3.13
5	5.37	122.53	54.067	0.1535	2.17
6	6.13	136.45	54.251	0.1518	1.47
7	6.98	146.89	54.933	0.1464	0.82
8	8	180.64	54.721	0.1478	–0.25

1 inch/mi = 0.0158 m/km.

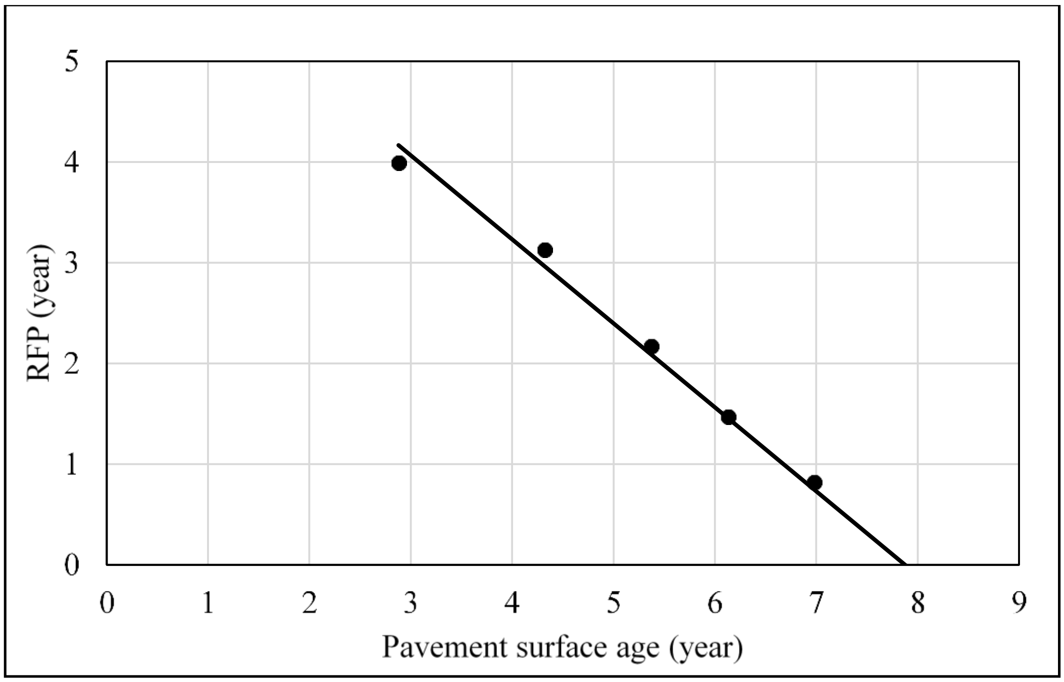
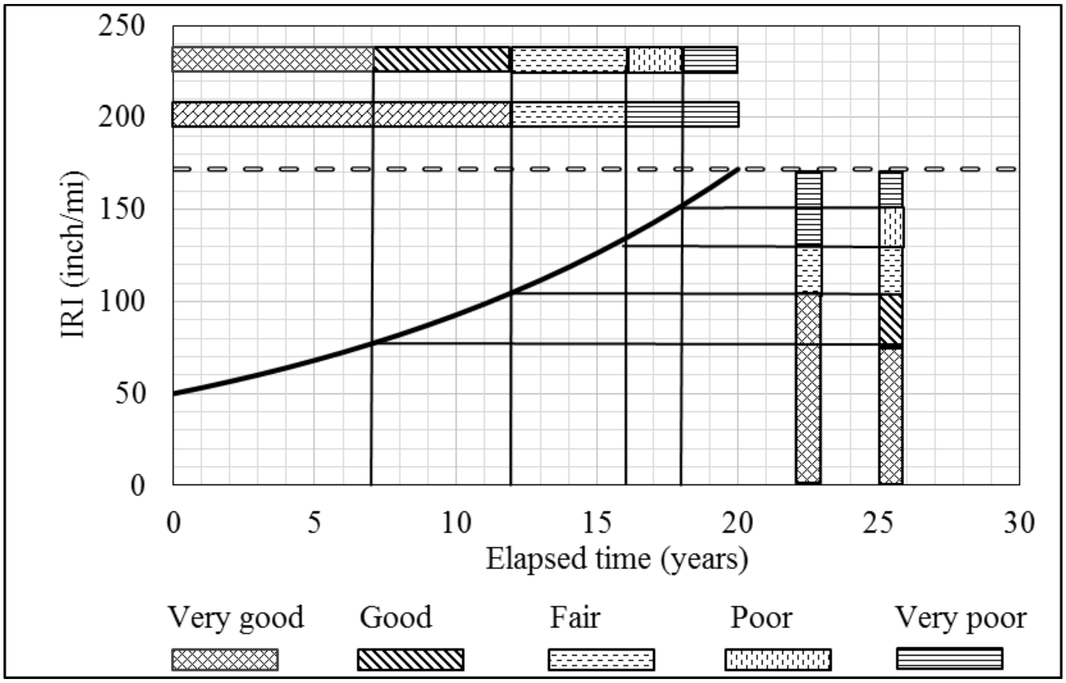


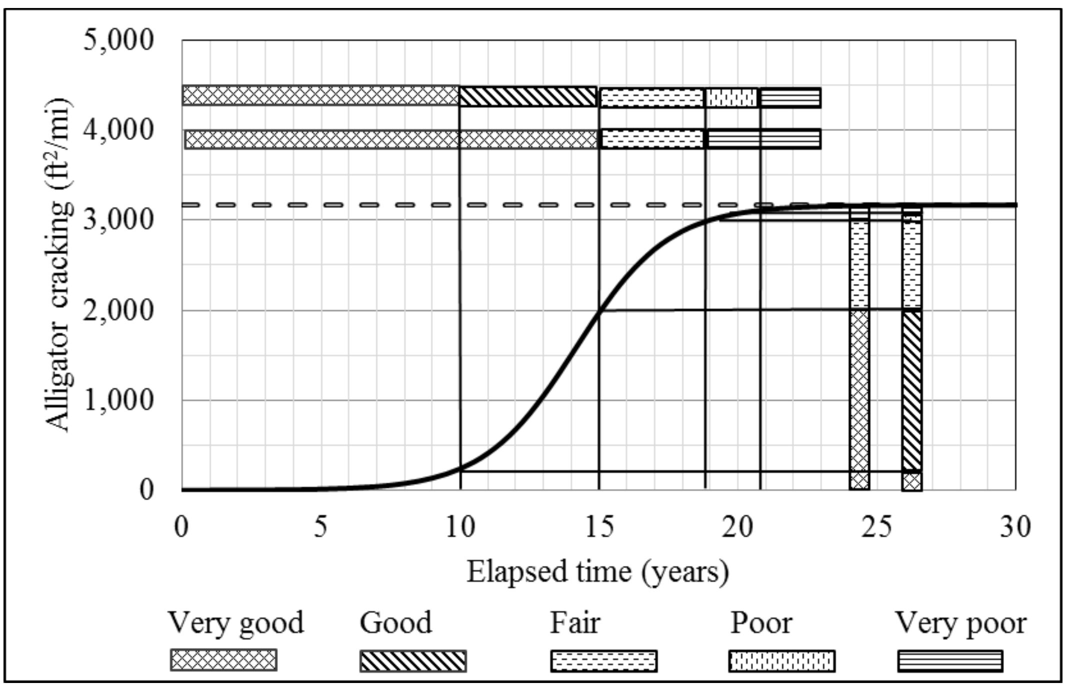
Figure 34. Graph. RFP versus pavement surface age for LTPP test section 0102 in Iowa.

The three- and five-level CS systems in terms of RFP and RSP are depicted for an idealized and untreated pavement section shown in figure 35 and figure 36. The threshold values in the figures are 172 inches/mi (2.73 m/km) and 3,168 ft^2 of alligator cracking per 0.1 mi (180 m^2 of alligator cracking per 0.1 km), respectively.



1 inch/mi = 0.0158 m/km.

Figure 35. Graph. RFP CSs.



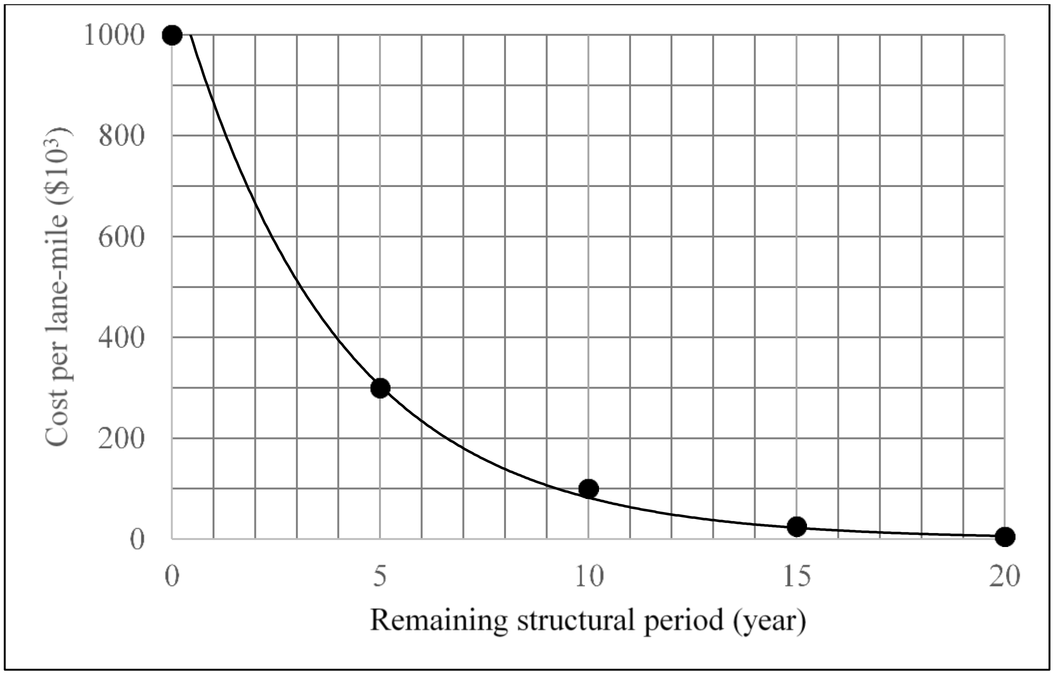
1 ft^2/mi = 0.0581 m^2/km

Figure 36. Graph. RSP CSs.

In addition, RFP and/or RSP can be used for the following purposes:

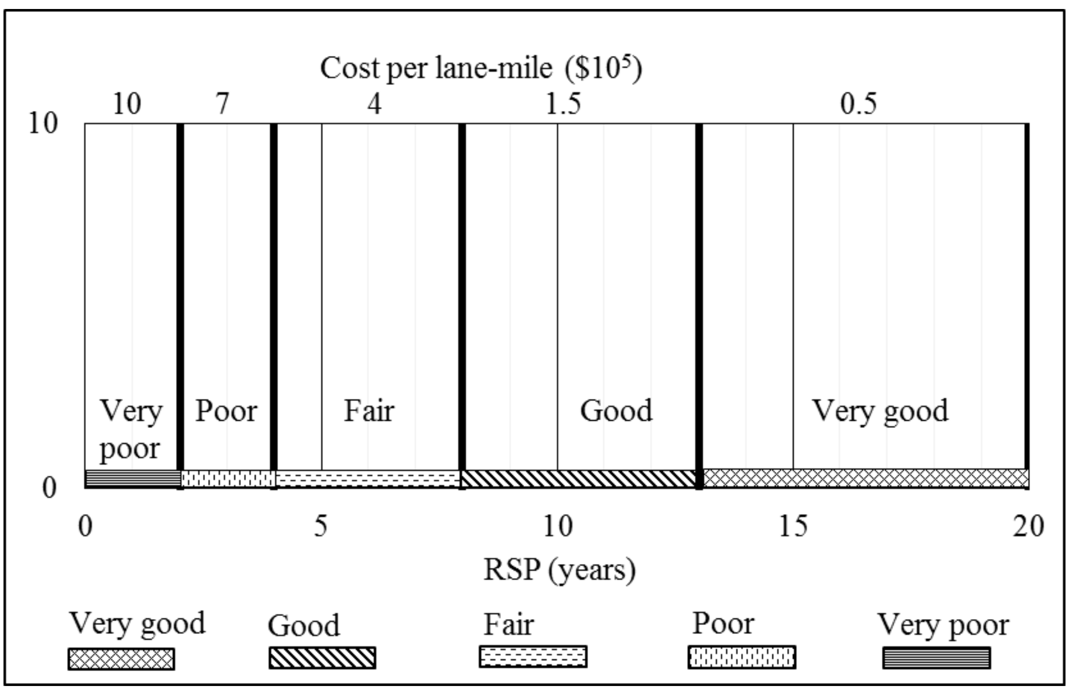
- Expressing the rating of a pavement section or a pavement network (see table 11 and table 12).

- Estimating the cost of pavement preservation, as shown in figure 37.
- Understanding the consequences of delayed actions, as shown in figure 38.



1 lane-mile = 1.61 lane-km.

Figure 37. Graph. Conceptualized cost of pavement preservation versus RSP.



1 lane-mi = 1.61 lane-km.

Figure 38. Graph. Correlations among RSP, cost of preservation, and descriptive pavement classification.

Figure 37 and figure 38 show that as RSP decreases, the pavement condition worsens and the cost of pavement preservation increases. For example, the average cost per lane-mile of maintaining the pavement when RSP was 13 or more years was about $50,000 per lane mi ($31,250 per lane km). If the pavement deteriorated to fair condition (RSP between 4 and 8 years), the average cost of pavement preservation would increase to $400,000 per lane-mi ($250,000 per lane km) (or eight times more). Such average cost data could be obtained from past pavement project records and shared with legislators and the general public. When the relationship between costs and the RSPs or the pavement classification descriptive terms is established, the data could be used to arrive at the optimum pavement preservation strategy at the network level and for LCCA at the project level.

The RSP concept and the benefits of preservation treatments can be demonstrated by considering a flexible pavement section that was designed and constructed to last 15 years (DSL = 15 years). After construction, the section was in very good condition. (RSP was estimated at 15 years.)

Over time, the section deteriorated (started showing some cracks), and the pavement condition dropped from very good to good in 6 years and to fair in 10 years, as shown in figure 39 and figure 40. When the pavement condition reached fair status (RSP = 5 years), a thin (less than 2.5-inch (63.5-mm) HMA overlay was applied, and the pavement surface condition was restored to very good. Over the next 7-year period, the pavement surface condition deteriorated again from very good to good and then to fair. At 17 years after the original construction, another thin overlay was applied, and the surface condition was restored to very good status. Five years after the second overlay, the pavement surface condition dropped from very good to good. At that time (22 years after construction), the section was subjected to thin mill-and-fill treatment, and the pavement surface condition was restored once again to very good condition. To summarize, the first HMA overlay was applied 10 years after the original construction (when the condition of the pavement surface reached the fair CS). A second overlay was applied when the surface of the first overlay reached the fair CS. Finally, a thin mill-and-fill treatment was applied when the second overlay treatment was still in good condition. As illustrated in figure 39 and figure 40, the HMA overlays provided a better surface condition initially and decreased the rate of deterioration of the lower layers (the original HMA layers). Indeed, the original asphalt layer was still in fair condition 22 years after construction.

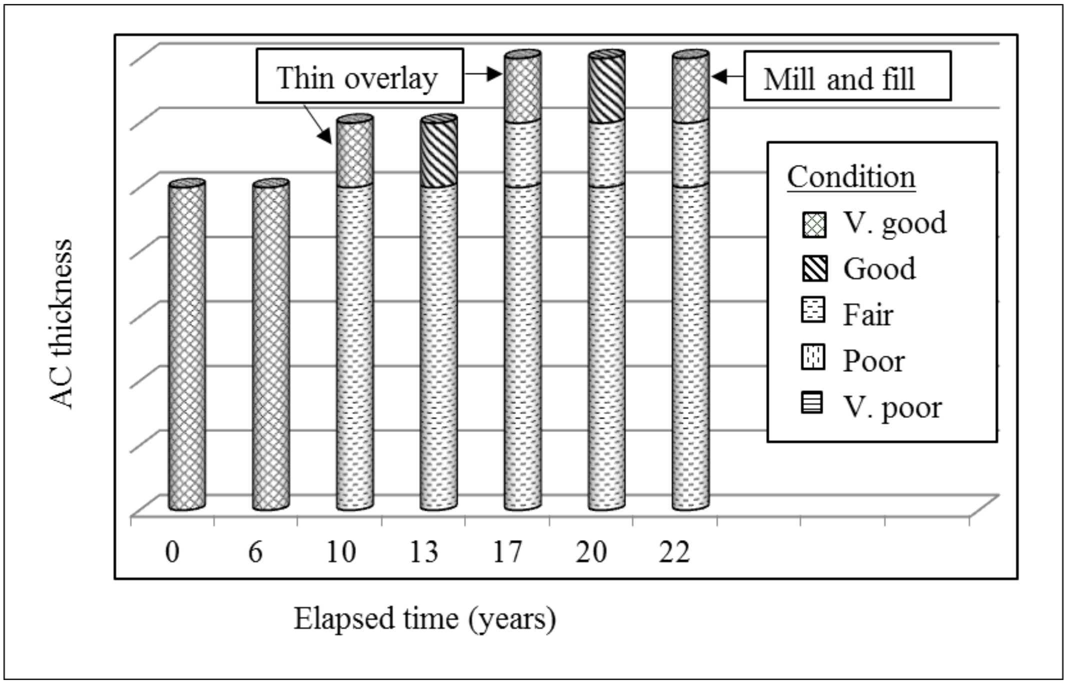


Figure 39. Graph. Example of pavement condition over time with two thin HMA overlays and one thin mill-and-fill action.

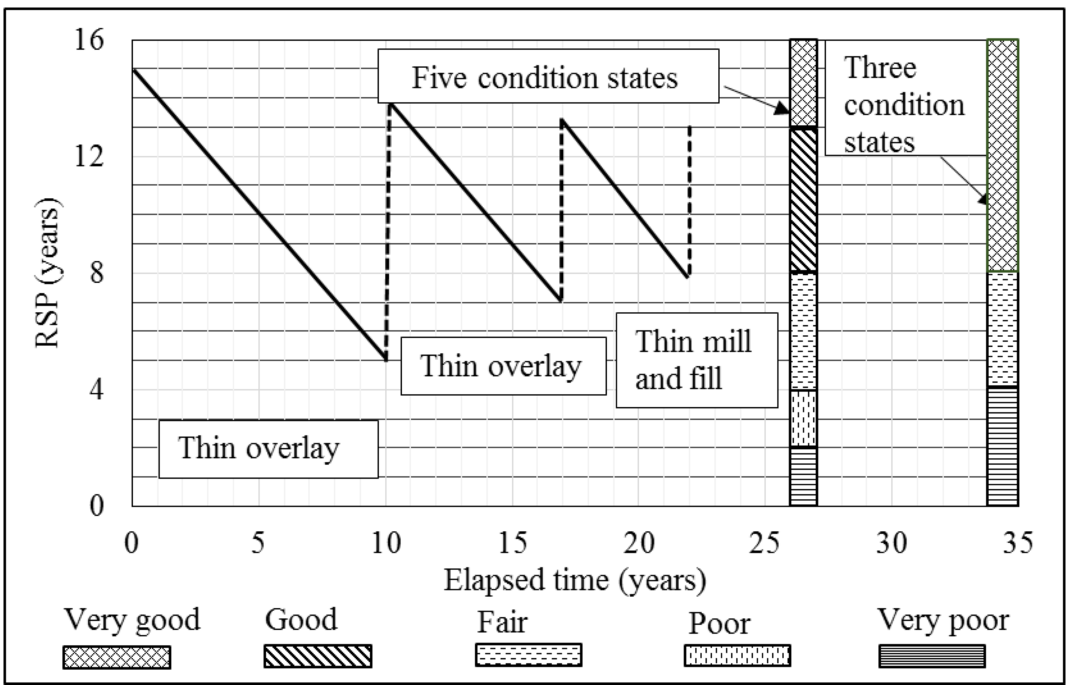


Figure 40. Graph. Example RSP over time with two thin HMA overlays and one mill-and-fill action.

It should be noted that the timing for the first and second overlays or any other treatment type should be selected after LCCA is conducted. Any pavement section can be treated at any time during its service life. Some sections may be treated when RSP is 15 years, while others may be treated when RSP is 8 years, and still others when RSP is at 3 years. Once again, the time and the type of the treatment should be selected based on the results of LCCA. In this regard, the following data are required for LCCA:

- For each applicable treatment type, the costs of the treatment when the pavement section is in each of the five CSs listed in table 12.
- The expected TL (the time in years until the pavement condition after treatment reaches the same status as that before treatment).

The preferred treatment type(s) and time of treatment are those that yield the minimum cost and maximum benefits.

RECOMMENDED THRESHOLD VALUES

Threshold values are defined herein as the magnitude of a measurable pavement condition or distress that constitutes the minimum level of pavement functionality acceptable to the agency and users or the minimum acceptable level of structural integrity. All threshold values should be established based on suitable engineering criteria of appropriate pavement performance measures. For example, functionality thresholds should be established based on ride quality and safety (such as IRI, skid resistance, and rut depth), whereas, structural thresholds should be established based on each cracking type, rutting or faulting. The units of measurement for the threshold values should be the same as those used in measuring the corresponding pavement condition and distresses. The engineering criteria for the threshold should include the impacts of their values on pavement condition and distress, the lifecycle cost, and the optimum timing for pavement preservation. The recommended threshold values for the calculation of RFP and RSP are presented in table 15 and table 16. The reasons for the selection of these values are shown in each table. Note that the recommended threshold values are flexible and can be adjusted based on the agency and user needs and constraints and on the posted speed limit or road class.

Table 15. Threshold values describing RFP.

Pavement Condition Type	Threshold Values Used in the Analyses		AASHTO MEPDG[37]
	Threshold Value	Explanation	
IRI	172 inches/mi (2.7 m/km)	Minimum acceptable ride quality at 55 mi/h (90 km/h); driver speed and comfort may be reduced above this value	Interstate: 160 inches/mi (2.5 m/km) Primary: 200 inches/mi (3.16 m/km) Secondary: 200 inches/mi (3.16 m/km)
Skid resistance	To be determined by the agency	Depends on the method of measurement and pavement type	Not included
Rut depth	0.5 inches (12.7 mm)	Maximum allowable depth to control hydroplaning potential in wet conditions at 55 mi/h (90 km/h)	Interstate: 0.40 inches (10.6 mm) Primary: 0.50 inches (12.7 mm) Others (< 45 mi/h (72 km/h)): 0.65 inches (16.5 mm)

Table 16. Threshold values describing RSP.

Pavement Condition or Distress Type	Threshold Values Used in the Analyses: Value	Threshold Values Used in the Analyses: Explanation	AASHTO MEPDG[37]
Alligator cracking	1,267 ft^2/0.1 mi (73 m^2/0.1 km)	Twenty percent of the lane area cracked (assuming 12-ft (3.66-m) lane width)	Interstate: 10-percent lane area Primary: 20-percent lane area Secondary: 35-percent lane area
Longitudinal cracking	1,056 ft/0.1 mi (200 m/0.1 km)	Two cracks along the entire section length	Not included
Transverse cracking (JPCP)	264 ft/0.1 mi (50 m/0.1 km)	Two thirds of the slabs are cracked (assuming 16-ft (4.88-m)-long slab	Interstate: 10 percent Primary: 15 percent Secondary: 20 percent
Transverse cracking (HMA)	350 ft/0.1 mi (67 m/0.1 km)	Lane is divided into 12-ft (3.66-m) squares, assuming 12-ft (3.66-m) lane width and even crack spacing of 12 ft (3.66 m)	Interstate: 500 ft/mi (94.7 m/km) Primary: 700 ft/mi (132.6 m/km) Secondary: 700 ft/mi (132.6 m/km)
Faulting	0.25 inches (6.35 mm), average over 0.1 mi (100 m)	Dowel bars have likely sheared, or concrete around dowels has deteriorated and may be spalled	Interstate: 0.15 inches (3.8 mm) Primary: 0.20 inches (5.8 mm) Secondary: 0.25 inches (6.4 mm)
Deflection	See chapter 7	Certain deflection or differential deflection could indicate structural or load transfer issues or void beneath the slab	Not included

The threshold values listed in table 15 are representative of the average minimum level of serviceability that the State transportation departments strive to provide their users. Therefore, these values are somewhat subjective; where one State transportation department may strive to provide pavement with no more than 150 inches/mi (2.4 m/km) IRI, another may set the maximum acceptable pavement roughness at 225 inches/mi (3.6 m/km) IRI for its pavements. Neither value is wrong, right, or otherwise; the value should be determined by the State transportation department by considering the users' needs and expectations as well as the practicality and the lifecycle costs associated with maintaining its pavement network given the agency constraints. Safety-related threshold values, such as rut depth and skid resistance, should also be determined by the State transportation department based on an assessment of the typical driving conditions, speeds, and vehicle characteristics and their role in a risk assessment analysis. For example, the potential for hydroplaning increases in wet climates, on roads with minimal cross-slope for surface drainage, and in areas with higher speed limits. The maximum allowable rut depth should be determined to provide reasonably safe travel for most roadway travelers in an economically feasible manner.

Further, the threshold values do not imply that the State transportation department must or should wait to take action until the pavement sections reach the threshold values, nor do they imply that a roadway must be closed to traffic if the threshold is surpassed. The threshold value is a management tool that helps planners and managers evaluate, assess, and make reasonable and potentially cost-effective decisions regarding the conditions and serviceability of the pavement network.

On the other hand, the engineering criteria for most of the structural threshold values listed in table 16 are much more difficult to establish for various reasons, including the following:

- The lack of sufficient long-term pavement performance databases that can be used to analyze the impact of the threshold levels on the lifecycle cost and the health of the pavement network. The most critical information that is insufficient to support the analyses is cost data.

- The total yearly cost of preserving the pavement network.

- The constraints of the road authorities regarding budget level, political pressure, increased demand, and increasing cost over time.

- The relationship between the engineering criteria (the threshold values) and the road class. A typical highway authority manages several classes of roads that have various traffic demands. If different threshold values are established for different road classes, communicating the values becomes problematic.

Nevertheless, the pavement community generally agrees that pavement preservation and maintenance actions applied over time are more cost effective than allowing pavements to deteriorate until reconstruction is required. Reconstruction is rarely required because pavements can be preserved indefinitely. The problem lies in where the tipping points occur in the economics of preservation, maintenance, and rehabilitation. Typical pavement structures are subjected to reconstruction after numerous cycles of pavement preservation, maintenance, and rehabilitation. The number of these cycles and the corresponding lifecycle costs are functions of the pavement preservation strategy used and the timing of the various pavement preservation cycles.

The establishment of the RSP threshold values should be based on the assumption that RSP would flag the pavement sections for preservation actions at the proper time. For example, consider a flexible pavement section that has begun to develop block cracking (top-down type cracks) owing to surface aging. Several surface treatments, such as mill and fill or HMA overlay, could reduce or eliminate the block cracking and its rate of propagation. If the threshold value for block cracking is set very high or if RSP is allowed to decrease to zero and beyond the threshold value, the block cracks extend in depth and the cost effectiveness of these treatments generally decreases until the conditions (the tipping point) for reconstruction are reached. This tipping point could be when the cracks pass the mid-depth of the asphalt layer or when they penetrate the entire asphalt layer. The specific condition(s) under which reconstruction becomes most cost effective depends on many factors, including the following:

- **User costs**: User costs can be summarized as the travel costs associated with driver delay and the vehicle operating costs VOCs (fuel and vehicle wear). The magnitude of the user costs for a given pavement project has many factors, including the type and length of traffic control and detouring, the conditions of the roadway, and the traffic volume.

- **Availability of funds:** State transportation department funds are limited, and some pavement projects have higher priority than others for various reasons. Hence, pavement treatments may be applied sooner or later than optimum, and the cost effectiveness of the treatment may be affected. For example, a given pavement section may have reached the tipping point, without budget constraints, the proper fix can be applied. With budget constraints (short funding) a less-expensive stopgap treatment may be applied.

- **Ancillary work required:** Federal and State regulation and policy often require standardization of ancillary transportation items when a pavement project is undertaken. The requirements can be contingent on the type of work being performed. For example, a roadway reconstruction requires update of vertical and horizontal curves, bridge clearance, guard rails etc., while an HMA overlay may not require any ancillary updates.

- **Pavement location (urban or rural):** The location of the pavement segment affects the costs of equipment mobilization and worker travel. Likewise, the amount of traffic and the number of access points also affects the costs. Highly trafficked roadways may necessitate detour routes that require improvements to handle the increased traffic, while the cost of traffic control can also be affected by traffic volumes and the number of driveways and entrance/exit ramps present.

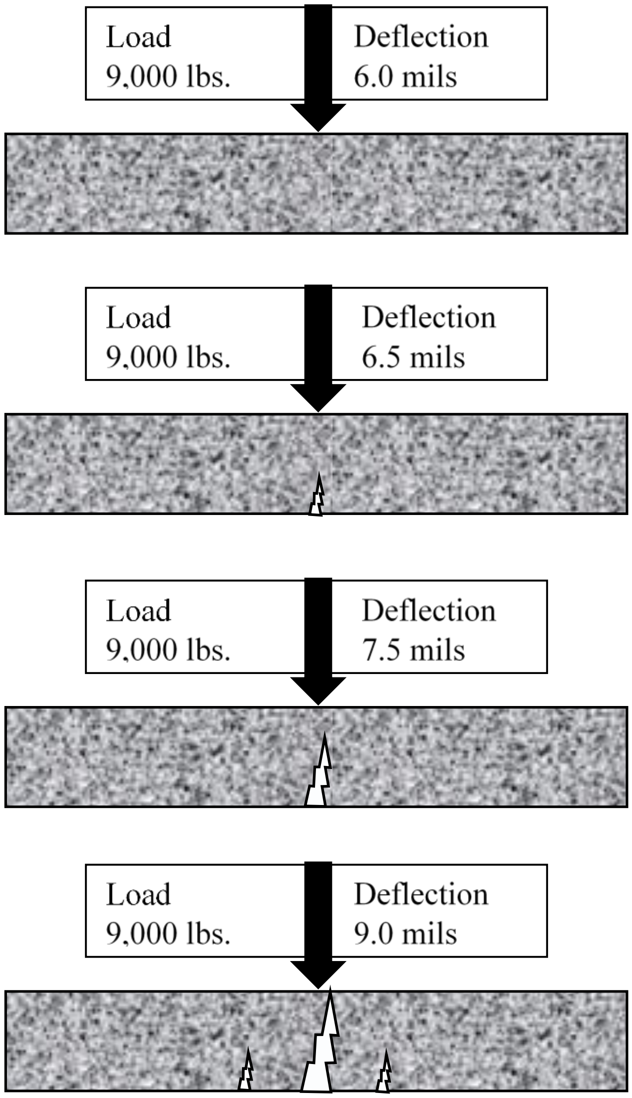
- **Pavement treatment benefits:** The benefit of a given treatment depends on many factors, such as the before treatment conditions and rates of deterioration, construction and material quality, and the anticipated traffic and environmental loading. The inherent variability in the materials and construction quality often yield differing treatment benefits within the boundaries of a given pavement project.

The concept of structural integrity thresholds is even further complicated by the false notion that pavements that have reached the threshold value must be reconstructed. Some pavement sections reach the threshold for structural integrity yet may still provide acceptable level of service. For example, for a concrete pavement that has 100 percent of slabs with two or more transverse cracks, it would likely be more cost effective to reconstruct than to perform full-depth patching at each transverse crack. However, if the cracks are not faulted, the roadway may still have an acceptable IRI. In this scenario, the pavement has an RFP greater than 0 years while its RSP is 0 years. The preferred alternative in this scenario would be to schedule pavement reconstruction for when RFP reaches 0 years. In other words, an RFP of 0 years implies that action is needed but an RSP of 0 years does not necessarily imply that reconstruction is needed immediately. However, the latter is a function of the threshold value and the type of distress. For example, a pavement rut of typical depth can be removed using certain treatments. However, if the rut is due to shear failure in the lower pavement layers, reconstruction may be required to eliminate the causes of rutting. Likewise, if the threshold value is set too high (for example, 100-percent alligator cracking), reconstruction may be required.

The concept of long-life pavement can also add an extra nuance to the RSP concept. The idea behind long-life pavement is to construct a significant pavement structure that will resist structural deterioration due to traffic and environmental loading, throughout the pavement cross section. Pavement deterioration in long-life pavement would be limited to areas near the pavement surface (upper few inches), which can be perpetually replaced (i.e., with mill-and-fill treatment). In this scenario, RSP is virtually constant because any structural deterioration is periodically repaired.

One last important note on RFPs and RSPs is the concept of negative RFP or RSP. A value of zero implies that either the pavement is providing less than the standard level of service or the pavement structure has deteriorated to the point (depending on the threshold value) where major rehabilitation or reconstruction may be the most cost-effective treatment option. A negative value of RFP or RSP indicates the number of overdue years passed after the due date for major rehabilitation or reconstruction. This information is not of particular use to pavement managers, because a pavement section with 0 or –5 years RFP or RSP yields the same conclusion—reconstruction or heavy rehabilitation might be needed. For this reason, RFP or RSP could be limited to 0. Note that there is no technical upper limit on RFPs or RSPs. However, the maximum value should be reasonably set based on the average DSL of the pavement structure.

Finally, the previous discussions are primarily based on the use of data observed or measured to characterize the pavement surface. Unfortunately, the damage has already occurred by the time the distresses appear on the pavement surface. An early indicator of impending surface distress would support early actions and the selection of cost-effective pavement treatments. Such an early indication could come from the pavement deflection data measured using an FWD. The measured deflection and the rates of change over time could indicate the beginning of pavement deterioration before surface manifestation. Hence, it may be possible to establish deflection thresholds and incorporate FWD data into the RSP concept, as discussed in chapter 8. It is envisioned that a relationship may exist between deflection and its rate of change and the initiation and growth of pavement surface defects. For example, consider the increasing deflection likely to be measured on a pavement segment experiencing the initiation and propagation of alligator cracking, as shown in figure 41. The envisioned data in the figure indicate that the pavement deflection increases as the cracks initiate and propagate upward.



1 lb = 0.454 kg.
1 mil = 25.4 microns.

Figure 41. Illustration. Envisioned change in deflection through the progression of alligator cracking.

Unfortunately, such a relationship between crack initiation and increases in the measured pavement deflection has not received significant attention. The main reason is that the crack initiation time is not known; most cracking data start when the cracks appear on the pavement surface. Further, no deflection threshold value has been established that indicates the initiation of cracks. The difficulty in developing a deflection threshold is that each pavement structure is designed to have a different deflection under the same load. For example, a section of interstate freeway may be designed to deflect about 4 to 8 mil (101.6 to 203.2 microns) under 9,000 lb (4,086 kg) (half of an 18-kips single-axle load) while a suburban arterial may be designed to deflect 20 to 32 mil (508 to 813 microns) under the same load. Further, the frequency of pavement loading has an enormous impact on the initiation of alligator cracking and deflection.

Because no applicable relationships between deflection and pavement surface distress have been developed or published, in this study, the LTPP deflection data were analyzed to explore the potential for developing such relationship. The measured pavement deflections of several LTPP flexible and rigid pavement test sections were analyzed in an attempt to establish threshold values based on the pavement structural integrity. Results of the analyses are discussed in chapter 8 of this report. It should be noted, however, that most, if not all, State transportation departments collect FWD data at the project level as needed. The data are not collected on a regular basis at the network level.

FLEXIBILITY OF THE PAVEMENT RATING SYSTEMS

The dual pavement condition rating systems are designed to be adaptable to the needs and constraints of the users. The dual systems are based on three types of information: (1) time-series pavement condition and distress data, (2) threshold values, and (3) applications of the results, which can be adjusted by any State transportation department to work for almost any dataset and for many different tasks. The three information types are the following:

- **Data**: The data of the dual condition rating systems are the pavement condition and distress types included in the development of the rating. Recall that the dual rating is based on both functionality and structural integrity. The user may decide how to describe the pavement function and structural integrity. For example, this report uses the IRI to describe ride quality, as is used by the LTPP Program and most State transportation departments. However, an agency may choose to use another measurement or index, such as the ride quality index in place of IRI. The rating process would be essentially the same with different data.

  Likewise, the pavement conditions and distresses used to comprise the functional and structural integrity rating could have a wide range from user to user. For example, this report uses IRI and rut depth to rate the pavement function and uses alligator, longitudinal, and transverse cracking, and either rut depth or faulting to describe structural integrity. However, an agency may choose to use only traffic load or wheelpath related distresses such as alligator cracking and transverse cracking (rigid pavement). Finally, an agency may choose to include additional data that were not included in the ratings in this report (such as edge cracking and block cracking). The data elements collected by the State transportation departments are not consistent and some may have more or less available data for use in the dual rating systems.

  For example, pavement surface friction data are not often available at the network level. However, a State transportation department with significant friction data may choose to include the data in its functional rating or in a safety rating system. The addition or subtraction of the rating systems data does not affect the process of the rating systems. The rating is based on the minimum RFPs and RSPs, respectively, regardless of the number of elements.

- **Thresholds**: The dual pavement condition rating systems use pavement condition and distress threshold values and CSs. Both of these are flexible in nature and can be molded to fit the needs of any State transportation department. Important considerations include the following:

- The pavement condition and distress types and thresholds presented in table 15 and table 16 are not set in stone. They are based on those values available in various literature, the state of the practice of State transportation departments, and the experience and opinion of the research team.

- Thresholds can be modified and calibrated to the needs of the interested State transportation departments. For example, the recommended IRI threshold of 172 inches/mi (2.7 m/km) listed in table 15 is based on providing a comfortable ride on roads with speed limits of 55 mi/h (90 km/h) or greater. An agency may choose a higher or lower value such as 160 or 200 inches/mi (2.5 or 3.16 m/km) based on the road users' inputs or the managers of the agency and in some cases the legislators.

- Similarly, the interested agency may choose to use different threshold values for the structural integrity. The recommended values in this report, listed in table 16, are based on crack saturation or the point where preservation treatments other than heavy rehabilitation or reconstruction are no longer cost effective. These threshold values are highly variable and depend on numerous factors. Hence, the values are anticipated to be modified by the interested agencies based on their specific scenarios.

- The ranges in years of RFP and RSP listed in table 11 and expanded in table 12 are designed to describe both the pavement conditions and rates of deterioration and to provide sufficient time for planners and managers to scope pavement sections for the application of cost-effective pavement preservation treatments. Again, the interested agency may choose to modify these ranges to fit its needs. For example, the RSP range in years for the poor rating of 0 to less than 4 years is based on the required time to select, program, finance, and bid a major rehabilitation or reconstruction project. That is, if a project has an RSP of less than 4 years and it was selected for major rehabilitation, the time required to finish the paperwork to approve, design, plan, finance, establish specifications, and bid the project varies from 2 to 5 years depending on the State transportation department. Hence, 4 years is recommended so that by the time of construction, RSP is near 0 years. This scenario implies that any interested State transportation department could modify the ranges of the rating scale to fit its needs and based on its own practice. However, if changes or modifications are made, the process of the rating systems would be essentially the same with different ranges of RFP and/or RSP.

- **Applications**: The applications of the dual rating systems are open and unlimited. Because the terms good, fair, and poor and the corresponding colors green, yellow, and red are easy to interpret and can be understood by the majority of stakeholders, the public could be informed of the CSs of the entire network or specific routes or sections. Legislators could use the ratings to determine future funding levels and directives. Planners and upper managers could use them to allocate funds or to select regions or routes for treatment. Pavement managers could use them to flag pavement sections for treatment or to assess the future needs of the pavement network. The specific uses of the dual rating systems are numerous and can be established by the interested agency.

The dual rating systems cannot be used alone for the selection of treatment categories unless the boundaries of these categories are established. Figure 42 depicts the classic S-shaped curve for alligator cracking (expressed in the figure as percent of the total area). The figure also shows the following boundaries for three treatment categories:

- **Window 1 (W1):** Do nothing or light maintenance where the extent of alligator cracking varies from 0.0 to 2 percent. This extent corresponds to RSP CS 3 (more than 8 years).

- **Window 2 (W2):** Potential preservation actions where the extent of alligator cracking varies from 2 to about 18 percent. This extent corresponds to RSP CS 2 (4 to 8 years).

- **Window 3 (W3):** Potential heavy rehabilitation or reconstruction actions where the extent of alligator cracking exceeds 18 percent. This extent corresponds to RSP CS 1 (0 to 4 years).

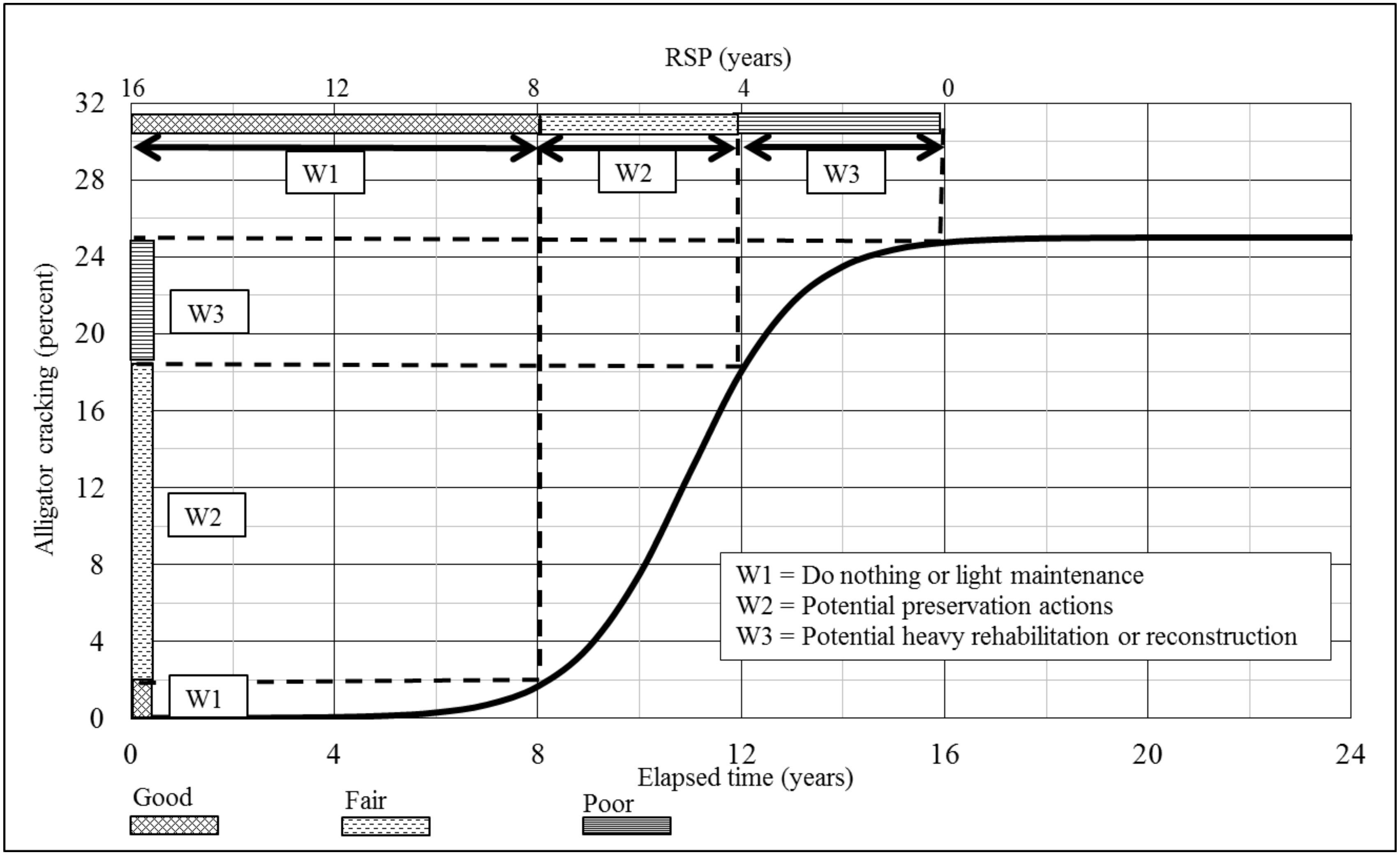


Figure 42. Graph. Idealized S-shaped curve for alligator cracking showing three windows (threshold values) for various treatment actions.

The final selection of the treatment category and treatment type within a given category should be accomplished after the actual pavement condition and distress data and results of the forensic investigation of the causes of distresses are carefully examined.

The previous discussion implies that the RSP and RFP spectrum could be divided into various ranges to aid in the selection of pavement preservation type and estimation of cost. For example, an RSP longer than 8 years implies light maintenance, whereas RSPs between 4 and 8 years imply preservation treatments (i.e., no preservation treatment should be applied to pavement section having an RSP of less than 4 years). Finally, an RSP shorter than 4 years implies rehabilitation and/or reconstruction. The interested State transportation department could assign an average cost to each of these RSP ranges based on its cost data. When such cost estimates can be accomplished based on the ranges of RSP, the data indicate that early preservation (at high RSP) yields the least lifecycle cost.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This chapter presented the basis for and development of the dual pavement condition rating systems. Topics covered included the condition classification, dual pavement condition rating systems, recommended threshold values, and a discussion of the flexibility of the systems. The various topics were evaluated, and the research team made the following recommendations:

- Pavement condition rating should be based on current conditions and distresses as well as the pavement's rates of deterioration. It is recommended that accurate planning and management be based on the pavement conditions and rates of deterioration.

- Dual pavement condition rating systems were developed to express the pavement conditions and distresses over time based on the new RFP and RSP concept. RFP and RSP express the time until certain functional and structural thresholds are reached. It is recommended that the dual rating system be adopted and calibrated by the stakeholders.

- Threshold values were provided for calculation of RFP and RSP. The values were based on minimum level of service to the user (functional), and loss of structural integrity (structural). It is recommended to adopt these or similar threshold values and evaluate them as more pavement condition and distress data become available.

- The dual pavement condition rating systems were found to be flexible and could be easily adopted. It is recommended that the dual rating system be adopted and tailored to fit agency needs and constraints.

- The RSP and RFP spectrum could be divided to three or five ranges. The estimated average cost to fix a unit length pavement section could be assigned to each range (see figure 37 and figure 38). This would assist the agency in establishing an optimum preservation strategy based on LCCA.

CHAPTER 4. DATA MINING AND SYNTHESIS

DATA SOURCES

The data used in this study were obtained from the LTPP database Standard Data Release 28.0. The database contains six volumes consisting of the primary dataset, data compilation views, FWD measurements, profile data, traffic data, and LTPP Traffic Analysis Software tables. Each of the six volumes contains various data elements for the more than 2,500 pavement test sections included in the LTPP Program. At the time of this study, about 1,700 test sections had been de-assigned or decommissioned from the LTPP Program over time, but nearly 800 remained active under the various experiments. Planning and scheduling has been taking place under the direction of FHWA to establish additional experiments and test sections to study different/new topics. Table 17 and table 18 list the number of active test sections under the SPS and GPS experiments, respectively, at the time of this report. The data from both active and de-assigned test sections were extracted from the database and arranged in special format for analyses. The detailed data extraction is presented in later sections in this chapter. Results of the analyses are presented and discussed in chapters 5, 6, and 7.

Table 17. Active LTPP SPS test sections as of January 2014.

	SPS Number									Total SPS
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	
Number of active test sections	53	186	0	0	53	18	0	59	43	412

Table 18. Active LTPP GPS test sections as of January 2014.

	GPS Number									Total GPS
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	
Number of active test sections	13	8	67	16	30	174	51	0	13	372

In addition, the pavement management databases from three State transportation departments—CDOT, WSDOT, and LADOTD—were requested and received. From each database, several pavement projects were identified, and their data were downloaded and formatted for analyses. Each of the selected projects was subjected to certain treatments in the past. The data for each project included the location reference systems, the time-series pavement conditions and distresses, the time and types of treatments that were performed in the past, and, in some cases, the cost of the treatments. The data were analyzed and the results are discussed in chapter 9.

AUTOMATED AND MANUAL PAVEMENT DISTRESS DATA

The monitoring module within the primary dataset of the LTPP database contained time-series pavement distress data (rut depth, cracking, and so forth). The data were collected using two different survey procedures: manual (visual observations) and semi-automated (videotape).

Table 19 and table 20 list the number of manual and semi-automated surveys conducted for each test section in the SPS-1 experiment. The data for all other test sections in the SPS and all test sections in the GPS experiments were submitted to FHWA and are available from the LTPP Customer Support Services.(79) After detailed examination of the manual and the semi-automated pavement distress and condition data, the manual data were selected for data modeling and analyses. The semi-automated data were not used for the following reasons:

- The number of available manual data points was much greater than for the semi-automated data. The manual data had been collected over the entire duration of the LTPP Program, while the semi-automated data were only collected between 1989 and 2004. Hence, less semi-automated data were collected.

- The two sets of data were not compatible enough to be combined and analyzed as a function of time. The few semi-automated data points generally did not align with the trends indicated by the manual data over time.

- The variability of the time-series semi-automated pavement distress and condition data was much greater than that of the manual data.

It is important to note that similar findings were reported by another team of researchers.(76)

Table 19. Number of manual and semi-automated surveys for test sections 0101–0112 in LTPP SPS-1 experiment.

	0101		0102		0103		0104		0105		0106		0107		0108		0109		0110		0111		0112	
State (Code)	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**
Alabama (1)	14	5	14	6	9	5	9	5	9	6	9	5	4	8	9	6	9	6	9	5	9	5	9	6
Delaware (10)	9	4	13	4	9	4	9	4	10	4	9	4	9	4	9	4	9	4	9	4	9	4	9	4
Florida (12)	10	2	10	2	10	2	10	2	10	2	10	2	10	2	10	2	10	2	10	2	10	2	10	2
Iowa (19)	7	7	6	7	5	7	6	7	6	7	6	7	7	7	6	7	6	7	6	7	6	7	6	7
Kansas (20)	3	2	3	2	8	7	7	7	8	7	8	7	3	2	8	7	8	7	8	7	8	7	8	7
Nevada (32)	26	6	8	6	8	6	11	6	8	6	11	6	11	6	11	6	11	6	11	6	11	6	11	6
New Mexico (35)	9	4	9	4	9	4	9	4	9	4	9	4	9	4	9	4	9	4	9	4	9	4	9	4
Ohio (39)	1	1	2	1	4	3	11	6	3	1	8	6	2	1	4	4	4	4	4	4	7	6	11	6

M = Manual.
SA = Semiautomatic.

Table 20. Number of manual and semi-automated surveys for test sections 0113–0124 in LTPP SPS-1 experiment.

	0113		0114		0115		0116		0117		0118		0119		0120		0121		0122		0123		0124	
State (Code)	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**	**M**	**SA**
Arizona (4)	24	6	23	6	10	6	10	6	10	6	10	6	10	6	10	6	10	6	10	6	10	6	10	6
Arkansas (5)	8	5	8	5	8	5	8	5	8	4	8	5	6	5	6	5	6	5	8	5	8	5	8	5
Michigan (26)	—	—	—	—	13	3	13	3	13	3	6	2	—	—	6	2	6	2	—	—	13	3	13	3
Montana (30)	13	2	20	2	13	2	13	2	13	2	13	2	13	2	13	2	13	2	13	2	13	2	13	2
Nebraska (31)	2	2	13	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
Oklahoma (40)	1	5	2	5	4	5	11	5	3	5	8	5	2	5	4	5	4	5	4	5	11	5	11	5
Texas (48)	13	3	13	3	13	3	13	3	13	3	13	3	13	3	13	3	13	3	13	3	13	3	13	3
Virginia (51)	12	1	19	4	8	4	8	4	8	4	8	4	8	4	9	4	11	4	8	4	8	4	8	4
Wisconsin (55)	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5

— No data available.
M = Manual.
SA = Semiautomatic.

DATA EXTRACTION

To facilitate the analyses of this study, the following specific data items were extracted from the LTPP database for each test section of the SPS and GPS experiments and formatted for time-series analyses:

- Inventory.
- Time-series pavement condition and distress.
- Time and type of pavement rehabilitation, preservation, and maintenance actions.
- Traffic.
- FWD data, including those collected along the test sections in the SMP.
- Climatic region.

Inventory Data

All inventory data, including construction history of the test sections, their opening dates to traffic, lane widths, number of lanes, pavement layer types and thicknesses, and subgrade information, were obtained from the inventory module. Some of the tables specifically used for this purpose were INV_AGE, INV_GENERAL, INV_ID, INV_LAYER, INV_SUBGRADE.

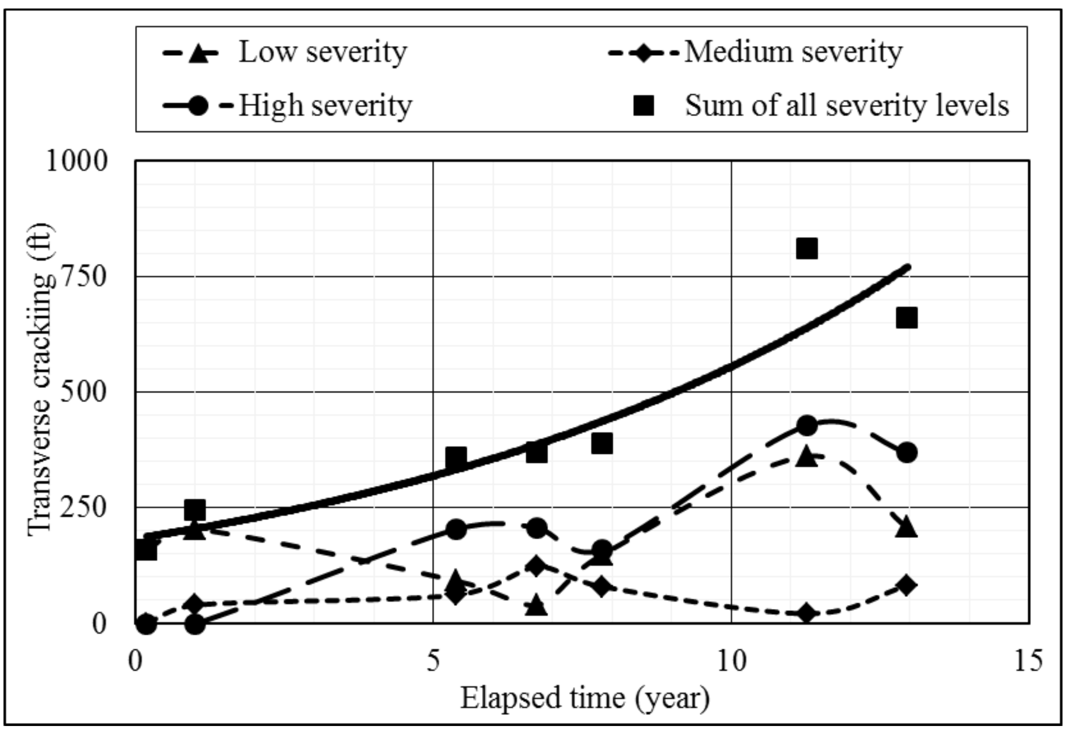
Time-Series Pavement Condition and Distress Data

The time-series pavement condition and distress data used in the analyses (presented in chapter 5) included transverse, longitudinal, and alligator cracking; rut depth; IRI; and faulting. These data were extracted from their respective files and reorganized in a spreadsheet format for analyses. The pavement condition and distress data were obtained from the following LTPP tables under the monitoring module of the LTPP database:

- **Cracking (MON_DIS_AC_REV, MON_DIS_CRCP_REV, and MON_DIS_JPCC_REV):** The cracking data were classified and stored in the database using three severity levels—low, medium, and high—as described by the *LTPP Distress Identification Manual*.(80) The difficulty with such data was that the crack severity rating was a function of several variables, including the following:

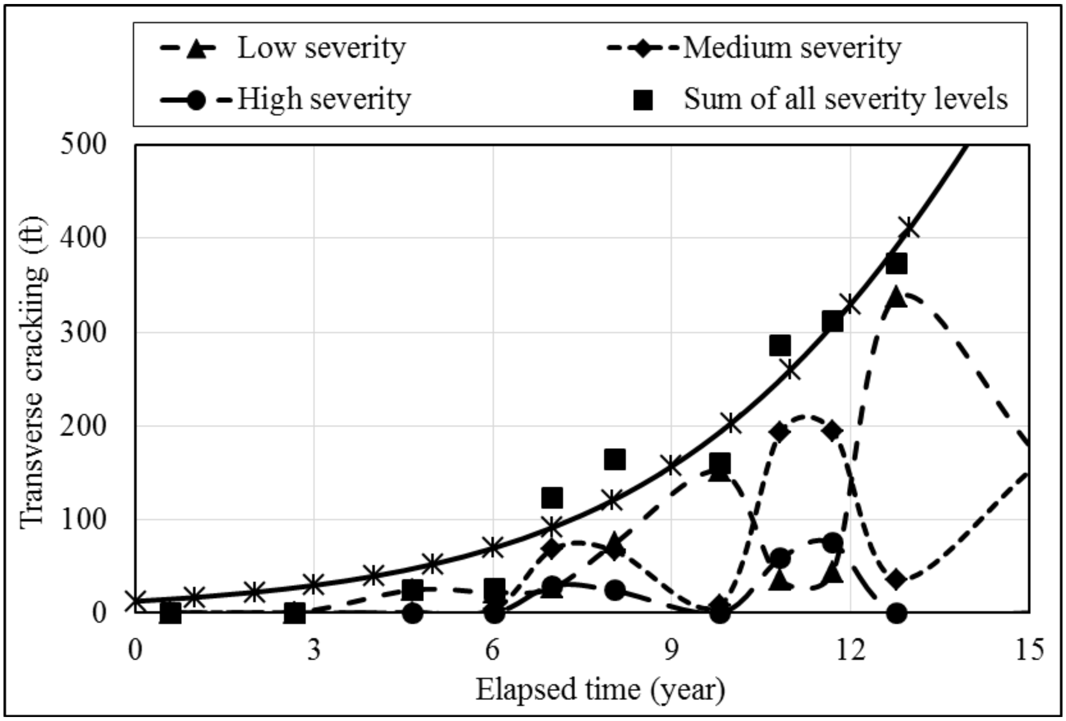
    - The pavement temperature at the time of the distress survey. The crack width, which is part of the severity level assignment, is a function of the pavement temperature. In general, the crack width increases as the temperature decreases.

    - The subjective judgment of the surveyor who reviewed and observed the cracks. Such subjective judgment is a function of the degree of training and experience of the surveyors. Further, the same pavement segment is likely to be surveyed by different surveyors over time. Thus, a crack may be labeled high severity in one year and medium the next year or vice versa. Figure 43 and figure 44 depict, respectively, the time-series low, medium, and high severity transverse cracking data of SPS-3 test section A330 and SPS-5 test section 0502 in California. Examination of the figures indicated that the sum of the time-series low, medium, and high severity cracking data was more consistent over time than the individual severity levels of cracking and therefore more suitable for modeling. Further, various attempts were made to analyze the data per severity level. In each attempt, the data for a significant number of test sections were eliminated from the analyses because of their high variability over time. In addition, the transverse cracking data were classified and stored in the LTPP database as either unsealed or sealed cracking at each severity level, while the longitudinal cracking data were classified and stored at each severity level as sealed and unsealed cracks in the wheelpath or non-wheelpath. The sealed and unsealed cracks had the same effect on the pavement structural integrity; the only difference was that sealed cracks retarded water infiltration, which might slow the rate of deterioration. Further, the wheelpath and nonwheelpath longitudinal cracks differed in their potential causes. Wheelpath longitudinal cracks in flexible pavement were likely to be either the start of top-down cracking due to pavement-tire interaction or the first appearance of alligator cracks on the pavement surface. In fact, for some test sections, the extent of longitudinal cracking, from one survey cycle to the next decreased substantially as alligator cracking was recorded for the first time, indicating that the longitudinal cracks were reclassified as alligator cracks. Therefore, for flexible pavements, longitudinal cracking in the wheelpath were combined with alligator cracking to facilitate the analyses of the data. Note that the selection of treatment type was based on the severity and location of the cracks, but the condition rating of the pavement was not affected by such information.



1 ft = 0.305 m.

Figure 43. Graph. Transverse cracking versus elapsed time for LTTP SPS-3 test section A330 in California.



1 ft = 0.305 m.

Figure 44. Graph. Transverse cracking versus elapsed time for LTTP SPS-5 test section 0502 in California.

- **Roughness (MON_PROFILE_MASTER):** The time-series pavement roughness data were computed into IRIs and stored in the database as left wheelpath IRI and right wheelpath IRI. The average of the two values was considered equivalent to the effect of roughness on the traveling vehicle. Hence, the average IRI was considered in the analyses.

- **Faulting (MON_DIS_JPCC_FAULT):** For transverse joints, pavement faulting data were stored as edge faulting and wheelpath faulting. The wheelpath faulting data were used in the analyses because faulting at the edges could be influenced more by warping and/or curling. In addition, the average faulting among all joints within a given test section was calculated and used in the analyses.

- **Rut Depth (MON_T_PROF_INDEX_SECTION):** In the initial stages of the LTPP Program, rut depth measurements were made using a 4-ft (1.2-m) straightedge reference under the assumption that wheel-path depressions were not wider than 4 ft (1.2 m). These rut depth measurements can be found in the MON_RUT_DEPTH_ POINT table. However, in many instances, the wheel-path depressions were wider than 4 ft (1.2) m. Hence, transverse profile measurements were chosen by the LTPP Program over straightedge measurements to account for this.(59) The transverse profile data were used by the LTPP Program to calculate the mean and the maximum rut depth in each wheelpath. The average of the two means and the average of the two maximum rut depth data were calculated, and the former was used in the analyses in this study.

Pavement Rehabilitation, Preservation, and Maintenance Data

Pavement rehabilitation, preservation, and maintenance data were extracted from the MAINT_REHAB module of the LTPP database. The treatments performed on the LTPP test sections were classified and stored under different tables, namely MNT_IMP and RHB_IMP. After downloading the data, the research team organized them such that the treatment information for each LTPP test section could be easily retrieved and analyzed. For all LTPP test sections that received one or more treatments and for each treatment type, the pavement condition and distress data were organized in two different groups—before treatment and after treatment. Such grouping was crucial to accurately model the pavement performance before and after treatment and to estimate the treatment benefits.

Traffic Data

The research team extracted the traffic data for each test section from the traffic module of the LTPP database. The equivalent single-axle load (ESAL) data in the TRF_ESAL_COMPUTED table were used to group the various test sections. The groupings were used to assess the impact of various variables on pavement performance, the longevity of the pavement sections, and the effectiveness of the pavement treatments.

FWD Data

The research team extracted FWD data from the FWD measurements folder of the LTPP database to analyze the possible relationships between the measured deflection and pavement distress and/or conditions. The peak pavement deflection measured at each of the seven or nine sensors for different loads (different drop heights) were extracted and organized for analyses. Further, the time-series LTEs were also extracted. Finally, the FWD data of all test sections included in the SMP were extracted and organized for analyses. Because, for each test site included in the SMP, the FWD data were collected at different time and temperature, the data were analyzed to do the following:

- Check the accuracy of existing temperature correction procedures and develop a global temperature correction function.
- Determine whether the time-dependent FWD data could be used to establish a threshold value to trigger structural treatment or as an indicator of the structural integrity of the pavement sections.

Climatic Data

North America has four climatic regions: DF, DNF, WF, and WNF. The climatic regions were obtained from the TRF_ESALS_INPUTS_SUMMARY table of the traffic module in the LTPP database. The criterion established by the LTPP Program to identify wet and dry climates was based on annual precipitation. Regions with annual precipitation of less than 20 inches (50.8 cm) per year were considered dry. The classification of freeze or no freeze was based on the Freezing Index. Test sites in regions where the annual Freezing Index was greater than 150 degree-days were considered in a freezing climatic region.

STATUS OF THE CONDITION AND DISTRESS DATA

As stated in the previous section, for each LTPP test section in the SPS and GPS experiments, and for each pavement treatment type, all available before treatment and after treatment condition and distress data were downloaded and organized in spreadsheet format for analyses. Table 21 summarizes the alligator, longitudinal, and transverse cracking data of all SPS-1 test sections located in Montana (State code 30). Each row in the table represents one pavement test section. The columns indicate the treatment types and the number of pavement condition and distress surveys (i.e., number of time-series data points) that have been conducted before and after each treatment. For example, test section 0113 was subjected to crack sealing, aggregate seal coat, and two additional crack sealing treatments since its assignment to the LTPP Program. The number 5 under the first before treatment column indicates that five time-series data points (surveys) were available in the LTPP database taken before the first crack sealing treatment was applied. The number 2 under the after treatment/before treatment column indicates that two data points were available in the database taken after the first crack sealing treatment and before the aggregate sealing treatment. The numbers under the other after treatment/before treatment columns indicate the number of data points available in the LTPP database that were taken after the previous treatment and before the next treatment. Finally, four time-series data points were available in the LTPP database taken after the last crack sealing treatment. Note that the number of before treatment data points taken before the first crack sealing application and the number of after treatment data points taken after the last crack sealing treatment were greater than three, and hence, the data can be modeled as a function of time.

A similar summary for each SPS and GPS experiment in each State was made. The summaries and the information listed in table 21 were used to identify the test sections and the treatments for which three or more time-series condition or distress data points were available that were collected before and/or after a particular treatment.

Table 21. Number of cracking data points available before treatment and after treatment for LTTP SPS-1 test sections in Montana.

State (Code)	SHRP ID	BT	Treatment Type	AT/BT	Treatment Type	AT/BT	Treatment Type	AT/BT	Treatment Type	AT
Montana (30)	0113	5	CS	2	ASC	1	CS	1	CS	4
	0114	10	CS	3	ASC	2	CS	1	CS	4
	0115	5	CS	2	ASC	1	CS	1	CS	4
	0116	5	CS	2	ASC	1	CS	1	CS	4
	0117	5	CS	2	ASC	1	CS	1	CS	4
	0118	5	CS	2	ASC	1	CS	1	CS	4
	0119	5	CS	2	ASC	1	CS	1	CS	4
	0120	5	CS	2	ASC	1	CS	1	CS	4
	0121	5	CS	2	ASC	1	CS	1	CS	4
	0122	5	CS	2	ASC	1	CS	1	CS	4
	0123	5	CS	2	ASC	1	CS	1	CS	4
	0124	5	CS	2	ASC	1	CS	1	CS	4

CS = Crack sealing.
ASC = Aggregate seal coat.
BT = Before treatment.
AT = After treatment.
AT/BT = After treatment for the previous treatment and before treatment for the last treatment.

STATUS OF THE MAINTENANCE AND REHABILITATION DATA

As stated earlier, the maintenance and rehabilitation actions and their time of application were compiled for analysis. The number of test sections in the LTTP SPS and GPS experiments with and without treatments are summarized in table 22 and table 23.

Table 22. Number of LTTP SPS test sections with available treatment data in the database.

Treatment Status	Number of Sections for each SPS Experiment Number Designation									Total
	1	2	3	4	5	6	7	8	9	
With treatments	163	106	408	192	202	169	38	13	102	1,393
Without treatments	82	101	37	28	2	1	1	40	35	327
Total	245	207	445	220	204	170	39	53	137	1,720

Table 23. Number of LTTP GPS test sections with available treatment data in the database.

Treatment Status	Number of Sections for each GPS Experiment Number Designation								Total
	1	2	3	4	5	6	7	9	
With treatments	197	121	96	56	50	50	27	12	609
Without treatments	36	23	37	13	35	15	8	13	180
Total	233	144	133	69	85	65	35	25	789

Test sections in the LTTP SPS-1 through -7 and LTTP GPS-6, -7, and -9 were included in the analyses to assess the impacts of design variables and treatment benefits. The available data for the untreated test sections were used as control sections or to estimate the service period of the test section.

ANALYSES PROCEDURES

Most of the proposed analyses for the evaluation of the effectiveness of the various pavement treatments are based on the determination and evaluation of the relationships between the before and after treatment pavement performance. Such pavement performance is a function of the available time-dependent pavement condition and distresses data and the corresponding rates of pavement deterioration. To model, with some degree of certainty, the condition and distress data over time using nonlinear mathematical functions, a minimum of three time-series data points are required before and/or after treatment. Two or fewer data points do not define the parameters of the nonlinear mathematical functions representing the data. Examination of the available data points in the LTPP database indicates that, for a significant number of test sections, only two data points were available before and/or after treatment. To enhance the number of available data and to increase the number of test sections that could be analyzed, several actions were proposed in the interim report, discussed during the project meeting in Washington, DC, and implemented in the analyses. The actions taken are described in the following subsections.

Addition of One Data Point Immediately After Certain Treatments

Often, the pavement conditions and distresses were not measured immediately after construction or after treatment application. Depending on the treatment type, the condition and distress values after some treatment actions can be logically and reasonably assumed. Therefore, for all newly constructed SPS-1 and -2 test sections and for all other test sections where AC overlay or mill-and-fill treatments were applied, one can reasonably assume that at 0.01 years (3 days) after construction, the initial value of the rut depth, faulting, and the total length of each crack type would be negligible. Because a 0.0 data point is not allowed in the mathematical functions reported in the literature and used in modeling the data, the initial pavement distress and condition at the elapsed time of 0.01 years after construction were assigned the following insignificant values:

- **Rut depth**: 0.01 mm for the LTPP data and 0.01 inches for the State data.
- **Transverse cracks**: 0.01 m for the LTPP data and 0.01 ft for the State data.
- **Longitudinal cracks**: 0.01 m for the LTPP data and 0.01 ft for the State data.
- **Alligator cracks**: 0.01 m^2 for the LTPP data and 0.01 ft^2 for the State data.

This assumption supports the addition of one extra data point for use in the analyses of pavement performance. Unfortunately, no initial IRI can be reasonably assumed. To illustrate, in Oklahoma, skin patching was applied to 12 SPS-1 test sections. The LTPP database contained more than three time-series pavement condition and distress data points that were collected after the skin patching was performed, but only two data points were available that were collected before the treatment. Because all SPS-1 test sections were newly constructed, one data point could be assumed indicating that at 0.01 years after construction, the magnitudes of rut depth, crack length, and faulting were the same as those listed in the previous bullets. The addition of such data points made the analyses of the before treatment pavement performance possible. Once again, such an assumption was reasonable and logical because for flexible pavements, the smooth-drum rollers typically used in the compaction of the original HMA or overlays or mill-and-fill treatments produce smooth and flat pavement surfaces with no rutting or cracking. The LTPP treatments that were considered for this action are listed in table 24. This addition of a data point immediately after treatment was applied only to pavement segments where only two before treatment and/or after treatment data points were available. If fewer than two data points were available, the procedure would not yield three data points and hence it was not used. The addition of such data points significantly enhanced the number of available pavement segments for analyses. Note that no data points were added to any pavement segment that was subjected to any treatments not listed in table 24.

Table 24. Condition or distress type eligible for data addition for different treatments.

Pavement Treatment Type	LTPP Treatment Code	Pavement Condition or Distress Type Eligible for Data Addition: IRI	Rut Depth	Longitudinal Cracking	Transverse Cracking	Alligator Cracking	Faulting
Grinding surface	12	—	—	—	—	—	X
Reconstruction (removal and replacement)	18	—	X	X	X	X	X
AC overlay	19	—	X	X	X	X	X
PCC overlay	20	—	N/A	X	X	N/A	X
Surface treatment, single layer	28	—	—	X	X	X	—
Surface treatment, double layer	29	—	—	X	X	X	—
Surface treatment, three or more layers	30	—	X	X	X	X	—
Aggregate seal coat	31	—	—	X	X	X	—
Hot-mix recycled AC	43	—	X	X	X	X	X
Cold-mix recycled AC	44	—	X	X	X	X	X
Heater scarification, surface recycled AC	45	—	X	X	X	X	X
Recycled PCC	48	—	N/A	X	X	N/A	X
Mill off AC and overlay with AC	51	—	X	X	X	X	X
Mill off AC and overlay with PCC	52	—	N/A	X	X	N/A	X
Mill existing pavement and overlay with hot-mix recycled AC	55	—	X	X	X	X	X
Mill existing pavement and overlay with cold-mix recycled AC	56	—	X	X	X	X	X

— Indicates not eligible for data point addition.
N/A = Not applicable.

Table 25 summarizes the status of the cracking data of all SPS-1 test sections located in the DNF region. Similar tables for all pavement condition and distress data types and other LTPP experiments and climatic regions were submitted to FHWA and are available from the LTPP Customer Support Services.[79]

Table 25. Summary of cracking data for LTPP SPS-1 test sections located in the DNF region.

A	B	C	D	E	F	G	H	I	J	K	L	M	N
		Treatment Data		Number of Treatment Applications							Number of Treatment Applications That Can Be Analyzed Before and After Treatment		
					Three or More Data Points			One Assigned Data Point After Treatment (0.01 Years)					
Climatic Region	State (Code)	Type	Number of Test Sections	Total	BT and AT	BT Only	AT Only	BT and AT	BT Only	AT Only	BT and AT	BT Only	AT Only
DNF	Arizona (4)	CS	6	8	2	4	2	0	0	0	2	4	2
		FDP	1	1	0	1	0	0	0	0	0	1	0
		PHP	1	1	0	1	0	0	0	0	0	1	0
		SS	6	6	4	0	2	0	0	0	4	0	2
	New Mexico (35)	GS	2	2	0	2	0	0	0	2	2	0	0
	Oklahoma (40)	MPSP	1	1	1	0	0	0	0	0	1	0	0
		SP	12	12	0	0	12	0	12	0	12	0	0
	Texas (48)	ACOL	12	12	0	1	1	0	0	0	0	1	1
		ASC	11	11	0	0	0	0	0	11	0	0	11
		GS	3	3	0	0	3	0	3	0	3	0	0
		MOAC	12	12	12	0	0	0	0	0	12	0	0
		MPSP	11	11	0	11	0	0	0	0	0	11	0

ACOL = AC overlay.
ASC = Aggregate seal coat.
CS = Crack sealing.
FDP = Full-depth patching.
GS = Grinding surface.
MOAC = Mill and overlay with AC.
MPSP = Machine premix spot patching.
PHP = Pot holes patching.
SP = Skin patching.
SS = Slurry seal.
BT = Before treatment.
AT = After treatment.

The following describes the data that appear in each of the columns in table 25:

- **Column A**: Climatic region.
- **Column B**: State and State code.
- **Column C**: Treatment type.
- **Column D**: Number of SPS-1 test sections.
- **Column E**: Number of times treatments were applied. When the number in column E is greater than the number in column D, it implies that at least one test section received the treatment more than one time.
- **Column F**: Number of treatment applications for which three or more time-series data points are available that were collected before and after treatment.
- **Column G**: Number of treatment applications for which three or more time-series data points are available that were collected before treatment only.
- **Column H**: Number of treatment applications where three or more time-series data points are available that were collected after treatment only.
- **Column I**: Number of treatment applications where one data point can be logically assumed to have been collected immediately after treatment (as stated earlier as 0.01 year), which yields three time-series data points before and after treatment.
- **Column J**: Number of treatment applications where one data point can be logically assumed to have been collected immediately after treatment (i.e., the addition of the 0.1-year point described earlier in this section), which yields three time-series data points that were collected before treatment only.
- **Column K**: Number of treatment applications where one data point can be logically assumed to have been collected immediately after treatment (i.e., the addition of the 0.1-year point described earlier in this section), which yields three time-series data points that were collected after treatment only.
- **Column L**: Number of test sections that can be analyzed before and after treatment.
- **Column M**: Number of test sections that can be analyzed before treatment only.
- **Column N**: Number of test sections that can be analyzed after treatment only.

At the time of this report, there were 1,555 LTPP test sections (supplemental sections were not included) in the SPS-1 through -7 and in the GPS-6, -7, and -9 experiments. The majority of these test sections (1,301) were treated at least 1 time during their assignment period. The total number of treatment applications was 2,674 (some test sections received more than 1 treatment).

For new construction (SPS-1 and -2 test sections) and for overlay and mill-and-fill treatments, one rut depth, cracking length, and faulting data point was added at 0.01 years after construction.

Table 26 lists the number of test sections that could be analyzed before and after treatment, the number of test sections that could be analyzed before treatment only, and the number of test sections that could be analyzed after treatment after the addition of this data point.

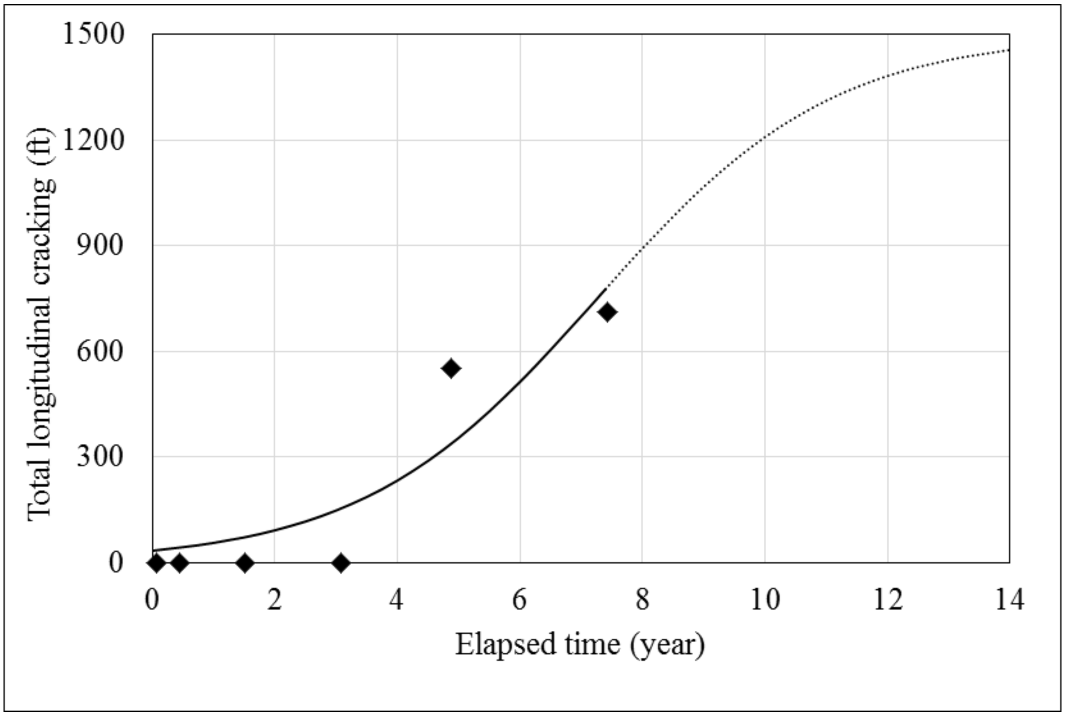
Table 26. Summary of treatments applied to LTTP SPS-1 through -7 and GPS-6, -7, and -9 sections analyzed in this study.

Number of Test Sections	Number of Treated Test Sections	Number of Treatment Applications	Pavement Distress/ Condition	Number of Treatment Applications Analyzed		
				BT and AT	BT Only	AT Only
1,555	1,301	2,674	Cracking	278	463	925
			IRI	468	558	911
			Rut depth	394	453	747
			Faulting	42	70	108
Total Analyzed				1,182	1,544	2,691

BT = Before treatment.
AT = After treatment.

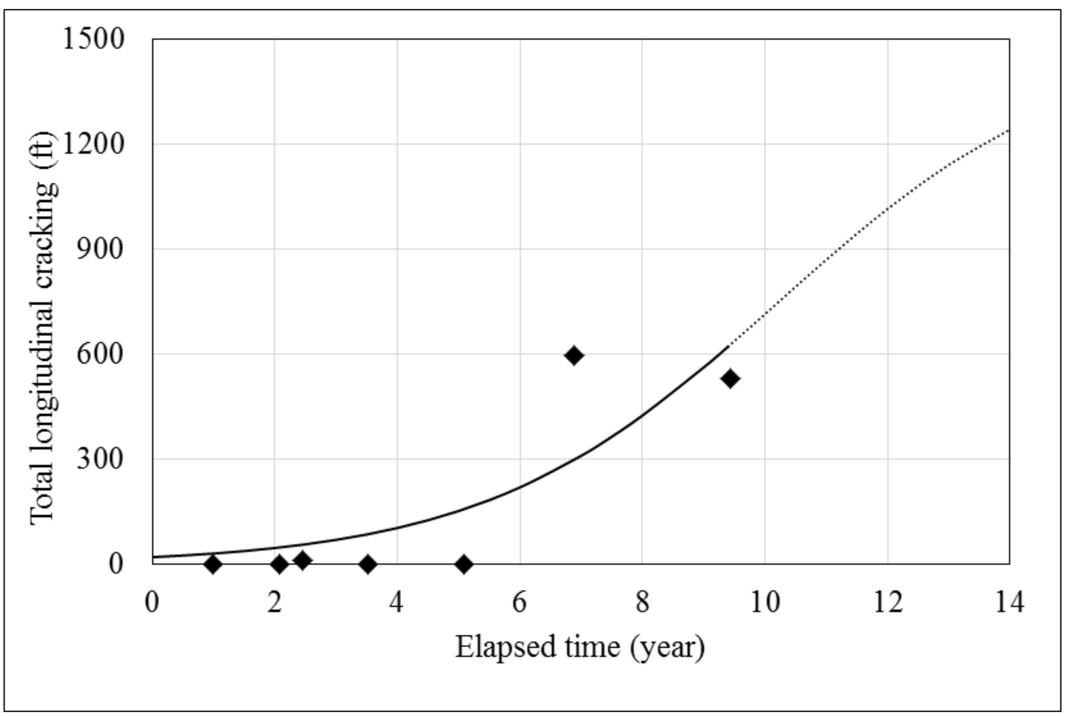
Using the Control Section Data for Before Treatment Conditions

Several of the LTPP experiments, including SPS-3 through -6, were designed with a control section (untreated) adjacent to the test sections (which were subjected to various treatment types). The control section was subjected to almost the same traffic and environmental loading and had almost identical structure and subgrade support characteristics. For this reason, the performance data of each control section could be used to represent the before treatment performance data of the adjacent test sections when only two or fewer before treatment data points are available in the database for the given test section. For example, SPS-3 test section A310 (see figure 45) in Maryland had only one cracking data point (not shown in the figure) collected before an overlay treatment was performed. Six cracking data points were available after the overlay was performed. To analyze the before treatment conditions of test section A310, the performance data of the control section A340 (see figure 46), which was not subjected to treatment, was used to represent test section A310 before-treatment performance.



1 ft = 0.305 m.

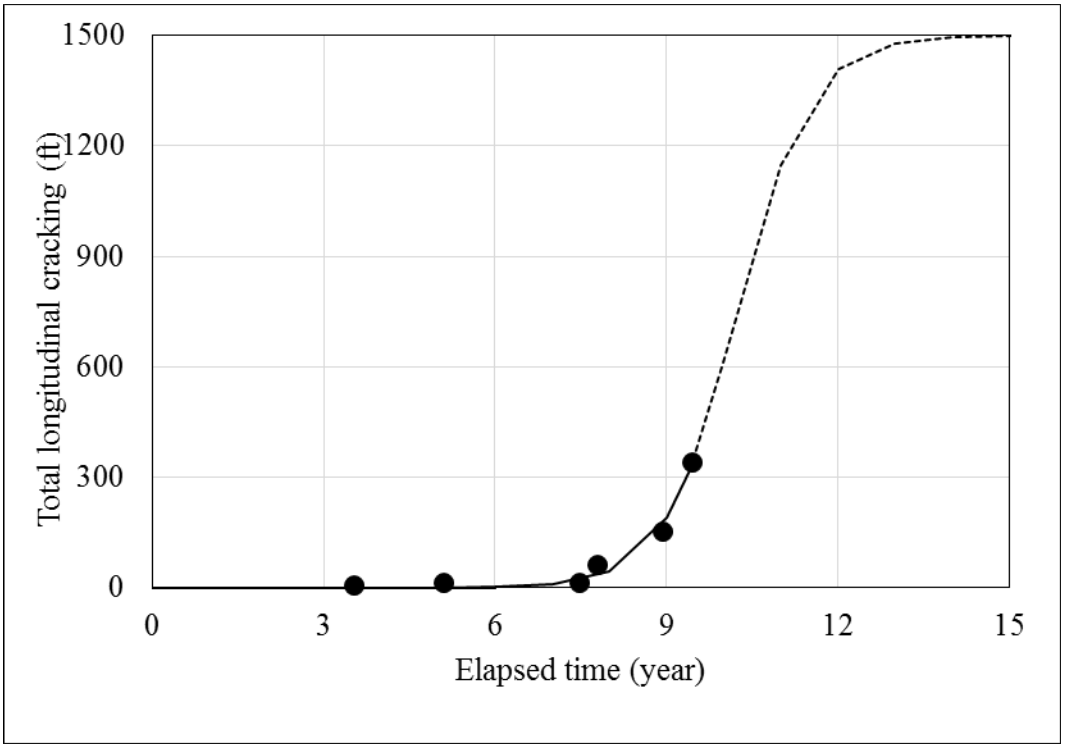
Figure 45. Graph. Total longitudinal cracking versus time for LTTP SPS-3 test section A310 in Maryland.



1 ft = 0.305 m.

Figure 46. Graph. Total longitudinal cracking versus time for LTPP SPS-3 control section A340 in Maryland.

In some cases, where no control sections were assigned, the linked GPS test sections associated with the SPS sections were used as control sections.(60) Linked GPS test sections were under the GPS experiment and were located adjacent to the SPS test sections. They had traffic loading and structure similar to the SPS test sections to which they were linked. Some of the linked GPS sections were also treated. However, the before treatment data (see figure 47) could still be used as the before treatment data for the SPS-3 test section.

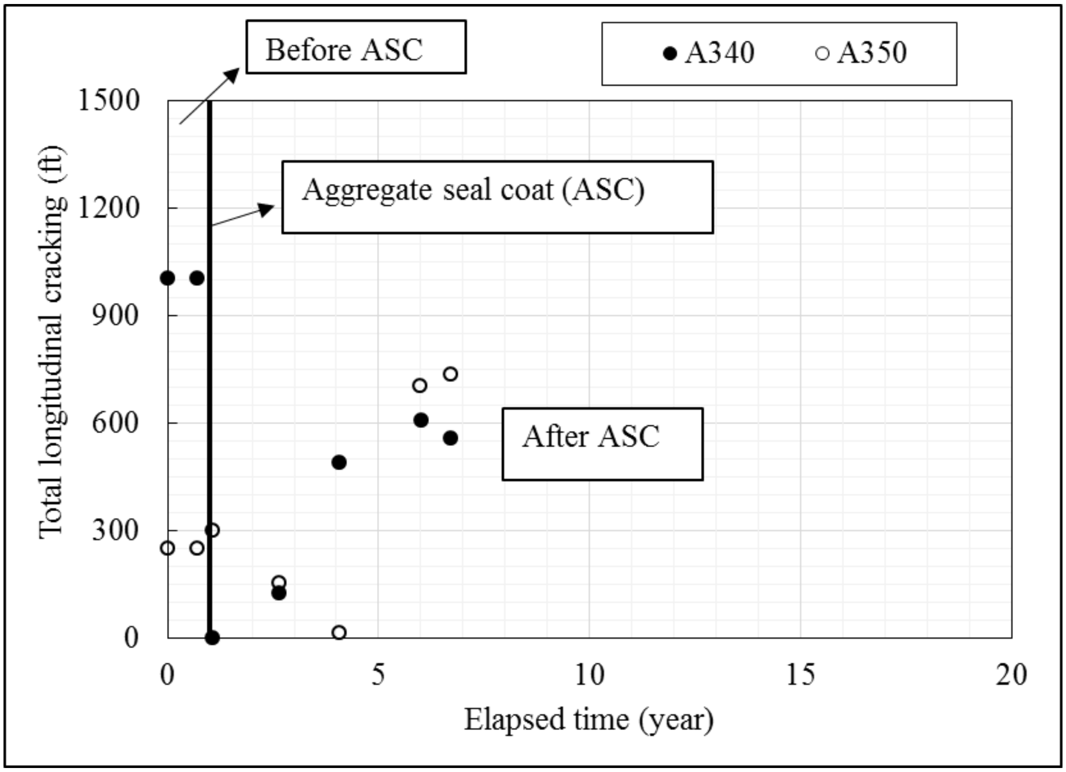


1 ft = 0.305 m.

Figure 47. Graph. Total longitudinal cracking versus elapsed time for LTPP GPS-1634 linked test section to SPS-3 experiment in Maryland.

Cracking data collected before the overlay could also be used as before treatment data for test section A310. Comparisons of the pavement condition and distress data between the control and the linked section that were related to the conditions of the test section were made to verify whether the data of the control and/or linked sections were indeed similar to the available before treatment data points of the test section. Finally, if the data of the control section represented the before treatment data of the test section, the performance of the two sections were compared to determine the benefits of the treatment applied to the test section

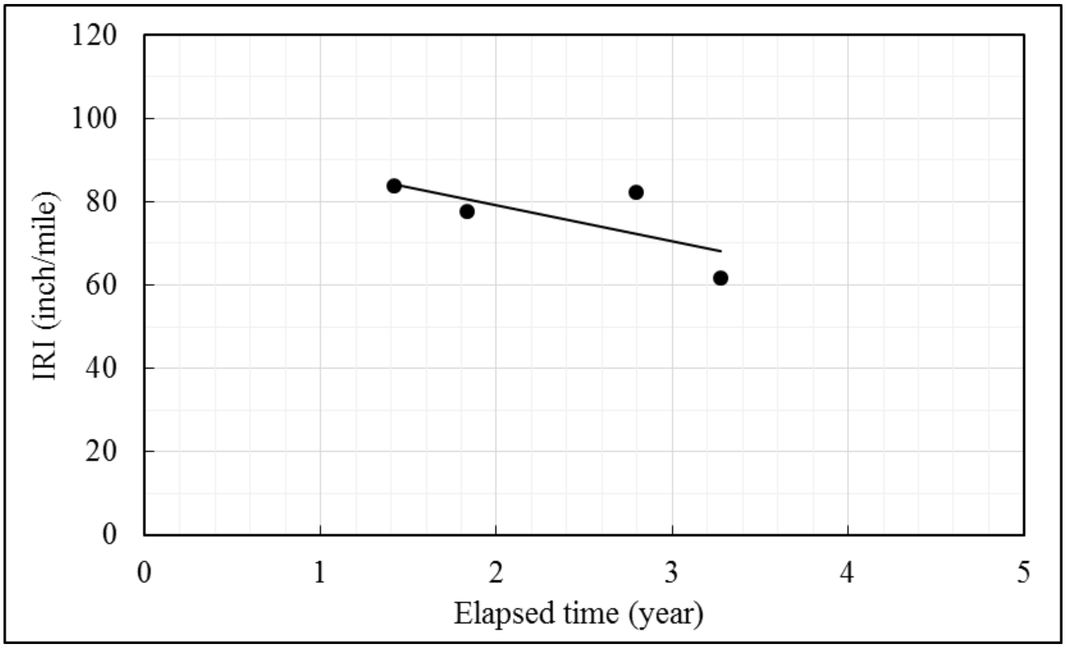
For some test sections, such as LTTP SPS-3 test section A350 in New York, the reported before treatment longitudinal cracking was about 984 ft (300 m), as shown in figure 48. The longitudinal cracking data of the associated control section A340 indicated 197 to 328 ft (60 to 100 m) of cracking, as shown by the open symbols in figure 48. It should be noted that the control section was not subjected to any treatment. Nevertheless, in this and similar cases, the data from the control sections were not used because they were not representative of the before treatment pavement performance of the test section in question. Note that the use of the control section data in place of the before treatment data significantly increased the number of available test sections for analysis.



1 ft = 0.305 m.

Figure 48. Graph. Total longitudinal cracking versus elapsed time for LTTP SPS-3 test section A350 and A340 control section in New York.

Although a treated pavement section may have had three data points collected before treatment and/or after treatment, it may or may not have been accepted for analysis. The before treatment and/or the after treatment time-series data of some of these test sections indicated that the pavement condition and/or distress was improving over time without the application of any treatment, as shown in figure 49.



1 inch/mi = 0.0158 m/km.

Figure 49. Graph. IRI versus elapsed time for LTTP SPS-1 test section 0119 in Texas.

In the absence of a pavement treatment, most pavement sections deteriorate over time. When the pavement condition and/or distress data indicated improvement over time, without the application of treatment, the data precipitated negative parameters of the pavement performance model (negative slope) (i.e., improved condition and/or distress without treatment). Such pavement condition and distress trends could occur for various reasons including the following:

- Human error and/or inaccuracy while collecting the data. The subjectivity of assigning distress severity levels and estimating the extent of the distress could generate inaccuracies in the time-series data. Properly calibrated sensor measured data would not exhibit this problem because no human subjectivity was involved.

- Data inaccuracy due to the employed equipment, such as calibration, malfunction, or changing equipment type between surveys over time.

- Environmental conditions from one data collection cycle to the next. For example, the crack opening was wider on cold days than on warmer days as a result of thermal expansion and contraction. This could change the assigned crack severity level and/or the observance of the length of the cracks. Likewise, temperature differential could cause curling on certain days, which would influence the measured pavement roughness in terms of IRI.

When the pavement condition and/or distress showed improvement over time without any treatment application, the data yielded infinite RFPs and/or RSPs, and the pavement performance could not be assessed. For example, the data in figure 49 and the associated exponential equation indicate improvement of the IRI over time and a negative exponential power component. That is, according to the given data and the equation, the IRI would never reach the threshold value, and therefore RFP was infinite, which was not practical. Consequently, any time-series condition or distress data showing improvement over time without the application of treatment was not included in the analyses of the pavement performance.

Severity Level

Finally, the analyses of cracking data in this study were based on the sum of the data for low, medium, and high severity levels.

SUMMARY AND CONCLUSIONS

This chapter presented the various data elements of the more than 2,500 test sections included in the LTPP Program that were downloaded from the 6 data volumes housed in the LTPP database Standard Data Release 28.0. The data were organized in a special format and readied for analyses. In addition, the pavement management databases from three State transportation departments—CDOT, WSDOT, and LADOTD—were requested and received. From each database, several pavement projects that were subjected to certain treatments in the past were identified, and their data were downloaded from the respective databases and formatted for analyses. Based on thorough and exhaustive review of the data, the research team reached the following conclusions:

- At the time of this study, 800 test sections were still active, whereas 1,700 were decommissioned.
- The manually collected pavement distress data were more numerous and consistent than the semi-automated data. The manual data were used in the analyses of the structural pavement performance of each test section.
- The database for some of the test sections contained only two data points over time before and or after treatment. For those test sections that were subjected to overlay or mill-and-fill treatments, an initial data point was added to rut depth and to each cracking type data.
- The addition of one initial data point was based on engineering logic and increased the number of test sections that could be analyzed.
- A total of 2,674 treatments were applied to 1,301 test sections of the 1,555 test sections included in the SPS-1 through -7 and GPS-6, -7, and -9 experiments.
- For each treated test section, the data were organized for the analyses of pavement performance before, after, or before and after treatment, depending on the data availability.

CHAPTER 5. LTPP DATA ANALYSES OF FLEXIBLE PAVEMENTS

For all LTPP test sections in SPS-1 through -7 and GPS-6, -7, and -9, the time-series pavement conditions, distresses, and some of the FWD data were downloaded and organized in spreadsheet format for analyses. The data from these LTPP test sections and a few pavement sections from the CDOT, LADOTD, and WSDOT databases were modeled using the proper mathematical function and were subjected to analyses. The procedures and the results of the analyses of the LTPP flexible pavement condition and distress data are presented in this chapter, while the procedure and results for rigid pavement condition and distress data are presented in chapter 6. Results of the analyses of the FWD data are presented and discussed in chapter 7. Finally, the results of the analyses of the CDOT, LADOTD, and WSDOT data are presented and discussed in chapter 9. The information in this chapter is arranged in the following sections:

- Modeling of the Time-Series Pavement Condition and Distress Data.
- Impacts of Climatic Regions, Drainage, and AC Thickness on Pavement Condition and Distress Using the LTPP SPS-1 Test Sections.
- Summary, Conclusions, and Recommendations, LTPP SPS-1.
- Results of the Analyses of the LTPP SPS-3 Test Sections.
- Summary, Conclusions, and Recommendations, LTPP SPS-3.
- Results of the Analyses of the LTPP SPS-5 Test Sections.
- Summary, Conclusions, and Recommendations, LTPP SPS-5.
- Results of the Analyses of the LTPP GPS-6 Test Sections.
- Summary, Conclusions, and Recommendations, LTPP GPS-6.
- ORCSE Method.
- Summary, Conclusions, and Recommendations, ORCSE Model.

MODELING THE TIME-SERIES PAVEMENT CONDITION AND DISTRESS DATA

The time-series pavement condition and distress data of all test sections in the SPS-1 through -7 and GPS–6, -7, and -9 experiments were downloaded, organized, and modeled using the proper mathematical functions based on the type of pavement condition (IRI) or distress (rut depth, and cracking). The mathematical functions listed in table 27 and shown in figure 50 were selected based on reported trends and mechanisms of pavement deterioration.(37,81,78) For example, rutting typically occurs early in the asphalt pavement’s life, and its accumulation rate decreases over time as the pavement materials densify under traffic loads.

Table 27. Description of the mathematical functions used in the analyses of the pavement distress and condition data.

Pavement Condition or Distress	Function Type
IRI (inches/mi (m/km))	Exponential function
Rut depth (inches (mm))	Power function
Cracking (length, area, or percent)	Logistic (S-shaped)

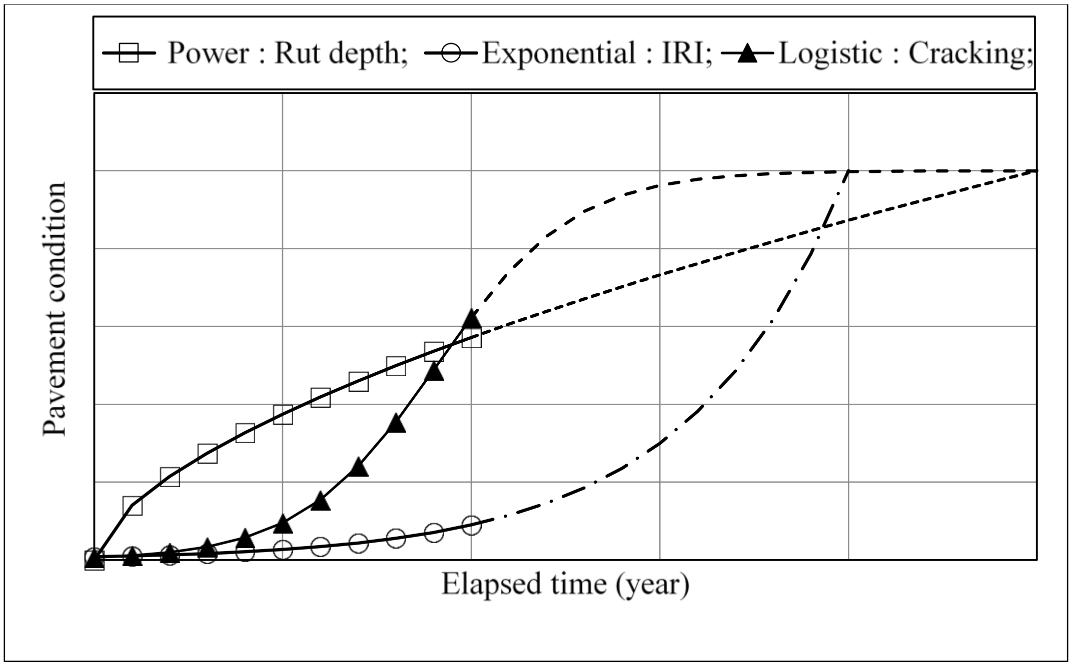


Figure 50. Graph. Exponential, power, and logistic (S-shaped) curves.

Therefore, a power function is typically used to model the time-series rut depth data. On the other hand, pavement roughness usually increases exponentially as the pavement ages, deteriorates, and becomes uneven causing increases in the dynamic effects of traffic loads. Hence, an exponential function is typically used to model the pavement roughness (IRI). Finally, the propagation of pavement cracks usually follows three stages. In the first stage, a few cracks appear in the early pavement life; their number and length increase exponentially. In the second stage, the number and length of cracks increase almost linearly over time. In this stage, a few new cracks are initiated and most existing cracks approach their maximum possible lengths (lane width or the pavement section length). In the third stage, the number of cracks and their length reach equilibrium as shown by the logistic curve in figure 50. Given this scenario, the modeling of crack propagation over time could be achieved using two different functions, depending on the availability of the data. If the cracking data are available over a short period of time after construction (stage one data only), an exponential function could be used to model the data. On the other hand, if the cracking data are available when the pavement is old (stage three only), a power function could be used. The modeling of the crack propagation using the logistic function cannot be confidently achieved unless at least four data points are available spanning the three crack propagation stages. To reduce the effect of the problem and to increase the number of test sections included in the analyses, one crack saturation point was assumed for each type of cracking. The assumed crack saturation points used throughout this study are listed in table 28. The assumption of the crack saturation points was based on engineering logic. For example, the saturation point for alligator cracking is the entire surface area of the pavement section, whereas the saturation point for longitudinal cracking is three cracks along the entire pavement section length. Note that the square, circle, and triangle symbols in figure 50 represent measured data. The solid portions of the curves are fit to the measured data while the dashed portions are forecasted based on the mathematical function fitting the data. It should be noted that the three mathematical models fit the LTPP pavement condition and distress data for all test sections and State data.

Table 28. Crack saturation values used in the analyses of the pavement cracking data.

Cracking Type	Saturation Value			Reason
	Per 500 ft (152.4 m) LTPP Test Section	Per 0.1 km	Per 0.1 mi	
Alligator cracking	5,906 ft^2 (549 m^2)	360 m^2	6,336 ft^2	100-percent of section cracked (12-ft (3.66-m) lane width)
Longitudinal cracking[1]	1,500 ft (457.2 m)	300 m	1,584 ft	Three cracks along the entire section length
Transverse cracking (length), flexible pavements	500 ft (152.4 m)	100 m	528 ft	One crack every 12 ft (3.65 m)
Number of transverse cracks, flexible pavements[1]	42	28	44	One crack each 3.65 m (12 ft)
Transverse cracking (length), rigid pavements	375 ft (114 m)	75 m	396 ft	One crack per slab (16-ft (4.87-m) joint spacing)
Number of transverse cracks, rigid pavements[1]	31	21	33	One crack per slab (16-ft (4.87-m) joint spacing)

[1]Data included for convenience. The analyses were conducted using the measured crack lengths and alligator cracked areas.

After selecting the proper mathematical function, the least squares regression technique was used to determine the statistical parameters of the selected mathematical functions. The least squares regression technique is based on minimizing the sum of the squared differences (error) between the calculated and the measured data.(3) To expedite the analyses, a MATLAB®-based computer program was written to complete the following functions for each pavement condition and distress dataset of each LTPP test section and for each pavement treatment type (see the program flowchart in figure 51):

- Read all available time-series data from a Microsoft® Excel spreadsheet.
- Separate the data into two parts, before and after treatment.

- Check the available number of time-series data points before and after treatment.

- If three or more data points are available that were collected before and/or after the treatment, use the proper mathematical function to fit the data and obtain the statistical parameters of the function.

- Organize the results (section identification, treatment date and type, climatic region, traffic, the last collected before treatment data point, the first collected after treatment data point, the number of before treatment and after treatment data points, and the statistical parameters) in a Microsoft® Excel spreadsheet format.

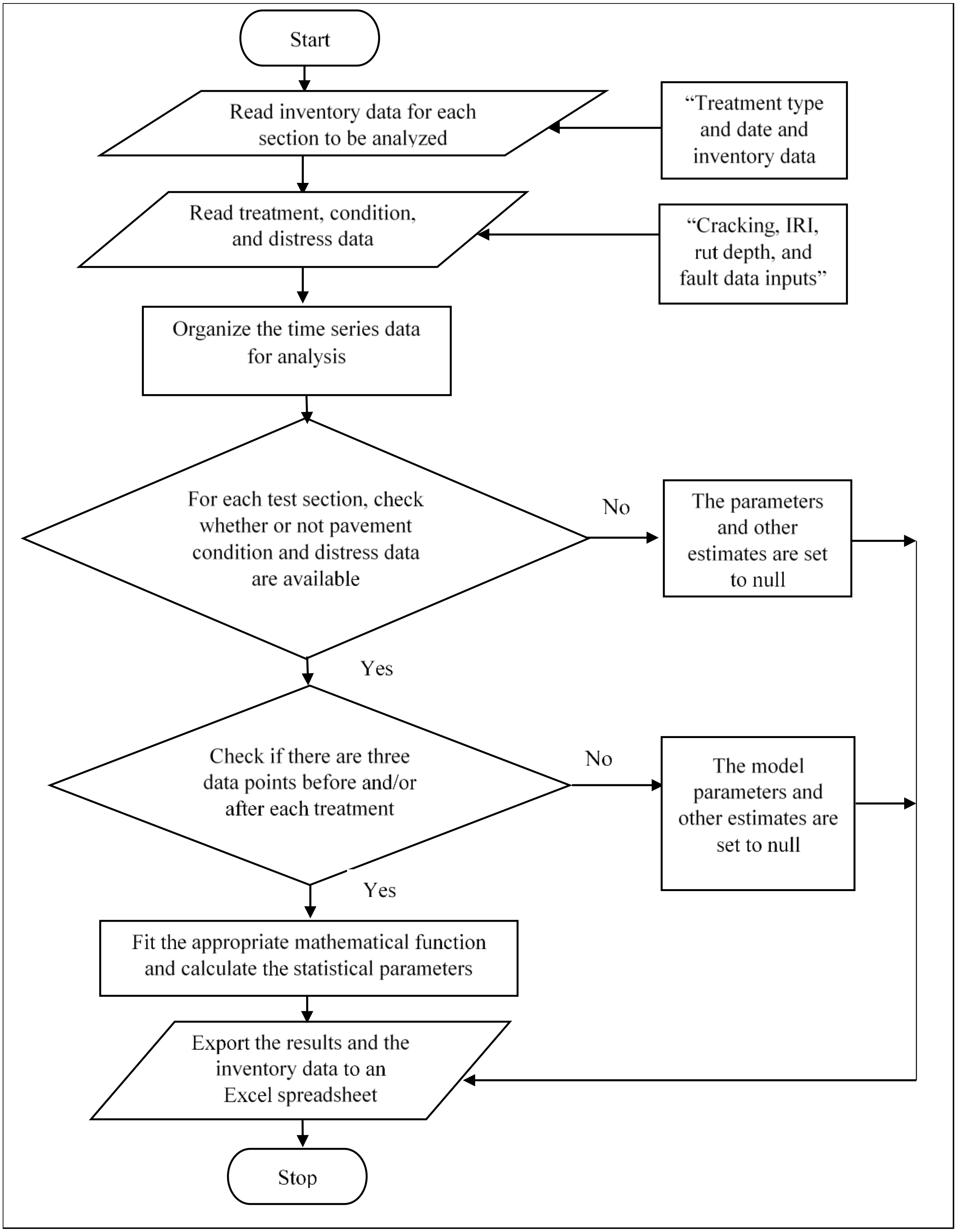


Figure 51. Illustration. Flowchart of the MATLAB® program.

The MATLAB® output data were then subjected to further analyses to estimate the following parameters for each test section and for each treatment type:

- The before treatment RFP, RSP, and CS.
- The after treatment RFP, RSP, and CS.
- Any changes in RFP or RSP resulting from the pavement treatment. The change in RFP or RSP is defined as the differences between the before treatment RFPs or RSPs and the after treatment RFPs or RSPs.
- The time period for the reoccurrence of the previous pavement conditions (the last collected data point before the application of the preservation treatment).
- Instantaneous change in the pavement conditions and distresses due to treatment (also called PJ).

IMPACTS OF CLIMATIC REGIONS, DRAINAGE, AND AC THICKNESS ON PAVEMENT PERFORMANCE USING THE LTPP SPS-1 TEST SECTIONS

Recall that the main objective of the SPS-1 experiment is to study the effects of the conditions in climatic regions and the following structural factors on pavement performance:[65]

- Presence or absence of a drainage layer.
- AC thickness (4 or 7 inches (102 or 178 mm)).
- Base type (dense-graded aggregate base, asphalt-treated base, permeable asphalt-treated base, or a combination thereof).
- Base thickness (8, 12, or 16 inches (203, 305, or 406 mm).

The analyses of the impacts of the various variables were accomplished in the following steps:

- **Step 1**: For each pavement test section in the SPS-1 experiment, each of the time-dependent pavement condition (IRI) and distress (rut depth, and alligator, transverse, and longitudinal cracking) data were used to calculate RFP and RSP of that section from the time of construction to the time when the pavement condition or distress reached the appropriate threshold values. The reason for calculating RFP and RSP from the construction data (surface age is 0 years) was that the dates of construction and last data collection for different test sections were not the same. This implies that the reference time for all SPS-1 test sections was taken as the date of construction.
- **Step 2**: For each pavement condition and distress type, the resulting RFPs and RSPs and other inventory data (such as SHRP ID, State, AC thickness, drainage, base type and thickness, and so forth) were then organized into a Microsoft® Excel spreadsheet format.

- **Step 3**: For each SHRP ID and for each pavement condition and distress type, the minimum and maximum RFPs and RSPs and their averages were calculated and listed in the Microsoft® Excel spreadsheet.

- **Step 4:** The data were then divided into the following climatic regions, groups, and subgroups. The main objective of the division was to separate the design variables affecting pavement performance.

    - **1—Climatic Regions**: The results were divided into four climatic regions: WF, WNF, DF, and DNF.

    - **2—AC Thickness Groups**: The results in each climatic region were then divided into two groups based on the thickness of the AC (4 and 7 inches (102 or 178 mm)).

    - **3—Drainage Subgroup**: The results in each AC thickness group were then divided into two drainage subgroups (presence and absence of drainage). It should be noted that various attempts were made to divide the results in each drainage subgroup into base thickness and base type subgroups and into three traffic levels. Unfortunately, none of the attempts was successful because any further division yielded an insignificant number of test sections in each subgroup such that no decisions could be made with any level of certainty. Therefore, the impacts of the base thickness and type were not studied any further.

The impacts of the conditions in the four climatic regions (WF, WNF, DF, and DNF), the AC thickness (4 and 7 inches (102 or 178 mm)), and drainable and undrainable bases on the pavement performance in terms of RFP and RSP were analyzed. The detailed results of the analyses (the calculated and rounded minimum, maximum, and average RFPs or RSPs for the SPS-1 test sections) were submitted to FHWA and are available from the LTPP Customer Support Services.[79] For convenience, the detailed results are summarized in table 29 through table 34.

The data in table 29 through table 34 address the impact of the climatic regions, the AC thickness, and drainable and undrainable bases on the pavement performance (in terms of RFPs and RSPs) of the SPS-1 test sections. The figures in the tables (which are rounded to whole numbers) indicate the differences in years in RFPs or RSPs of the SPS-1 test sections having the column heading parameters compared with RFPs and RSPs of the SPS-1 test sections having the row heading parameters. Thus, in table 29 through table 33, the diagonal indicated by asterisks represents the line of symmetry. If the table were folded along the diagonal, the aligned numbers from above and from below the diagonal would be the same but with different sign. The proper reading of the data in the tables is illustrated in the two examples following the tables.

Table 29. Summary of the results of analyses of the impacts of design factors on RFP of LTPP SPS-1 test sections based on IRI (years).

Climatic Region, AC Thickness, and Drainage Subgroup			**Climatic Region, AC Thickness, and Drainage Subgroup**															
			WF				WNF				DF				DNF			
			4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	4-inch AC	D	*	–5	0	–2	2	2	2	2	0	2	2	2	2	1	2	2
		ND	5	*	4	3	7	7	7	7	5	7	7	7	7	6	7	7
	7-inch AC	D	0	–4	*	–1	2	2	2	2	1	2	2	2	2	1	2	2
		ND	2	–3	1	*	4	3	4	4	2	4	4	4	4	3	4	4
WNF	4-inch AC	D	–2	–7	–2	–4	*	0	0	0	–2	0	0	0	0	–1	0	0
		ND	–2	–7	–2	–3	0	*	0	0	–1	0	0	0	0	–1	0	0
	7-inch AC	D	–2	–7	–2	–4	0	0	*	0	–2	0	0	0	0	–1	0	0
		ND	–2	–7	–2	–4	0	0	0	*	–2	0	0	0	0	–1	0	0
DF	4-inch AC	D	0	–5	–1	–2	2	1	2	2	*	2	2	2	2	1	2	2
		ND	–2	–7	–2	–4	0	0	0	0	–2	*	0	0	0	–1	0	0
	7-inch AC	D	–2	–7	–2	–4	0	0	0	0	–2	0	*	0	0	–1	0	0
		ND	–2	–7	–2	–4	0	0	0	0	–2	0	0	*	0	–1	0	0
DNF	4-inch AC	D	–2	–7	–2	–4	0	0	0	0	–2	0	0	0	*	–1	0	0
		ND	–1	–6	–1	–3	1	1	1	1	–1	1	1	1	1	*	1	1
	7-inch AC	D	–2	–7	–2	–4	0	0	0	0	–2	0	0	0	0	–1	*	0
		ND	–2	–7	–2	–4	0	0	0	0	–2	0	0	0	0	–1	0	*

* Indicates the line of symmetry along the diagonal of the table.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 30. Summary of the results of analyses of the impacts of design factors on RFP/RSP of LTPP SPS-1 test sections based on rut depth (years).

Climatic Region, AC Thickness, and Drainage Subgroup			Climatic Region, AC Thickness, and Drainage Subgroup															
			WF				WNF				DF				DNF			
			4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	4-inch AC	D	*	–5	–1	–3	5	5	5	5	4	6	6	6	6	6	6	6
		ND	5	*	4	2	10	10	10	10	9	11	11	11	11	11	11	11
	7-inch AC	D	1	–4	*	–3	6	6	5	6	5	7	7	7	7	7	7	7
		ND	3	–2	3	*	8	9	8	9	7	9	9	9	9	9	9	9
WNF	4-inch AC	D	–5	–10	–6	–8	*	0	0	1	–1	1	1	1	1	1	1	1
		ND	–5	–10	–6	–9	0	*	0	0	–1	1	1	1	1	1	1	1
	7-inch AC	D	–5	–10	–5	–8	0	0	*	1	–1	1	1	1	1	1	1	1
		ND	–5	–10	–6	–9	–1	0	–1	*	–2	1	1	1	1	1	1	1
DF	4-inch AC	D	–4	–9	–5	–7	1	1	1	2	*	2	2	2	2	2	2	2
		ND	–6	–11	–7	–9	–1	–1	–1	–1	–2	*	0	0	0	0	0	0
	7-inch AC	D	–6	–11	–7	–9	–1	–1	–1	–1	–2	0	*	0	0	0	0	0
		ND	–6	–11	–7	–9	–1	–1	–1	–1	–2	0	0	*	0	0	0	0
DNF	4-inch AC	D	–6	–11	–7	–9	–1	–1	–1	–1	–2	0	0	0	*	0	0	0
		ND	–6	–11	–7	–9	–1	–1	–1	–1	–2	0	0	0	0	*	0	0
	7-inch AC	D	–6	–11	–7	–9	–1	–1	–1	–1	–2	0	0	0	0	0	*	0
		ND	–6	–11	–7	–9	–1	–1	–1	–1	–2	0	0	0	0	0	0	*

* Indicates the line of symmetry along the diagonal of the table.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 31. Summary of the results of analyses of the impacts of design factors on RSP of LTPP SPS-1 test sections based on alligator cracking (years).

Climatic Region, AC Thickness, and Drainage Subgroup			**Climatic Region, AC Thickness, and Drainage Subgroup**															
			WF				WNF				DF				DNF			
			4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	4-inch AC	D	*	–4	–3	–3	0	0	2	2	–3	–5	–2	–3	–5	–6	–1	–2
		ND	4	*	1	1	4	3	5	5	0	–2	2	1	–1	–2	3	2
	7-inch AC	D	3	–1	*	0	3	2	4	4	–1	–3	1	0	–2	–3	2	1
		ND	3	–1	0	*	3	3	5	5	0	–2	1	0	–1	–3	2	2
WNF	4-inch AC	D	0	–4	–3	–3	*	–1	1	1	–4	–6	–2	–3	–5	–6	–1	–2
		ND	0	–3	–2	–3	1	*	2	2	–3	–5	–2	–3	–4	–6	–1	–1
	7-inch AC	D	–2	–5	–4	–5	–1	–2	*	0	–5	–7	–4	–5	–6	–8	–3	–3
		ND	–2	–5	–4	–5	–1	–2	0	*	–5	–7	–4	–4	–6	–8	–3	–3
DF	4-inch AC	D	3	0	1	0	4	3	5	5	*	–2	1	1	–1	–3	2	2
		ND	5	2	3	2	6	5	7	7	2	*	3	2	1	–1	4	4
	7-inch AC	D	2	–2	–1	–1	2	2	4	4	–1	–3	*	–1	–2	–4	1	0
		ND	3	–1	0	0	3	3	5	4	–1	–2	1	*	–2	–3	2	1
DNF	4-inch AC	D	5	1	2	1	5	4	6	6	1	–1	2	2	*	–1	3	3
		ND	6	2	3	3	6	6	8	8	3	1	4	3	1	*	5	4
	7-inch AC	D	1	–3	–2	–2	1	1	3	3	–2	–4	–1	–2	–3	–5	*	0
		ND	2	–2	–1	–2	2	1	3	3	–2	–4	0	–1	–3	–4	0	*

* Indicates the line of symmetry along the diagonal of the table.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 32. Summary of the results of analyses of the impacts of design factors on RSP of LTPP SPS-1 test sections based on longitudinal cracking (years).

Climatic Region, AC Thickness, and Drainage Subgroup			Climatic Region, AC Thickness, and Drainage Subgroup															
			WF				WNF				DF				DNF			
			4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	4-inch AC	D	*	–1	–1	0	5	6	6	5	1	0	–1	0	6	5	6	7
		ND	1	*	0	1	6	7	7	6	1	1	0	1	7	6	6	8
	7-inch AC	D	1	0	*	1	6	7	7	6	2	1	0	1	7	6	7	8
		ND	0	–1	–1	*	5	6	6	5	0	0	–1	0	6	5	5	7
WNF	4-inch AC	D	–5	–6	–6	–5	*	1	1	0	–5	–5	–6	–5	1	0	0	2
		ND	–6	–7	–7	–6	–1	*	0	–2	–6	–6	–7	–6	–1	–1	–1	1
	7-inch AC	D	–6	–7	–7	–6	–1	0	*	–1	–6	–6	–7	–6	0	–1	–1	1
		ND	–5	–6	–6	–5	0	2	1	*	–4	–5	–5	–5	1	0	1	2
DF	4-inch AC	D	–1	–1	–2	0	5	6	6	4	*	0	–1	–1	5	4	5	7
		ND	0	–1	–1	0	5	6	6	5	0	*	–1	0	6	5	5	7
	7-inch AC	D	1	0	0	1	6	7	7	5	1	1	*	1	7	6	6	8
		ND	0	–1	–1	0	5	6	6	5	1	0	–1	*	6	5	6	7
DNF	4-inch AC	D	–6	–7	–7	–6	–1	1	0	–1	–5	–6	–7	–6	*	–1	0	1
		ND	–5	–6	–6	–5	0	1	1	0	–4	–5	–6	–5	1	*	1	2
	7-inch AC	D	–6	–6	–7	–5	0	1	1	–1	–5	–5	–6	–6	0	–1	*	2
		ND	–7	–8	–8	–7	–2	–1	–1	–2	–7	–7	–8	–7	–1	–2	–2	*

* Indicates the line of symmetry along the diagonal of the table.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 33. Summary of the results of analyses of the impacts of design factors on RSP of LTPP SPS-1 test sections based on transverse cracking (years).

Climatic Region, AC Thickness, and Drainage Subgroup			Climatic Region, AC Thickness, and Drainage Subgroup															
			WF				WNF				DF				DNF			
			4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC		4-inch AC		7-inch AC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	4-inch AC	D	*	1	2	1	3	3	4	3	–3	2	–3	0	–1	2	2	0
		ND	–1	*	2	1	2	2	3	2	–4	1	–4	0	–1	1	1	0
	7-inch AC	D	–2	–2	*	–1	1	0	1	1	–6	0	–5	–2	–3	–1	0	–2
		ND	–1	–1	1	*	2	1	2	2	–5	1	–4	–1	–2	0	1	–1
WNF	4-inch AC	D	–3	–2	–1	–2	*	–1	0	0	–7	–1	–6	–3	–4	–1	–1	–3
		ND	–3	–2	0	–1	1	*	1	1	–6	0	–5	–2	–3	–1	0	–2
	7-inch AC	D	–4	–3	–1	–2	0	–1	*	0	–7	–1	–6	–3	–4	–2	–1	–3
		ND	–3	–2	–1	–2	0	–1	0	*	–7	–1	–6	–3	–4	–1	–1	–3
DF	4-inch AC	D	3	4	6	5	7	6	7	7	*	6	1	4	3	5	6	4
		ND	–2	–1	0	–1	1	0	1	1	–6	*	–5	–2	–3	0	0	–2
	7-inch AC	D	3	4	5	4	6	5	6	6	–1	5	*	3	2	5	5	3
		ND	0	0	2	1	3	2	3	3	–4	2	–3	*	–1	1	2	0
DNF	4-inch AC	D	1	1	3	2	4	3	4	4	–3	3	–2	1	*	2	3	1
		ND	–2	–1	1	0	1	1	2	1	–5	0	–5	–1	–2	*	0	–1
	7-inch AC	D	–2	–1	0	–1	1	0	1	1	–6	0	–5	–2	–3	0	*	–2
		ND	0	0	2	1	3	2	3	3	–4	2	–3	0	–1	1	2	*

* Indicates the line of symmetry along the diagonal of the table.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 34. Summary of the results of analyses of the effects of climatic regions on the performance of the LTPP SPS-1 test sections.

Condition or Distress Type	Climatic Region	Climatic Regions and the Percent of Test Sections Where RFP/RSP Was Better, Equal, or Worse Than Other Climatic Regions											
		WF			WNF			DF			DNF		
		Better	Same	Worse	Better	Same	Worse	Better	Same	Worse	Better	Same	Worse
IRI	WF	—	—	—	58	4	38	56	28	17	58	42	0
	WNF	38	4	58	—	—	—	77	6	17	8	84	8
	DF	17	28	56	17	6	77	—	—	—	17	72	11
	DNF	0	42	58	8	84	8	11	72	17	—	—	—
Rut depth	WF	—	—	—	83	17	0	82	13	5	73	27	0
	WNF	0	17	83	—	—	—	23	68	9	9	91	0
	DF	5	13	82	9	68	23	—	—	—	0	100	0
	DNF	0	27	73	0	91	9	0	100	0	—	—	—
Alligator cracking	WF	—	—	—	67	8	25	42	4	54	42	8	50
	WNF	25	8	67	—	—	—	38	8	54	13	0	87
	DF	54	4	42	54	8	38	—	—	—	46	17	38
	DNF	50	8	42	87	0	13	38	17	46	—	—	—
Longitudinal cracking	WF	—	—	—	88	8	4	46	4	50	92	8	0
	WNF	4	8	88	—	—	—	8	29	63	42	42	16
	DF	50	4	46	63	29	8	—	—	—	67	29	4
	DNF	0	8	92	16	42	42	4	29	67	—	—	—
Transverse cracking	WF	—	—	—	54	33	13	42	12	46	46	21	33
	WNF	13	33	54	—	—	—	38	54	8	7	50	42
	DF	46	12	42	8	54	38	—	—	—	42	46	12
	DNF	33	21	46	42	50	7	12	46	42	—	—	—

— Indicates no data.

- **Example 1:** This example illustrates how to read the data in table 29 through table 33 using example data from table 29. The first four numbers in the first row below the top headings imply the following:
    - The average RFP of test sections located in the WF region having 4-inch (102-mm)-thick AC layers and undrainable bases was 5 years shorter (–5) than compatible test sections with drainable bases.
    - The average RFP of test sections located in the WF region having 7-inch (178-mm)-thick AC layers and drainable bases was the same (0 years) as those having 4-inch (102-mm) thick AC layers with drainable bases.
    - The average RFP of test sections located in the WF region having 7-inch (178-mm)-thick AC layers and undrainable bases was 2 years shorter (2 years) than those having 4-inch (102-mm)-thick AC layers with drainable bases.
    - The average RFP of test sections located in the WNF region having 4-inch (102-mm)-thick AC layers and drainable bases was 2 years longer (2 years) than those located in the WF region having 4-inch (102-mm)-thick AC layers and drainable bases.
- **Example 2:** This example illustrates how to read the data in table 34. The data in the table address the impact of the climatic regions on the pavement condition (IRI) and distresses (rut depth and alligator, longitudinal, and transverse cracking). The numbers in the table indicate the percent of the test sections with column heading parameters that performed better, the same, or worse compared with the test sections having the row heading parameters. For example, for IRI, the six numbers (three numbers in each of the top two populated rows in the three columns under the heading WF) and for rut depth, the six numbers (three numbers in each of the next two populated rows) under the same three columns heading (WF) imply the following:
    - In terms of IRI, 38 percent of the test sections located in the WF region performed better than those located in the WNF region, 4 percent performed the same, and 58 percent performed worse.
    - In terms of IRI, 17 percent of the test sections located in the WF region performed better than compatible sections located in the DF region, 28 percent performed the same, and 56 percent performed worse.
    - In terms of rut depth, none of the test sections located in the WF region performed better than those located in the WNF region, 17 percent performed the same, and 83 percent performed worse.
    - In terms of rut depth, 5 percent of the test sections located in the WF region performed better than compatible sections located in the DF region, 13 percent performed the same, and 82 percent performed worse.

Consistent with this explanation of table 29 through table 33 and table 34, in the following subsections, the discussion of the results shown in those tables is organized according to the pavement condition and distress.

IRI

The average calculated RFPs listed in table 29 indicate that the differences between the average RFP of test sections located in different climatic regions and that have 4- or 7-inch (102- or 178-mm)-thick AC layers with drainable and undrainable bases varied from –1 to 7 years. Because the 1-year difference was not significant and was within the data variability, it was considered a value of zero in the following discussion. Nevertheless, the data in table 29 indicate the following:

- In the WF region, the average RFP of test sections having 4-inch (102-mm) AC thickness and undrainable bases was about 5 years shorter than the average RFP of compatible test sections with drainable bases. The 5-year difference decreased to 2 years when the AC thickness of the undrainable test sections increased from 4 to 7 inches (102 to 178 mm). That is, the average RFP of test sections having 7-inch (178-mm)-thick AC layers and undrainable bases was about 3 years longer than test sections having 4-inch (102-mm)-thick AC layers and undrainable bases. Finally, for test sections having 7-inch (178-inch)-thick AC layers and drainable bases, the average RFP was the same as test sections having 4-inch (102-mm)-thick AC layers and drainable bases and 4 years longer compared with test sections having 4-inch (102-mm)-thick AC layers and undrainable bases.

- Also in the WF region, the average RFP of the test sections having 4-inch (102 mm)-thick and 7-inch (178-mm)-thick AC layers and drainable and undrainable bases was 2 to 7 years shorter than compatible test sections located in the other three climatic regions.

- In the WNF, DF, and DNF climatic regions, RFP of the test sections having 4-inch (102 mm)-thick and 7-inch (178-mm)-thick AC layers and drainable and undrainable bases was 2 to 7 years longer than compatible test sections located in the WF region.

- In the WNF region, RFP/RSP of the test sections having 4-inch (102 mm)-thick and 7-inch (178-mm)-thick AC layers and drainable and undrainable bases was almost the same.

- In the DF region, the average RFP of the test sections having 4-inch (102-mm)-thick AC layers and drainable bases was about 2 years shorter than test sections having 4-inch (102 mm)-thick and 7-inch (178-mm)-thick AC layers and drainable and undrainable bases located in the WNF and DNF climatic regions.

- In the DNF region, test sections having 4-inch (102 mm)-thick and 7-inch (178-mm)-thick AC layers and drainable and undrainable bases performed almost the same. That is, the existence of drainable bases and thicker AC layers did not affect the pavement performance in terms of IRI.

The impact of the climatic regions on pavement performance in terms of IRI is summarized in table 34. The data in the table indicate the following:

- Fifty-eight percent of the test sections located in the WF regions performed worse, 4 percent performed the same, and 38 percent performed better than compatible test sections located in the WNF regions.

- Fifty-six percent of the test sections located in the WF regions performed worse, 28 percent performed the same, and 17 percent performed better than compatible test sections located in the DF regions.

- Seventy-seven percent of the test sections located in the DF region performed better than compatible sections located in the WNF region while test sections located in the DNF and WNF performed almost the same.

There are several significant initial conclusions regarding pavement performance that could be drawn from the findings listed in table 29 and table 34. However, these conclusions would be based on pavement roughness only. To make these conclusions a part of the global pavement performance, the research team decided to include them in the summary, conclusions, and recommendations section that follows the discussion of all the other distress types.

Rut Depth

The average calculated RFPs or RSPs listed in table 30 indicate the following:

- In the WF regions, the average RFP/RSP of the SPS-1 test sections having 4-inch (102-mm)-thick AC layers and undrainable bases was 5 years shorter than the average RSP of compatible test sections with drainable bases. Further, the average RFP/RSP of SPS-1 test sections having 7-inch (178-mm)-thick AC layers and undrainable bases was 3 years shorter than compatible sections with drainable bases.

- All SPS-1 test sections having 4- or 7-inch (102- to 178-mm)-thick AC layers and drainable or undrainable bases located in the WNF, DF, and DNF climatic regions performed significantly better than compatible test sections located in the WF regions. The differences in RFP/RSP varied from 5 to 11 years.

The impact of the climatic regions on pavement performance in terms of rut depth is summarized in table 34. The data in the table indicate the following:

- Eighty-three percent, 82 percent, and 73 percent of the SPS-1 test sections located in the WF regions performed worse than compatible test sections located, respectively, in the WNF, DF, and DNF regions.

- Almost all SPS-1 test sections located in the DNF region performed the same as compatible test sections located in the WNF and DF climatic regions.

The conclusions are included in the summary, conclusions, and recommendations section following the discussion of the other distress types.

Alligator Cracking

The average calculated RSPs listed in table 31 indicate the following, on average:

- In the WF regions, the average RSP of test sections having 4-inch (102-mm)-thick AC layers and undrainable bases was 4 years shorter than the average RSP of test sections having 4-inch (102-mm)-thick AC layers and drainable bases. Further, RSP of SPS-1 test sections having 7-inch (178-mm)-thick AC layers and drainable and undrainable bases was almost the same.

- SPS-1 test sections having 4- and 7-inch (102- to 178-mm)-thick AC layers and drainable or undrainable bases located in the WNF region had longer RSPs compared with compatible test sections located in the WF region.

- All SPS-1 test sections located in the DNF regions had shorter RSPs than those located in the WNF region. The differences in RSP varied from 1 to 8 years.

The impact of the climatic regions on pavement performance in terms of alligator cracking is summarized in table 34. The data in the table indicate the following:

- Sixty-seven percent of the SPS-1 test sections located in the WF regions showed worse, 8 percent showed the same, and 25 percent showed better performance than compatible test sections located in the WNF region.

- Fifty-four percent and 50 percent of the SPS-1 test sections located in the WF regions performed better than compatible test section located, respectively in the DF and DNF regions. Forty-two percent performed worse than compatible test sections located in the DF and DNF regions. Hence, statistically speaking, SPS-1 test sections located in the WF region had slightly better performance than those in the DF and DNF regions.

- Forty-six percent of the SPS-1 test sections located in the DNF regions performed better than compatible test sections located in the DF region, whereas 38 percent performed worse.

- Finally, 87 percent of the SPS-1 test sections located in the DNF region performed worse than compatible test sections located in the WNF region, and only 13 percent performed better.

The conclusions are included in the summary, conclusions, and recommendations section following the discussion of the other distress types.

Longitudinal Cracking

The average calculated RSPs listed in table 32 indicate the following, on average:

- All SPS-1 test sections having 4-inch (102-mm)-thick and 7-inch (178-mm)-thick AC layers and drainable and undrainable bases and located in WF climatic region showed almost the same RSP in terms of longitudinal cracking.

- RSPs of most SPS-1 test sections having 4-inch (102-mm)-thick and 7-inch (178-mm)-thick AC layers and drainable and undrainable bases located in the WNF, DF, and DNF regions were longer than compatible test sections located in the WF region. The differences in RSP varied from 1 to 8 years.

- RSPs of almost all SPS-1 test sections having 4-inch (102-mm)-thick and 7-inch (178-mm)-thick AC layers and drainable and undrainable bases located in the DF and DNF regions were shorter than the RSP of compatible test sections located in the WNF region. The differences in RSPs varied from 1 to 7 years.

- Finally, the RSP of all SPS-1 test sections located in the DNF region was longer than the RSP of compatible sections located in the DF region. The differences in RSPs varied from 1 to 7 years.

The impact of the climatic regions on pavement performance in terms of longitudinal cracking is summarized in table 34. The data in the table indicate the following:

- Eighty-eight percent and 92 percent of the SPS-1 test sections located in the WF regions performed worse than compatible test sections located, respectively, in the WNF and DNF regions.

- Statistically speaking, the performance of SPS-1 test sections located in the WF region was almost the same as those located in the DF region. The data indicate that 50 percent performed better, and 46 percent performed worse.

- Forty-two percent of the SPS-1 test sections located in the DNF regions performed better than compatible test sections located in the WNF region, 42 percent performed the same, and only 16 percent performed worse.

- Sixty-seven percent of the SPS-1 test sections located in the DNF regions performed better than compatible test sections located in the DF regions while 29 percent performed the same.

Once again, the conclusions are included in the summary and conclusions section following the discussion of the other distress types.

Transverse Cracking

The average calculated RSPs listed in table 33 indicate the following:

- In the WF region, the average RSPs of test sections having 4- and 7-inch (102- and 178-mm)-thick AC layers and undrainable bases were almost the same as compatible sections with drainable bases. Further, the average RSP for SPS-1 test sections having 7-inch (178-mm)-thick AC layers and drainable bases was 2 years longer than test sections having 4-inch (102-mm)-thick AC layers and drainable bases.

- All SPS-1 test sections located in the WNF regions had 1- to 4-year longer RSPs than compatible sections located in the WF region.

- The majority of the SPS-1 test sections located in the DF and DNF regions had shorter RSPs than compatible sections located in the WF and WNF regions.

- The majority of the SPS-1 test sections having 4- and 7-inch (102- and 178-mm)-thick AC layers and drainable and undrainable bases and located in the DNF region had longer RSPs than compatible test sections located in the DF region.

The impact of the climatic regions on pavement performance in terms of transverse cracking is summarized in table 34. The data in the table indicate the following:

- Fifty-four percent of the SPS-1 test sections located in the WF regions performed worse, 13 percent performed better, and 33 percent performed the same as compatible test sections located in the WNF region.

- Statistically speaking, the performance of SPS-1 test sections located in the WF and DF and DNF regions was the same.

- Forty-two percent of the SPS-1 test sections located in the WNF regions performed better, 50 percent performed the same, and 7 percent performed worse than compatible test sections located in the DNF region.

- Forty-two percent of the SPS test sections located in the DNF regions performed better, 46 percent performed the same, and 12 percent performed worse than compatible test sections located in the DF region.

Summary, Conclusions, and Recommendations for LTPP SPS-1

The performance of each SPS-1 test section was analyzed using the available time-series IRI; rut depth; alligator, longitudinal, and transverse cracking data; and the proper mathematical functions. The results of the analyses were then expressed in terms of RFP for IRI, RFP/RSP for rut depth, and RSP for each cracking type. The test sections and their performance (RFP and RSP) were then tabulated using the SHRP IDs, climatic regions, AC thicknesses, and drainable or undrainable bases. Based on the results, the following conclusions were drawn:

- The climate in WF regions had a significant impact on pavement performance in terms of IRI, rut depth, and cracking. This conclusion was expected because of the repeated volume changes caused by freezing and thawing. These effects were reported previously by many researchers. Results of the analyses suggest that base drainage and AC thickness should be carefully examined in the pavement design in the WF region.

- Use of drainable bases decreased the impact on pavement performance in WF regions. This conclusion was expected, and it was reported in the 1993 *AASHTO Guide for Design of Pavement Structures*.[36]

- Increasing the thickness of the AC layer from 4 to 7 inches (102 to 178 mm) increased the frost protection of the lower layers, and hence it decreased the impact of the WF region. However, this option was not a cost-effective one.

- In the WF region, the inclusion of drainable base had slightly more favorable impact on pavement performance than increasing the AC thickness from 4 to 7 inches (102 to 178 mm).

- The other three climatic regions (WNF, DF, and DNF) did not affect the pavement performance in terms of rutting potential.

- The DF region had more adverse effects on cracking potential than those in the DNF region. This could be attributed to higher oxidation (aging) potential of the AC layer in the DF region.

- In the DNF regions, there were significantly higher adverse effects on cracking potential than in the WNF region. This could be attributed to higher solar radiation in the DNF region, which oxidizes the asphalt binder and makes AC more susceptible to cracking.

- The inclusion of drainable bases in the DF and DNF regions did not affect pavement performance in terms of RFP or RSP. This was expected because the volume and frequency of available water were low, and most rainfall occurred over short periods of time with most water running off the surface and not penetrating the pavement layers.

- In the WNF and DNF regions, the pavement performance in terms of IRI and rut depths of most SPS-1 test sections having 4- and 7-inch (102- and 178-mm)-thick AC layers and drainable and undrainable bases was very much similar. This scenario was substantially different for cracking potential as stated in previous conclusions. Similarly, in the DNF and DF regions, the pavement performance in terms of IRI and rut depths of most SPS-1 test sections having 4- and 7-inch (102- and 178-mm)-thick AC layers and drainable and undrainable bases was similar, Once again, this scenario was substantially different for cracking potential.

IMPACTS OF MAINTENANCE TREATMENTS ON PAVEMENT PERFORMANCE USING THE LTPP SPS-3 TEST SECTIONS

The main objective of SPS-3 experiment was to compare the performance of different maintenance treatments on flexible pavements compared with the control (untreated) test sections. The 81 SPS-3 test sites were initiated between 1990 and 1991 and were distributed across the United States and Canada. Each of the SPS-3 test sites consisted of 4 test sections for a total of 324 test sections. Fifty-one of the 81 test sites had control sections labeled 340. Each of the other 30 sites were linked to a GPS test section (listed in table 35 along with their SHRP ID), which could be used as control sections.[60]

Table 35. Linked GPS sections that serve as control sections.[60]

Site ID	Linked GPS Section ID
04_A300	4_1036
04_B300	4_1021
04_D300	4_1016
05_A300	5_3071
08_B300	8_2008
12_A300	12_9054
12_B300	12_3997
12_C300	12_4154
16_A300	16_1020
16_B300	16_1021
16_C300	16_1010
28_A300	28_1802
30_A300	30_1001
32_A300	32_1021
32_C300	32_2027
40_B300	40_1015
40_C300	40_4088
47_A300	47_3101
47_B300	47_3075
47_C300	47_1023
48_D300	48_2172
48_G300	48_1169
49_A300	49_1004
49_B300	49_1017
49_C300	49_1006
53_A300	53_1008
53_B300	53_1501
53_C300	53_1801
56_A300	56_1007
56_B300	56_7775

Each of the four SPS-3 test sections in each test site was subjected to one of the following treatments (note that the numbers in parentheses are the LTPP designation of the treatment; for example, the designation of the joint and crack sealing is 410):

- Thin overlay (310).
- Slurry seal (320).
- Crack seal (330).
- Aggregate seal coat; chip seal (350).

Several variables affect the performance of the treated pavement sections. These include climatic region, traffic, subgrade type, and the before treatment pavement condition and distress.

Unfortunately, these variables could be separated to analyze the effects of each on pavement performance. The reason is that separating the variables yields statistically insignificant numbers of test sections to be used in the analyses.

To illustrate, table 36 lists the number of test sections that were available for analyses based on separation of the following variables:

- Four treatment types.
- One pavement condition (IRI).
- Four pavement distress types.
- Four climatic regions (WF, WNF, DF, and DNF).
- Three traffic levels.

Table 36. Number of test sections that have before treatment and after treatment pavement condition, distress, and traffic data.

Condition or Distress Type	Treatment Type	Number of Test Sections by Climatic Region and Traffic Level											
		WF			WNF			DF			DNF		
		L	M	H	L	M	H	L	M	H	L	M	H
IRI	Thin overlay	8	4	4	8	2	6	3	4	3	1	0	1
	Slurry seal	6	4	4	6	3	6	3	3	2	1	0	1
	Crack seal	7	4	4	2	1	6	3	3	2	1	0	1
	Aggregate seal coat	5	4	4	7	2	5	3	3	3	0	1	1
Rut depth	Thin overlay	4	2	2	4	2	7	2	1	2	0	0	1
	Slurry seal	4	1	2	4	2	8	2	2	2	1	0	1
	Crack seal	4	1	3	2	0	6	3	2	2	0	0	1
	Aggregate seal coat	1	2	1	5	1	9	2	2	1	0	0	1
Alligator cracking	Thin overlay	4	2	4	4	2	5	1	0	0	0	0	0
	Slurry seal	1	0	3	5	0	5	0	0	0	0	0	0
	Crack seal	1	0	1	3	2	2	0	0	0	0	0	0
	Aggregate seal coat	2	0	3	4	0	3	0	0	0	0	0	0
Longitudinal cracking	Thin overlay	4	2	4	4	2	5	1	0	0	0	0	0
	Slurry seal	2	0	3	5	0	5	0	0	0	0	0	0
	Crack seal	1	0	3	2	3	3	0	0	0	0	0	0
	Aggregate seal coat	3	0	3	4	0	4	0	0	0	0	0	0
Transverse cracking	Thin overlay	4	2	4	4	2	5	1	0	0	0	0	0
	Slurry seal	1	0	3	5	0	4	0	0	0	0	0	0
	Crack seal	2	0	3	3	2	3	0	0	0	0	0	0
	Aggregate seal coat	2	0	3	4	0	3	0	0	0	0	0	0

Note: For each pavement condition and distress type, the test section was analyzed if the database contained at least one data point before treatment and/or three or more data points after treatment that could be modeled.
L = low traffic (0 to 60,000 yearly ESAL).
M = medium traffic (61,000 to 120,000 yearly ESAL).
H = high traffic (> 120,000 yearly ESAL).

It can be seen from the table that in some cells, especially in the DF and DNF regions and for some pavement distress types, the number of available test sections for analyses was not significant (ranges from 0 to 2). Therefore, the analyses were conducted to assess the impact of each treatment type in each climatic region and for each pavement condition and distress type. That is, the data were not separated based on traffic level, type of base and subbase, or type of roadbed.

Nevertheless, the analyses of the impacts of each of the four treatment types on pavement performance were accomplished using the following steps:

- **Step 1**: For each treated pavement test section in the SPS-3 experiment, the available pavement condition (IRI) and distress data in the LTPP database were used to calculate RFP and RSP of that section after treatment.

- **Step 2**: For each SPS-3 test section, each pavement condition and distress type, and for each pavement treatment type, the minimum and maximum RFPs and RSPs were calculated and tabulated. Further, the averages of RFP and RSP of all test sections located in the same climatic region were also calculated and tabulated.

- **Step 3**: The time-dependent pavement condition and distress data of each control section and/or linked GPS section in the SPS-3 experiment were used to calculate RFP and RSP of that section after the assignment date.

Results of the analyses are discussed per pavement condition and distress type in the next five subsections.

IRI

Listed in table 37 are the calculated minimum, maximum, and average RFPs based on IRI data for the SPS-3 test sections that were subjected to the same treatment type and located in the same climatic region. The table also includes the same data for the associated control sections. To assist the reader in interpreting the data, the numbers listed in the first row of the table, for example, indicate the following:

- There were 19 SPS-3 test sections in the WF region that were subjected to thin overlay and accepted for analyses. The minimum, maximum, and average RFP of the 19 SPS-3 test sections were 4, 20, and 16 years, respectively.

- There were 21 control sections in the WF region with a minimum RFP of 0 years, a maximum RFP of 19 years, and an average RFP of 11 years.

- The difference in the average RFP of the test sections and the average RFP of the control sections was 5 years. That is, on average, RFP of the treated sections was 5 years longer than the control sections.

Table 37. Impacts of various maintenance treatments and control section on pavement performance in terms of RFP based on IRI.

Climatic Region	Treatment Type	Remaining Functional Period (Years)								Difference in RFP (Years)
		Test Sections				Control Sections				
		Number of Sections	Min	Max	Average	Number of Sections	Min	Max	Average	
WF	Thin overlay	19	4	20	16	21	0	19	11	5
WNF		23	8	20	18	29	3	19	14	4
DF		13	5	20	17	13	2	19	12	5
DNF		3	3	13	9	4	3	18	11	–2
WF	Slurry seal	15	0	20	12	21	0	19	11	1
WNF		22	4	20	19	29	3	19	15	4
DF		13	4	20	14	13	2	19	13	1
DNF		2	9	20	15	4	3	18	11	4
WF	Crack seal	18	0	20	11	21	0	19	11	0
WNF		12	1	20	16	29	3	19	14	2
DF		13	3	20	15	13	2	18	12	3
DNF		4	6	20	14	4	3	18	11	3
WF	Aggregate seal coat	16	0	20	13	21	0	19	11	2
WNF		21	14	20	19	29	3	19	15	4
DF		13	1	20	14	13	2	19	13	1
DNF		3	4	10	7	4	3	18	11	4

Max = Maximum.
Min = Minimum.

Examination of the results of the analyses listed in table 37 indicates the following:

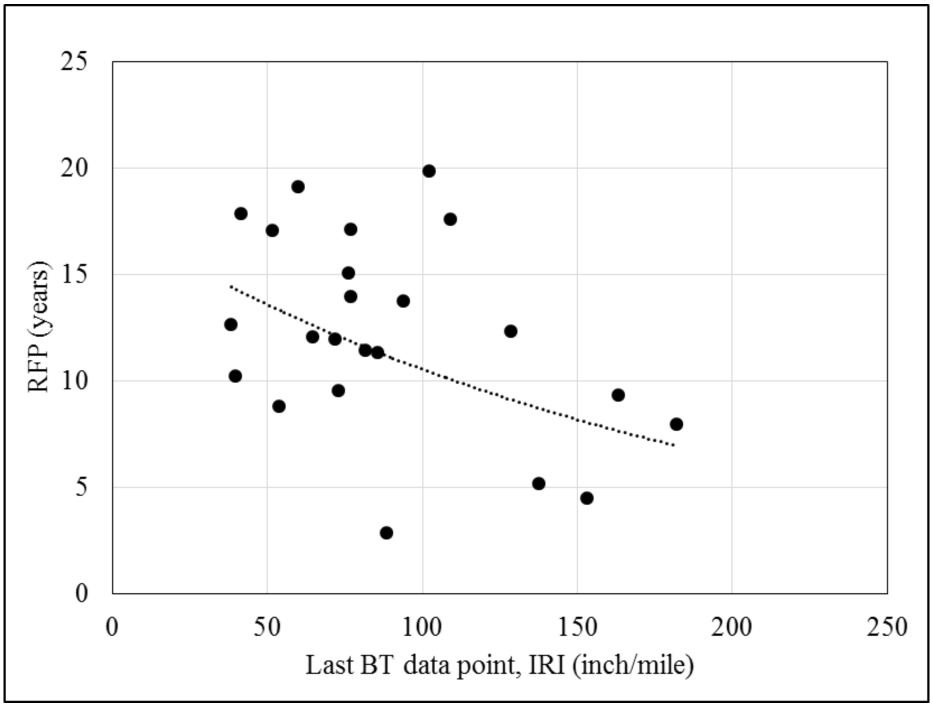
- Test sections that were subjected to thin overlay treatment performed better than the control sections by 5, 4, and 5 years in the WF, WNF, and DF regions, respectively, while they performed worse in the DNF region by 2 years. The reason for the latter was that the construction of the overlay caused increases in the IRI of all test sections in the DNF region. For example, the IRI of test section 04B310 increased from 88.564 inches/mi (1.3978 m/km) before the overlay to 95.901 inches/mi (1.5136 m/km) after the overlay.

- Test sections that were subjected to slurry seal performed better than the control sections by 1, 4, 1, and 4 years in the WF, WNF, DF, and DNF regions, respectively.

- Test sections that were subjected to crack seal performed better than the control sections by 2, 3, and 3 years in the WNF, DF, and DNF regions, respectively. Further, crack sealing had no impact on pavement performance in the WF region.

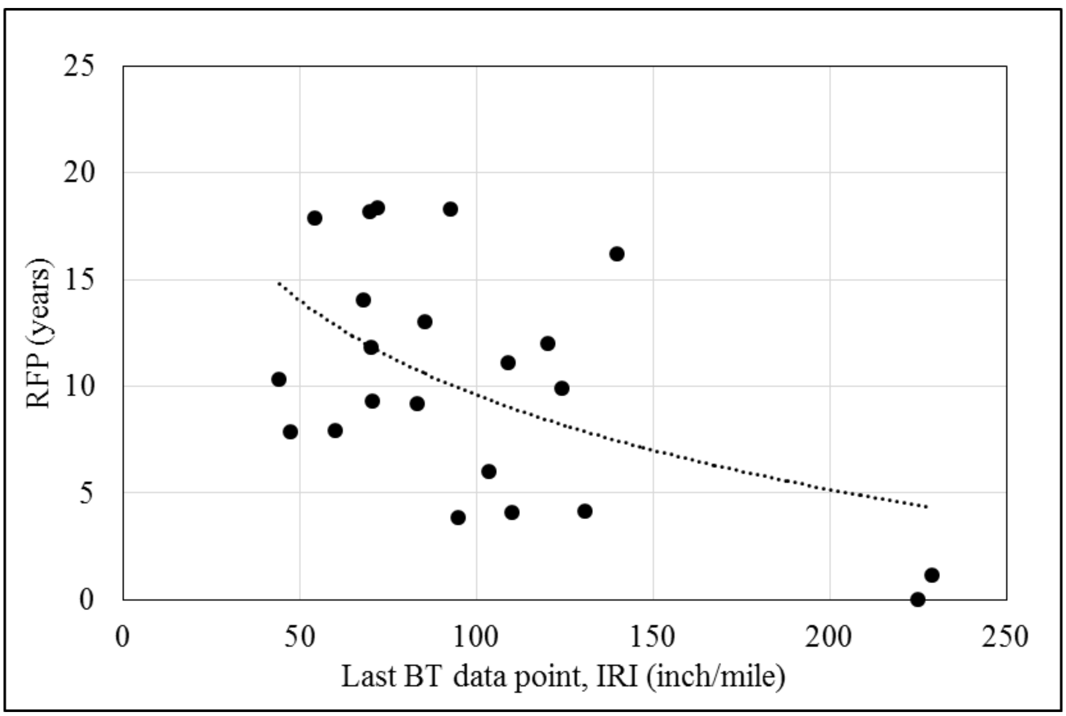
- Test sections that were subjected to aggregate seal coat performed better than the control sections by 2, 4, and 1 year in the WF, WNF, and DF regions, respectively, while they performed worse in the DNF regions by 4 years. Once again, the reason for the decreasing performance in the DNF regions was that construction of the aggregate seal coat increased the IRI of two of the three test sections.

For some of the SPS-3 test sections, the LTPP database contained one or more IRI data points before the sections were subjected to maintenance treatments. To assess the impact of the before treatment pavement conditions on the after treatment pavement performance, for each maintenance treatment type, RFPs after treatment were plotted against the last collected IRI data point before treatment. The results are shown in figure 52 through figure 55 for thin overlay, slurry seal, crack seal, and aggregate seal coat, respectively. Although the data in the figures are widely scattered, the general trend is that the higher the IRI is before treatment, the lower the RFP is after treatment. This finding was expected and supports the notion that maintaining pavement sections in good conditions pays higher dividends than treating deteriorated sections. Nevertheless, the scattering of the data in figure 52 through figure 55 was likely caused by differences in the original pavement cross sections, pavement materials, roadbed soil, climatic region, and traffic level. Unfortunately, the number of test sections subjected to the same traffic level bracket was so small that no decision regarding the impacts of traffic could be made with any level of certainty. Note the solid best fit curves in figure 52 through figure 55 are not intended to model the data. They show only the global trend, and therefore the inclusion of statistics such as standard error would be meaningless. As stated previously, the data in the figures are a function of many other variables that were not included in the analyses. Because separation of variables yielded data for few test sections (two or fewer), no decision could be made with any degree of certainty.



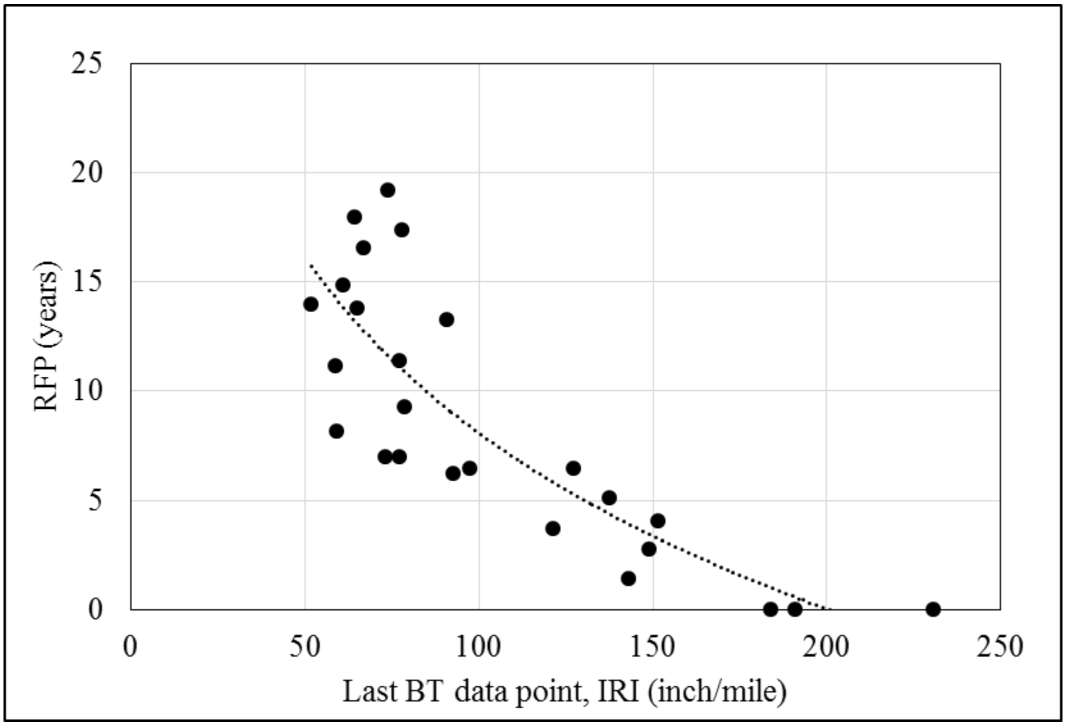
1 inch/mi = 0.0158 m/km.

Figure 52. Graph. After-treatment RFP versus before-treatment IRI of LTPP SPS-3 test sections subjected to thin overlay.



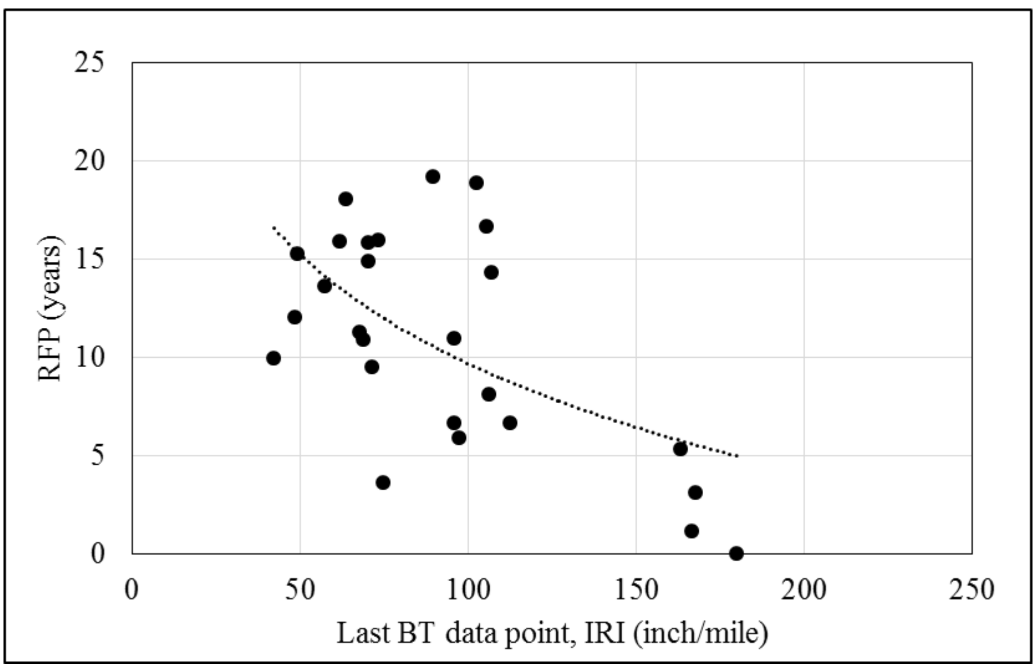
1 inch/mi = 0.0158 m/km.

Figure 53. Graph. After-treatment RFP versus before-treatment IRI of LTPP SPS-3 test sections subjected to slurry seal.



1 inch/mi = 0.0158 m/km.

Figure 54. Graph. After-treatment RFP versus before-treatment IRI of LTPP SPS-3 test sections subjected to crack seal.



1 inch/mi = 0.0158 m/km.

Figure 55. Graph. After-treatment RFP versus before-treatment IRI of LTPP SPS-3 test sections subjected to aggregate seal coat.

Rut Depth

Listed in table 38 are the calculated minimum, maximum, and average RFPs/RSPs based on rut depth data for the SPS-3 test sections that were subjected to the same treatment type and located in the same climatic region. The table also includes the same data for the associated control sections. The following summarizes the results:

- Test sections subjected to thin overlay performed better than the control sections by 7, 8, and 7 years in the WF, WNF, DF, respectively, and worse by 1 year in the DNF region.
- Test sections subjected to slurry seal performed better than the control sections by 2, 5, 4, and 3 years in the WF, WNF, DF, and DNF regions, respectively.
- Test sections subjected to crack seal performed better than the control sections by 1, 5, 7, and 2 years in the WF, WNF, DF, and DNF regions, respectively.
- Test sections subjected to aggregate seal coat performed better than the control sections by 3, 3, 5, and 9 years in the WF, WNF, DF, and DNF regions, respectively.

Similar to the IRI analyses, RSPs after treatment were plotted against the last measured rut depth data point before treatment. The results were submitted to FHWA and are available from the LTPP Customer Support Services.[(79)] The data indicate that deeper before treatment rut depths led to lower after treatment RFP/RSP or better performance in terms of rut depth after treatment. The scattering of data in the figures is mainly due to differences in the original pavement cross sections, pavement materials, roadbed soil, climatic region, and traffic level.

Table 38. Impacts of various maintenance treatments and control section on pavement performance in terms of RFP/RSP based on rut depth.

Climatic Region	Treatment Type	Remaining Functional/Structural Period (Years)								Difference in RFP/RSP (Years)
		Test Sections				Control Sections				
		Number of Sections	Min	Max	Average	Number of Sections	Min	Max	Average	
WF	Thin overlay	18	13	20	19	14	0	18	12	7
WNF		21	5	20	19	16	0	19	11	8
DF		8	9	20	19	8	0	19	12	7
DNF		4	0	20	10	3	0	16	11	–1
WF	Slurry seal	12	1	20	14	14	0	18	12	2
WNF		19	0	20	16	16	0	19	11	5
DF		10	1	20	16	8	0	19	12	4
DNF		3	1	20	14	3	0	16	11	3
WF	Crack seal	11	0	20	13	14	0	18	12	1
WNF		9	0	20	16	16	0	19	11	5
DF		9	2	20	18	8	0	18	11	7
DNF		3	0	20	13	3	0	16	11	2
WF	Aggregate seal coat	11	0	20	15	14	0	18	12	3
WNF		22	0	20	14	16	0	19	11	3
DF		8	0	20	17	8	0	19	12	5
DNF		2	0	20	20	3	0	16	11	9

Max = Maximum.
Min = Minimum.

Alligator Cracking

Listed in table 39 are the calculated minimum, maximum, and average RSPs based on alligator cracking for the SPS-3 test sections and located in the same climatic region. The table also includes the same data for the associated control sections. The results listed in the table indicate the following:

- Test sections subjected to thin overlay performed better than the control sections by 2 years in the WF and WNF regions, while they performed worse by 1 and 9 years in the DF and DNF regions, respectively.

- Test sections subjected to slurry seal performed worse than the control sections by 1, 3, and 7 years in the WF, DF, and DNF regions, respectively, while they performed the same in the WNF region.

- Test sections subjected to crack seal performed better than the control sections by 1 year in the DF region, while they performed worse by 2, 1, and 15 years in the WF, WNF, and DNF regions.

- Test sections subjected to aggregate seal coat performed better than the control sections by 2, 2, and 1 years in the WF, WNF, and DNF regions, respectively, while they performed worse by 1 year in the DF region.

Similar to the IRI and rut depths, for each treatment type, the RSPs of the test sections after treatment were plotted against the last collected alligator cracking data points before treatment. The results were submitted to FHWA and are available from the LTPP Customer Support Services.[79] In summary, the data indicate that as the alligator cracking increased, the after treatment RSPs decreased. That is, the data indicate that, on average, treating pavement sections at an early stage paid higher dividends than delayed treatment.

Table 39. Impacts of various maintenance treatments and control section on pavement performance in terms of RSP based on alligator cracking.

Climatic Region	Treatment Type	Remaining Structural Period (Years)								Difference in RSP (Years)
		Test Sections				Control Sections				
		Number of Sections	Min	Max	Average	Number of Sections	Min	Max	Average	
WF	Thin overlay	21	2	20	10	15	0	16	8	2
WNF		24	3	20	11	20	0	17	9	2
DF		11	4	20	11	8	6	18	12	–1
DNF		3	0	15	7	1	16	16	16	–9
WF	Slurry seal	16	0	20	7	15	0	16	8	–1
WNF		30	2	20	10	20	0	17	10	0
DF		10	3	20	9	8	6	18	12	–3
DNF		2	0	18	9	1	16	16	16	–7
WF	Crack seal	10	0	20	6	15	0	16	8	–2
WNF		22	0	20	8	20	0	17	9	–1
DF		11	0	20	13	8	5	18	12	1
DNF		2	0	2	1	1	16	16	16	–15
WF	Aggregate seal coat	15	2	20	10	15	0	16	8	2
WNF		18	4	20	12	20	0	17	10	2
DF		9	6	20	11	8	6	18	12	–1
DNF		2	13	20	17	1	16	16	16	1

Max = Maximum.
Min = Minimum.

Longitudinal Cracking

Listed in table 40 are the calculated minimum, maximum, and average RSPs based on longitudinal cracking for the SPS-3 test sections that were subjected to the same treatment type and located in the same climatic region. The table also includes the same data for the associated control sections. The results listed in the table indicate the following:

- Test sections subjected to thin overlay performed better than the control sections by 1 and 2 years in the DF and DNF regions, respectively, while they performed worse than the control sections by 3 years in the WF region. They performed the same in the WNF region.

- Test sections subjected to slurry seal performed better than the control sections by 2 and 3 years in the DF and DNF regions, while they performed worse than the control sections by 2 years in the WF region. They performed the same in the WNF region.

- Test sections subjected to crack seal performed better than the control sections by 1 year in the WNF region, while they performed worse by 5, 1, and 11 years in the WF, DF, and DNF regions, respectively.

- Test sections subjected to aggregate seal coat performed better than the control sections by 2, 4, and 7 years in the WNF, DF, and DNF regions, respectively, while they performed worse by 2 years in the WF region.

Once again, for each treatment type, the RSPs of the test sections after treatment were plotted against the last measured longitudinal cracking data point before treatment were submitted to FHWA and are available from the LTPP Customer Support Services.[79] It can be seen from the figures that, on average, the higher the longitudinal cracking length was before treatments, the lower the RSP was after treatments.

Table 40. Impacts of various maintenance treatments and control section on pavement performance in terms of RSP based on longitudinal cracking.

Climatic Region	Treatment Type	Remaining Structural Period (Years)								Difference in RSP (Year)
		Test Sections				Control Sections				
		Number of Sections	Min	Max	Average	Number of Sections	Min	Max	Average	
WF	Thin overlay	23	2	20	9	12	4	19	12	–3
WNF		26	5	20	12	20	0	18	12	0
DF		12	2	20	15	6	5	18	14	1
DNF		2	15	15	15	2	10	16	13	2
WF	Slurry seal	13	3	20	10	12	4	19	12	–2
WNF		30	4	20	13	20	0	18	13	0
DF		10	8	20	16	6	5	18	14	2
DNF		1	16	16	16	2	10	16	13	3
WF	Crack seal	12	0	20	7	12	4	19	12	–5
WNF		14	1	20	13	20	0	18	12	1
DF		7	1	20	13	6	5	18	14	–1
DNF		1	2	2	2	2	10	16	13	–11
WF	Aggregate seal coat	21	3	20	10	12	4	19	12	–2
WNF		19	5	20	14	20	0	18	12	2
DF		9	8	20	18	6	5	18	14	4
DNF		1	20	20	20	2	10	16	13	7

Max = Maximum.
Min = Minimum.

Transverse Cracking

Listed in table 41 are the calculated minimum, maximum, and average RSPs based on transverse cracking for the SPS-3 test sections that were subjected to the same treatment type and located in the same climatic region. The table also includes the same data for the associated control sections. The data in the table indicate the following:

- Test sections subjected to thin overlay performed better than the control sections by 1, 2, and 12 years in the WF, DF, and DNF regions, respectively, while they performed the same as the control sections in the WNF region.
- Test sections subjected to slurry seal performed better than the control sections by 2 and 5 years in the DF and DNF regions, while they performed the same as the control sections in the WF and WNF regions.
- Test sections subjected to crack seal performed better than the control sections in the DF region by 1 year, while they performed the same in the WF and WNF regions, respectively. Note that insufficient data were available to make a comparison in the DNF region.
- Test sections subjected to aggregate seal coat performed better than the control sections by 2, 1, 1, and 5 years in the WF, WNF, DF, and DNF regions, respectively.

For each treatment type, the RSPs of the test sections after treatment were plotted against the last measured transverse cracking data point before treatment were submitted to FHWA and are available from the LTPP Customer Support Services.[79] It can be seen from the figures that the lower the cumulative transfer cracks value was, the higher the RSP was after treatment.

Table 41. Impacts of various maintenance treatments and control section on pavement performance in terms of RSP based on transverse cracking.

Climatic Regions	Treatment Type	Remaining Structural Period (Years)								Difference in RSP (Year)
		Test Sections				Control Sections				
		Number of Sections	Min	Max	Average	Number of Sections	Min	Max	Average	
WF	Thin overlay	22	1	20	9	18	0	16	8	1
WNF		24	5	20	12	20	3	17	12	0
DF		12	2	20	13	8	0	18	11	2
DNF		2	12	20	16	1	4	4	4	12
WF	Slurry seal	14	0	20	8	18	0	16	8	0
WNF		30	3	20	12	20	3	17	12	0
DF		7	2	20	13	8	0	18	11	2
DNF		1	9	9	9	1	4	4	4	5
WF	Crack seal	14	0	20	8	18	0	16	8	0
WNF		18	0	20	12	20	3	16	12	0
DF		9	0	20	12	8	0	18	11	1
DNF		0	0	0	—	1	4	4	4	NC
WF	Aggregate seal coat	17	0	20	10	18	0	16	8	2
WNF		16	3	20	13	20	3	16	12	1
DF		8	1	20	12	8	0	18	11	1
DNF		1	9	9	9	1	4	4	4	5

— Indicates no data.
NC = Could not be compared.

Summary, Conclusions, and Recommendations for LTPP SPS-3

Table 42 summarizes the impacts of the four SPS-3 maintenance treatments on the pavement performance.

Table 42. Summary of the impact of treatment type on pavement performance compared with the control sections (years).

Treatment Type	Condition or Distress Type	Climatic Region			
		WF	WNF	DF	DNF
Thin overlay	IRI	5	4	5	–2
	Rut depth	7	8	7	–1
	Alligator cracking	2	2	–1	–9
	Longitudinal cracking	–3	0	1	2
	Transverse cracking	1	0	2	12
Slurry seal	IRI	1	4	1	4
	Rut depth	2	5	4	3
	Alligator cracking	–1	0	–3	–7
	Longitudinal cracking	–2	0	2	3
	Transverse cracking	0	0	2	5
Crack seal	IRI	0	2	3	3
	Rut depth	1	5	7	2
	Alligator cracking	–2	–1	1	–15
	Longitudinal cracking	–5	1	–1	–11
	Transverse cracking	0	0	1	NC
Aggregate seal coat	IRI	2	4	1	–4
	Rut depth	3	3	5	9
	Alligator cracking	2	2	–1	1
	Longitudinal cracking	–2	2	4	7
	Transverse cracking	2	1	1	5

NC = Could not be compared.

The number in each cell of the table expresses the average increase in RFP or RSP of the test sections compared with the control sections. It should be noted that the sections in the DNF region were too few in number to draw any reliable conclusions. Also, in many instances, the control sections were not truly representative of the test sections that had undergone treatments in terms of pavement condition and distress. Nevertheless, the data in the table indicate the following:

- The thin overlay treatment improved the pavement performance in terms of IRI and rut depth in all climatic regions except the DNF region. This could be related to construction issues and, perhaps, the relatively high solar radiation (accelerated oxidation/aging) in the DNF region.

- In general, the thin overlay treatment did not improve the pavement performance in terms of alligator, longitudinal, and transverse cracking. This was mainly because of the high rate of reflective cracking. Immediately after treatment, all cracks were hidden by the thin overlay.

- One or few years later, most cracks were reflected through the overlay, which implies a relatively high rate of deterioration and hence a short RSP. The exception is the DNF region, where the 12-year increase in the average RSP of the two test sections compared with the one control section was mainly attributable to the limited number of sections. That is, the conclusion was not reliable because of the limited number of test sections and control sections.

- The slurry seal treatment improved the pavement performance in terms of IRI and rut depth but did not have much impact on alligator, longitudinal, and transverse cracking. The increase in RSP based on transverse cracking in the DNF region was likely attributable to the limited number of sections (one test section and one control section).

- Crack sealing appeared to improve the pavement performance in terms of rutting. This was expected because crack sealing decreases water infiltration, which increased the stiffness of the lower pavement layers. The pavement performance in terms of IRI was improved in the WNF, DF, and DNF regions. However, it did not improve the pavement performance in terms of cracking.

- The aggregate seal coat appeared to improve the pavement performance in all climatic regions in terms of IRI, rut depth, and cracking. This improvement varied from about 1 to 5 years. In terms of IRI, the decrease in RFP of 4 years in the DNF region was highly likely owing to three reasons: construction quality, the good ride quality of one of the three control sections, and limited number of sections.

In general, the worse the pavement conditions were before treatment, the shorter the benefits of treatment were in terms of RFP or RSP.

IMPACT OF REHABILITATION TREATMENTS ON PAVEMENT PERFORMANCE USING LTPP SPS-5 TEST SECTIONS

Once again, one of the objectives of this study was to analyze the benefits of the various rehabilitation treatments applied to the SPS-5 test sections. Unfortunately, for some test sections, the LTPP database did not have enough time-series pavement condition and distress data to conduct the analyses. In one scenario, some of the test sections were subjected to a second treatment and only one or two data points were available. In another scenario, the measured IRI, rut depth, and/or cracking data showed improvement in the pavement condition and/or distresses over time without treatment. After an exhaustive search of the database, it was found that the database had an adequate number of time-series pavement condition and distress data for the evaluation of the benefits of the following rehabilitation treatments:

- Thin (2-inch (51-mm)) and thick (4-inch (102-mm)) AC overlay using recycled asphalt mixes.

- Thin (2-inch (51-mm)) and thick (4-inch (102-mm)) AC overlay using virgin asphalt mixes.

- Thin (2-inch (51-mm)) and thick (4-inch (102-mm)) mill and fill using recycled asphalt mixes.

- Thin (2-inch (51-mm)) and thick (4-inch (102-mm)) mill and fill using virgin asphalt mixes.

After identifying the types of treatments that could be analyzed, the time-dependent pavement condition and distress data were then organized per treatment type, climatic region, and per pavement condition and distress type. The data were then analyzed, RFPs and RSPs of each treated test section accepted for analyses, and the corresponding control and/or linked sections were calculated. For each pavement condition (IRI) and distress type (rut depth and alligator, longitudinal, and transverse cracking), RFP/RSP of the treatment and the treatment benefits are listed in table 43 through table 52. The benefits are listed per climatic region and pavement condition and distress type and are summarized in table 53. The summary of the treatment benefits listed in table 53 is divided based on the pavement condition and distress type and on the treatment type. However, the discussion that follows that table is based on the benefits for the pavement condition and distress type.

Table 43. Impacts of various treatments and control section on pavement performance in terms of RFP based on IRI for virgin AC mixes (years).

Climatic Region	State (Code)	Control Section RFP (Years)	Virgin AC Mix											
			Overlay						Mill and Fill					
			Thin			Thick			Thin			Thick		
			RFP	B1	B2	RFP	B1	B2	RFP	B1	B2	RFP	B1	B2
WF	Maine (23)	17	20	3	20	20	3	20	20	3	20	20	3	20
	Minnesota (27)	5	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	New Jersey (34)	10	20	10	20	20	10	20	20	10	20	20	10	20
	Alberta (81)	16	ND	—	ND	20	4	20	20	4	20	20	4	20
	Manitoba (83)	ND	20	—	18	20	—	19	NS	—	NS	20	—	20
WNF	Alabama (1)	15	20	5	18	20	5	20	20	5	20	20	5	20
	Florida (12)	12	20	8	20	20	8	20	20	8	20	20	8	20
	Georgia (13)	NCS	20	—	20	20	—	15	20	—	18	20	—	12
	Maryland (24)	18	20	2	11	15	–3	12	20	2	20	20	2	20
	Mississippi (28)	13	20	7	6	20	7	20	20	7	12	20	7	20
	Missouri (29)	ND	NS	—	NS	NS	—	NS	NS	—	NS	NS	—	NS
	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Texas (48)	ND	20	—	10	NS	—	NS	NS	—	NS	NS	—	NS
DF	Colorado (8)	ND	NS	—	NS	20	—	20	17	—	ND	20	—	ND
	Montana (30)	4	16	12	6	20	16	20	20	16	20	20	16	N/A
DNF	Arizona (4)	17	20	3	20	20	3	20	20	3	20	20	3	20
	California (6)	0	10	10	6	20	20	10	20	20	15	20	20	20
	New Mexico (35)	ND	20	—	ND	20	—	ND	20	—	ND	20	—	ND

— Indicates could not be estimated.
Thin = 2 inches (51 mm).
Thick = 4 inches (102 mm).
B1= Change in functional period (CFP).
B2 = Functional condition reoccurrence period.
ND = No data.
NCS = No control section.
NS = Model has a negative slope.
N/A = Not applicable.

Table 44. Impacts of various treatments and control section on pavement performance in terms of RFP based on IRI for recycled AC mixes (years).

Climatic Region	State (Code)	Control Section RFP (Years)	Recycled AC Mix											
			Overlay						Mill and Fill					
			Thin			Thick			Thin			Thick		
			RFP	B1	B2	RFP	B1	B2	RFP	B1	B2	RFP	B1	B2
WF	Maine (23)	17	NS	—	NS	NS	—	NS	NS	—	NS	NS	—	NS
	Minnesota (27)	5	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	New Jersey (34)	10	20	10	20	20	10	20	20	10	20	20	10	20
	Alberta (81)	16	20	4	18	20	4	20	20	4	17	20	4	20
	Manitoba (83)	ND	20	—	20	NS	—	NS	20	—	20	20	—	20
WNF	Alabama (1)	15	20	5	17	20	5	20	20	5	20	20	5	10
	Florida (12)	12	20	8	16	20	8	20	20	8	20	20	8	20
	Georgia (13)	NCS	20	—	15	20	—	15	20	—	14	20	—	11
	Maryland (24)	18	ND	—	ND	20	2	15	20	2	12	20	2	20
	Mississippi (28)	13	20	7	20	20	7	20	20	7	20	20	7	20
	Missouri (29)	ND	NS	—	NS	NS	—	NS	NS	—	NS	NS	—	NS
	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Texas (48)	ND	20	—	15	20	—	20	20	—	20	20	—	16
DF	Colorado (8)	ND	20	—	20	NS	—	NS	20	—	ND	20	—	20
	Montana (30)	4	12	8	6	20	16	20	15	11	3	20	16	20
DNF	Arizona (4)	17	13	–4	8	20	3	20	16	–1	14	20	3	20
	California (6)	0	11	11	10	20	20	10	9	9	8	20	20	18
	New Mexico (35)	ND	20	—	ND	20	—	ND	20	—	ND	20	—	ND

— Indicates could not be estimated.
Thin = 2 inches (51 mm).
Thick = 4 inches (102 mm).
B1= CFP.
B2 = Functional condition reoccurrence period.
ND = No data.
NCS = No control section.
NS = Model has a negative slope.

Table 45. Impacts of various treatments and control section on pavement performance in terms of RFP/RSP based on rut depth for virgin AC mixes (years).

Climatic Region	State (Code)	Control Section RFP/RSP (Years)	Virgin AC Mix											
			Overlay						Mill and Fill					
			Thin			Thick			Thin			Thick		
			RFP/ RSP	B1	B2	RFP/ RSP	B1	B2	RFP/ RSP	B1	B2	RFP/ RSP	B1	B2
WF	Maine (23)	0	20	20	20	12	11	15	8	8	13	10	10	11
	Minnesota (27)	16	NS	—	NS	NS	—	NS	NS	—	NS	NS	—	NS
	New Jersey (34)	NS	20	—	20	20	—	20	NS	—	NS	NS	—	NS
	Alberta (81)	ND	ND	—	ND	20	—	ND	20	—	ND	20	—	ND
	Manitoba (83)	ND	20	—	ND	20	—	ND	20	—	ND	20	—	ND
WNF	Alabama (1)	NS	20	—	ND	20	—	ND	20	—	ND	20	—	ND
	Florida (12)	12	20	8	20	20	8	20	20	8	20	20	8	20
	Georgia (13)	NCS	20	—	20	20	—	20	20	—	20	20	—	20
	Maryland (24)	ND	NS	—	NS	20	—	3	NS	—	NS	20	—	20
	Mississippi (28)	0	15	15	20	4	4	5	8	8	13	3	3	5
	Missouri (29)	NS	20	—	20	20	—	13	20	—	20	20	—	20
	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Texas (48)	14	20	6	20	20	6	20	20	6	18	20	6	18
DF	Colorado (8)	NS	20	—	13	20	—	13	20	—	20	18	—	20
	Montana (30)	NS	20	—	20	14	—	7	10	—	10	17	—	13
DNF	Arizona (4)	NS	20	—	ND	NS	—	NS	NS	—	NS	NS	—	NS
	California (6)	ND	20	—	6	20	—	20	20	—	20	20	—	20
	New Mexico (35)	ND	20	—	ND	20	—	ND	20	—	ND	20	—	ND

— Indicates could not be estimated.
Thin = 2 inches (51 mm).
Thick = 4 inches (102 mm).
B1= Change in functional period.
B2 = Functional condition reoccurrence period.
ND = No data.
NCS = No control section.
NS = Model has a negative slope.

Table 46. Impacts of various treatments and control section on pavement performance in terms of RFP/RSP based on rut depth for recycled AC mixes (years).

Climatic Region	State (Code)	Control Section RFP/RSP (Years)	Recycled AC Mix											
			Overlay						Mill and Fill					
			Thin			Thick			Thin			Thick		
			RFP/ RSP	B1	B2	RFP/ RSP	B1	B2	RFP/ RSP	B1	B2	RFP/ RSP	B1	B2
WF	Maine (23)	0	15	15	16	14	14	15	12	12	18	10	10	15
	Minnesota (27)	16	20	4	20	NS	—	NS	20	4	20	NS	—	NS
	New Jersey (34)	NS	20	—	20	NS	—	NS	NS	—	NS	NS	—	NS
	Alberta (81)	ND	NS	—	NS	20	—	ND	20	—	ND	NS	—	NS
	Manitoba (83)	ND	20	—	ND	20	—	ND	20	—	ND	20	—	ND
WNF	Alabama (1)	NS	20	—	ND	20	—	ND	20	—	ND	20	—	ND
	Florida (12)	12	20	8	20	20	8	11	20	8	20	20	8	20
	Georgia (13)	NCS	20	—	20	20	—	20	20	—	20	20	—	20
	Maryland (24)	ND	4	—	1	2	—	0	9	—	2	3	—	1
	Mississippi (28)	0	9	9	20	6	6	12	9	9	20	4	4	9
	Missouri (29)	NS	20	—	20	20	—	3	20	—	20	20	—	1
	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Texas (48)	14	20	6	20	20	6	20	20	6	20	20	6	20
DF	Colorado (8)	NS	19	—	6	20	—	8	18	—	20	13	—	20
	Montana (30)	NS	16	—	18	20	—	20	12	—	10	20	—	20
DNF	Arizona (4)	NS	20	—	ND	20	—	ND	20	—	ND	NS	—	NS
	California (6)	ND	20	—	20	20	—	11	20	—	20	20	—	20
	New Mexico (35)	ND	NS	—	NS	20	—	ND	20	—	ND	20	—	ND

— Indicates could not be estimated.
Thin = 2 inches (51 mm).
Thick = 4 inches (102 mm).
B1= CFP.
B2 = Functional condition reoccurrence period.
ND = No data.
NCS = No control section.
NS = Model has a negative slope.

Table 47. Impacts of various treatments and control section on pavement performance in terms of RSP based on alligator cracking for virgin AC mixes (years).

Climatic Region	State (Code)	Control Section RSP (Years)	Virgin AC Mix											
			Overlay						Mill and Fill					
			Thin			Thick			Thin			Thick		
			RSP	B1	B2	RSP	B1	B2	RSP	B1	B2	RSP	B1	B2
WF	Maine (23)	10	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Minnesota (27)	16	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	New Jersey (34)	ND	20	—	20	20	—	15	20	—	13	18	—	10
	Alberta (81)	ND	ND	—	ND	13	—	ND	11	—	ND	14	—	ND
	Manitoba (83)	ND	9	—	0	13	—	0	11	—	0	13	—	0
WNF	Alabama (1)	0	20	20	0	20	20	15	20	20	20	20	20	20
	Florida (12)	12	20	8	11	20	8	20	20	8	20	19	8	20
	Georgia (13)	NCS	20	—	9	ND	—	ND	ND	—	ND	20	—	10
	Maryland (24)	ND	ND	—	ND	ND	—	ND	20	—	20	20	—	20
	Mississippi (28)	16	12	–4	7	10	–6	9	10	–6	5	9	–7	9
	Missouri (29)	NS	ND	—	ND	8	—	6	9	—	8	10	—	7
	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Texas (48)	NS	NS	—	NS	NS	—	NS	ND	—	ND	20	—	NS
DF	Colorado (8)	ND	6	—	4	7	—	3	6	—	4	8	—	1
	Montana (30)	ND	NS	—	NS	ND	—	ND	20	—	20	ND	—	ND
DNF	Arizona (4)	ND	9	—	ND	20	—	ND	20	—	ND	ND	—	ND
	California (6)	ND	5	—	ND	11	—	ND	8	—	ND	11	—	ND
	New Mexico (35)	ND	12	—	5	ND	—	ND	20	—	20	16	—	11

— Indicates could not be estimated.
Thin = 2 inches (51 mm).
Thick = 4 inches (102 mm).
B1= CFP.
B2 = Functional condition reoccurrence period.
ND = No data.
NCS = No control section.
NS = Model has a negative slope. .

Table 48. Impacts of various treatments and control section on pavement performance in terms of RSP based on alligator cracking for recycled AC mixes (years).

Climatic Region	State (Code)	Control Section RSP (Years)	Recycled AC Mix											
			Overlay						Mill and Fill					
			Thin			Thick			Thin			Thick		
			RSP	B1	B2	RSP	B1	B2	RSP	B1	B2	RSP	B1	B2
WF	Maine (23)	10	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
WF	Minnesota (27)	16	ND	—	ND	ND	—	ND	20	4	—	ND	—	ND
WF	New Jersey (34)	ND	11	—	19	15	—	0	12	—	0	17	—	1
WF	Alberta (81)	ND	7	—	ND	6	—	ND	7	—	ND	10	—	ND
WF	Manitoba (83)	ND	7	—	0	7	—	0	10	—	0	12	—	0
WNF	Alabama (1)	0	16	16	4	20	20	20	20	20	10	20	20	20
WNF	Florida (12)	12	20	8	16	20	8	20	20	8	20	20	8	20
WNF	Georgia (13)	—	20	—	20	ND	—	ND	20	—	16	ND	—	ND
WNF	Maryland (24)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
WNF	Mississippi (28)	16	7	–10	0	11	–5	5	6	–10	0	9	–8	4
WNF	Missouri (29)	NS	ND	—	ND	ND	—	ND	20	—	17	ND	—	ND
WNF	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
WNF	Texas (48)	NS	20	—	NS	20	—	NS	20	—	NS	20	—	NS
DF	Colorado (8)	ND	6	—	0	6	—	4	20	—	0	7	—	5
DF	Montana (30)	ND	ND	—	ND	3	—	7	4	—	5	6	—	6
DNF	Arizona (4)	ND	4	—	ND	13	—	ND	15	—	ND	20	—	ND
DNF	California (6)	ND	4	—	ND	8	—	ND	3	—	ND	20	—	ND
DNF	New Mexico (35)	ND	20	—	0	10	—	8	15	—	0	10	—	9

— Indicates could not be estimated.
Thin = 2 inches (51 mm).
Thick = 4 inches (102 mm).
B1= CFP.
B2 = Functional condition reoccurrence period.
ND = No data.
NS = Model has a negative slope.

Table 49. Impacts of various treatments and control section on pavement performance in terms of RSP based on longitudinal cracking for virgin AC mixes (years).

Climatic Region	State (Code)	Control Section RSP (Years)	Virgin AC Mix											
			Overlay						Mill and Fill					
			Thin			Thick			Thin			Thick		
			RSP	B1	B2	RSP	B1	B2	RSP	B1	B2	RSP	B1	B2
WF	Maine (23)	8	9	0	8	9	1	8	9	1	8	9	1	9
	Minnesota (27)	9	6	–3	2	11	2	6	8	–1	5	8	–1	6
	New Jersey (34)	ND	20	—	NA	15	—	NA	18	—	NA	12	—	NA
	Alberta (81)	ND	ND	—	ND	20	—	ND	20	—	ND	17	—	ND
	Manitoba (83)	ND	20	—	0	14	—	0	20	—	0	17	—	0
WNF	Alabama (1)	ND	20	—	6	20	—	0	20	—	8	20	—	0
	Florida (12)	ND	20	—	0	20	—	0	17	—	14	20	—	14
	Georgia (13)	NCS	11	—	0	14	—	0	13	—	0	14	—	0
	Maryland (24)	ND	ND	—	ND	ND	—	ND	13	—	0	12	—	3
	Mississippi (28)	9	20	11	0	12	2	1	14	5	5	ND	—	ND
	Missouri (29)	4	5	1	5	10	6	10	10	5	10	18	13	19
	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Texas (48)	NS	11	—	ND	20	—	ND	17	—	ND	20	—	ND
DF	Colorado (8)	ND	5	—	4	6	—	4	7	—	5	9	—	6
	Montana (30)	ND	NS	—	NS	ND	—	ND	10	—	9	ND	—	ND
DNF	Arizona (4)	ND	20	—	ND	18	—	ND	20	—	ND	20	—	ND
	California (6)	ND	11	—	ND	11	—	ND	10	—	ND	16	—	ND
	New Mexico (35)	ND	12	—	7	10	—	8	11	—	5	10	—	8

— Indicates could not be estimated.
Thin = 2 inches (51 mm).
Thick = 4 inches (102 mm).
B1= CFP.
B2 = Functional condition reoccurrence period.
ND = No data.
NCS = No control section.
NS = Model has a negative slope.

Table 50. Impacts of various treatments and control section on pavement performance in terms of RSP based on longitudinal cracking for recycled AC mixes (years).

Climatic Region	State (Code)	Control Section RSP (Years)	Recycled AC Mix											
			Overlay						Mill and Fill					
			Thin			Thick			Thin			Thick		
			RSP	B1	B2	RSP	B1	B2	RSP	B1	B2	RSP	B1	B2
WF	Maine (23)	8	10	2	10	9	0	8	10	2	10	9	1	9
	Minnesota (27)	9	9	0	3	20	11	11	9	1	7	13	5	10
	New Jersey (34)	ND	NS	—	NS	12	—	N/A	NS	—	NS	12	—	N/A
	Alberta (81)	ND	13	—	ND	14	—	ND	12	—	ND	13	—	ND
	Manitoba (83)	ND	NS	—	NS	20	—	0	20	—	0	16	—	0
WNF	Alabama (1)	ND	20	—	20	20	—	0	20	—	0	20	—	0
	Florida (12)	ND	20	—	20	19	—	19	20	—	1	20	—	18
	Georgia (13)	NCS	11	—	1	14	—	0	13	—	0	14	—	0
	Maryland (24)	ND	7	—	5	ND	—	ND	16	—	7	20	—	6
	Mississippi (28)	9	20	11	4	20	11	0	9	0	7	14	5	0
	Missouri (29)	4	13	9	10	10	5	7	13	9	6	13	8	11
	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Texas (48)	NS	10	—	ND	12	—	ND	12	—	ND	12	—	ND
DF	Colorado (8)	ND	7	—	4	6	—	4	7	—	6	6	—	5
	Montana (30)	ND	ND	—	ND	ND	—	ND	ND	—	ND	19	—	8
DNF	Arizona (4)	ND	NS	—	NS	19	—	ND	20	—	ND	18	—	ND
	California (6)	ND	10	—	ND	10	—	ND	8	—	ND	9	—	ND
	New Mexico (35)	ND	10	—	9	8	—	6	10	—	6	8	—	5

— Indicates could not be estimated.
Thin = 2 inches (51 mm).
Thick = 4 inches (102 mm).
B1= CFP.
B2 = Functional condition reoccurrence period.
ND = No data.
NCS = No control section.
NS = Model has a negative slope.
N/A = Not applicable.

Table 51. Impacts of various treatments and control section on pavement performance in terms of RSP based on transverse cracking for virgin AC mixes (years).

Climatic Region	State (Code)	Control Section RSP (Years)	Virgin AC Mix											
			Overlay						Mill and Fill					
			Thin			Thick			Thin			Thick		
			RSP	B1	B2	RSP	B1	B2	RSP	B1	B2	RSP	B1	B2
WF	Maine (23)	NS	ND	—	ND	11	—	10	11	—	10	ND	—	ND
	Minnesota (27)	16	2	–15	5	13	–4	15	6	–10	13	13	–3	14
	New Jersey (34)	ND	18	—	5	20	—	15	20	—	13	20	—	14
	Alberta (81)	ND	ND	—	ND	20	—	ND	17	—	ND	11	—	ND
	Manitoba (83)	ND	20	—	0	10	—	2	20	—	0	14	—	0
WNF	Alabama (1)	10	20	10	0	20	10	11	20	10	9	20	10	4
	Florida (12)	ND	19	—	7	20	—	7	20	—	0	20	—	2
	Georgia (13)	NCS	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Maryland (24)	ND	ND	—	ND	ND	—	ND	20	—	5	20	—	20
	Mississippi (28)	8	12	4	6	11	3	9	14	6	5	13	5	11
	Missouri (29)	NS	9	—	8	ND	—	ND	10	—	9	14	—	11
	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Texas (48)	NS	12	—	ND	20	—	ND	20	—	ND	20	—	ND
DF	Colorado (8)	ND	10	—	6	10	—	8	15	—	5	NS	—	NS
	Montana (30)	ND	20	—	20	20	—	20	20	—	20	20	—	20
DNF	Arizona (4)	ND	14	—	ND	19	—	ND	19	—	ND	19	—	ND
	California (6)	ND	9	—	ND	12	—	ND	9	—	ND	13	—	ND
	New Mexico (35)	ND	15	—	10	ND	—	ND	12	—	11	17	—	16

— Indicates could not be estimated.
Thin = 2 inches (51 mm).
Thick = 4 inches (102 mm).
B1= CFP.
B2 = Functional condition reoccurrence period.
ND = No data.
NCS = No control section.
NS = Model has a negative slope.

Table 52. Impacts of various treatments and control section on pavement performance in terms of RSP based on transverse cracking for recycled AC mixes (years).

Climatic Region	State (Code)	Control Section RSP (Years)	Recycled AC Mix											
			Overlay						Mill and Fill					
			Thin						Thin			Thick		
			RSP	B1	B2	RSP	B1	B2	RSP	B1	B2	RSP	B1	B2
WF	Maine (23)	NS	ND	—	ND	ND	—	ND	20	—	9	ND	—	ND
	Minnesota (27)	17	8	–8	5	13	–4	9	10	–7	7	9	–8	8
	New Jersey (34)	ND	13	—	1	17	—	8	20	—	0	20	—	10
	Alberta (81)	ND	12	—	ND	14	—	ND	9	—	ND	9	—	ND
	Manitoba (83)	ND	17	—	0	17	—	0	20	—	0	8	—	6
WNF	Alabama (1)	10	19	9	0	20	10	7	20	10	2	20	10	7
	Florida (12)	ND	20	—	14	ND	—	ND	20	—	10	20	—	20
	Georgia (13)	NCS	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Maryland (24)	ND	14	—	10	8	—	7	20	—	20	ND	—	ND
	Mississippi (28)	8	8	0	6	9	1	3	10	2	7	10	2	5
	Missouri (29)	NS	ND	—	ND	ND	—	ND	20	—	15	ND	—	ND
	Oklahoma (40)	ND	ND	—	ND	ND	—	ND	ND	—	ND	ND	—	ND
	Texas (48)	NS	11	—	ND	15	—	ND	13	—	ND	17	—	ND
DF	Colorado (8)	ND	6	—	4	ND	—	ND	6	—	4	ND	—	ND
	Montana (30)	ND	ND	—	ND	20	—	20	ND	—	ND	NS	—	NS
DNF	Arizona (4)	ND	NS	—	NS	8	—	ND	9	—	ND	11	—	ND
	California (6)	ND	10	—	ND	10	—	ND	8	—	ND	8	—	ND
	New Mexico (35)	ND	17	—	3	15	—	8	17	—	2	11	—	9

— Indicates could not be estimated.
Thin = 2-inches (51 mm).
Thick = 4 inches (102 mm).
B1= CFP.
B2 = Functional condition reoccurrence period.
ND = No data.
NCS = No control section.
NS = Model has a negative slope.

Table 53. Summary of benefits of various rehabilitation treatments (years).

					Distress							
			Condition		Rut		Cracking					
Treatment	Thickness		IRI		Depth		Alligator		Longitudinal		Transverse	
Type	(Inches)	Statistic	B1	B2	B1	B2	B1	B2	B1	B2	B1	B2
Overlay, virgin AC mix	2	Min	2	6	6	6	–4	0	–3	0	–15	0
		Max	12	20	20	20	20	20	11	8	10	20
		Average	7	14	12	18	8	7	2	3	0	7
	4	Min	–3	10	4	3	–6	0	1	0	–4	2
		Max	20	20	11	20	20	20	6	10	10	20
		Average	7	18	7	14	7	10	3	4	3	11
Overlay, recycled AC mix	2	Min	–4	6	4	1	–10	0	0	1	–8	0
		Max	11	20	15	20	16	20	11	20	9	14
		Average	6	15	8	17	5	7	5	9	0	5
	4	Min	2	10	6	0	–5	0	0	0	–4	0
		Max	20	20	14	20	20	20	11	19	10	20
		Average	8	18	9	12	8	8	7	6	2	8
Mill and fill, virgin AC mix	2	Min	2	12	6	10	–6	0	–1	0	–10	0
		Max	20	20	8	20	20	20	5	14	10	20
		Average	8	19	8	17	7	13	2	6	2	8
	4	Min	2	12	3	5	–7	0	–1	0	–3	0
		Max	20	20	10	20	20	20	13	19	10	20
		Average	8	19	7	17	7	11	4	6	4	11
Mill and fill, recycled AC mix	2	Min	–1	3	4	2	–10	0	0	0	–7	0
		Max	11	20	12	20	20	20	9	10	10	20
		Average	6	16	8	17	6	7	3	5	2	7
	4	Min	2	10	4	1	–8	0	1	0	–7	5
		Max	20	20	10	20	20	20	8	18	10	20
		Average	8	18	7	15	7	8	5	6	2	9

1 inch = 25.4 mm.
B1 = Changes in functional or structural period (CFP/CSP) in years.
B2 = Functional or structural condition reoccurrence period (FCROP/SCROP) in years.
Max = Maximum.
Min = Minimum.

IRI

The benefits data listed in table 53 under the heading “IRI” indicate that the averages of the CFPs (labeled “B1” in the table) of all eight treatments were similar and equaled about 7 years. This was expected because a proper construction of 2- and 4-inch (51-and 102-mm) overlays and 2- and 4-inch (51- and 102-mm) mill-and-fill treatments result in smooth pavement surface and almost the same rate of deterioration. Further, the average functional condition reoccurrence period (FCROP) in years of any of the eight treatments was about 17 years (i.e., 17 years after applying any of the eight treatments, the IRI of the treated pavement would be the same as it was just before treatment).

Rut Depth

The benefits data listed in table 53 under the heading “Rut Depth” indicate that the benefits in terms of RSP (column labeled “B1”) and the structural condition reoccurrence period (SCROP) (column labeled “B2”) for AC overlays and mill-and-fill treatments using virgin and recycled asphalt mixes were statistically similar. Note that the SCROP is the same as TL as previously defined.[5]

Alligator Cracking

The benefits data listed in table 53 under the heading “Alligator Cracking” indicate that the B1s of the eight treatments varied slightly depending on the thickness of the overlay and the type of the AC mix. On average, each treatment caused an increase in RSP of about 6 years. (This varied from a high of 20 years to a low of 10 years.) The latter was mainly the result of the condition of the control sections (i.e., no alligator cracking). Thus, the minimum and maximum change in structural period (CSP) should not be taken seriously; they are for information only. The average CSP, on the other hand, was a good measure of the benefits of each treatment. Further, the average structural period of the 2-inch (51-mm)-thick virgin AC overlay was 1 to 3 years lower than the 4 inch (102-mm) virgin AC overlay. The type of AC mix (virgin and recycled) appeared not to affect the SCROP.

Longitudinal Cracking

The benefits data listed in table 53 under the heading “Longitudinal Cracking” indicate that the average CFP/CSP of the test sections subjected to 2- and 4-inch (51- and 102-mm)-thick overlays and mill and fill treatments using virgin and recycled AC mixes appeared to have had the lowest CSPs (2 to 4 years), while the CSP for the recycled mixes was about 2 years longer. Further, the average SCROP of each of the four mill-and-fill treatments was about 6 years.

Transverse Cracking

The benefits data listed in table 53 under the heading “Transverse Cracking” varied and depended on the thickness of the AC overlay. The 2-inch (51-mm)-thick AC overlay yielded a CSP of 0 years, whereas the 4-inch (102-mm) AC overlay yielded, on average, a CSP of 3 years. This was expected because the thin 2-inch (51-mm)-thick overlay has minor resistance to reflective cracking. The average SCROP of the 2-inch (51-mm)-thick overlay or mill and fill was about 7 years, whereas the average SCROP of the 4-inch (102-mm) overlay or mill and fill was about 10 years.

Summary, Conclusions and Recommendations for LTPP SPS-5

Based on data availability in the LTPP database, eight rehabilitation treatments were included in the analyses of the treatment benefits. The benefits were estimated by comparing RFP and RSP of the test sections and RFP and RSP (CFP/CSP) of the control or linked sections In addition, the FCROP/SCROP (the time in years from the treatment to the year during which the pavement condition or distresses are the same as those before treatment) were also used as calculated indicators of benefits. Based on the results of the analyses, the following conclusions were drawn:

- On average, the impact of 2- and 4-inch (51- and 102-mm)-thick virgin or recycled AC overlay on the pavement performance was almost the same.
- The 2-inch (51-mm)-thick AC overlay (virgin or recycled mix) did not provide long-term remediation of transverse cracking. The cracks in the lower pavement structure typically reflected through the overlay in a few years.
- On average, the benefits of the 2-inch (51-mm)-thick AC overlay in terms of alligator cracking were slightly less than those of the 4-inch (102-mm)-thick overlay.
- The minimum or maximum CFP or CSP should not be used as an indicator of benefits because these values are also a function of the conditions and distresses of the control or linked sections.

Based on the results of the data, the research team strongly recommends the following:

- Establish a solid criterion for the selection of the control sections. This criterion should be based on the similarity of the pavement condition and distresses, traffic, and material types to the test sections. Perhaps each control or linked section should border the test section in question. Ideally, the roughness, rut depth, cracking, and any other condition measures of the control and test sections before treatment should be similar.
- Obtain the history of the selected control or linked section and the test sections and keep it in the database. This information should include construction and treatment history as well as pavement condition and distress data.
- Measure the pavement condition and distress data no more than 1 month before the application of a treatment and no more than 1 month after the completion of the treatment.
- Collect the pavement condition and distress data more frequently (once a year or less) and for a longer time period (6 years is recommended) before the treatment is applied.

IMPACTS OF PAVEMENT TREATMENTS ON PAVEMENT PERFORMANCE USING THE LTPP GPS-6 TEST SECTIONS

The LTPP GPS-6 experiment contained flexible pavement test sections that were overlain prior to their assignment to the LTPP Program. The experiment also included test sections that were moved from other LTPP experiments after they were subjected to either AC overlay or mill-and-fill treatments. The test sections in the GPS-6 experiment were classified as GPS-6A, -6B, -6C, -6D, and -6S. The following list explains each of the classifications:

- **GPS-6A:** The test sections under this classification were part of the original LTPP design. They were subjected to AC overlay prior to their assignment to the LTPP Program.

- **GPS-6B:** The test sections under this classification were also part of the original LTPP design. They were subjected to AC overlay following assignment to the LTPP Program.

- **GPS-6C, -6D, and -6S:** The test sections under these classifications did not have an experimental design associated with them. They were moved to one of these three classifications from other LTPP experiments after they were subjected to rehabilitation actions according to the following scheme:

  - If overlain with recycled AC mixes, they were moved to the GPS-6C classification.

  - If overlain using virgin AC mixes, they are moved to the GPS-6D classification.

  - If milled and filled using virgin or recycled AC mixes, they are moved to the GPS-6S classification.

After an extensive search of the database, all of the test sections in the GPS-6 experiment that had three or more before treatment and three or more after treatment time-series pavement condition and/or distress data points were grouped according to the following variables:

- Two treatment types (AC overlay and mill and fill).
- AC mix type (virgin and recycled).
- Thickness types (thin ≤ 2.5 inches (63.5 mm) and thick > 2.5 inches (63.5 mm)).
- Four climatic regions (WF, WNF, DF, and DNF).
- One pavement condition (IRI).
- Four pavement distress types.

Therefore, the analyses were conducted to assess the impacts of each treatment type and AC mix type and thickness on the pavement performance (IRI, rut depth, and cracking) in each climatic region using RFP and RSP of each treated test section before and after treatment.

For each test section, the treatment benefits were expressed in terms of the CFP or CSP, which were the difference between the after treatment RFP or RSP and the before treatment RFP or RSP. The minimum and maximum CFPs and CSPs and their averages for all test sections located in the same climatic region were calculated and listed in table 54 through table 58 depending on the pavement condition and distress type. The data in the five tables are discussed in the follow subsections per pavement condition and distress type.

Table 54. Impacts of various treatments on pavement performance in terms of CFP based on IRI (years).

Treatment Type	Mix Type	Thickness	WF No.	WF CFP (Year) Min	WF CFP (Year) Max	WF CFP (Year) Avg	WNF No.	WNF CFP (Year) Min	WNF CFP (Year) Max	WNF CFP (Year) Avg	DF No.	DF CFP (Year) Min	DF CFP (Year) Max	DF CFP (Year) Avg	DNF No.	DNF CFP (Year) Min	DNF CFP (Year) Max	DNF CFP (Year) Avg
Overlay	Virgin	Thin	6	4	20	11	16	6	17	10	3	6	17	12	0	0	0	—
		Thick	6	5	20	13	5	10	14	12	4	3	20	13	1	14	14	14
	Recycled	Thin	0	0	0	—	4	2	20	10	0	0	0	—	0	0	0	—
		Thick	1	13	13	13	3	4	14	8	1	–6	–6	–6	0	0	0	—
Mill and fill	Virgin	Thin	4	–4	9	4	19	–7	12	7	3	6	14	11	1	13	13	13
		Thick	1	19	19	19	4	4	14	9	0	0	0	—	7	5	20	12
	Recycled	Thin	2	10	13	12	2	11	16	13	0	0	0	—	0	0	0	—
		Thick	3	10	20	15	7	3	19	12	0	0	0	—	1	13	13	13

— Indicates no data.
Avg = Average.
No. = Number of test sections.
Min = Minimum.
Max = Maximum.
Thin = ≤ 2.5 inches (63.5 mm).
Thick = > 2.5 inches (63.5 mm).

Table 55. Impacts of various treatments on pavement performance in terms of CFP/CSP based on rut depth (years).

Treatment Type	Mix Type	Thickness	Climatic Regions															
			WF				WNF				DF				DNF			
			No.	CFP/CSP (Year)			No.	CFP/CSP (Year)			No.	CFP/CSP (Year)			No.	CFP/CSP (Year)		
				Min	Max	Avg		Min	Max	Avg		Min	Max	Avg		Min	Max	Avg
Overlay	Virgin	Thin	6	9	20	13	12	–2	20	10	1	8	8	8	0	0	0	—
		Thick	6	5	20	13	3	8	20	12	3	–2	9	5	2	10	13	9
	Recycled	Thin	0	0	0	—	2	4	10	7	0	0	0	—	0	0	0	—
		Thick	0	0	0	—	1	14	14	14	0	0	0	—	0	0	0	—
Mill and fill	Virgin	Thin	11	1	20	11	20	0	20	16	2	12	16	14	0	0	0	—
		Thick	5	3	20	13	2	20	20	20	0	0	0	—	6	2	15	9
	Recycled	Thin	0	0	0	—	2	11	20	15	0	0	0	—	0	0	0	—
		Thick	6	9	20	13	12	–2	20	10	1	8	8	8	0	0	0	—

— Indicates no data.
Avg = Average.
No. = Number of test sections.
Min = Minimum.
Max = Maximum.
Thin = ≤ 2.5 inches (63.5 mm).
Thick = > 2.5 inches (63.5 mm).

Table 56. Impacts of various treatments on pavement performance in terms of CSP based on alligator cracking (years).

Treatment Type	Mix Type	Thickness	WF No.	WF CSP (Year) Min	WF CSP (Year) Max	WF CSP (Year) Avg	WNF No.	WNF CSP (Year) Min	WNF CSP (Year) Max	WNF CSP (Year) Avg	DF No.	DF CSP (Year) Min	DF CSP (Year) Max	DF CSP (Year) Avg	DNF No.	DNF CSP (Year) Min	DNF CSP (Year) Max	DNF CSP (Year) Avg
Overlay	Virgin	Thin	3	11	20	15	4	5	20	13	0	0	0	—	0	0	0	—
		Thick	0	0	0	—	2	8	13	10	0	0	0	—	1	6	6	6
	Recycled	Thin	0	0	0	—	1	4	4	4	0	0	0	—	0	0	0	—
		Thick	0	0	0	—	1	10	10	10	0	0	0	—	0	0	0	—
Mill and fill	Virgin	Thin	3	7	8	7	5	0	20	6	1	11	11	11	0	0	0	—
		Thick	0	0	0	—	1	4	4	4	0	0	0	—	2	12	17	15
	Recycled	Thin	0	0	0	—	0	0	0	—	0	0	0	—	0	0	0	—
		Thick	3	11	20	15	4	5	20	13	0	0	0	—	0	0	0	—

— Indicates no data.
Avg = Average.
No. = Number of test sections.
Min = Minimum.
Max = Maximum.
Thin = ≤ 2.5 inches (63.5 mm).
Thick = > 2.5 inches 63.5 mm).

Table 57. Impacts of various treatments on pavement performance in terms of CSP based on longitudinal cracking (years).

Treatment Type	Mix Type	Thickness	WF No.	WF CSP (Year) Min	WF CSP (Year) Max	WF CSP (Year) Avg	WNF No.	WNF CSP (Year) Min	WNF CSP (Year) Max	WNF CSP (Year) Avg	DF No.	DF CSP (Year) Min	DF CSP (Year) Max	DF CSP (Year) Avg	DNF No.	DNF CSP (Year) Min	DNF CSP (Year) Max	DNF CSP (Year) Avg
Overlay	Virgin	Thin	2	0	18	9	5	–3	20	7	0	0	0	—	0	0	0	—
		Thick	0	0	0	—	2	5	8	7	0	0	0	—	0	0	0	—
	Recycled	Thin	0	0	0	—	1	3	3	3	0	0	0	—	0	0	0	—
		Thick	0	0	0	—	1	5	5	5	0	0	0	—	0	0	0	—
Mill and fill	Virgin	Thin	4	–3	5	0	16	–10	20	5	2	10	12	11	0	0	0	—
		Thick	0	0	0	—	1	14	14	14	0	0	0	—	2	0	12	6
	Recycled	Thin	0	0	0	—	1	10	10	10	0	0	0	—	0	0	0	—
		Thick	1	17	17	17	2	14	14	14	0	0	0	—	0	0	0	—

— Indicates no data.
Avg = Average.
No. = Number of test sections.
Min = Minimum.
Max = Maximum.
Thin = ≤ 2.5 inches (63.5 mm).
Thick = > 2.5 inches (63.5 mm).

Table 58. Impacts of various treatments on pavement performance in terms of CSP based on transverse cracking (years).

Treatment Type	Mix Type	Thickness	WF No.	WF CSP (Year) Min	WF CSP (Year) Max	WF CSP (Year) Avg	WNF No.	WNF CSP (Year) Min	WNF CSP (Year) Max	WNF CSP (Year) Avg	DF No.	DF CSP (Year) Min	DF CSP (Year) Max	DF CSP (Year) Avg	DNF No.	DNF CSP (Year) Min	DNF CSP (Year) Max	DNF CSP (Year) Avg
Overlay	Virgin	Thin	3	5	12	8	10	–2	17	6	0	0	0	—	0	0	0	—
		Thick	2	8	20	14	3	7	14	11	0	0	0	—	1	4	4	4
	Recycled	Thin	0	0	0	—	1	3	3	3	0	0	0	—	0	0	0	—
		Thick	0	0	0	—	1	3	3	3	0	0	0	—	0	0	0	—
Mill and fill	Virgin	Thin	0	0	0	—	5	–6	16	5	1	3	3	3	0	0	0	—
		Thick	0	0	0	—	1	5	5	5	0	0	0	—	3	0	12	7
	Recycled	Thin	0	0	0	—	2	10	11	11	0	0	0	—	0	0	0	—
		Thick	2	8	11	9	3	8	16	13	0	0	0	—	0	0	0	—

— Indicates no data.
Avg = Average.
No. = Number of test sections.
Min = Minimum.
Max = Maximum.
Thin = ≤ 2.5 inches (63.5 mm).
Thick = > 2.5 inches (63.5 mm).

IRI

The data in table 54 indicate the following, on average:

- The thin and thick overlays using virgin AC mix extended the pavement functional period by about 11 and 13 years, respectively.
- The thick overlays using recycled AC mix extended the pavement functional period by 13 years in the WF region and by 8 years in the WNF region. The construction of this overlay type on the single test section in the DF region caused 6 years loss in the pavement functional period. The reason was the rough pavement surface after construction.
- The CFPs of the thin mill and fill treatment using virgin AC mix were 4, 7, 11, and 13 years in the WF, WNF, DF, and DNF regions, respectively, while the CFPs of the thick mill-and-fill treatment using virgin AC mixes are 19, 9, and 12 years in the WF, WNF, and DNF regions, respectively.
- The thin mill-and-fill treatment using recycled AC mix extended the pavement functional period by about 12 years in the WF and WNF regions, Whereas the thick mill-and-fill treatment using recycled AC mix extended the pavement functional period by about 13 years in the WF, WNF, and DNF regions, respectively.

Rut Depth

The data in table 55 indicate the following, on average:

- The thin and thick overlays using virgin AC mix extended the pavement structural period by about 13, 11, 7, and 9 years in the WF, WNF, DF, and DNF regions, respectively.
- The thin and thick overlays using recycled AC mix in the WNF region extended the pavement structural period by 7 and 14 years, respectively.
- The thin mill-and-fill treatment using virgin AC mix extended the pavement structural period by 11, 16, and 14 years in the WF, WNF, and DF regions, respectively. On the other hand, the thick mill-and-fill treatment using virgin AC mix extended the pavement structural period by 13, 20, and 9 years in the WF, WNF, and DNF regions, respectively.
- The thick mill-and-fill treatment using recycled AC mix extended the pavement structural period by 15 years in the WNF regions, whereas the thick mill-and-fill treatment using recycled AC mix caused a 5-year loss in the pavement structural period in the WF region. This was most likely due to inadequate compaction during the construction of the AC fill.

Alligator Cracking

The data in table 56 indicate the following, on average:

- The thin virgin AC mix overlays extended the pavement structural period by about 14 years in the WF and WNF regions while the thick overlay using virgin AC mix extended the pavement structural period by 10 and 6 years in WNF and DNF regions, respectively.

- In the WNF region, the thin and thick recycled AC mix overlays extended the pavement structural period by 4 and 10 years, respectively.

- The thin mill-and-fill treatment using virgin AC mix extended the pavement structural period by about 6 years in the WF and WNF regions and by 11 years in the DF region, whereas the thick mill-and-fill treatment using virgin AC mix extended the pavement structural period by 4 and 14 years in the WNF and DNF regions, respectively.

- The thick mill-and-fill treatment using recycled AC mix extended the pavement structural period by about 13 years in the WF and WNF regions.

Longitudinal Cracking

The data in table 57 indicate the following, on average:

- The thin overlays using virgin AC mix extended the pavement structural period by about 8 years in the WF and WNF regions. Likewise, the thick overlays using virgin AC mix extended the pavement structural period by 8 years in the WNF region.

- The thin and thick overlays using recycled AC mix extended the pavement structural period by about 4 years in the WNF region.

- The thin mill and fill treatment using virgin AC mix extended the pavement structural period by 5 years in the WF and WNF regions and by 1 years in the DF region, whereas the thick mill-and-fill treatment using virgin AC mix extended the pavement structural period by 14 and 6 years in the WNF and DNF regions, respectively.

- The thick mill-and-fill treatment using recycled AC mix extended the pavement structural period by about 15 years in the WF and WNF regions.

Transverse Cracking

The data in table 58 indicate the following, on average:

- The thin overlays using virgin AC mix extended the pavement structural period by about 7 years in the WF and WNF regions, whereas the thick overlays using virgin AC mix extended the pavement structural period by 14, 11, and 4 years in the WF, WNF, and DNF regions, respectively.

- The thin and thick overlays using recycled AC mix extended the pavement structural period by 3 years in the WNF region.

- The thin mill-and-fill treatment using virgin AC mix extended the pavement structural period by 5 years in the WNF region. On the other hand, the thick mill-and-fill treatment using virgin AC mix extended the pavement structural period by about 6 years in the WNF and DNF regions.

- The thin mill-and-fill treatment using recycled AC mix extended the pavement structural period by 11 years in the WNF region. The thick mill-and-fill treatment using recycled AC mix extended the pavement structural period by nine and thirteen years in the WF and WNF regions, respectively.

Impact of the Before Treatment Condition and Distress on the Performance of the Pavement After Treatment for LTPP GPS-6

Several attempts were made to analyze the impacts of the before treatment pavement condition (IRI) and distresses (rut depths and cracking) on the pavement performance after treatment. Examples of the results for thin and thick virgin AC overlay and for IRI and transverse cracking are shown in the T^2Ms in table 59 through table 62. Although only 27 and 16 test sections could be analyzed for thin and thick overlays using virgin AC mix, respectively, the results were logical and expected. The data in table 59 and table 60 indicate that the before treatment pavement condition (IRI) had minute to no effects on the RFP of the test sections. This was more pronounced for the thick AC overlay than for the thin AC overlay. That is if the AC overlay was constructed properly, it would produce a smooth pavement surface. Certainly thicker AC overlays will be constructed using two or more courses. The greater the number of the overlay courses, the smoother the final pavement surface is. The implication herein is that, if a pavement section is to be treated based on high IRIs (low ride quality), then the AC overlay should be constructed using at least two courses (two lifts). Otherwise, the original rough pavement surface could be milled to a smoother surface and then subjected to a single course (one lift) AC overlay.

Table 59. Functional T^2M for thin overlay using virgin AC mix (IRI, number of LTPP test sections).

Row Designation	Column Designation											
	A	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**	**J**	**K**	**L**
	RFP Before and After Thin Overlay Using Virgin AC Mix Based on IRI											
	Before Treatment				**After Treatment**							
	RFP CS and Number and Percent of Pavement Sections in Each CS				**RFP CS (Code and RFP Ranges (Years)) and Number of LTPP Test Sections Transferred From Each Before Treatment CS to the Indicated After Treatment CSs**					**Weighted Average FCROP, CFP, and After Treatment RFP of the Treatment (Years)**		
	CS		**LTPP Test Sections**									
	RFP Condition Code	**RFP Ranges (Years)**	**Number**	**Percent**	**1**	**2**	**3**	**4**	**5**	**FCROP**	**CFP**	**RFP**
A	1	< 2	4	15	< 2	2 to < 4	4 to < 8	8 to < 13	≥ 13	14	11	12
B	2	2 to < 4	2	7	0	**0**	0	0	2	20	13	16
C	3	4 to < 8	2	7	0	0	**0**	0	2	18	10	16
D	4	8 to < 13	15	56	0	0	0	**1**	14	10	6	16
E	5	≥ 13	4	15	0	0	0	0	**4**	9	0	16
F	Total		27	100	0	0	1	2	24	12	6	15

Note: Bolding indicates no gain.

Table 60. Functional T^2M for thick overlay using virgin AC mix (IRI, number of LTPP test sections).

Row Designation	A	B	C	D	E	F	G	H	I	J	K	L
	RFP Before and After Thick Overlay Using Virgin AC Mix Based on IRI											
	Before Treatment				**After Treatment**							
	RFP CS and Number and Percent of Pavement Sections in Each CS				**RFP CS (Code and RFP Ranges (Years)) and Number of LTPP Test Sections Transferred From Each Before Treatment CS to the Indicated After Treatment CSs**					**Weighted Average FCROP, CFP, and After Treatment RFP of the Treatment (Years)**		
	CS		**LTPP Test Sections**		**1**	**2**	**3**	**4**	**5**			
	RFP Condition Code	**RFP Ranges (Years)**	**Number**	**Percent**	**< 2**	**2 to < 4**	**4 to < 8**	**8 to < 13**	**≥ 13**	**FCROP**	**CFP**	**After Treatment RFP**
A	1	< 2	4	25	**0**	0	0	0	4	20	15	16
B	2	2 to < 4	0	0	0	**0**	0	0	0	—	—	—
C	3	4 to < 8	4	25	0	0	**0**	0	4	20	10	16
D	4	8 to < 13	5	31	0	0	0	**0**	5	10	6	16
E	5	≥ 13	3	19	0	0	0	0	**3**	18	0	16
F	Total		16	100	0	0	0	0	16	16	8	16

Note: Bolding indicates no gain.
— Indicates no data.

Table 61. Structural T²M for thin overlay using virgin AC mix (transverse cracking, number of LTPP test sections).

Row Designation	Column Designation											
	A	B	C	D	F	G	H	I	J	K	L	M
	RSP Before and After Thin Overlay Using Virgin AC Mix Based on Transverse Cracking											
	Before Treatment				After Treatment							
	RSP CS and Number and Percent of Pavement Sections in Each CS				RSP CS (Code and RSP Ranges (Years)) and Number of LTPP Test Sections Transferred From Each Before Treatment RSP CS to the Indicated After Treatment RSP CSs					Weighted Average SCROP, CSP, and After Treatment RSP of the Treatment (Years)		
	CS		LTPP Test Sections		1	2	3	4	5			
	RSP Condition Code	RSP Ranges (Years)	Number	Percent	< 2	2 to < 4	4 to < 8	8 to < 13	≥ 13	SCROP	CSP	After Treatment RSP
A	1	< 2	3	23	**1**	0	0	2	0	9	6	7
B	2	2 to < 4	2	15	0	**0**	0	0	2	12	13	16
C	3	4 to < 8	4	31	0	1	**0**	1	2	7	5	11
D	4	8 to < 13	3	23	0	0	1	**0**	2	6	3	13
E	5	≥ 13	1	8	0	0	0	1	**0**	6	–6	10
F	Total		13	100	1	1	1	4	6	8	5	11

Note: Bolding indicates no gain.

Table 62. Structural T^2M for thick overlay using virgin AC mix (transverse cracking, number of LTPP test sections).

Row Designation	Column Designation											
	A	**B**	**C**	**D**	**F**	**G**	**H**	**I**	**J**	**K**	**L**	**M**
	RSP Before and After Thick Overlay Using Virgin AC Mix Based on Transverse Cracking											
	Before Treatment				**After Treatment**							
	RSP CS and Number and Percent of Pavement Sections in Each CS				**RSP CS (Code and RSP Ranges in Years) and Number of LTPP Test Sections Transferred From Each Before Treatment RSP CS to the Indicated AT RSP CSs**					**Weighted Average SCROP, CSP, and After Treatment RSP of the Treatment (Years)**		
	CS		**LTPP Test Sections**									
	RSP Condition Code	**RSP Ranges (Years)**	**Number**	**Percent**	**1** **< 2**	**2** **2 to < 4**	**3** **4 to < 8**	**4** **8 to < 13**	**5** **≥ 13**	**SCROP**	**CSP**	**After Treatment RSP**
A	1	< 2	3	60	**0**	0	0	1	2	13	13	14
B	2	2 to < 4	0	0	0	**0**	0	0	0	—	—	—
C	3	4 to < 8	2	40	0	0	**0**	2	0	10	4	10
D	4	8 to < 13	0	0	0	0	0	**0**	0	—	—	—
E	5	≥ 13	0	0	0	0	0	0	**0**	—	—	—
F	Total		5	100	0	0	0	3	2	12	9	12

Note: Bolding indicates no gain.
— Indicates no data.

Unfortunately, there were fewer test sections available for analyses of the structural period. Table 61 and table 62 for thin and thick virgin AC overlay, respectively, list the results of the analyses of the impacts of transverse cracking before treatment on the pavement performance in terms of transverse cracking after treatment. There were only 13 test sections for thin AC overlay and only 5 for the thick AC overlay. When these limited sections were distributed among the five CSs before treatment, the number of test sections in each CS became statistically insignificant to support reliable conclusions. However, the limited data indicated that the longer the length of transverse cracks before treatment was, the worse the pavement performance after treatment was. Further, the thick AC overlay performed better than the thin overlay; it retarded reflective cracking better.

Summary, Conclusions, and Recommendations for LTPP GPS-6

The performance of pavement rehabilitation is a function of many variables, including the type of rehabilitation, the material used, construction, traffic, and climate. Results of the analyses of the GPS-6 test sections confirmed that. Although the GPS-6 test sections did not represent enough data for detailed analyses of each variable, several cautious conclusions could be drawn given the limited number of test sections. These conclusions include the following:

- In each climatic region, the impacts of the thin and thick overlay or mill-and-fill treatment on IRI were almost the same. This was expected because good quality construction could decrease the pavement roughness substantially regardless of the overlay thickness.

- Thin and thick overlays and mill-and-fill treatments had a similar impact on rut depths. Once again, this was expected because most pavement rutting occurs early in the pavement life and can be removed during the treatment. One word of caution is that poor compaction of the overlay could precipitate early rutting in the AC overlay.

- The impact of the AC overlay and mill-and-fill treatments on pavement performance in terms of alligator and longitudinal cracking could not be assessed with an acceptable degree of certainty because of the limited number of test sections.

- The impact of thin and thick overlay on pavement performance in terms of transverse cracking was very much as expected; the thicker the overlay or the mill-and-fill treatment was, the more the reflective transverse cracking was retarded and the longer the pavement structural period was.

- The effects of the conditions in the different climatic regions on the pavement condition and distress could not be fully assessed because of the limited number of test sections in each climatic region.

Based on the results of the analyses, the following is strongly recommended:

- The data collection frequency on newly designed and constructed or newly rehabilitated LTPP test sections should be increased to a minimum of once a year, twice a year for test sections subjected to light rehabilitation, and three times a year for test section subjected to maintenance treatments.

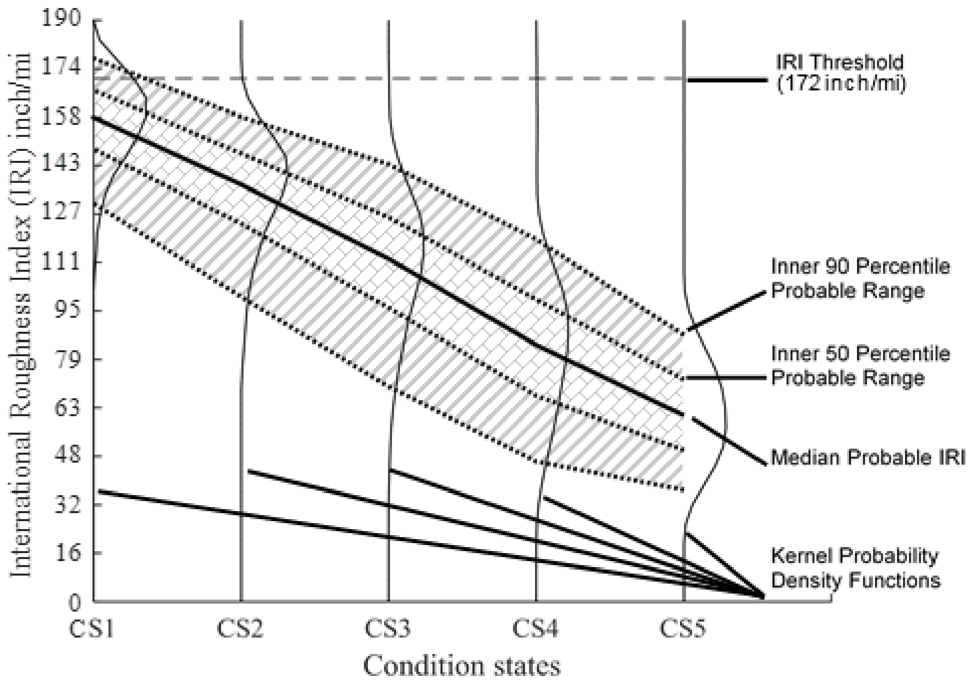
- The construction process should be documented, and the quality control data should be included in the database.
- Future analyses of pavement condition and distress data should be based on the new pavement rating and classification systems RFP and RSP.
- The benefits of pavement rehabilitation and/or maintenance treatments should be measured in terms of RFP or RSP, CFP or CSP, and FCROP or SCROP.

ORCSE METHOD

The procedures for the analyses for flexible pavement sections described earlier in this chapter provide a basis for evaluating pavement sections for which there is sufficient data to meet the data quality control requirements outlined in chapter 4. As discussed, not all State transportation departments maintain sufficient pavement management system (PMS) data to use these outlined methods or may not regularly monitor all pavement sections to the same rigorous standards. The estimation of a pavement section RFP or RSP from a single condition or distress record would be extremely beneficial for these State transportation departments to use on a network and individual section basis. The remainder of chapter 5 presents a novel method developed using the LTPP data for estimation of RFP or RSP for pavement sections that have experienced lower levels of monitoring or sections that may not yet have sufficient data records for modeling owing to age. Herein, the method is referred to as the ORCSE method. It is very important to note that the development of this method was not a part of the original study or study objective. Although it is included herein to show that such analyses are possible, it requires further study and calibration, and it must be emphasized that probabilistic issues are very important and should be considered in a more balanced and comprehensive PMS.

ORCSE Method and Development Procedures Using LTPP Data

The ORCSE method is a probabilistic model used to estimate the probability of a pavement section being part of a specific CS (RFP or RSP range) based on a single condition or distress record as shown in figure 56. This probability is developed using multiple probability distributions functions calibrated to local or regional pavement design groups, specific designs, construction practices, and other design or management factors. Similar to the local calibration recommended for MEPDG use in the FHWA's *Local Calibration of the MEPDG Using Pavement Management Systems, Final Report, Volume I*, the ORCSE method provides the best estimates of a pavement section CS when calibrated specifically to one or multiple of these design or management constraints as long as those constraints are significant factors in pavement performance.[82]



1 inch/mi = 0.0158 m/km.

Figure 56. Graph. ORCSE model probability graph from LTPP SPS-1 before treatment evaluation.

The development of the ORCSE model for a group of pavement sections using an entire database of newly constructed pavements should follow eight steps. The flowchart in figure 57 illustrates the process. It is important to note that the ORCSE model should be tested using model validation techniques to ensure consistent results when using it to estimate the RFP/RSP of pavement sections.

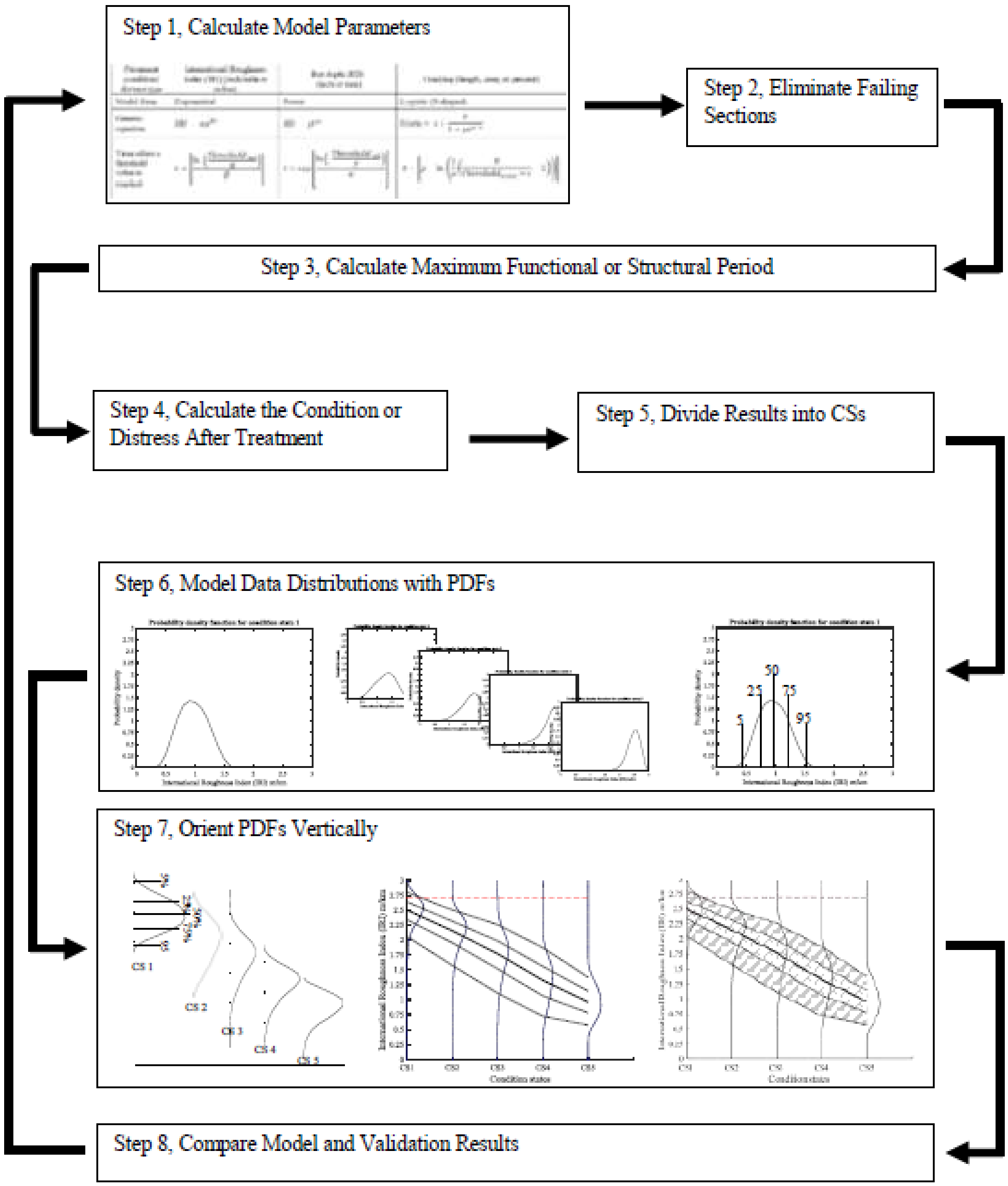


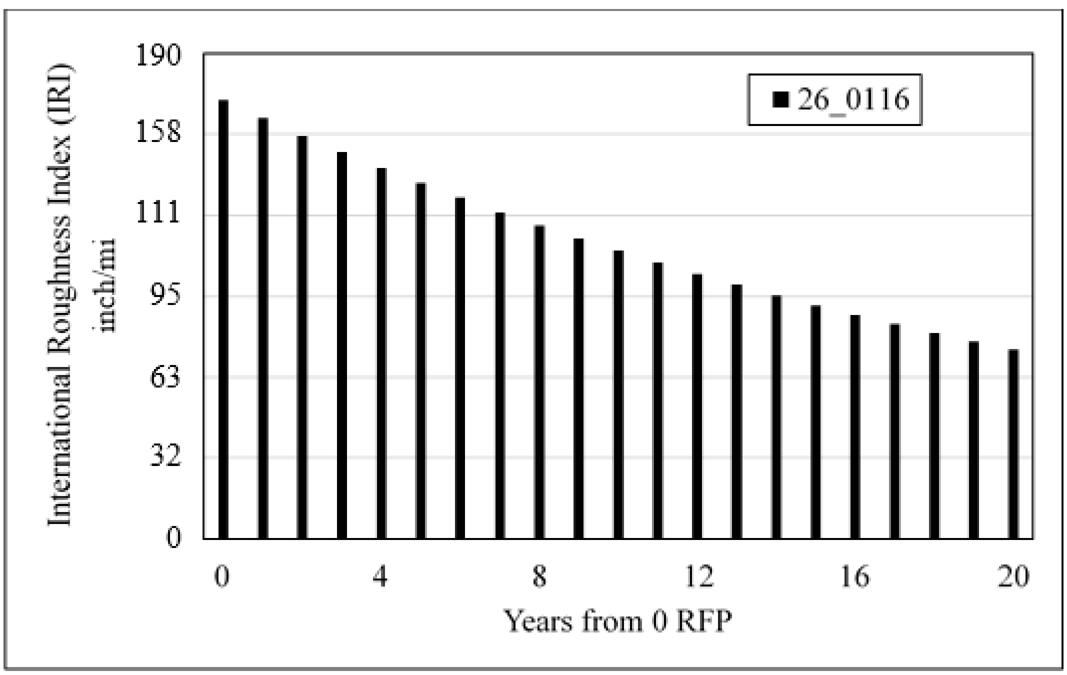
Figure 57. Illustration. Flowchart of ORCSE model steps with examples from LTPP SPS-1 before treatment evaluation.

Note that although steps 1 through 3 were discussed in greater detail in previous sections of this chapter and are the basis for the RFP/RSP approach recommended in this report, they are summarized here with minor modifications to show the complete process:

- **Step 1**: For each pavement section and collection of condition or distress data, calculate the model parameters of the appropriate condition or distress model (see table 27).

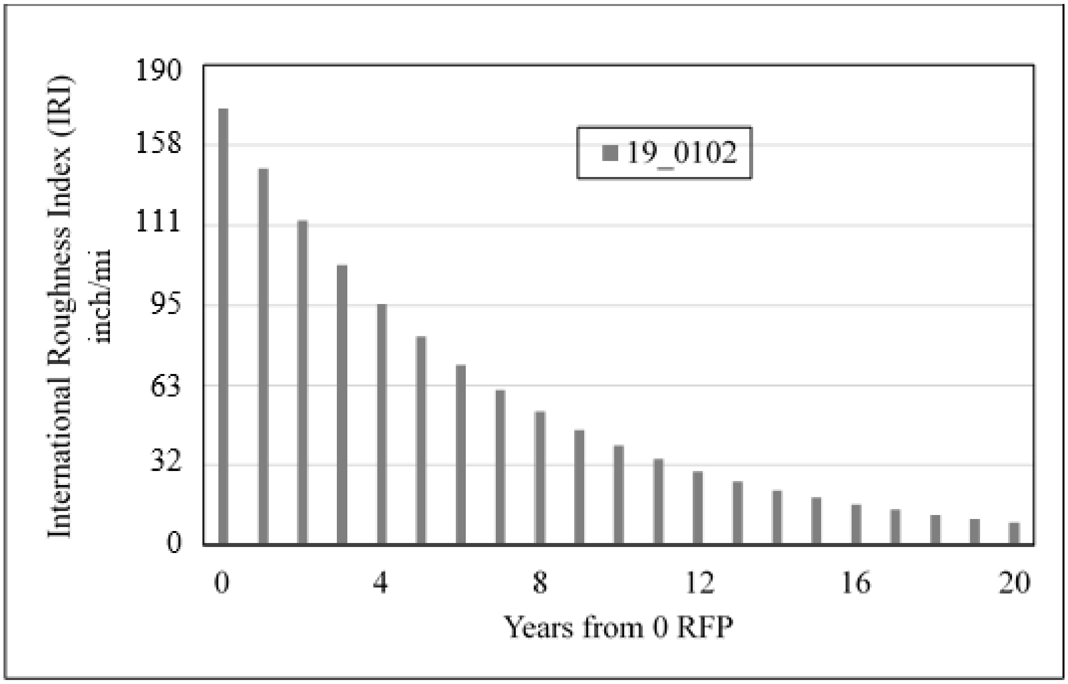
- **Step 2**: Eliminate sections that fail to meet preestablished criteria, and, for the accepted sections, separate a portion of your accepted data for modeling and a portion for validation as outlined by the selected validation method. These criteria could include the following:
  - Negative condition or distress growth.
  - Construction-related failures observed early in pavement life or field observations of poor construction.
  - Poor model fitness for the distress data.

  It should be noted that a model could be constituted specifically of poorly constructed pavement sections to provide an estimate for other sections that might have similar construction deficiencies.
- **Step 3**: For sections that pass step 2, calculate the maximum functional or structural period using the previously fit model parameters (step 1) for each section.
- **Step 4:** For the maximum functional or structural period calculated in step 3, calculate the condition or distress back to the date of construction for each individual section on a yearly basis. If condition or distress values are not available for the period immediately after construction, consider using back propagating to the time of construction (figure 58 to figure 61).



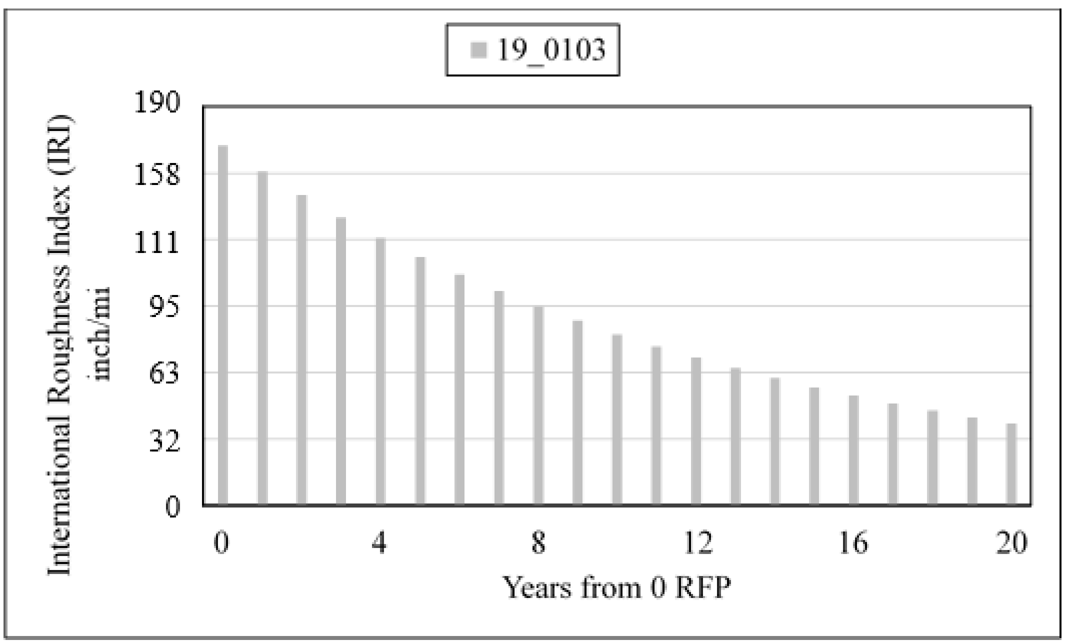
1 inch/mi = 0.0158 m/km.

Figure 58. Graph. Backpropagation of IRI data for SHRP ID 26_0116 from an RFP of 0 years.



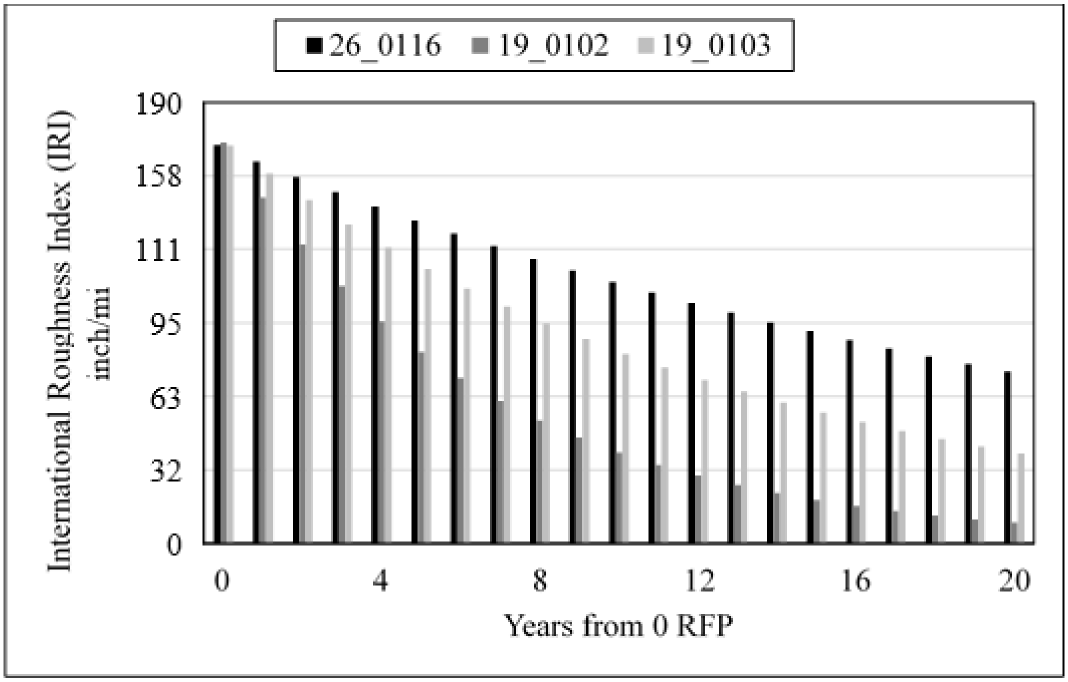
1 inch/mi = 0.0158 m/km.

Figure 59. Graph. Backpropagation of IRI data for SHRP ID 19_0102 from an RFP of 0 years.



1 inch/mi = 0.0158 m/km.

Figure 60. Graph. Backpropagation of IRI data for SHRP ID 19_0103 from an RFP of 0 years.



1 inch/mi = 0.0158 m/km.

Figure 61. Graph. Backpropagation of IRI data for SHRP IDs 26_0116, 19_0102, and 19_0103 displaying variation in IRI growth from an RFP of 0 years.

- **Step 5:** Divide the condition or distress values into their respective CSs based on ranges of RFP or RSP (see chapter 3). Recall that CS 1 corresponds to the RFP or RSP range from 0 to 2 years, and CS 5 corresponds to an RFP or RSP of 13 or more years.

- **Step 6**: For each of the grouped CS data collections (CS 1, CS 2, CS 3, CS 4, and CS 5), model the data distributions using a user-preferred probability density function (PDF), which has been tested for fitness, and calculate the 5th, 25th, 50th, 75th, and 95th percentile for each PDF. These percentiles will be associated with the median, middle 50, and middle 90 percent of pavement section behaviors per CS.

- **Step 7**: Rotate each PDF vertically and orient each over its respective CS, connect each equivalent percentile range with line segments, and display the condition or distress threshold if it is applicable or assists in use.

- **Step 8**: Compare results between model and validation data groups using a selected cross validation technique to ensure similar results with any subset of data, ensuring accurate estimates of RFP and RSP for pavement sections to be assessed.

Specific details for individual steps, as applied for this report, include the following:

- An exponential model was applied to evaluated IRI data.

- The same elimination data as discussed early in this report for IRI data were applied as well as one additional criterion—pavements with functional and structural periods longer than 25 years were removed because of the typically low fitness of the models and early-life records, which prevented construction of a complete lifecycle model. This is an important aspect of this method, and best results should be observed if model fitting is completed on sections that have substantially surpassed the established threshold before maintenance or rehabilitation occurred.

- For model validation, a repeated random subsampling validation was used with an 80/20 model-validation split. Modeling and validation were completed five times.

- The PDF used was Epanechnikov Kernels PDF modeling, which produced the closest fit to the observed data while providing secondary benefits of allowing visual identification of possible sub-probability groups or divergent behaviors within the larger generalized sample. Divergent or bimodal behaviors might be indicative of construction issues or premature failure of multiple sections otherwise experiencing similar condition or distress growth. Although this modeling method was selected, any PDF or Kernel method may be appropriate after evaluation of the goodness of fit.

The ORCSE model graph may also be represented as a table considering the CS probability for a set of condition or distress values or range of values. An example with averaged weighted probabilities per 16 inches/mi (0.25 m/km) IRI range can be reviewed in table 63. Note that shading could be added to the table to emphasize the best match.

Table 63. ORCSE model table example from LTPP SPS-1 before treatment evaluation.

IRI Range (inches/mi)	Probability of a CS or RFP Bracket for Selected IRI Ranges (percent)				
	CS 1 or RFP < 2 years	CS 2 or RFP 2 to < 4 years	CS 3 or RFP 4 to < 8 years	CS 4 or RFP 8 to < 13 years	CS 5 or RFP ≥ 13 years
16–32	0	0	0	3	97
32–48	0	0	3	14	83
48–63	0	0	4	19	77
63–79	0	0	9	39	52
79–95	0	3	19	66	11
95–111	0	12	49	37	1
111–127	3	28	59	10	0
127–143	13	61	22	3	0
143–158	55	36	6	3	0
158–171	88	8	1	2	0

1 inch/mi = 0.0158 m/km.

The procedure for creating a table from the ORCSE model graph is outlined in the following four steps:

- For each analysis grouping (i.e., SPS-1 all climates), delineate the experienced condition or distress ranges desired into subdistress ranges (16 inches/mi (0.25 m/km) intervals for this report using IRI).

- For each of the subdistress values, record probabilities from the previously created ORCSE model for each of the five CSs.

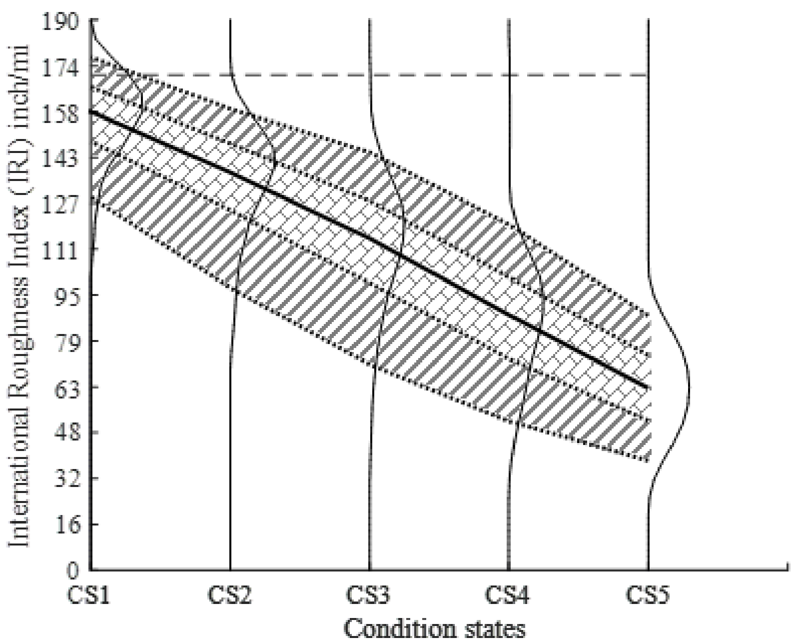
- For each subdistress range (i.e., 16 to 32 inches/mi (0.25 to 0.50 m/km)), average and enter PDF probabilities into a preliminary ORCSE table.

- Modify the preliminary table by weighting the probabilities for each subdistress across all five CS columns. This results in a cumulative 100-percent probability for each subdistress row and helps to emphasize the differences in raw CS probabilities.

For example, the IRI range of 16 to 32 inches/mi (0.25 to 0.50 m/km) for CS 5 may have 24- and 30-percent probability, respectively. The average probability for the range is thus the average of 24 and 30 percent—27 percent. If it is assumed that for the remaining CS groups, whose percent probabilities are 20, 12, 8, and 1 percent for CS 4, CS 3, CS 2, and CS 1, respectively, the weighted probabilities can be found. Weighted probabilities are calculated for each CS as the averaged probability of the selected CS divided by the summation of all CSs for the subdistress range (16 to 32 inches/mi (0.25 to 0.50 m/km)). For this example, the weighted values are 40, 29, 18, 12, and 1 percent probability for CS 5, CS 4, CS 3, CS 2, and CS 1, respectively. It can then be best estimated that for an IRI between 16 to 32 inches/mi (0.25 and 0.50 m/km), the pavement has the greatest chance of being in CS 5 with a greater than or equal to 13-year RFP.

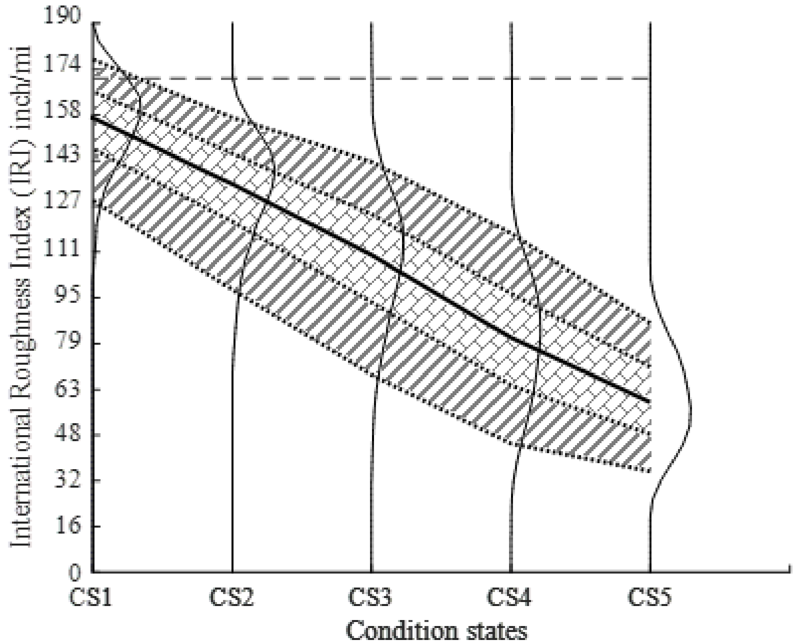
Results of ORCSE Predictions and Analyses of LTPP SPS-1 Test Sections

Based on the procedures provided in chapter 5, analyses were completed for ORCSE models based on the same LTPP SPS-1 data collection. The results were generated in both ORCSE model graphs and tables for a total of five iterations, a randomly selected sample group from the complete dataset, to provide model validation. Figure 62 to figure 66 depict the resulting ORCSE graphs for the respective resampling.



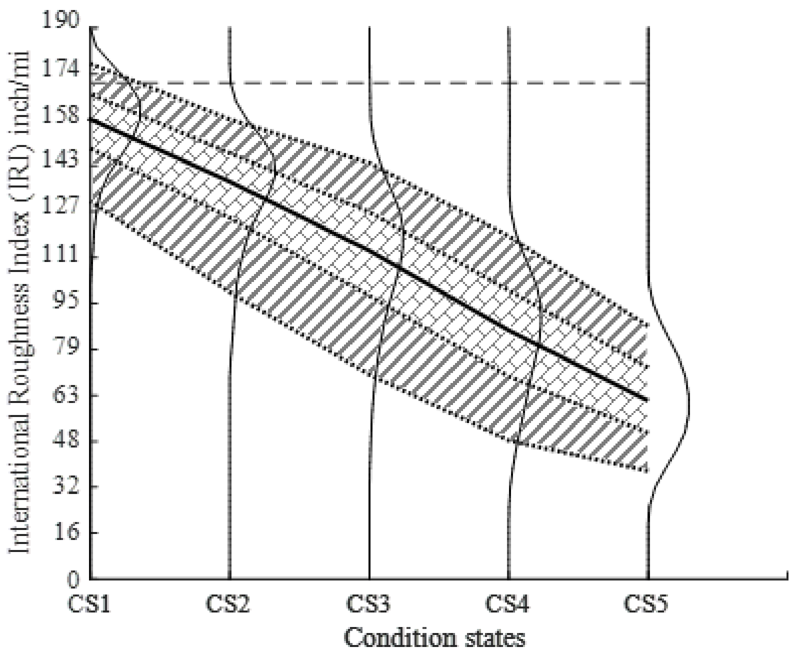
1 inch/mi = 0.0158 m/km.

Figure 62. Graph. ORCSE model graph for LTPP SPS-1 virgin pavement analysis for iteration 1.



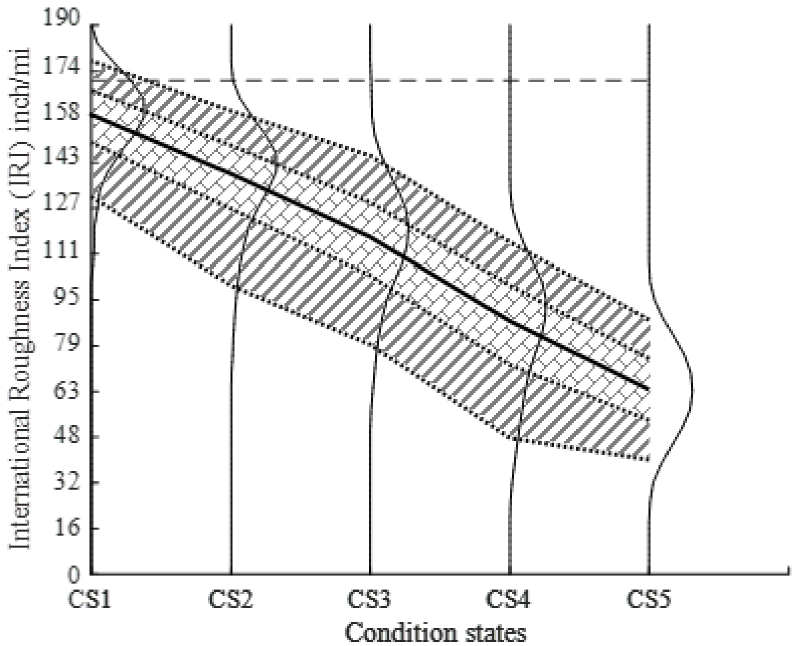
1 inch/mi = 0.0158 m/km.

Figure 63. Graph. ORCSE model graph for LTPP SPS-1 virgin pavement analysis for iteration 2.



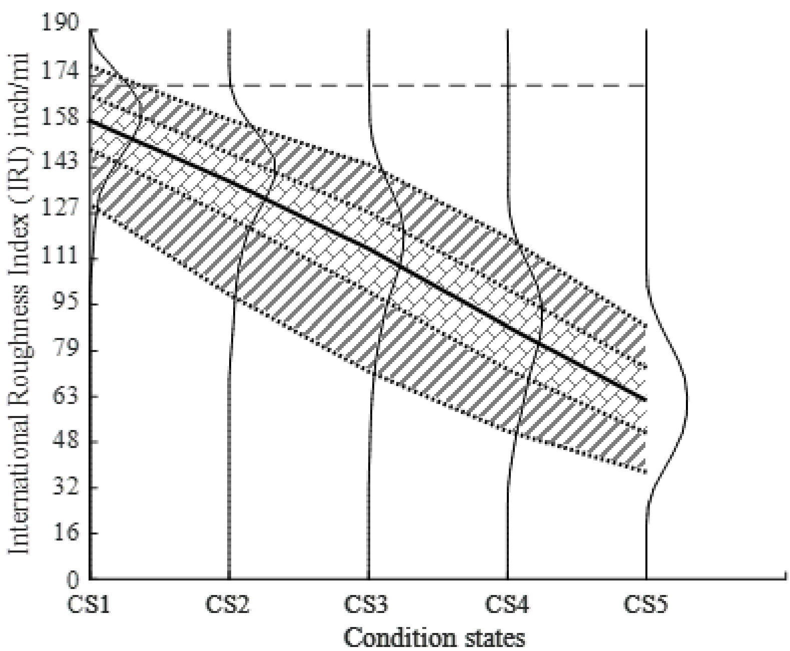
1 inch/mi = 0.0158 m/km.

Figure 64. Graph. ORCSE model graph for LTPP SPS-1 virgin pavement analysis for iteration 3.



1 inch/mi = 0.0158 m/km.

Figure 65. Graph. ORCSE model graph for LTPP SPS-1 virgin pavement analysis for iteration 4.



1 inch/mi = 0.0158 m/km.

Figure 66. Graph. ORCSE model graph for LTPP SPS-1 virgin pavement analysis for iteration 5.

Results of Validation of ORCSE Predictions and Analyses of LTPP SPS-1 Test Sections

Table 64 through table 68 list each of the resulting ORCSE models for the respective sampling.

Table 64. ORCSE model table for LTPP SPS-1 virgin pavement analysis for iteration 1.

IRI Range (inches/mi)	Probability of a CS or RFP Bracket for Selected IRI Ranges (percent)				
	CS 1 or RFP < 2 Years	CS 2 or RFP 2 to < 4 Years	CS 3 or RFP 4 to < 8 Years	CS 4 or RFP 8 to < 13 Years	CS 5 or RFP ≥ 13 Years
16–32	0	0	0	3	97
32–48	0	0	3	14	83
48–63	0	0	4	19	77
63–79	0	0	9	39	52
79–95	0	3	19	66	11
95–111	0	12	49	37	1
111–127	3	28	59	10	0
127–143	13	61	22	3	0
143–158	55	36	6	3	0
158–171	88	8	1	2	0

1 inch/mi = 0.0158 m/km.

Table 65. ORCSE model table for LTPP SPS-1 virgin pavement analysis for iteration 2.

IRI Range (inches/mi)	Probability of a CS or RFP Bracket for Selected IRI Ranges (percent)				
	CS 1 or RFP < 2 Years	CS 2 or RFP 2 to < 4 Years	CS 3 or RFP 4 to < 8 Years	CS 4 or RFP 8 to < 13 Years	CS 5 or RFP ≥ 13 Years
16–32	0	0	0	19	81
32–48	0	0	2	21	77
48–63	0	0	3	25	71
63–79	0	0	10	51	38
79–95	0	3	28	59	10
95–111	0	13	59	27	1
111–127	3	35	53	9	0
127–143	15	65	17	3	0
143–158	62	31	5	3	0
158–171	90	6	1	2	0

1 inch/mi = 0.0158 m/km.

Table 66. ORCSE model table for LTPP SPS-1 virgin pavement analysis for iteration 3.

IRI Range (inches/mi)	Probability of a CS or RFP Bracket for Selected IRI Ranges (percent)				
	CS 1 or RFP < 2 Years	CS 2 or RFP 2 to < 4 Years	CS 3 or RFP 4 to < 8 Years	CS or RFP 4 8 to < 13 Years	CS 5 or RFP ≥ 13 Years
16–32	0	0	0	21	79
32–48	0	0	2	20	78
48–63	0	0	3	22	74
63–79	0	0	9	44	47
79–95	0	3	21	65	11
95–111	0	12	53	34	1
111–127	3	29	59	9	0
127–143	13	63	21	3	0
143–158	56	36	5	3	0
158–171	90	7	1	2	0

1 inch/mi = 0.0158 m/km.

Table 67. ORCSE model table for LTPP SPS-1 virgin pavement analysis for iteration 4.

IRI Range (inches/mi)	Probability of a CS or RFP Bracket for Selected IRI Ranges (percent				
	CS 1 or RFP < 2 Years	CS 2 or RFP 2 to < 4 Years	CS 3 or RFP 4 to < 8 Years	CS 4 or RFP 8 to < 13 Years	CS 5 or RFP ≥ 13 Years
16–32	0	0	0	34	66
32–48	0	0	0	27	73
48–63	0	0	1	25	75
63–79	0	0	4	40	55
79–95	0	3	14	71	13
95–111	0	12	47	39	1
111–127	3	25	65	7	0
127–143	12	62	26	0	0
143–158	53	41	7	0	0
158–171	89	10	1	0	0

1 inch/mi = 0.0158 m/km.

Table 68. ORCSE model table for LTPP SPS-1 virgin pavement analysis for iteration 5.

IRI Range (inches/mi)	Probability of a CS or RFP Bracket for Selected IRI Ranges (percent)				
	CS 1 or RFP < 2 years	CS 2 or RFP 2 to < 4 years	CS 3 or RFP 4 to < 8 years	CS 4 or RFP 8 to < 13 years	CS 5 or RFP ≥ 13 years
16–32	0	0	0	2	98
32–48	0	0	2	13	84
48–63	0	0	3	19	78
63–79	0	0	9	40	50
79–95	0	3	19	68	11
95–111	0	12	50	37	1
111–127	3	27	60	10	0
127–143	13	62	22	3	0
143–158	54	37	6	3	0
158–171	89	8	1	2	0

1 inch/mi = 0.0158 m/km.

The ORCSE model figures visually indicate that each model iteration produced similar results, with the most significant variation in iteration 4 primarily adjusting the inner 90-percent pavement section behavior. These outer behaviors can be used to identify outliers existing in the accepted condition and distress model data as well as to visually approximate the variety of aging behavior in each modeled group. This is completed by assessing the width of the distribution at each CS—the wider the distribution is, the wider the rates of pavement condition or distress change are.

The ORCSE model tables list results similar to those shown in the figures. The tables present the same maximum probability trends for each IRI range to CS group and approximately match the change in distribution width for each CS. While not demonstrated here, the change in distribution width for each CS can also be used to approximate the variety of sections present for a modeled pavement section set. The greater the number of nonzero cells per row is, the lower the maximum relative probability per row is and the wider the range of section behavior.

The five iterations used to evaluate the consistency of the ORCSE model results were based on an 80/20 model-validation split using repeated random subsampling validation with five model iterations, as presented earlier. This numeric assessment of the models can be completed by comparing the difference between CS ranges estimated using ORCSE figures and tables with the modeled functional or structural period per section for each year the pavement section is modeled. This analysis provides the approximate number and percent of wrong predictions for similar pavements (i.e., those that have a similar design or management characteristics as those modeled) as well as details about which CS groups are most commonly incorrectly estimated by the ORCSE method.

Individual iteration analyses are presented in table 69 through table 73. These analyses can also be presented as a total number of differences for all five iterations, an average, and a median difference across each iteration. All three analyses are presented in table 74 through table 76.

Table 69. ORCSE validation of estimated versus modeled CS groups for iteration 1.

ORCSE Estimated CS	Modeled CS					Difference in CS			Incorrect Estimates (Percent)
	CS 1	CS 2	CS 3	CS 4	CS 5	Min	Max	Total	
CS 1	219	1	0	0	0	–1	–1	1	0
CS 2	0	217	3	0	0	–1	–1	3	1
CS 3	0	1	214	3	2	–2	1	6	3
CS 4	0	0	9	200	11	–1	1	20	9
CS 5	0	0	3	13	204	1	2	16	7

Min = Minimum.
Max = Maximum.

Table 70. ORCSE validation of estimated versus modeled CS groups for iteration 2.

ORCSE Estimated CS	Modeled CS					Difference in CS			Incorrect Estimates (Percent)
	CS 1	CS 2	CS 3	CS 4	CS 5	Min	Max	Total	
CS 1	240	0	0	0	0	0	0	0	0
CS 2	3	237	0	0	0	1	1	3	1
CS 3	0	4	236	0	0	1	1	4	2
CS 4	0	0	14	226	0	1	1	14	6
CS 5	0	0	0	95	145	1	1	95	40

Min = Minimum.
Max = Maximum.

Table 71. ORCSE validation of estimated versus modeled CS groups for iteration 3.

ORCSE Estimated CS	Modeled CS					Difference in CS			Incorrect Estimates (Percent)
	CS 1	CS 2	CS 3	CS 4	CS 5	Min	Max	Total	
CS 1	160	0	0	0	0	0	0	0	0
CS 2	0	159	1	0	0	–1	–1	1	1
CS 3	0	0	158	2	0	–1	–1	2	1
CS 4	0	0	0	155	5	–1	–1	5	3
CS 5	0	0	0	18	142	1	1	18	11

Min = Minimum.
Max = Maximum.

Table 72. ORCSE validation of estimated versus modeled CS groups for iteration 4.

ORCSE Estimated CS	Modeled CS					Difference in CS			Incorrect Estimates (Percent)
	CS 1	CS 2	CS 3	CS 4	CS 5	Min	Max	Total	
CS 1	220	0	0	0	0	0	0	0	0
CS 2	2	214	4	0	0	–1	1	6	3
CS 3	0	3	208	7	2	–2	1	12	5
CS 4	0	0	9	194	17	–1	1	26	12
CS 5	0	0	1	16	203	1	2	17	8

Min = Minimum.
Max = Maximum.

Table 73. ORCSE validation of estimated versus modeled CS groups for iteration 5.

ORCSE Estimated CS	Modeled CS					Difference in CS			Incorrect Estimates (Percent)
	CS 1	CS 2	CS 3	CS 4	CS5	Min	Max	Total	
CS 1	199	1	0	0	0	–1	–1	1	1
CS 2	0	198	2	0	0	–1	–1	2	1
CS 3	0	1	195	2	2	–2	1	5	3
CS 4	0	0	7	185	8	–1	1	15	8
CS 5	0	0	8	27	165	1	2	35	18

Min = Minimum.
Max = Maximum.

Table 74. ORCSE summarized validation of estimated versus modeled CS groups for all iterations.

ORCSE Estimated CS	Modeled CS					Difference in CS			Incorrect Estimates (Percent)
	CS 1	CS 2	CS 3	CS 4	CS 5	Min	Max	Total	
CS 1	1,038	2	0	0	0	–1	0	2	0
CS 2	5	1,025	10	0	0	–1	1	15	1
CS 3	0	9	1,011	14	6	–2	1	29	3
CS 4	0	0	39	960	41	–1	1	80	8
CS 5	0	0	12	169	859	1	2	181	17

Min = Minimum.
Max = Maximum.

Table 75. ORCSE averaged validation of estimated versus modeled CS groups, all iterations.

ORCSE Estimated CS	Modeled CS					Difference in CS			Incorrect Estimates (Percent)
	CS 1	CS 2	CS 3	CS 4	CS 5	Min	Max	Total	
CS 1	208	0	0	0	0	0	0	1	0
CS 2	1	205	2	0	0	–1	0	3	1
CS 3	0	2	202	3	1	–1	1	6	3
CS 4	0	0	8	192	8	–1	1	16	7
CS 5	0	0	2	34	172	1	2	36	17

Min = Minimum.
Max = Maximum.

Table 76. ORCSE median validation of estimated versus modeled CS groups, all iterations.

ORCSE Estimated CS	Modeled CS					Difference in CS			Incorrect Estimates (Percent)
	CS 1	CS 2	CS 3	CS 4	CS 5	Min	Max	Total	
CS 1	219	0	0	0	0	0	0	0	0
CS 2	0	214	2	0	0	–1	–1	2	1
CS 3	0	1	208	2	2	–2	1	5	3
CS 4	0	0	9	194	8	–1	1	17	8
CS 5	0	0	1	18	165	1	2	19	11

Min = Minimum.
Max = Maximum.

The validation results indicate that for the most critical CS groups, CS 1 and CS 2, representing from the time the condition or distress threshold was reached to approximately 4 years beforehand, there were very few ORCSE estimation errors. The total errors during that period were only 17 across 2,080 total estimates, or less than 1 percent of all CS 1 and CS 2 estimates. For CS 1, the maximum difference was an overestimate of 1 CS group by the ORCSE estimation, representing a change from 0 to 2 years to 2 to 4 years. CS 2 also experienced the maximum estimate difference of 1 CS group with both occurrences of over and under-predicting CS groups compared with those sections modeled at CS 2. It is important to note that a change from one CS group to another in estimation may not be the entire width of the group. For an overestimation of CS 2 and CS 3, the modeled remaining years may be 5 while the estimated remaining years are 4, a difference of only 1 year.

The remaining CS groups experienced higher estimation errors, ranging from 1 to 3 percent for CS 3, 3 to 12 percent for CS 4, and 7 to 40 percent for CS 5, with average and median errors for each typically near the lower to middle of these ranges indicating that the majority of interactions experienced strong results. Further, the maximum CS difference in estimation between the ORCSE model method and the distress projection from the beginning of this chapter was only two CSs of the five total, again indicating a typically well-fit estimation of pavement section CS.

Summary, Conclusions, and Recommendations for the ORCSE Model

The ORCSE method, developed using LTPP data for estimation of RFP or RSP, was applied to pavement sections that had experienced lower levels of monitoring or sections that might not yet have had sufficient data records for modeling owing to age. Modeling and validation of this novel method indicate the following:

- The ORCSE method was successful in predicting pavement sections CSs throughout the pavement sections' entire functional or structural period.
- The majority of incorrect estimates by the ORCSE method occurred soon after treatment when additional time was available to continue recording condition or distress data and perform comprehensive section modeling.
- The ORCSE method average estimation error for the most critical CSs—CS 1 and CS 2—was less than 1 percent.
- The ORCSE method was resilient when using different data selections for calibration based on repeated subsampling validation.

It is recommended that the ORCSE method be expanded to address additional conditions and distresses as well as applied to a wider range of LTPP and State transportation department data to further verify its successful prediction of CS groups. There is a significant potential benefit to local roadway owners as well as Federal roadway managers and State transportation departments in the use of the ORCSE method when planning pavement preservation, rehabilitation, and reconstruction.

CHAPTER 6. LTPP DATA ANALYSES OF RIGID PAVEMENTS

Chapter 5 presented the results of the analyses of the time-series condition and distress data of the LTPP flexible pavement test sections. This chapter presents the results of the analyses of the time-series condition and distress data of the LTPP rigid pavement test sections. Once again, the data used in this study were obtained from the LTPP database Standard Data Release 28.0.

IMPACTS OF MAINTENANCE TREATMENTS ON PAVEMENT CONDITION AND DISTRESS USING THE LTPP SPS-2 TEST SECTIONS

The main objective of the SPS-2 experiment was to study the effects of the following:(75)

- Climatic region.
- Drainage.
- Slab thickness (8 and 11 inches (203 and 279 mm)).
- The 14-day concrete flexural strength (551 and 899 lbf/inch^2 (3.8 and 6.2 MPa)).
- Slab width (12 and 14 ft (3.66 and 4.27 m)).
- Base type (dense-graded aggregate base, asphalt-treated base, permeable asphalt-treated base, and a combination thereof).

Analysis Steps

In this study, the analyses of the impacts of the various design variables were accomplished using the following steps:

- **Step 1**: For each pavement test section in the LTPP SPS-2 experiment, the time-dependent pavement condition (IRI) and distress (transverse and longitudinal cracking) data were downloaded from the LTPP database, organized, and analyzed. Results of the analyses included the RFP and RSP of each test section calculated as the time period from the time of construction to the time when the pavement condition or distress reached the appropriate threshold values. The reason for calculating RFP and RSP from the construction data (surface age is 0 years) was that the dates of construction and the dates of the last data collection for the SPS-2 test sections were not the same. The implication of this is that the reference time for each SPS-2 test section was taken as the date of construction.

- **Step 2**: For each pavement condition and distress type, the resulting RFPs and RSPs and other inventory data (such as SHRP ID, State, slab thickness, drainage, slab width, concrete flexural strength, and so forth) were then organized in a Microsoft® Excel spreadsheet format.

- **Step 3:** For each SHRP ID and for each pavement condition and distress type, the minimum and maximum RFPs and RSPs and their averages were calculated and listed in the Microsoft® Excel spreadsheets.

- **Step 4**: The data were then organized into the following groups and subgroups and in table 77. The main objective of the division was to separate the design variables affecting pavement performance.

    - **Climatic region groups**: The results of the analyses were organized into the four climatic regions—WF, WNF, DF, and DNF.

    - **Slab thickness subgroups**: The results of the analyses in each climatic region were then organized into two subgroups based on the slab thicknesses of 8 and 11 inches (203 and 279 mm).

    - **Slab width subgroups**: The results of the analyses in each slab thickness subgroup were then organized into two subgroups based on the slab widths of 12 and 14 ft (3.66 and 4.27 m).

    - **Concrete flexural strength subgroups**: The results in each slab width subgroup were then organized into two subgroups based on the concrete 14day flexural strength of 551 and 899 lbf/inch^2 (3.8 and 6.2 MPa).

    - **Drainage subgroups**: The results in each concrete flexural strength subgroup were further divided into two drainage subgroups (presence and absence of drainage).

Table 77. Analysis subgroups and the number of test sections available for analyses in the LTPP SPS-2 experiment in each subgroup.

Condition/ Distress Type	Lane Width (ft)	Slab Strength (lbf/inches^2)	Slab Thickness (inches)	Number of Available Test Sections Based on Climatic Region and Presence of Aggregate Base Drainage							
				WF		WNF		DF		DNF	
				D	ND	D	ND	D	ND	D	ND
IRI	12	500	8	1	4	1	4	2	2	1	2
			11	3	8	1	2	3	2	1	2
		900	8	3	7	1	2	3	2	1	1
			11	1	3	2	3	1	2	1	2
	14	500	8	3	6	1	2	3	2	1	2
			11	1	4	1	4	1	2	1	2
		900	8	2	4	0	4	2	2	1	2
			11	2	7	0	2	2	2	1	2
Longitudinal cracking	12	500	8	2	3	1	4	2	3	1	2
			11	4	8	1	2	1	2	1	2
		900	8	4	8	1	2	0	2	1	2
			11	2	3	2	4	1	2	1	2
	14	500	8	4	7	1	2	1	2	1	2
			11	2	4	2	4	2	2	1	2
		900	8	2	4	0	4	2	2	1	2
			11	4	7	0	2	1	2	1	2
Transverse tracking	12	500	8	2	3	1	4	2	2	1	2
			11	4	8	1	2	1	2	1	2
		900	8	4	8	1	2	0	2	1	2
			11	2	3	2	4	1	2	1	2
	14	500	8	4	7	1	2	1	2	1	2
			11	2	4	2	4	2	2	1	2
		900	8	2	4	0	4	2	2	1	2
			11	4	7	0	2	1	2	1	1

1 ft = 0.305 m.
1 lbf/inch^2 = .00690 MPa.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Analyses Results

The detailed analysis results were submitted to FHWA and are available from the LTPP Customer Support Services.[79] For convenience, the detailed results are summarized in table 78 through table 89. Because there are many design variables, four tables were populated to summarize the impacts of the design variables on RFP or RSP for each pavement condition or distress. Each table summarizes the impacts of the climatic region, slab thickness, drainable bases, and a combination of slab width and concrete flexural strength on the RFP or RSP of test sections based on one condition (IRI) or one distress (longitudinal or transverse cracks). For example, table 78 summarizes the results of the analyses of the impacts of design factors on RFP based on the IRI of SPS-2 test sections having slab width 12 ft (3.66 m) and concrete flexural strength 551 lbf/inch^2 (3.8 MPa), while table 79 summarizes the results of analyses of the impacts of design factors on RFP based on the IRI of SPS-2 test sections having slab width 14 ft (4.27 m) and concrete flexural strength 551 lbf/inch^2 (3.8 MPa). The numbers in the tables indicate the differences in years in RFPs or RSPs of the SPS-2 test sections having the top heading parameters relative to RFPs and RSPs of the SPS-2 test sections having the side heading parameters. The explanation of the listed numbers in the tables is the same as that included in chapter 5 in table 29 through table 33. For convenience, the following paragraphs explain the listed numbers in table 78 through table 89 using the data from the first and second rows in table 78.

Table 78. Summary of the results of the analyses of the impacts of design factors on RFP based on IRI of LTPP SPS-2 test sections with slab width of 12 ft (3.66 m) and concrete flexural strength of 551 lbf/inch^2 (3.8 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	0	0	–2	0	0	0	0	0	0	0	0	0	–3	0	0
		ND	0	*	0	–1	0	0	0	0	0	0	0	0	0	–3	0	0
	11	D	0	0	*	–2	0	0	0	0	0	0	0	0	0	–3	0	0
		ND	2	1	2	*	2	2	2	2	2	2	2	2	2	–2	2	2
WNF	8	D	0	0	0	–2	*	0	0	0	0	0	0	0	0	–3	0	0
		ND	0	0	0	–2	0	*	0	0	0	0	0	0	0	–3	0	0
	11	D	0	0	0	–2	0	0	*	0	0	0	0	0	0	–3	0	0
		ND	0	0	0	–2	0	0	0	*	0	0	0	0	0	–3	0	0
DF	8	D	0	0	0	–2	0	0	0	0	*	0	0	0	0	–3	0	0
		ND	0	0	0	2	0	0	0	0	0	*	0	0	0	–3	0	0
	11	D	0	0	0	–2	0	0	0	0	0	0	*	0	0	–3	0	0
		ND	0	0	0	–2	0	0	0	0	0	0	0	*	0	–3	0	0
DNF	8	D	0	0	0	–2	0	0	0	0	0	0	0	0	*	–3	0	0
		ND	3	3	3	2	3	3	3	3	3	3	3	3	3	*	3	3
	11	D	0	0	0	–2	0	0	0	0	0	0	0	0	0	–3	*	0
		ND	0	0	0	–2	0	0	0	0	0	0	0	0	0	–3	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 79. Summary of the results of analyses of the impacts of design factors on RFP based on IRI of LTPP SPS-2 test sections with slab width 14 ft (4.27 m) and concrete flexural strength of 551 lbf/inch^2 (3.8 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	–4	0	0	0	–1	0	0	0	0	0	0	0	0	0	–2
		ND	4	*	4	4	4	3	4	4	4	4	4	4	4	4	4	2
	11	D	0	–4	*	0	0	–1	0	0	0	0	0	0	0	0	0	–2
		ND	0	–4	0	*	0	–1	0	0	0	0	0	0	0	0	0	–2
WNF	8	D	0	–4	0	0	*	–1	0	0	0	0	0	0	0	0	0	–2
		ND	1	–3	1	1	1	*	1	1	1	1	1	1	1	1	1	–1
	11	D	0	–4	0	0	0	–1	*	0	0	0	0	0	0	0	0	–2
		ND	0	–4	0	0	0	–1	0	*	0	0	0	0	0	0	0	–2
DF	8	D	0	–4	0	0	0	–1	0	0	*	0	0	0	0	0	0	–2
		ND	0	–4	0	0	0	–1	0	0	0	*	0	0	0	0	0	–2
	11	D	0	–4	0	0	0	–1	0	0	0	0	*	0	0	0	0	–2
		ND	0	–4	0	2	0	–1	0	0	0	0	0	*	0	0	0	–2
DNF	8	D	0	–4	0	0	0	–1	0	0	0	0	0	0	*	0	0	–2
		ND	0	–4	0	0	0	–1	0	0	0	0	0	0	0	*	0	–2
	11	D	0	–4	0	0	0	–1	0	0	0	0	0	0	0	0	*	–2
		ND	2	–2	2	2	2	1	2	2	2	2	2	2	2	2	2	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = drainable base.
ND = undrainable base.

Table 80. Summary of the results of analyses of the impacts of design factors on RFP based on IRI of LTPP SPS-2 test sections with slab width 12 ft (3.66 m) and concrete flexural strength of 899 lbf/inch^2 (6.2 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	–4	0	0	0	–2	0	0	NC	0	NC	0	0	0	0	0
		ND	4	*	4	4	4	2	4	4	NC	4	NC	4	4	4	4	4
	11	D	0	–4	*	0	0	–2	0	0	NC	0	NC	0	0	0	0	0
		ND	0	–4	0	*	0	–2	0	0	NC	0	NC	0	0	0	0	0
WNF	8	D	0	–4	0	0	*	–2	0	0	NC	0	NC	0	0	0	0	0
		ND	2	–2	2	2	2	*	2	2	NC	2	NC	2	2	2	2	2
	11	D	0	–4	0	0	0	–2	*	0	NC	0	NC	0	0	0	0	0
		ND	0	–4	0	0	0	–2	0	*	NC	0	NC	0	0	0	0	0
DF	8	D	NC	NC	NC	0	NC	NC	NC	NC	*	NC	NC	NC	NC	NC	NC	NC
		ND	0	–4	0	0	0	–2	0	0	NC	*	NC	0	0	0	0	0
	11	D	NC	NC	NC	0	NC	NC	NC	NC	NC	NC	*	NC	NC	NC	NC	NC
		ND	0	–4	0	0	0	–2	0	0	NC	0	NC	*	0	0	0	0
DNF	8	D	0	–4	0	0	0	–2	0	0	NC	0	NC	0	*	0	0	0
		ND	0	–4	0	0	0	–2	0	0	NC	0	NC	0	0	*	0	0
	11	D	0	–4	0	0	0	–2	0	0	NC	0	NC	0	0	0	*	0
		ND	0	–4	0	0	0	–2	0	0	NC	0	NC	0	0	0	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.
NC = Could not be compared.

Table 81. Summary of the results of analyses of the impacts of design factors on RFP based on IRI of LTPP SPS-2 test sections with slab width 14 ft (4.27 m) and concrete flexural strength of 899 lbf/inch^2 (6.2 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	0	0	0	NC	0	NC	0	0	0	NC	0	0	0	0	0
		ND	0	*	0	0	NC	0	NC	0	0	0	NC	0	0	0	0	0
	11	D	0	0	*	0	NC	0	NC	0	0	0	NC	0	0	0	0	0
		ND	0	0	0	*	NC	0	NC	0	0	0	NC	0	0	0	0	0
WNF	8	D	NC	NC	NC	NC	*	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
		ND	0	0	0	0	NC	*	NC	0	0	0	NC	0	0	0	0	0
	11	D	NC	NC	NC	NC	NC	NC	*	NC	NC	NC	NC	NC	NC	NC	NC	NC
		ND	0	0	0	0	NC	0	NC	*	0	0	NC	0	0	0	0	0
DF	8	D	0	0	0	0	NC	0	NC	0	*	0	NC	0	0	0	0	0
		ND	0	0	0	0	NC	0	NC	0	0	*	NC	0	0	0	0	0
	11	D	NC	NC	NC	0	NC	NC	NC	NC	NC	NC	*	NC	NC	NC	NC	NC
		ND	0	0	0	0	NC	0	NC	0	0	0	NC	*	0	0	0	0
DNF	8	D	0	0	0	0	NC	0	NC	0	0	0	NC	0	*	0	0	0
		ND	0	0	0	0	NC	0	NC	0	0	0	NC	0	0	*	0	0
	11	D	0	0	0	0	NC	0	NC	0	0	0	NC	0	0	0	*	0
		ND	0	0	0	0	NC	0	NC	0	0	0	NC	0	0	0	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.
NC = Could not be compared.

Table 82. Summary of the results of analyses of the impacts of design factors on RSP based on longitudinal cracking of LTPP SPS-2 test sections with slab width 12 ft (3.66 m) and concrete flexural strength of 551 lbf/inch^2 (3.8 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	–2	0	0	0	–1	0	0	0	–3	0	0	0	–7	0	0
		ND	2	*	2	2	2	1	2	2	2	–1	2	2	2	–5	2	2
	11	D	0	–2	*	0	0	–1	0	0	0	–3	0	0	0	–7	0	0
		ND	0	–2	0	*	0	–1	0	0	0	–3	0	0	0	–7	0	0
WNF	8	D	0	–2	0	0	*	–1	0	0	0	–3	0	0	0	–7	0	0
		ND	1	–1	1	1	1	*	1	1	0	–2	1	1	1	–6	1	1
	11	D	0	–2	0	0	0	–1	*	0	0	–3	0	0	0	–7	0	0
		ND	0	–2	0	0	0	–1	0	*	0	–3	0	0	0	–7	0	0
DF	8	D	0	–2	0	0	0	0	0	0	*	–3	0	0	0	–6	0	0
		ND	3	1	3	7	3	2	3	3	3	*	3	3	3	–4	3	3
	11	D	0	–2	0	0	0	–1	0	0	0	–3	*	0	0	–7	0	0
		ND	0	–2	0	0	0	–1	0	0	0	–3	0	*	0	–7	0	0
DNF	8	D	0	–2	0	0	0	–1	0	0	0	–3	0	0	*	–7	0	0
		ND	7	5	7	7	7	6	7	7	6	4	7	7	7	*	7	7
	11	D	0	–2	0	0	0	–1	0	0	0	–3	0	0	0	–7	*	0
		ND	0	–2	0	0	0	–1	0	0	0	–3	0	0	0	–7	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 83. Summary of the results of analyses of the impacts of design factors on RSP based on longitudinal cracking of LTPP SPS-2 test sections with slab width 14 ft (4.27 m) and concrete flexural strength of 551 lbf/inch^2 (3.8 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	–3	0	0	0	–7	0	0	0	0	0	–3	0	0	0	0
		ND	3	*	3	3	3	–4	3	3	3	3	3	1	3	3	3	3
	11	D	0	–3	*	0	0	–7	0	0	0	0	0	–3	0	0	0	0
		ND	0	–3	0	*	0	–7	0	0	0	0	0	–3	0	0	0	0
WNF	8	D	0	–3	0	0	*	–7	0	0	0	0	0	–3	0	0	0	0
		ND	7	4	7	7	7	*	7	7	7	7	7	5	7	7	7	7
	11	D	0	–3	0	0	0	–7	*	0	0	0	0	–3	0	0	0	0
		ND	0	–3	0	0	0	–7	0	*	0	0	0	–3	0	0	0	0
DF	8	D	0	–3	0	0	0	–7	0	0	*	0	0	–3	0	0	0	0
		ND	0	–3	0	0	0	–7	0	0	0	*	0	–3	0	0	0	0
	11	D	0	–3	0	0	0	–7	0	0	0	0	*	–3	0	0	0	0
		ND	3	–1	3	0	3	–5	3	3	3	3	3	*	3	3	3	3
DNF	8	D	0	–3	0	0	0	–7	0	0	0	0	0	–3	*	0	0	0
		ND	0	–3	0	0	0	–7	0	0	0	0	0	–3	0	*	0	0
	11	D	0	–3	0	0	0	–7	0	0	0	0	0	–3	0	0	*	0
		ND	0	–3	0	0	0	–7	0	0	0	0	0	–3	0	0	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 84. Summary of the results of analyses of the impacts of design factors on RSP based on longitudinal cracking of LTPP SPS-2 test sections with slab width 12 ft (3.66 m) and concrete flexural strength of 899 lbf/inch^2 (6.2 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	–2	0	–2	0	–3	0	0	NC	0	0	0	0	0	0	0
		ND	2	*	2	–1	2	–1	2	2	NC	2	2	2	2	2	2	2
	11	D	0	–2	*	–2	0	–3	0	0	NC	0	0	0	0	0	0	0
		ND	2	1	2	*	2	–1	2	2	NC	2	2	2	2	2	2	2
WNF	8	D	0	–2	0	–2	*	–3	0	0	NC	0	0	0	0	0	0	0
		ND	3	1	3	1	3	*	3	3	NC	3	3	3	3	3	3	3
	11	D	0	–2	0	–2	0	–3	*	0	NC	0	0	0	0	0	0	0
		ND	0	–2	0	–2	0	–3	0	*	NC	0	0	0	0	0	0	0
DF	8	D	NC	NC	NC	–2	NC	NC	NC	NC	*	NC	NC	NC	NC	NC	NC	NC
		ND	0	–2	0	–2	0	–3	0	0	NC	*	0	0	0	0	0	0
	11	D	0	–2	0	–2	0	–3	0	0	NC	0	*	0	0	0	0	0
		ND	0	–2	0	–2	0	–3	0	0	NC	0	0	*	0	0	0	0
DNF	8	D	0	–2	0	–2	0	–3	0	0	NC	0	0	0	*	0	0	0
		ND	0	–2	0	–2	0	–3	0	0	NC	0	0	0	0	*	0	0
	11	D	0	–2	0	–2	0	–3	0	0	NC	0	0	0	0	0	*	0
		ND	0	–2	0	–2	0	–3	0	0	NC	0	0	0	0	0	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.
NC = Could not be compared.

Table 85. Summary of the results of analyses of the impacts of design factors on RSP based on longitudinal cracking of LTPP SPS-2 test sections with slab width 14 ft (4.27 m) and concrete flexural strength of 899 lbf/inch^2 (6.2 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	0	0	0	NC	0	NC	0	–1	0	0	–3	0	0	0	0
		ND	0	*	0	0	NC	0	NC	0	–1	0	0	–3	0	0	0	0
	11	D	0	0	*	0	NC	0	NC	0	–1	0	0	–3	0	0	0	0
		ND	0	0	0	*	NC	0	NC	0	–1	0	0	–3	0	0	0	0
WNF	8	D	NC	NC	NC	NC	*	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
		ND	0	0	0	0	NC	*	NC	0	–1	0	0	–3	0	0	0	0
	11	D	NC	NC	NC	NC	NC	NC	*	NC	NC	NC	NC	NC	NC	NC	NC	NC
		ND	0	0	0	0	NC	0	NC	*	–1	0	0	–3	0	0	0	0
DF	8	D	1	1	1	0	NC	1	NC	1	*	1	1	–2	1	1	1	1
		ND	0	0	0	0	NC	0	NC	0	–1	*	0	–3	0	0	0	0
	11	D	0	0	0	0	NC	0	NC	0	–1	0	*	–3	0	0	0	0
		ND	3	3	3	0	NC	3	NC	3	2	3	3	*	3	3	3	3
DNF	8	D	0	0	0	0	NC	0	NC	0	–1	0	0	–3	*	0	0	0
		ND	0	0	0	0	NC	0	NC	0	–1	0	0	–3	0	*	0	0
	11	D	0	0	0	0	NC	0	NC	0	–1	0	0	–3	0	0	*	0
		ND	0	0	0	0	NC	0	NC	0	–1	0	0	–3	0	0	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.
NC = Could not be compared.

Table 86. Summary of the results of analyses of the impacts of design factors on RSP based on transverse cracking of LTPP SPS-2 test sections with slab width 12 ft (3.66 m) and concrete flexural strength of 551 lbf/inch^2 (3.8 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	–6	1	–1	1	–4	1	1	1	1	1	1	1	–12	1	1
		ND	6	*	6	5	6	2	6	6	6	6	6	6	6	–6	6	6
	11	D	–1	–6	*	–2	0	–4	0	0	0	0	0	0	0	–12	0	0
		ND	1	–5	2	*	2	–3	2	2	2	2	2	2	2	–11	2	2
WNF	8	D	–1	–6	0	–2	*	–4	0	0	0	0	0	0	0	–12	0	0
		ND	4	–2	4	3	4	*	4	4	4	4	4	4	4	–8	4	4
	11	D	–1	–6	0	–2	0	–4	*	0	0	0	0	0	0	–12	0	0
		ND	–1	–6	0	–2	0	–4	0	*	0	0	0	0	0	–12	0	0
DF	8	D	–1	–6	0	–2	0	–4	0	0	*	0	0	0	0	–12	0	0
		ND	–1	–6	0	11	0	–4	0	0	0	*	0	0	0	–12	0	0
	11	D	–1	–6	0	–2	0	–4	0	0	0	0	*	0	0	–12	0	0
		ND	–1	–6	0	–2	0	–4	0	0	0	0	0	*	0	–12	0	0
DNF	8	D	–1	–6	0	–2	0	–4	0	0	0	0	0	0	*	–12	0	0
		ND	12	6	12	11	12	8	12	12	12	12	12	12	12	*	12	12
	11	D	–1	–6	0	–2	0	–4	0	0	0	0	0	0	0	–12	*	0
		ND	–1	–6	0	–2	0	–4	0	0	0	0	0	0	0	–12	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 87. Summary of the results of analyses of the impacts of design factors on RSP based on transverse cracking of LTPP SPS-2 test sections with slab width 14 ft (4.27 m) and concrete flexural strength of 551 lbf/inch^2 (3.8 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	–2	0	0	0	–5	0	0	0	0	0	0	0	–1	0	–1
		ND	2	*	2	2	2	–3	2	2	2	2	2	2	2	0	2	0
	11	D	0	–2	*	0	0	–5	0	0	0	0	0	0	0	–1	0	–1
		ND	0	–2	0	*	0	–5	0	0	0	0	0	0	0	–1	0	–1
WNF	8	D	0	–2	0	0	*	–5	0	0	0	0	0	0	0	–1	0	–1
		ND	5	3	5	5	5	*	5	5	5	5	5	5	5	3	5	3
	11	D	0	–2	0	0	0	–5	*	0	0	0	0	0	0	–1	0	–1
		ND	0	–2	0	0	0	–5	0	*	0	0	0	0	0	–1	0	–1
DF	8	D	0	–2	0	0	0	–5	0	0	*	0	0	0	0	–1	0	–1
		ND	0	–2	0	1	0	–5	0	0	0	*	0	0	0	–1	0	–1
	11	D	0	–2	0	0	0	–5	0	0	0	0	*	0	0	–1	0	–1
		ND	0	–2	0	1	0	–5	0	0	0	0	0	*	0	–1	0	–1
DNF	8	D	0	–2	0	0	0	–5	0	0	0	0	0	0	*	–1	0	–1
		ND	1	0	1	1	1	–3	1	1	1	1	1	1	1	*	1	0
	11	D	0	–2	0	0	0	–5	0	0	0	0	0	0	0	–1	*	–1
		ND	1	0	1	1	1	–3	1	1	1	1	1	1	1	0	1	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.

Table 88. Summary of the results of analyses of the impacts of design factors on RSP based on transverse cracking of LTPP SPS-2 test sections with slab width 12 ft (3.66 m) and concrete flexural strength of 899 lbf/inch^2 (6.2 MPa).

			Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
Climatic Region	PCC Slab Thickness (inches)	Base Type	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	–1	–2	–5	1	–5	1	1	NC	1	1	1	1	1	1	1
		ND	1	*	0	–3	2	–4	2	2	NC	2	2	2	2	2	2	2
	11	D	2	0	*	–3	3	–4	3	3	NC	3	3	3	3	3	3	3
		ND	5	3	3	*	6	–1	6	6	NC	6	6	6	6	6	6	6
WNF	8	D	–1	–2	–3	–6	*	–6	0	0	NC	0	0	0	0	0	0	0
		ND	5	4	4	1	6	*	6	6	NC	6	6	6	6	6	6	6
	11	D	–1	–2	–3	–6	0	–6	*	0	NC	0	0	0	0	0	0	0
		ND	–1	–2	–3	–6	0	–6	0	*	NC	0	0	0	0	0	0	0
DF	8	D	NC	NC	NC	–6	NC	NC	NC	NC	*	NC	NC	NC	NC	NC	NC	NC
		ND	–1	–2	–3	–6	0	–6	0	0	NC	*	0	0	0	0	0	0
	11	D	–1	–2	–3	–6	0	–6	0	0	NC	0	*	0	0	0	0	0
		ND	–1	–2	–3	–6	0	–6	0	0	NC	0	0	*	0	0	0	0
DNF	8	D	–1	–2	–3	–6	0	–6	0	0	NC	0	0	0	*	0	0	0
		ND	–1	–2	–3	–6	0	–6	0	0	NC	0	0	0	0	*	0	0
	11	D	–1	–2	–3	–6	0	–6	0	0	NC	0	0	0	0	0	*	0
		ND	–1	–2	–3	–6	0	–6	0	0	NC	0	0	0	0	0	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.
NC = Could not be compared.

Table 89. Summary of the results of analyses of the impacts of design factors on RSP based on transverse cracking of LTPP SPS-2 test sections with slab width 14 ft (4.27 m) and concrete flexural strength of 899 lbf/inch^2 (6.2 MPa).

Climatic Region	PCC Slab Thickness (inches)	Base Type	Differences Between RFP of the Top Heading and RFP of the Side Heading (Year)															
			WF				WNF				DF				DNF			
			8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC		8-inch PCC		11-inch PCC	
			D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND	D	ND
WF	8	D	*	2	2	2	NC	2	NC	2	5	2	2	0	2	3	2	2
		ND	2	*	4	4	NC	4	NC	4	3	4	4	2	4	1	4	4
	11	D	–2	4	*	0	NC	0	NC	0	7	0	0	2	0	5	0	0
		ND	–2	4	0	*	NC	0	NC	0	7	0	0	2	0	5	0	0
WNF	8	D	NC	NC	NC	NC	*	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
		ND	–2	4	0	0	NC	*	NC	0	7	0	0	2	0	5	0	0
	11	D	NC	NC	NC	NC	NC	NC	*	NC	NC	NC	NC	NC	NC	NC	NC	NC
		ND	–2	4	0	0	NC	0	NC	*	7	0	0	2	0	5	0	0
DF	8	D	5	3	7	0	NC	7	NC	7	*	7	7	5	7	2	7	7
		ND	–2	4	0	5	NC	0	NC	0	7	*	0	2	0	5	0	0
	11	D	–2	4	0	0	NC	0	NC	0	7	0	*	2	0	5	0	0
		ND	0	2	2	0	NC	2	NC	2	5	2	2	*	2	3	2	2
DNF	8	D	–2	4	0	0	NC	0	NC	0	7	0	0	2	*	5	0	0
		ND	3	1	5	5	NC	5	NC	5	2	5	5	3	5	*	5	5
	11	D	–2	4	0	0	NC	0	NC	0	7	0	0	2	0	5	*	0
		ND	–2	4	0	0	NC	0	NC	0	7	0	0	2	0	5	0	*

* Indicates the diagonal of the matrix where the top and the side headings are the same.
1 inch = 25.4 mm.
D = Drainable base.
ND = Undrainable base.
NC = Could not be compared.

The following list describes the RFP of the SPS-2 test sections having 11-inch (279-mm)-thick slab and undrainable bases in the WF region:

- Two years less than the RFP of test sections located in the WF region and having 8-inch (203-mm)-thick slab and drainable bases.
- An insignificant 1 year less than the RFP of test sections located in the WF region and having 8-inch-thick slab and undrainable bases.

In the DNF region, the RFP of the SPS-2 test sections having 8-inch (203-mm)-thick slab and undrainable bases is 3 years less than the RFP of test sections located in the WF region and having 8-inch (203-mm)-thick slab and either drainable or undrainable bases.

It should be noted that the results listed in table 78 to table 89 are further summarized in table 90 based on the relative performance of comparable SPS-2 test sections. In this context, the term comparable implies SPS-2 test sections having the same slab thickness and slab width, the same concrete flexural strength, and similar bases. The summarized data in table 90 address the impact of the climatic regions on pavement performance in terms of functional condition (RFP based on IRI) and structural condition (RSP based on longitudinal and transverse cracking). The values in the table indicate the percent of the test sections, having the heading parameters that performed better, the same, or worse than the test sections having the side heading parameters. In the following sections, these values are presented and discussed for each pavement condition and distress type.

Table 90. Summary of the results of the analyses of the effects of climatic region on the performance of the LTPP SPS-2 test sections.

Condition/ Distress Type	Climatic Region	Percent of SPS-2 Test Sections Located in One Climatic Region That Performed Better, the Same, or Worse Than Compatible Test Sections Located in Other Regions											
		WF			WNF			DF			DNF		
		Better	Same	Worse	Better	Same	Worse	Better	Same	Worse	Better	Same	Worse
IRI	WF	—	—	—	26	70	4	32	68	0	26	65	9
	WNF	4	70	26	—	—	—	10	90	0	9	82	9
	DF	0	68	32	0	90	10	—	—	—	0	90	10
	DNF	9	65	26	9	82	9	10	90	0	—	—	—
Longitudinal Cracking	WF	—	—	—	13	78	9	14	68	18	17	79	4
	WNF	9	78	13	—	—	—	14	72	14	78	18	4
	DF	18	68	14	14	72	14	—	—	—	18	77	5
	DNF	4	79	17	4	18	78	5	77	18	—	—	—
Transverse Cracking	WF	—	—	—	39	44	17	48	43	9	39	35	26
	WNF	17	44	39	—	—	—	18	77	5	13	64	23
	DF	9	43	48	5	77	18	—	—	—	9	64	27
	DNF	26	35	39	23	64	13	27	64	9	—	—	—

— Indicates no data.

IRI

The data listed in the IRI rows in table 90 indicate that the pavement performance based on IRI, of the majority of the SPS-2 test sections, was not affected by the climatic regions. The various findings leading to this conclusion are detailed as follows:

- In the WF region, 70, 68, and 65 percent of the SPS-2 test sections performed the same as comparable test sections located in the WNF, DF, and DNF regions, respectively, while 26, 32, and 26 percent performed worse.

- In the WNF region, 90 and 82 percent of the SPS-2 test sections performed the same as comparable test sections located in the DF and DNF regions, respectively, while only 10 and 9 percent performed worse.

- In the DF region, 90 percent of the SPS-2 test section performed the same as comparable test sections located in the DNF region, and only 10 percent performed better.

Longitudinal Cracking

The data listed in the longitudinal cracking rows in table 90 indicate the following:

- In the WF region, 78, 68, and 79 percent of the SPS-2 test sections performed the same as comparable test sections located in the WNF, DF, and DNF regions, respectively, while 13, 14, and 17 percent performed worse, and 9, 18, and 4 percent performed better. In the WNF region, 72 and 18 percent of the SPS-2 test sections performed the same as comparable test sections located in the DF and DNF regions, respectively, while 14 and 78 percent performed worse, and 14 and 4 percent performed better.

- In the DF region, 77 percent of the SPS-2 test section performed the same as comparable test sections located in the DNF region, and 18 percent performed worse.

Transverse Cracking

The data listed in the transverse cracking block of table 90 indicate the following:

- In the WF region, 44, 43, and 35 percent of the SPS-2 test sections performed the same as comparable test sections located in the WNF, DF, and DNF regions, respectively, while 17, 9, and 26 percent performed better, and 39, 48, and 39 percent performed worse.

- In the WNF region, the majority (77 and 64 percent) of the SPS-2 test sections performed the same as comparable test sections located in the DF and DNF regions, respectively, while few percentages performed either better or worse.

- Likewise, in the DF region, the majority of the SPS-2 test sections performed the same as comparable test sections located in the DNF region, while 27 percent performed better, and 9 percent performed worse.

Summary, Conclusions, and Recommendations for LTPP SPS-2

The available data in the LTPP database Standard Data Release 28.0 regarding the LTPP SPS-2 experiment were downloaded, organized, and analyzed. The intent was to study the impact of each design variable on pavement performance. When the data were divided into various groups based on separation of variables, the number of test sections under each variable was statistically insignificant. However, for each test section, the resulting RFPs and RSPs are listed in table 78 through table 89. Because of the limited number of SPS-2 test sections under each variable, the impact of the design variables on pavement performance was not analyzed or discussed any further. Rather, the data were summarized in table 90, and the impacts of the climatic region on pavement performance were presented after that table. Based on the analyses results, the following conclusions were drawn:

- On average, the pavement performance in terms of IRI was not affected by the climatic region, although the data indicated that SPS-2 test sections located in the WF region performed slightly worse than compatible test sections located in the other three climatic regions.

- On average, the majority of the SPS-2 test sections located in the WNF region performed worse in terms of longitudinal cracking than those in the DNF region. This was mainly due to the impact of excessive moisture on pavement performance.

- The WF region had a more damaging impact on pavement performance in terms of transverse cracking than on those in the WNF, DF, and DNF regions. This was expected, owing to the combined effects of subfreezing temperatures and moisture.

IMPACTS OF MAINTENANCE TREATMENTS ON PAVEMENT CONDITION AND DISTRESS USING THE LTPP SPS-4 TEST SECTIONS

The main objective of the LTPP SPS-4 experiment was to compare the performance of rigid pavement test sections subjected to selected maintenance treatments to the performance of untreated test sections or the control sections. The 34 SPS-4 test sites were initiated between 1990 and 1995 and are distributed across the United States and Canada. Each of the SPS-4 test sites consisted of three test sections. One each of two sections at each site was subjected to one or the other of the following two treatments:

- Joint and crack sealing (410).
- Joint undersealing (420).

(Note that the numbers in parentheses are the LTPP designation of the treatment. For example, the designation of the joint and crack sealing is 410.) The third section was counted as a control section that was not treated in accordance with the original experimental design. However, only 10 of the 34 test sites contained a test section that was joint undersealed, bringing the total number of test sections and control sections to 78.

Several variables affect the performance of the treated pavement sections. These include climatic region, traffic, subgrade type, etc. Similar to the SPS-3 experiment, unfortunately, in some scenarios, if these variables were separated, the number of test sections available for analyses became insignificant. To illustrate, table 91 lists the number of test sections available for analyses based on the separation of the following variables:

- Two treatment types.
- One pavement condition (IRI).
- Two pavement distress types (longitudinal and transverse cracking).
- Four climatic regions (WF, WNF, DF, and DNF).
- Three traffic levels.

Table 91. Number of test sections that have after treatment pavement condition and distress and traffic data.

Condition or Distress Type	Treatment Type	Number of Test Sections Subjected to Each of Three Traffic Levels in the Various Climatic Regions											
		WF			WNF			DF			DNF		
		L	M	H	L	M	H	L	M	H	L	M	H
IRI	Joint and crack sealing	0	0	7	1	3	6	0	0	2	0	0	2
	Joint undersealing	0	0	0	0	3	3	0	0	1	0	0	1
	Control section	0	0	7	1	3	7	0	0	1	0	0	2
Longitudinal cracking	Joint and crack sealing	0	0	0	0	2	3	0	0	0	0	0	0
	Joint undersealing	0	0	0	0	1	1	0	0	0	0	0	0
	Control section	0	0	1	0	2	0	0	0	1	0	0	2
Transverse cracking	Joint and crack sealing	0	0	1	0	1	3	0	0	1	0	0	0
	Joint undersealing	0	0	0	0	0	2	0	0	0	0	0	0
	Control section	0	0	2	0	1	2	0	0	1	0	0	1

Note: For each pavement condition and distress type, a test section was analyzed only if it exhibited any condition or distress and had three or more data points after treatment that could be modeled.
L = Low traffic (0 to 60,000 ESAL/year).
M = Medium traffic (61,000 to 120,000 ESAL/year).
H = High traffic (> 120,000 ESAL/year).

It can be seen that for longitudinal and transverse cracking, the number of SPS-4 test sections that were available for analyses was statistically insignificant in all climatic regions. Therefore, the analyses were conducted to assess the impact of each treatment type in each climatic region and for each pavement condition and distress type. That is, the data were not separated based on traffic level or by the type of subbase or subgrade. Nevertheless, the analyses of the impacts of each of the two treatment types on pavement performance were accomplished using the following steps:

- **Step 1**: For each treated pavement test section in the SPS-4 experiment, each of the available pavement condition (IRI) and distress data were used to calculate the RFP and RSP of that section from the time of the treatment to the time when the pavement condition or distress reach the prespecified threshold values.

- **Step 2:** For each pavement condition and distress type and for each pavement treatment type, the minimum and maximum RFPs and RSPs and their averages for all test sections located in the same climatic region were calculated and are listed in table 92 through table 94 depending on the pavement condition and distress type.

Results of the analyses are discussed per pavement condition and distress type in the three subsections following table 92 through table 94.

Table 92. Impacts of various maintenance treatments and control section on pavement performance in terms of RFP based on IRI.

Climatic Region	Treatment Type	RFP (Year) Test Sections Number of Test Sections	Test Sections Min	Test Sections Max	Test Sections Avg	Control Sections Number of Test Sections	Control Sections Min	Control Sections Max	Control Sections Avg	Difference in RFP (Year)
WF	Joint crack sealing	8	10	20	17	8	8	20	17	0
WNF	Joint crack sealing	10	6	20	18	11	0	20	15	3
DF	Joint crack sealing	4	1	16	9	2	11	13	12	3
DNF	Joint crack sealing	2	5	20	12	2	17	17	17	4
WF	Joint undersealing	0	—	—	—	8	8	20	17	NC
WNF	Joint undersealing	6	0	20	12	11	0	20	15	3
DF	Joint undersealing	1	3	3	3	2	11	13	12	10
DNF	Joint undersealing	1	2	2	2	2	17	17	17	14

—Indicates no data.
Min = Minimum.
Max = Maximum.
Avg = Average.
NC = Could not be compared.

Table 93. Impacts of various maintenance treatments and control section on pavement performance in terms of RSP based on longitudinal cracking.

Climatic Region	Treatment Type	RFP (Year)								Difference in RSP (Year)
		Test Sections				Control Sections				
		Number of Test Sections	Min	Max	Avg	Number of Test Sections	Min	Max	Avg	
WF	Joint crack sealing	0	—	—	—	1	20	20	20	NC
WNF		5	13	20	19	2	20	20	20	1
DF		0	—	—	—	1	20	20	20	NC
DNF		0	—	—	—	2	17	20	19	NC
WF	Joint undersealing	0	—	—	—	1	20	20	20	NC
WNF		2	20	20	20	2	20	20	20	0
DF		0	—	—	—	1	20	20	20	NC
DNF		0	—	—	—	2	17	20	19	NC

—Indicates no data.
Min = Minimum.
Max = Maximum.
Avg = Average.
NC = Could not be compared.

Table 94. Impacts of various maintenance treatments and control section on pavement performance in terms of RSP based on transverse cracking.

Climatic Region	Treatment Type	RFP (Year)								Difference in RSP (Year)
		Test Sections				Control Sections				
		Number of Test Sections	Min	Max	Avg	Number of Test Sections	Min	Max	Avg	
WF	Joint crack sealing	1	20	20	20	2	11	20	16	4
WNF		4	6	20	14	3	20	20	20	6
DF		1	19	19	19	1	20	20	20	1
DNF		0	—	—	—	1	14	14	14	NC
WF	Joint undersealing	0	—	—	—	2	11	20	16	NC
WNF		2	1	20	11	3	20	20	20	9
DF		0	—	—	—	1	20	20	20	NC
DNF		0	—	—	—	1	14	14	14	NC

—Indicates no data.
Min = Minimum.
Max = Maximum.
Avg = Average.
NC = Could not be compared.

IRI

The calculated minimum, maximum, and average RFPs based on IRI data for the SPS-4 test sections that were subjected to the same treatment type and for the associated control sections are listed in table 92. The data in the table indicate the following:

- Eight SPS-4 test sections in the WF region were subjected to joint and crack sealing and accepted for analyses. The minimum, maximum, and average RFPs of the eight SPS-4 test sections were 10, 20, and 17 years, respectively. In addition, there were eight control sections with minimum, maximum, and average RFPs of 8, 20, and 17 years, respectively. Thus, the difference between the average RFPs of the treated test sections and the control sections was 0 years. That is, joint crack sealing had no impact on pavement performance in the WF region.

- The average RFP of the 10 treated SPS-4 test sections located in the WNF region was 3 years longer than the average RFP of the 11 control sections located in the same region.

- The average RFP of the four treated SPS-4 test sections located in the DF region was 3 years shorter than the average RFP of the two control sections located in the same region.

- The average RFP of the two treated SPS-4 test sections located in the DNF region was 5 years shorter than the average RFP of the two control sections located in the same region.

- The joint undersealing treatment of the SPS-4 test sections in the WNF, DF, and DNF regions caused greater pavement roughness, and consequently, the average RFPs of the control sections in the three regions were substantially shorter than the test sections.

The main reason for the differences between the average RFPs of the test sections and the control sections is that the conditions of the control sections were not representative of the conditions of the test sections when they were subjected to treatments. For example, the IRI obtained from the first survey performed on the treated test section 06B420 was 156 inches/mi (2.4 m/km) while the IRI of the control section 06B430 was 123.5 inches/mi (1.9 m/km). Because the magnitude and the rates of deterioration of the two test sections were different, they precipitated differences in their RFPs.

Longitudinal Cracking

The calculated minimum, maximum, and average RSPs based on longitudinal cracking data for the SPS-4 test sections that were subjected to the same treatment type and for the associated control sections are listed in table 93. The data in the table indicate that neither treatment had any impact on the average RFPs of the SPS-4 test sections located in the WNF region. No test or control sections are located in the other three climatic regions.

Transverse Cracking

The calculated minimum, maximum, and average RSPs based on transverse cracking of the SPS-4 test sections that were subjected to the same treatment type and for the associated control sections are listed in table 94. The data in the table indicate the following:

- Joint and crack sealing had a positive impact on pavement performance in the WF region. The RFP of the one SPS-4 test section was 4 years longer than the average RFP of the two control sections, whereas, the same treatment caused losses in the average RFPs of the SPS-4 test sections located in the WNF and DF regions.

- The two SPS-4 test sections located in the WNF region and subjected to joint undersealing performed worse than the three control sections by 9 years.

Summary, Conclusions, and Recommendations for LTTP SPS-4

The available data in the LTPP database regarding the LTPP SPS-4 experiment were downloaded, organized, and analyzed. The intent was to study the impact of two maintenance treatments, joint and crack sealing (410) and joint undersealing (420), on pavement performance. When the data were separated based on traffic levels, the number of test sections that were available for analyses in each traffic level was statistically insignificant. Therefore, the data were grouped based on the two maintenance treatment types and the four climatic regions. For each group, the minimum, maximum, and average RFPs and RSPs for the test and control sections were calculated and are listed in table 92 through table 94. The impacts of the two maintenance treatments in each climatic region were presented in the section following the tables. Based on the results of the analyses, the following conclusions were drawn:

- On average, in terms of IRI, joint and crack sealing treatment had no impact on pavement performance in the WF region, a positive impact in the WNF region, and a negative impact in the DF and DNF regions.

- On average, in terms of IRI, joint undersealing treatment had a negative impact on pavement performance in the WNF, DF, and DNF regions.

- On average, the two maintenance treatments had no impact on pavement performance in terms of longitudinal cracking in the WNF region.

- Joint and crack sealing treatment had a positive impact on pavement performance in the WF region and a negative impact in the WNF region.

- Joint undersealing treatment had no impact on the pavement performance in terms of transverse cracking in the WNF region.

In summary, the research team concluded that joint and crack sealing was effective in the WF region and not effective in the other three climatic regions, and joint undersealing was not effective in any region.

IMPACTS OF REHABILITATION TREATMENTS ON PAVEMENT CONDITION AND DISTRESS USING THE LTPP SPS-6 TEST SECTIONS

The main objective of the SPS-6 experiment was to examine the effects of various rehabilitation treatments on the performance of rigid pavement test sections. The 14 SPS-6 test sites were initiated between 1989 and 1998 and are distributed across the United States and Canada. Each SPS-6 test site consisted of 1 control section and 7 treated test sections for a total of 112 test sections. Each of the seven treated sections was subjected to one of the following rehabilitation actions (note that the numbers in parenthesis are the LTPP designation of the rehabilitation actions):

- Minimum restoration (602).
- Minimum restoration with 4-inch (102-mm) AC overlay (603).
- Minimum restoration with 4-inch (102-mm) AC overlay and sawed and sealed joints in the AC (604).
- Maximum restoration (605).
- Maximum restoration with 4-inch (102-mm) AC overlay (606).
- Crack, break, and seat with 4-inch (102-mm) AC overlay (607).
- Crack, break, and seat with 8-inch (204-mm) AC overlay (608).

The minimum restoration action included limited patching, crack sealing, and joint stabilization. Further, diamond grinding was performed when faulting was considered too high. Maximum restoration included subsealing, subdrainage, joint repair and sealing, full-depth repairs and load transfer restoration, and diamond grinding. Cracking and seating was used for JPCP test sections while breaking and seating was performed for JRCP test sections.

For each SPS-6 test section subjected to one of the previously listed rehabilitation actions, the time-series pavement condition and distress data (collected after the rehabilitation action was taken and before the next treatment was applied) were used to calculate the RFPs and RSPs of that section. Thus, RFPs and RSPs expressed the pavement service period between rehabilitation and the time when the pavement condition or distress reached the prespecified threshold values. Similarly, the RFPs and RSPs of the control sections were also calculated. For each pavement condition (IRI) and distress type (rut depth and alligator, longitudinal, and transverse cracking), the treatment benefits were expressed in terms of the following:

- RFP or RSP of the treated pavement section.
- The difference in RFP or RSP of the treated pavement section and RFP or RSP of the associated control section. This difference was labeled CFP or CSP.

Results of the analyses of the treatment benefits are listed in table 95 through table 99 based on two climatic regions (no test sections were present in the DF and DNF regions), pavement type, and pavement condition and distress type. The data in the tables are discussed in the following subsections per pavement condition and distress type.

Table 95. RFP of control sections and the impact of treatment types on pavement performance in terms of RFP based on IRI.

Climatic Region	Pavement Type	State (Code)	Control Section RFP (Year)	Minimum Restoration and No AC Overlay		Minimum Restoration and AC Overlay				Maximum Restoration and No AC Overlay		Maximum Restoration and 4-inch AC Overlay		Crack/Break and Seat and AC Overlay			
						4 inches		4 inches With SS						4 inches		8 inches	
				RFP	CFP	RFP	CFP	RFP	CFP	RFP	CFP	RFP	CFP	RFP	CFP	RFP	CFP
WF	JPCP	AZ (04)	ND	ND	—	18	—	20	—	ND	—	20	—	20	—	20	—
		IN (18)	ND	NS	—	20	—	20	—	NS	—	20	—	14	—	20	—
		MO (29)	ND	20	—	20	—	10	—	16	—	15	—	20	—	20	—
		SD (46)	ND	ND	—	ND	—	ND	—	ND	—	ND	—	ND	—	ND	—
		Average	—	20	—	19	—	17	—	16	—	18	—	18	—	20	—
	JRCP	IL (17)	7	20	13	20	13	20	13	13	6	ND	—	20	13	20	13
		IA (19)	ND	ND	—	ND	—	ND	—	20	—	20	—	20	—	20	—
		MI (26)	11	ND	—	20	9	NS	—	3	8	20	9	NS	—	20	9
		MO (29)	ND	ND	—	20	—	20	—	ND	—	20	—	ND	—	20	—
		PA (42)	ND	ND	—	19	—	20	—	ND	—	19	—	20	—	20	—
		Average	9	20	11	20	11	20	11	12	3	20	11	20	11	20	11
WNF	JPCP	AL(01)	0	20	20	20	20	20	20	20	20	20	20	ND	—	20	20
		AR (05)	ND	20	—	20	—	20	—	ND	—	20	—	20	—	20	—
		CA (06)	0	ND	—	15	15	ND	—	7	7	ND	—	16	16	20	20
		TN (47)	ND	15	—	20	—	13	—	ND	—	20	—	ND		20	—
		Average	0	18	18	19	19	18	18	14	14	20	20	18	18	20	20
	JRCP	OK (40)	ND	ND	—	ND	—	ND	—	ND	—	ND	—	ND	—	ND	—

— Indicates could not be calculated.
1 inch = 25.4 mm.
SS = saw and seal of joints.
ND = no data.

Table 96. Impact of various treatments and control section on pavement performance in terms of RFP/RSP based on rut depth.

Climatic Region	Pavement Type	State (Code)	Control Section RSP (Year)	Minimum Restoration and No AC Overlay		Minimum Restoration and AC Overlay				Maximum Restoration and No AC Overlay		Maximum Restoration and 4-inch AC Overlay		Crack/Break and Seat and AC Overlay			
						4 inches		4 inches With SS						4 inches		8 inches	
				RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP
WF	JPCP	AZ (04)	N/A	N/A	—	20	—	20	—	N/A	—	NS	—	20	—	20	—
		IN (18)	N/A	N/A	—	20	—	20	—	N/A	—	20	—	20	—	20	—
		MO (29)	N/A	N/A	—	NS	—	NS	—	N/A	—	NS	—	NS	—	NS	—
		SD (46)	N/A	N/A	—	ND	—	ND	—	NA	—	ND	—	ND	—	ND	—
		Average	—	—	—	20	—	20	—	—	—	20	—	20	—	20	—
	JRCP	IL (17)	N/A	N/A	—	20	—	20	—	N/A	—	ND	—	NS	—	NS	—
		IA (19)	N/A	N/A	—	NS	—	20	—	N/A	—	NS	—	NS	—	20	—
		MI (26)	N/A	N/A	—	20	—	20	—	N/A	—	20	—	20	—	20	—
		MO (29)	N/A	N/A	—	20	—	20	—	N/A	—	20	—	ND	—	20	—
		PA (42)	N/A	N/A	—	20	—	20	—	N/A	—	20	—	20	—	20	—
		Average	—	—	—	20	—	20	—	—	—	20	—	20	—	20	—
WNF	JPCP	AL (01)	N/A	N/A	—	20	—	20	—	N/A	—	20	—	20	—	20	—
		AR (05)	N/A	N/A	—	20	—	20	—	N/A	—	20	—	20	—	20	—
		CA (06)	N/A	N/A	—	20	—	20	—	N/A	—	ND	—	20	—	20	—
		TN (47)	N/A	N/A	—	20	—	20	—	N/A	—	20	—	ND	—	20	—
		Average	—	—	—	20	—	20	—	—	—	20	—	20	—	20	—
	JRCP	OK (40)	N/A	N/A	—	20	—	20	—	N/A	—	20	—	20	—	20	—

— Indicates could not be calculated.
1 inch = 25.4 mm.
SS = Saw and seal of joints.
ND = No data.
N/A = Not applicable.
NS = Negative model slope (pavement condition and/or distress improving over time with no treatment).

Table 97. Impact of various treatments and control section on pavement performance in terms of RSP based on alligator cracking.

Climatic Region	Pavement Type	State (Code)	Control Section RSP (Year)	Minimum Restoration and No AC Overlay		Minimum Restoration and AC Overlay				Maximum Restoration and No AC Overlay		Maximum Restoration and 4-inch AC Overlay		Crack/Break and Seat and AC Overlay			
						4 inches		4 inches With SS						4 inches		8 inches	
				RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP
WF	JPCP	AZ (04)	N/A	N/A	—	13	—	20	—	N/A	—	19	—	19	—	20	—
		IN (18)	N/A	N/A	—	20	—	13	—	N/A	—	11	—	12	—	12	—
		MO (29)	N/A	N/A	—	ND	—	15	—	N/A	—	19	—	19	—	20	—
		SD (46)	N/A	N/A	—	ND	—	ND	—	N/A	—	ND	—	ND	—	ND	—
		Average	—	—	—	17	—	16	—	—	—	16	—	17	—	17	—
	JRCP	IL (17)	N/A	N/A	—	5	—	13	—	N/A	—	ND	—	ND	—	18	—
		IA (19)	N/A	N/A	—	ND	—	8	—	N/A	—	8	—	8	—	10	—
		MI (26)	N/A	N/A	—	20	—	ND	—	N/A	—	10	—	ND	—	20	—
		MO (29)	N/A	N/A	—	ND	—	20	—	N/A	—	ND	—	ND	—	16	—
		PA (42)	N/A	N/A	—	20	—	ND	—	N/A	—	ND	—	20	—	ND	—
		Average	—	—	—	15	—	14	—	—	—	9	—	14	—	16	—
WNF	JPCP	AL (01)	N/A	N/A	—	9	—	15	—	N/A	—	ND	—	3	—	7	—
		AR (05)	N/A	N/A	—	18	—	16	—	N/A	—	18	—	20	—	20	—
		CA (06)	N/A	N/A	—	7	—	ND	—	N/A	—	ND	—	5	—	7	—
		TN (47)	N/A	N/A	—	ND	—	12	—	N/A	—	ND	—	ND	—	ND	—
		Average	—	—	—	11	—	14	—	—	—	18	—	9	—	11	—
	JRCP	OK (40)	N/A	N/A	—	20	—	20	—	N/A	—	20	—	5	—	9	—

— Indicates could not be calculated.
1 inch = 25.4 mm.
SS = Saw and seal of joints.

ND = No data.
N/A = Not applicable.

Table 98. Impact of various treatments and control section on pavement performance in terms of RSP based on longitudinal cracking.

Climatic Region	Pavement Type	State (Code)	Control Section RSP (Year)	Minimum Restoration and No AC Overlay		Minimum Restoration and AC Overlay				Maximum Restoration and No AC Overlay		Maximum Restoration and 4-inch AC Overlay		Crack/Break and Seat and AC Overlay			
						4 inches		4 inches With SS						4 inches		8 inches	
				RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP
WF	JPCP	AZ (04)	ND	ND	—	7	—	12	—	ND	—	13	—	10	—	13	—
		IN (18)	ND	19	—	6	—	12	—	20	—	14	—	15	—	10	—
		MO (29)	20	ND	—	10	10	6	14	13	7	9	11	10	10	10	10
		SD (46)	ND	ND	—	ND	—	ND	—	ND	—	ND	—	ND	—	ND	—
		Average	20	19	1	8	12	10	10	17	3	12	8	12	8	11	9
	JRCP	IL (17)	13	ND	—	20	7	12	1	ND	—	ND	—	20	7	17	4
		IA (19)	ND	ND	—	ND	—	7	—	ND	—	3	—	0	—	1	—
		MI (26)	19	ND	—	10	9	ND	—	NS	—	10	9	9	10	9	10
		MO (29)	ND	ND	—	ND	—	19	—	ND	—	20	—	ND	—	20	—
		PA (42)	ND	ND	—	14	—	15	—	ND	—	14	—	14	—	17	—
		Average	16	—	—	15	1	13	3	—	—	12	4	11	5	13	3
WNF	JPCP	AL (01)	ND	20	—	5	—	6	—	ND	—	5	—	7	—	20	—
		AR (05)	ND	ND	—	10	—	10	—	ND	—	12	—	9	—	9	—
		CA (06)	ND	ND	—	6	—	7	—	ND	—	7	—	5	—	5	—
		TN (47)	ND	20	—	6	—	5	—	ND	—	5	—	ND	—	7	—
		Average	—	20	—	7	—	7	—	—	—	7	—	7	—	10	—
	JRCP	OK (40)	ND	ND	—	3	—	3	—	ND	—	3	—	3	—	3	—

— Indicates could not be calculated.
1 inch = 25.4 mm.
SS = Saw and seal of joints.
ND = No data.

Table 99. Impact of various treatments and control section on pavement performance in terms of RSP based on transverse cracking.

Climatic Region	Pavement Type	State (State Code)	Control Section RSP (Year)	Minimum Restoration and No AC Overlay		Minimum Restoration and AC Overlay				Maximum Restoration and No AC Overlay		Maximum Restoration and 4-inch AC Overlay		Crack/Break and Seat and AC Overlay			
						4 inches		4 inches With SS						4 inches		8 inches	
				RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP	RSP	CSP
WF	JPCP	AZ (04)	ND	ND	—	3	—	0	—	ND	—	7	—	8	—	12	—
		IN (18)	ND	ND	—	3	—	0	—	ND	—	3	—	3	—	16	—
		MO (29)	20	20	0	12	8	0	20	20	0	9	11	11	9	10	10
		SD (46)	ND	ND	—	ND	—	ND		ND	—	ND	—	ND	—	ND	—
		Average	20	20	0	6	14	0	20	20	0	6	14	7	13	13	7
	JRCP	IL (17)	5	0	5	20	15	20	15	0	5	ND	—	ND	—	20	15
		IA (19)	ND	ND	—	ND	—	NS		ND	—	NS	—	20	—	14	—
		MI (26)	0	ND	—	19	19	ND		0	0	15	15	14	14	12	12
		MO (29)	ND	ND	—	ND	—	NS		ND	—	20	—	ND	—	20	—
		PA (42)	ND	ND	—	20	—	NS		ND	—	20	—	20	—	ND	—
		Average	3	0	3	20	17	20	17	0	3	18	15	18	15	17	14
WNF	JPCP	AL (01)	20	20	0	7	13	0	20	12	8	5	15	4	16	20	0
		AK (05)	ND	ND	—	8	—	0	—	ND	—	8	—	10	—	13	—
		CA (06)	ND	ND	—	6	—	ND	—	ND	—	ND	—	11	—	12	—
		TN (47)	ND	20	—	13	—	1	—	ND	—	11	—	ND	—	ND	—
		Average	20	20	0	9	11	0	20	12	8	8	12	8	12	15	5
	JRCP	OK (40)	ND	ND	—	12	—	17	—	ND	—	17	—	4	—	20	—

— Indicates could not be calculated.
1 inch = 25.4 mm.
SS = Saw and seal of joints.
ND = No data.
NS = Negative model slope (pavement condition and/or distress improving over time with no treatment).

IRI

The data listed in table 95 indicate the following:

- Although the RFP and CFP of some treated pavement sections are listed in the table, the results were based on only one test section and one control section. Hence, no substantial discussion and/or conclusion could be made.

- Based on the limited number of test sections in each treatment type, the following appeared to be true, on average:

    - The maximum restoration and no AC overlay treatment type yielded the lowest RFP. This could be related to the treatment type or more likely the construction quality that yielded high pavement roughness.

    - The impact of rigid pavement type and climatic region on the performance of the treated test sections was similar.

Rut Depth

The data listed in table 96 indicate the following:

- Although the test sections that received AC overlay has rut depth measurements available in the LTPP database, the PCC control sections were not subjected to AC overlay and hence did not have rut depth data. Therefore, no CFPs/CSPs could be calculated.

- Once again, although RFPs/RSPs of most treated pavement sections are listed in the table, the data in each category (each cell in the table) were based on only one test section. Nevertheless, the data in the table indicate that RFPs/RSPs of all treated pavement sections, where a minimum of three data points were collected, was 20 years, regardless of the treatment type, pavement type, or climatic region.

Alligator Cracking

The data listed in table 97 indicate the following:

- The LTPP database had no alligator cracking data for any of the control sections. Once again, the reason was that the control sections were rigid pavement, while the test sections were composite pavements.

- The reported alligator cracking data on the test sections were highly likely top-down cracking. The reason was that in composite pavements, surface tensile stress and strain due to pavement-tire interaction was greater than the tensile stress and strain at the bottom of the AC overlay. Nevertheless, RSPs of most of the treated pavement sections listed in table 97 were based on only one test section per treatment type, pavement type, and climatic region. The data could not be compared with the control sections to extract treatment benefits because of the different pavement types. However, the differential benefits of the various treatments could be obtained by studying the minimum, maximum, and average values of RSP. The data in the table indicate the following:

- RSPs of the test sections subjected to minimum restoration and 4-inch (102-mm) AC overlay ranged from a low of 5 years to a high of 20 years, with an average of about 14 years.
- RSPs of the test sections subjected to minimum restoration, sawing and sealing the joints, and 4-inch (102-mm) AC overlay ranged from a low of 5 years to a high of 20 years, with an average of about 15 years.
- RSPs of the test sections subjected to maximum restoration and 4-inch (102-mm) AC overlay ranged from a low of 8 years to a high of 20 years, with an average of about 15 years.
- RSPs of the test sections subjected to crack, break and seat, and 4-inch (102-mm) AC overlay ranged from a low of 5 years to a high of 20 years, with an average of about 15 years.
- RSPs of the test sections subjected to crack, break, and seat and 8-inch (203-mm) AC overlay ranged from a low of 7 years to a high of 20 years, with an average of about 15 years.

These observations indicate that the pavement performance of the five treatments was almost the same and independent of pavement type and climatic region.

Longitudinal Cracking

The data listed in table 98 indicate the following:

- The LTPP database contained three or more time-series data points for only three control sections. The other 11 sections either had fewer than three data points or the control section was treated before three data points were collected.
- It appeared that none of treatment types applied to JPCP and JRCP test sections located in the WF region were effective. The performance of the treated sections was less than the performance of the control sections.
- For JPCP test sections located in the WNF region, it appeared that the minimum restoration and no AC overlay treatment yielded the highest RSP (20 years), while the crack and break and seat and 8-inch AC overlay yielded an RSP of 10 years. RSPs of the test sections that received each of the other four treatments were 7 years.

Transverse Cracking

The data listed in table 99 indicate the following:

- For the JRCP test section located in Illinois (WF region), the minimum restoration and no AC overlay treatment appeared not to address the transverse cracking problem; the RSP of the treated test section is 0 years. Further, RSPs of two test sections located in Illinois and Michigan and subjected to maximum restoration and no AC overlay treatment were also 0 years. The data from this limited number of test sections suggested that neither the minimum nor the maximum restoration with no overlay treatments addressed transverse cracking problems in JRCP test sections, or the treatment construction quality was not adequate, or a combination thereof. On the other hand, the two treatments appeared to be the right treatment for the four JPCP test sections located in the WF (two sections in Missouri) and WNF regions (two sections in Alabama).

- The other three treatments—minimum and maximum restoration with 4-inch (102-mm) AC overlay—appeared to be the right treatment for transverse cracking of JRCP test sections located in the WF region.

Summary, Conclusions, and Recommendations for LTPP SPS-6

The available data in the LTPP database Standard Data Release 28.0 regarding the LTPP SPS-6 experiment were downloaded, organized, and analyzed. The intent was to study the impact of seven maintenance treatments on pavement performance. Each of the 7 test sections of each of the 14 test sites was subjected to certain treatments. The measured condition and distress data for each test site and control section were analyzed, and RFP, RSP, CFP, and CSP were calculated. The results were then grouped per pavement type and climatic region for further analyses. Unfortunately, the IRI and distress data for many control sections and for some test sections did not support the analyses because of either the lack of three data points or improvement in the pavement condition and/or distress over time without the application of treatments. Consequently, the results for only a few test sections could be compared. Based on the limited number of test and control sections, the following conclusions were drawn:

- RFP values of the treated pavement sections were independent of pavement type and climatic region.

- The pavement performance based on rut depth of treated test sections was independent of the treatment type, pavement type, and climatic region.

- The alligator cracking data in the database were highly likely an advanced form of top-down cracking (top-down cracks are fatigue cracks that initiate at the pavement surface and, over time, propagate downward) where the transverse and longitudinal cracks resembled alligator cracking.

- The performance of the test sections in terms of longitudinal cracking was worse after subjecting the section to any of the seven treatment types.

- Minimum and maximum pavement restoration with no AC overlay treatments did not improve the performance of the JRCP test sections.

It should be noted that each of these conclusions should be accepted with cautious because they were based on the results of a few and sometimes on only one test section.

IMPACTS OF BONDED CONCRETE OVERLAYS ON PAVEMENT PERFORMANCE USING THE LTPP SPS-7 TEST SECTIONS

The main objective of the SPS-7 experiment is to study the effects of bonded concrete overlay thickness, surface preparation before concrete overlay, and the use of cement grout on the performance of PCC pavements. Four SPS-7 test sites were initiated between 1990 and 1992. Three of the four sites consisted of CRCP test sections while the fourth site consisted of JPCP test sections. Each of the four test sites had eight test sections and one control section, except the test site in Louisiana, where no control section was included. The eight test sections were subjected to one of the following treatments (the numbers in parenthesis are the LTPP designation of the treatment):

- Three-inch (76-mm) concrete overlay with milling and grouting (702).
- Three-inch (76-mm) concrete overlay with milling (703).
- Three-inch (76-mm) concrete overlay with shot blasting (704).
- Three-inch (76-mm) concrete overlay with shot blasting and grouting (705).
- Five-inch (127-mm) concrete overlay with shot blasting and grouting (706).
- Five-inch (127-mm) concrete overlay with shot blasting (707).
- Five-inch (127-mm) concrete overlay with milling (708).
- Five-inch (127-mm) concrete overlay with milling and grouting (709).

For each test section that was subjected to one of these treatments, the available time-series pavement condition and distress data from the time of treatment to that of the next treatment were used to calculate the RFP and RSP of that section. Hence RFP and RSP describe the time period between the treatment construction and the time when the pavement condition or distress reached the prespecified threshold values. The RFPs and RSPs of the control sections were also calculated. For each pavement condition (IRI) and distress type (longitudinal and transverse cracking), the treatment benefits were calculated based on the following:

- The RFP and RSP of each treated test section.

- The difference in the RFP or RSP of the treated test section and the RFP or RSP of the associated control section. The difference was labeled CFP and CSP.

Results of the analyses are listed in table 100 through table 102 and discussed in the following subsections based on pavement condition and distress type. It should be noted that, for each of the CRCP test sections, the total transverse crack length was calculated as the sum of half of the cumulative length of low severity transverse cracks, the total length of medium severity cracks, and the total length of high severity transverse cracks. The reason was that the signature of CRCP is the tightly spaced transverse cracks (also called shrinkage cracks). Some of these transverse cracks may open up over time, connect, and produce punch-outs. After careful observations of the CRCP transverse crack data, it was observed that for most CRCP test sections, the total length of the low-severity transverse cracks reported in the database exceeded the crack saturation point. Therefore, it was assumed that about half of the total length of the reported low-severity transverse cracks was open enough to be considered in the analyses. The other half were very tight shrinkage cracks.

Table 100. Impact of bonded concrete overlays on pavement performance in terms of RFP based on IRI.

Climatic Region	Existing Pavement Type	State (Code)	Control Section RFP (Year)	RFP and CFP (B1) of Treated Test Sections (Years)															
				Thin Bonded Overlay								Thick Bonded Overlay							
				Milling				Shot Blasting				Milling				Shot Blasting			
				G		NG		G		NG		G		NG		G		NG	
				RFP	B1	RFP	B1	RFP	B1	RFP	B1	RFP	B1	RFP	B1	RFP	B1	RFP	B1
WF	CRCP	Iowa (19)	20	20	0	20	0	NS	—	NS	—	20	0	20	0	NS	—	20	0
		Minnesota (27)	ND	20	—	20	—	20	—	20	—	20	—	20	—	20	—	17	—
	JPCP	Missouri (29)	10	20	10	16	6	20	10	10	0	20	10	20	10	19	9	20	10
WNF	CRCP	Louisiana (22)	NCS	20	—	20	—	20	—	20	—	20	—	20	—	20	—	NS	—

—Indicates could not be calculated.
G = Grouting.
NG = No grouting.
NCS = No control section.
ND = No data, no distress is observed, or fewer than three data points.
NS = Negative slope.
Thin = 3 inches (76 mm).
Thick = 5 inches (127 mm).
B1 = CFP.

Table 101. Impact of bonded concrete overlays on pavement performance in terms of RSP based on longitudinal cracking.

Climatic Region	Existing Pavement Type	State (State Code)	Control Section RFP (Year)	RSP and CSP (B1) of Treated Test Sections (Years)															
				Thin Bonded Overlay								Thin Bonded Overlay							
				Milling				Milling				Milling				Milling			
				G		NG		G		NG		G		NG		G		NG	
				RSP	B1	RSP	B1	RSP	B1	RSP	B1	RSP	B1	RSP	B1	RSP	B1	RSP	B1
WF	CRCP	Iowa (19)	ND	ND	—	20	—	ND	—	ND	—	ND	—	NS	—	ND	—	ND	—
		Minnesota (27)	ND	ND	—	ND	—	ND	—	20	—	20	—	ND	—	20	—	11	—
	JPCP	Missouri (29)	20	ND	—	18	2	20	0	20	0	20	0	20	0	13	7	20	0
WNF	CRCP	Louisiana (22)	NCS	NS	—	ND	—	ND	—	ND	—	ND	—	20	—	ND	—	20	—

—Indicates could not be calculated.
G = Grouting.
NG = No grouting.
NCS = No control section.
ND = No data, no distress was observed, or fewer than three data points.
NS = Negative slope.
Thin = 3 inches (76 mm).
Thick = 5 inches (127 mm).
B1 = CFP.

Table 102. Impact of bonded concrete overlays on pavement performance in terms of RSP based on transverse cracking.

Climatic Region	Existing Pavement Type	State (State Code)	Control Section RFP (Year)	RSP and CSP (B1) of Treated Test Sections (Years)															
				Thin Bonded Overlay								Thin Bonded Overlay							
				Milling				Milling				Milling				Milling			
				G		NG		G		NG		G		NG		G		NG	
				RSP	B1	RSP	B1	RSP	B1	RSP	B1	RSP	B1	RSP	B1	RSP	B1	RSP	B1
WF	CRCP	Iowa (19)	ND	9	—	7	—	6	—	6	—	0	—	0	—	3	—	7	—
		Minnesota (27)	ND	0	—	0	—	0	—	0	—	0	—	ND	—	0	—	2	—
	JPCP	Missouri (29)	20	0	20	11	9	9	11	0	20	0	20	0	20	0	20	0	20
WNF	CRCP	Louisiana (22)	NCS	0	—	0	—	2	—	0		ND	—	0	—	0	—	0	—

—Indicates could not be calculated.
G = Grouting.
NG = No grouting.
NCS = No control section.
ND = No data, no distress is observed, or fewer than three data points.
NS = Negative slope.
Thin = 3 inches (76 mm).
Thick = 5 inches (127 mm).
B1 = CFP.

IRI

Table 100 lists RFPs and CFPs of all LTPP CRCP test sections located in Iowa, Minnesota, and Louisiana and the JPCP test sections located in Missouri. The data in the table indicate the following:

- The measured time-dependent IRI data of three of the eight CRCP test sections in Iowa showed improvement in the IRI over time (negative slope) without the application of any treatment. Hence, RFP and CFP of those three test sections were not calculated.

- RFPs of the other 21 CRCP test sections in Iowa, Minnesota, and Louisiana were about 20 years (20 years for 20 sections and 17 years for 1 section in Minnesota). That is, the data indicate that the performance of the treated CRCP test sections was independent of the eight treatment types and the two climatic regions.

- RFPs and the CFPs of the eight JPCP test sections located in Missouri appeared to be related to the treatment type. RFPs of the two test sections that were not grouted and subjected to a 3-inch (76-mm) concrete overlay with milling (703) or with shot blasting (704) are 16 and 10 years, respectively. These RFPs were 20 and 50 percent lower than the other two test sections that were grouted and subjected to 3-inch (76-mm) overlay and the four test sections that were subjected to 5-inch (127-mm) concrete overlays with and without grouting.

- The maximum CFP of the JPCP test sections (10 years) is mainly due to the low RFP of the control section (10 years).

Longitudinal Cracking

Table 101 lists RSPs and CSPs of all LTPP CRCP test and control sections located in Iowa, Minnesota, and Louisiana, and the JPCP test sections located in Missouri. The data in the table indicate the following:

- Only one of the eight test sections in Iowa had adequate time-series longitudinal data to be analyzed. The RSP of that test section was 20 years. Another test section showed improvement in the length of longitudinal cracking over time without the application of any treatment (negative slope). The LTPP database contained 0.1-ft (30-mm)-long measured longitudinal cracking over time for the other six test sections and for the control section.

- The RSPs and CSPs for four test sections in Minnesota are listed in table 101. Once again, The LTPP database contained 0.1-ft (30-mm)-long measured longitudinal cracking over time for three test sections and for the control section and only two data points for one test section.

- The RSPs and CSPs for two test sections in Louisiana are listed in table 101. Once again, the LTPP database contained 0.1-ft (30-mm)-long measured longitudinal cracking over time for four test sections and only two data points for one test section. The data showed improvement over time in the length of longitudinal cracking without the application of any treatment.

- The RSPs and CSPs of six JPCP test sections located in Missouri appear to be independent of the treatment type The RSP of one test section that was subjected to 5-inch (127-mm) concrete overlay with shot blasting and grouting was 13 years, about 7 years shorter than the RSPs of the other test sections. This could be the exception and not the rule. Stated differently, no decision could be or should be drawn based on only one section.

Transverse Cracking

Table 102 lists the RSP and the CSP values of most LTPP CRCP test sections located in Iowa, Minnesota, and Louisiana and the JPCP test sections located in the State of Missouri. The LTPP database did not contain adequate data except for two CRCP test sections, one located in Minnesota and the other in Louisiana. The data in table 102 indicated that none of the eight treatments in the two climatic regions were successful in treating transverse cracking problems in CRCP. The time-series transverse cracking data indicated that the RSP value was 0 years for two test sections in Iowa, six test sections in Minnesota, and six test sections in Louisiana. Further, the RSP of only one test section in each of the two States was 2 years, while the RSP of six test sections in Iowa ranged from 3 to 9 years.

Summary, Conclusions, and Recommendations for LTPP SPS-7

The LTPP SPS-7 experiment was designed to study the effects of bonded concrete overlay thickness, surface preparation before concrete overlay, and the use of cement grout on the performance of PCC pavements. Such study would be based on comparison between the performance of the test sections and the performance of compatible control sections. The pavement condition and distress data for each test and control section were downloaded from the LTPP database, organized, and analyzed to obtain the performance of the sections. Results of the analyses are listed in table 100 through table 102. Based on the results of the analyses, the following conclusions were drawn:

- The IRI-based performance of the treated CRCP test sections was independent of the eight treatment types and the two climatic regions.

- The performance of the JPCP test sections subjected to 3-inch (76-mm) concrete overlay with milling (703) or with shot blasting (704) treatments appeared to be lower than the performance of the other JPCP test sections subjected to the other six treatments.

- Because of lack of an adequate number of data points in the LTPP database, no specific conclusions could be drawn regarding longitudinal cracking performance.

- None of the eight treatments were effective to treat transverse cracking problems of the CRCP test sections.

IMPACTS OF PAVEMENT TREATMENTS ON PAVEMENT PERFORMANCE USING THE LTPP GPS-7 TEST SECTIONS

The GPS-7 pavement test sections are composites that were overlain prior to their assignment to the LTPP Program. The experiment also includes rigid pavement test sections that were moved from other LTPP experiments after they were subjected to AC overlay or existing composite pavement test sections that were subjected to mill and fill. The test sections in the GPS-7 experiment are classified as GPS-7A, -7B, -7C, -7D, -7F, and -7S. The following explains each of the classifications:

- **GPS-7A:** The test sections under this classification were part of the original LTPP design. They were subjected to AC overlay prior to their assignment to the LTPP Program.
- **GPS-7B:** The test sections under this classification were also part of the original LTPP design. They were subjected to AC overlay following assignment to the LTPP Program.
- **GPS-7C, -7D, -7F, and -7S:** The test sections under these classifications did not have an experimental design associated with them. They were moved to GPS-7C, -7D, -7F, or -7S classification from other LTPP experiments after they were subjected to rehabilitation actions. The specific classification in the four GPS-7 experiments depended on the type of pavement rehabilitation detailed as follows:
    - If the rigid pavement test sections from other LTPP experiments were overlain with virgin AC mixes, they were moved to the GPS-7B classification.
    - If the rigid pavement test sections from other LTPP experiments were overlain or if the existing composite pavement test sections were overlain again using recycled AC mixes, they were moved to the GPS-7C classification.
    - If the existing composite pavement test sections were overlain again using conventional AC mixes, they were moved to the GPS-7D classification.
    - If the rigid pavement test sections from other LTPP experiments were subjected to crack and break and seat before being overlain using virgin or recycled AC mixes, they were moved to the GPS-7F classification.
    - If the existing composite pavement test sections from other LTPP experiments were subjected to mill and fill using virgin or recycled AC mixes, they were moved to the GPS-7S classification.

Unfortunately, the number of rigid and composite pavement test sections that had more than three condition and/or distress data points before they were subjected to overlay or mill-and-fill treatments was extremely low. Given that the behavior of rigid pavement test sections would be much different than that of a composite pavement test sections, they could not be grouped to increase the number of test sections for analyses. However, the LTPP test sections in the GPS-7 experiment that had three or more after treatment time-series pavement condition and/or distress data points were grouped according to the following variables:

- Two treatment types (AC overlay and mill and fill).
- AC mix type (virgin and recycled).
- Thickness types (thin ≤ 2.5 inches (63.5 mm) and thick > 2.5 inches (63.5 mm)).
- Four climatic regions (WF, WNF, DF, and DNF).
- One pavement condition (IRI).
- Four pavement distress types (rut depth, and alligator, longitudinal, and transverse cracking).

After grouping, the data were analyzed to assess, in each climatic region, the impacts of treatment type, AC mix type, and thickness on the calculated RFP and RSP based on IRI, rut depth, and cracking. It should be noted that the LTPP database contained no before treatment pavement condition and distress data for any test section. Therefore, only the RFP or RSP of the pavement sections were calculated. For each pavement condition and distress type, the average RFPs and/or RSPs of the test sections located in the same climatic region were calculated and are listed in table 103 through table 107. The data in the five tables are discussed in the following sections per pavement condition and distress type.

Table 103. Impacts of various treatment types on RFP of the test sections based on IRI.

Treatment Type	Mix Type	Thickness	Number of Test Sections and RFP Values in the Designated Climatic Region							
			WF		WNF		DF		DNF	
			No.	RFP (Year)	No.	RFP (Year)	No.	RFP (Year)	No.	RFP (Year)
Overlay	Virgin	Thin	6	18	0	—	0	—	0	—
		Thick	25	19	6	20	1	20	1	20
	Recycled	Thin	2	20	1	20	1	20	0	—
		Thick	0	—	0	—	1	20	1	20
Mill and fill	Virgin	Thin	3	20	2	17	1	20	0	—
		Thick	3	20	2	20	0	—	0	—
	Recycled	Thin	1	20	0	—	0	—	0	—
		Thick	0	—	0	—	0	—	0	—

— Indicates could not be calculated.
No. = Number of test sections.
Thin = ≤ 2.5 inches (63.5 mm).
Thick = > 2.5 inches (63.5 mm).

Table 104. Impacts of various treatment types on RFP/RSP of the test sections based on rut depth.

Treatment Type	Mix Type	Thickness	Number of Test Sections and RFP/RSP Values in the Designated Climatic Region							
			WF		WNF		DF		DNF	
			No.	RFP/RSP (Year)	No.	RFP/RSP (Year)	No.	RFP/RSP (Year)	No.	RFP/RSP (Year)
Overlay	Virgin	Thin	4	20	0	—	0	—	0	—
		Thick	20	20	6	20	0	—	0	—
	Recycled	Thin	0	—	0		1	6	0	—
		Thick	0	—	1	20	0	—	0	—
Mill and fill	Virgin	Thin	3	20	1	20	1	20	0	—
		Thick	3	20	1	20	0	—	0	—
	Recycled	Thin	1	20	0	—	0	—	0	—
		Thick	0	—	0	—	0	—	0	—

— Indicates could not be calculated.
No. = Number of test sections.
Thin = ≤ 2.5 inches (63.5 mm).
Thick = > 2.5 inches (63.5 mm).

Table 105. Impacts of various treatment types on RSP of test sections based on alligator cracking.

Treatment Type	Mix Type	Thickness	Number of Test Sections and RSP Values in the Designated Climatic Region							
			WF		WNF		DF		DNF	
			No.	RSP (Year)	No.	RSP (Year)	No.	RSP (Year)	No.	RSP (Year)
Overlay	Virgin	Thin	2	19	1	11	0	—	0	—
		Thick	13	14	4	12	0	—	1	10
	Recycled	Thin	1	9	1	12	0	—	0	—
		Thick	0	—	1	20	0	—	0	—
Mill and fill	Virgin	Thin	1	20	1	20	1	20	0	—
		Thick	2	12	0	—	0	—	0	—
	Recycled	Thin	0	—	0	—	0	—	0	—
		Thick	0	—	0	—	0	—	0	—

— Indicates could not be calculated.
No. = Number of test sections.
Thin = ≤ 2.5 inches (63.5 mm).
Thick = > 2.5 inches (63.5 mm).

Table 106. Impacts of various treatments on pavement performance in terms of RSP based on longitudinal cracking.

Treatment Type	Mix Type	Thickness	Number of Test Sections and RSP Values in the Designated Climatic Region							
			WF		WNF		DF		DNF	
			No.	RSP (Year)	No.	RSP (Year)	No.	RSP (Year)	No.	RSP (Year)
Overlay	Virgin	Thin	4	6	1	4	0	—	0	—
		Thick	17	8	6	8	0	—	2	10
	Recycled	Thin	0	—	0	—	1	8	0	—
		Thick	0	—	1	10	1	8	0	—
Mill and fill	Virgin	Thin	1	9	1	15	1	8	0	—
		Thick	3	6	2	13	0	—	0	—
	Recycled	Thin	2	5	0	—	0	—	0	—
		Thick	0	—	0	—	0	—	0	—

— Indicates could not be calculated.
No. = Number of test sections.
Thin = ≤ 2.5 inches (63.5 mm),
Thick = > 2.5 inches (63.5 mm).

Table 107. Impacts of various treatments on pavement performance in terms of RSP based on transverse cracking.

Treatment Type	Mix Type	Thickness	Number of Test Sections and RSP Values in the Designated Climatic Region							
			WF		WNF		DF		DNF	
			No.	RSP (Year)	No.	RSP (Year)	No.	RSP (Year)	No.	RSP (Year)
Overlay	Virgin	Thin	4	8	1	6	0	—	0	—
		Thick	16	11	6	17	0	—	1	13
	Recycled	Thin	1	3	1	7	0	—	0	—
		Thick	0	—	1	17	0	—	0	—
Mill and fill	Virgin	Thin	1	11	1	17	1	9	0	—
		Thick	4	16	2	16	0	—	0	—
	Recycled	Thin	2	12	0	—	0	—	0	—
		Thick	0	—	0	—	0	—	0	—

— Indicates could not be calculated.
No. = Number of test sections.
Thin = ≤ 2.5 inches (63.5 mm).
Thick = > 2.5 inches (63.5 mm).

IRI

Table 103 lists the average RFPs of test sections located in the same climatic zone and subjected to one of the four treatments listed in the table. The data indicate that the average RFP of the test sections was between 17 and 20 years.

Rut Depth

Table 104 lists the average RFPs/RSPs of test sections located in the same climatic zone and subjected to one of the four treatments listed in the table. The data indicate that except for one test section, the average RFP/RSP of all other sections was 20 years. Again, the exemption was one test section located in the DF region and subjected to thin overlay using recycled AC mixes. Its RFP/RSP was only 6 years. The reason for this RFP/RSP was highly likely problems associated with the AC mix or with construction of the overlay. The AC mix problems could be excessive binder content or unstable mix while the construction issue could be inadequate compaction of the overlay or the early opening of the road to traffic.

Alligator Cracking

Table 105 lists the average RSPs of test sections located in the same climatic zone and subjected to one of the four treatments listed in the table. It is important to note that the labeling as "Alligator Cracking" was highly likely not related to bottom-up fatigue cracks in composite pavements. The label was most likely related to advanced stages of top-down fatigue cracking. Nevertheless, the data in the table indicate that the average RSPs varied from 9 to 20 years detailed as follows:

- In the WF region, the average RSPs of the test sections that were subjected to thin overlay and mill-and-fill treatments using virgin AC mixes were 19 and 20 years, respectively, while the average RSPs of the test sections that were subjected to thick overlay and mill-and-fill treatments using virgin AC mixes were 14 and 12 years, respectively. Further, the RSP of thin overlay treatment using recycled AC mixes was only 9 years.

- The RSPs of the eight test sections located in the WNF region varied from 11 to 20 years.

- In the DF region, the RSP of the one test section subjected to thin mill-and-fill treatment using virgin AC mix was 20 years.

- In the DNF region, the RSP of the one test section subjected to thick overlay treatment using virgin AC mix was 10 years.

Because of the limited number of test sections subjected to a certain treatment and located in one climatic zone, and the lack of time-dependent pavement condition and distress data before treatment, no reliable conclusion could be drawn regarding the benefits of one treatment. For the same reasons the benefits of the various treatments could not be compared.

Longitudinal Cracking

Table 106 lists the average RSPs of test sections located in the same climatic zone and subjected to one of the four treatments listed in the table. The data in the table indicate that the average RSPs varied from 4 to 15 years detailed as follows:

- In the WF region, the average RSPs of the test sections that were subjected to thin overlay and mill-and-fill treatment treatments using virgin AC mixes were 6 and 9 years, respectively, while the average RSPs of the test sections that were subjected to thick overlay and mill-and-fill treatments using virgin AC mixes were 8 and 6 years, respectively. Furthermore, the RSP of thin mill-and-fill treatment using recycled AC mixes was only 5 years.

- The RSPs of the 11 test sections located in the WNF region varied from 4 to 15 years.

- In the DF region, the three test sections had an RSP of 8 years.

- In the DNF region, the average RSP of the test sections subjected to thick overlay treatment using virgin AC mix was 10 years.

Once again, because of the limited number of test sections subjected to a certain treatment and located in one climatic zone, and the lack of time-dependent pavement condition and distress data before treatment, no reliable conclusion could be drawn regarding the benefits of one treatment. For the same reasons, the benefits of the various treatments could not be compared.

Transverse Cracking

Table 107 lists the average RSPs of test sections located in the same climatic region and subjected to one of the four treatments listed in the table. The data in the table indicate that the average RSPs varied from 4 to 17 years, detailed as follows:

- In the WF region, the average RSPs of the test sections that were subjected to thin overlay and mill-and-fill treatments using virgin AC mixes were 8 and 11 years, respectively, while the average RSPs of the test sections that were subjected to thick overlay and mill-and-fill treatments using virgin AC mixes were 11 and 16 years, respectively. Furthermore, the RSPs of thin overlay and mill-and-fill treatment using recycled AC mixes were 3 and 12 years, respectively.

- The RSPs of the 12 test sections located in the WNF region varied from 6 to 17 years. The thicker the AC overlay was, the longer the RSP was.

- In the DF region, the one test section had an RSP of 9 years.

- In the DNF region, the one test section had an RSP of 13 years.

Similar to alligator and longitudinal cracking, because of the limited number of test sections subjected to a certain treatment and located in one climatic zone, and the lack of time-dependent pavement condition and distress data before treatment, no reliable conclusion could be drawn regarding the benefits of one treatment. For the same reasons, the benefits of the various treatments could not be compared.

Summary, Conclusions, and Recommendations for LTPP GPS-7

As noted earlier, the LTPP GPS-7 experiment consisted of the following types of sections:

- Composite pavement test sections that were subjected to AC overlay prior to their assignment to the LTPP Program.

- JPCP and JRCP test sections that were moved from other LTPP experiments after they were subjected to AC overlay.

- Composite pavement test sections that were parts of the other LTPP experiments that were milled and filled and moved to GPS-7.

All of the GPS-7 test sections that had more than three after-treatment data points were analyzed and then grouped based on the AC overlay thickness, surface preparation before the AC overlay, overlay type, and climatic regions. It should be noted that the before treatment data were those for rigid pavement, hence they were not included in the comparison of the pavement performance before and after treatment. Nevertheless, results of the analyses are listed in table 103 through table 107. Based on the results of the analyses, the following conclusions were drawn:

- The average RFPs of the test sections in all the climatic regions subjected to any of the treatments previously described was between 17 and 20 years.

- The average RFP/RSP of all but one test section in all the climatic regions was 20 years. The RFP/RSP of that one test section was 6 years, most likely owing to construction problems.

- Given the limited number of sections available for analyses, no reliable conclusions could be drawn for alligator, transverse, and longitudinal cracking.

CHAPTER 7. LTPP DEFLECTION DATA ANALYSES

FWD DEFLECTION DATA AND RFP AND RSP

Pavement deflection data are typically collected by State transportation departments at the project level and rarely at the network level. However, most LTPP test sections were subjected to deflection testing on a periodic basis. The deflection data were collected using the most common type of equipment, the FWD. FWD tests are often preferred over laboratory testing for several reasons, including the following:[78,83]

- FWD tests are nondestructive in nature, whereas laboratory tests require cores.
- The operational cost per FWD test is much lower than a laboratory test.
- FWD tests have short duration and can be designed to provide more coverage of the pavement network compared with laboratory tests, which are time consuming and limited to the locations where pavement cores were extracted.
- FWD data reflect the in-situ boundary conditions.

The FWD operates on two basic assumptions: the force of impact of a falling weight is considered a static load, and the roadbed soils act as an elastic body.[83] The deflection data collected from FWD testing are typically used for the following purposes:

- Backcalculating the moduli values of the pavement layers, which can be used to assess the structural capacity of the roadway and to facilitate treatment type selection and design.
- Assessing the variability of the structural capacity along and across the pavement sections.
- Analyzing LTE in rigid pavements.
- Determining the presence of voids under the pavement slabs.
- Studying the magnitude of slab curling and its impact on ride quality.

At the time of this report, research was exploring methods to increase the efficiency of deflection data collection by using a rolling wheel deflectometer (RWD). The RWD collects pavement deflection data at highway speeds and makes network-level data collection more feasible. It was reported that the results were used to flag structurally deficient pavement sections for further analyses and to estimate the structural number.[83,84] In this study, the LTPP deflection data were analyzed to determine whether the data could be used to estimate the RSP of pavement sections and to determine the critical time for pavement preservation.

The RSP algorithm is primarily based on the measured time-dependent pavement surface condition and distress data and their corresponding threshold values. Hence, the distress (such as cracking) must be visible from the pavement surface. During the development of the RFP and RSP concepts, it was envisioned that the pavement deflection data could be used to indicate impending distress and become a part of the RSP algorithm. Such algorithms would empower State transportation departments to take corrective actions prior to the manifestation of surface defects.

To incorporate deflection into the RSP algorithm, a deflection threshold value must be developed for each pavement section. To investigate the potential for the development of deflection threshold values, the measured FWD deflection data of various LTPP test sections were analyzed as described in the next few subsections.

At the outset, it was envisioned that the rate of change and the magnitude of the measured deflections were related to the measured pavement distresses such as alligator cracking. Because alligator cracks initiate at the bottom of the asphalt layer and propagate upward toward the pavement surface, the pavement system starts to weaken as the cracks initiate and before they reach the pavement surface. Therefore, it was assumed that flexible pavement deflections would start to increase before the appearance of alligator cracking on the pavement surface (i.e., the response of a pavement structure to load would increase as the pavement deteriorated, which could be used as an early warning of impending, surface alligator cracking). Similarly, for rigid pavements, increasing the magnitude of deflection could be a sign of deterioration of the concrete slab support. Such deterioration could lead to distresses such as transverse cracking, corner breaks, and so forth. Again, increasing deflection over time may provide a flag prior to the appearance of the pavement surface distress. Further, LTE across joints or cracks in rigid pavements was typically measured using the differential deflection across joints or cracks. Increasing relative differential deflection implied lower LTE. It was also envisioned that the rate of change of LTE across joints or cracks in rigid pavements could be related to the rate of change of faulting. Joints with a good load transfer mechanism, such as dowel bars, would have almost 100-percent LTE and little or no faulting, whereas joints without dowel bars or with damaged or sheared dowel bars, or cracks with no aggregate interlock, would have minimal LTE and increased probability of faulting. Therefore, decreasing LTE over time could be used to warn of impending surface defects.

TIME-SERIES FWD DATA MODELING—FLEXIBLE PAVEMENT

Once again, it was envisioned that an analysis of the time-series deflection data could provide indication of relationships between pavement deflection and pavement condition or distress. To investigate such potential relationships, FWD deflection data from the LTPP SMP test sections were plotted as a function of time following the procedures discussed in the following subsections. A partial record of the inventory data for the SMP test sections are listed in table 108. The deflection data were analyzed to determine trends in the measured deflection over time. Such trends, if they existed, would provide a tool for pavement managers to estimate future pavement conditions and distress before surface defects such as cracking occurred. The SMP test sections were used in this analysis because deflection data were collected on a much more frequent basis than the other LTPP test sections. However, because the FWD tests were performed at various times of the year and at different temperatures, the measured deflection was adjusted to account for the material properties and their relationships to temperature. The details of these adjustments are presented in the following subsections.

Table 108. LTPP SMP partial inventory data.

SHRP ID	AC Thickness (inches)	Base Thickness (inches)	Subbase Thickness (inches)	Roadbed Soil Type	Most Recent Traffic (ESAL/year)	Climatic Region
010101	7.4	7.9	—	—	—	WNF
010102	4.2	12.0	—	—	—	WNF
040113	4.4	7.5	—	—	337,000	DNF
040114	6.8	12.0	—	—	308,000	DNF
041024	10.8	6.3	—	Coarse-grained soil: clayey sand with gravel	—	DNF
081053	4.6	5.4	23.5	Fine-grained soils: lean inorganic clay	42,000	DF
091803	7.1	12.0	—	Coarse-grained soils: well-graded sand with silt and gravel	36,000	WF
100102	4.1	11.8	39.0	Coarse-grained soils: poorly graded sand	—	WNF
131031	10.6	8.8	—	Coarse-grained soil: silty sand	—	WNF
131005	7.6	8.8	—	Coarse-grained soil: clayey sand	—	WNF
161010	10.7	5.4	—	Coarse-grained soil: silty sand	235,000	DF
231026	7.2	17.6	—	Coarse-grained soil: silty sand with gravel	—	WF
241634	3.6	4.8	13.0	Fine-grained soils: silt	—	WNF
251002	7.8	4.0	4.9	Coarse-grained soils: poorly graded sand with silt	—	WF
271018	4.4	5.2	—	Coarse-grained soils: poorly graded sand with silt	—	WF
271028	9.6	0.0	—	Coarse-grained soils: poorly graded sand with silt	112,000	WF
276251	7.4	10.2	—	Coarse-grained soils: poorly graded sand with silt	70,000	WF
281802	3.1	4.9	1.6	Coarse-grained soils: poorly graded sand	98,000	WNF
281016	7.6	19.3	—	—	52,000	WNF
308129	3.0	22.8	—	Fine-grained soils: gravelly lean clay with sand	39,000	DF
310114	6.6	12.0	—	—	95,000	WF
331001	8.4	19.3	14.4	Coarse-grained soils: poorly graded sand with silt	61,000	WF
351112	5.4	6.4	—	—	51,000	DNF
360801	5.0	8.4	—	—	2,000	WF
371028	1.6	8.2	—	Coarse-grained soils: poorly graded sand with silt	93,000	WNF

SHRP ID	AC Thickness (inches)	Base Thickness (inches)	Subbase Thickness (inches)	Roadbed Soil Type	Most Recent Traffic (ESAL/year)	Climatic Region
404165	2.7	5.5	—	—	151,000	WNF
460804	6.9	12.0	—	—	3,000	DF
469187	5.5	6.0	3.0	Fine-grained soils: lean inorganic clay	—	DF
481077	4.9	10.4	—	Fine-grained soils: sandy silt	132,000	WNF
481068	10.9	6.0	8.0	Fine-grained soils: sandy lean clay	—	WNF
481122	3.0	15.6	8.4	Coarse-grained soil: clayey sand	51,000	WNF
481060	7.5	12.3	6.0	—	323,000	WNF
483739	1.5	11.4	7.4	Coarse-grained soils: poorly graded sand	52,000	WNF
491001	5.1	5.8	—	Coarse-grained soil: silty sand	—	DF
501002	8.5	25.8	—	—	92,000	WF
510113	4.0	7.9	6.0	—	265,000	WNF
510114	7.3	11.9	6.0	Fine-grained soils: sandy silty clay with gravel	253,000	WNF
561007	2.8	6.2	—	Coarse-grained soil: silty sand with gravel	5,000	DF
831801	4.4	5.6	13.2	Coarse-grained soil: silty sand	372,000	WF
871622	5.7	6.7	26.3	Fine-grained soils: sandy silt	271,000	WF
906405	2.8	9.0	2.5	Coarse-grained soil: silty sand	90,000	DF

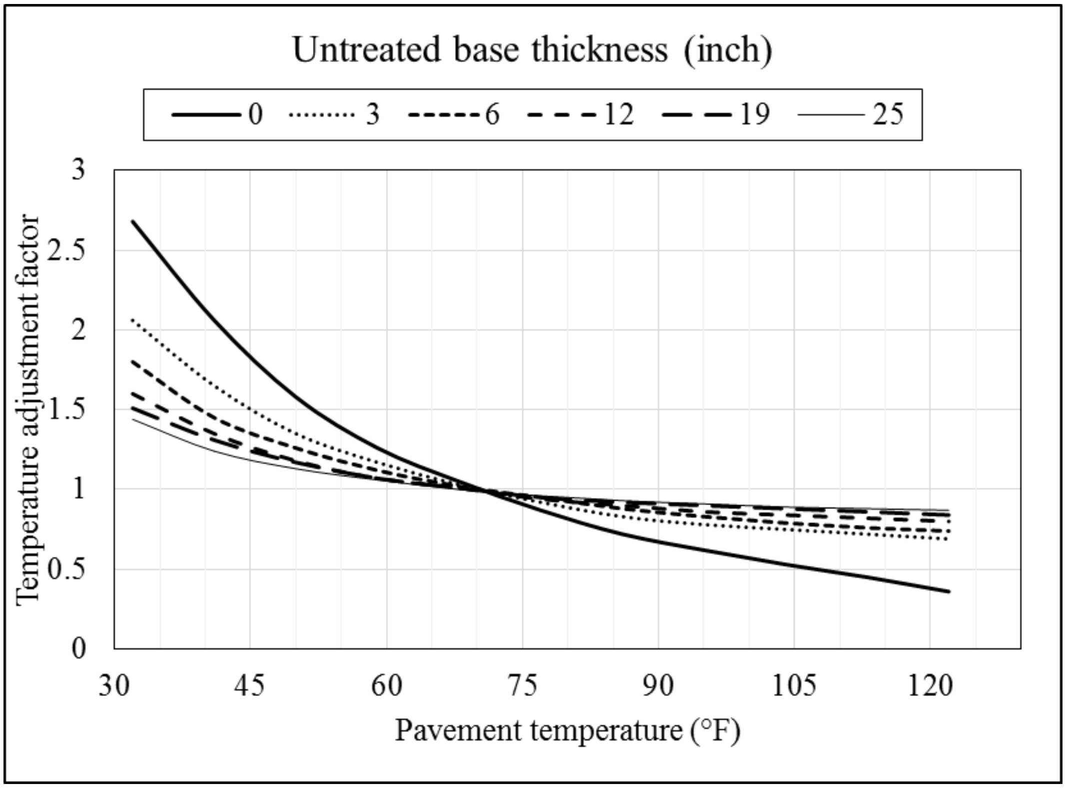
— Indicates no data were available.
1 inch = 25.4 mm.

Temperature-Deflection Correlation

Some researchers have developed correlations between temperature and the measured flexible pavement deflections. At the time of this report, the most publicized temperature correlation model was that of the AI.[85] In this model, the measured pavement deflection basin was adjusted based on the measured pavement temperature and the untreated base thickness. Stated differently, the AI temperature adjustment factor (TAF) was used as a multiplier to adjust the entire measured deflection basin to a standard temperature of 70 ºF (21 ºC). This made the implementation of the AI method at the network level relatively easy. Other models, such as the BELLS, had also been developed and verified based on the LTPP SMP data.[86] These models were developed using the measured air temperatures during the FWD testing, the measured pavement surface temperatures, and the temperatures of the asphalt mat measured at various depths from the pavement surface. The use of these models facilitated the adjustment of the backcalculated layer moduli values to different temperatures. However, the data required to use the models is not readily available in the current or historic deflection records of most State transportation departments and would require additional effort to obtain.

In this study, the main purpose of the analyses of the deflection data was to identify relationships, if any, between the measured pavement deflection and the measured pavement condition data, to determine whether the deflection data could be used to estimate the optimum time for pavement preservation. To conduct the analyses, the measured pavement deflections must be adjusted to the standard temperature of 70 ºF (21 ºC). Therefore, the first step in the analyses was to validate the existing temperature adjustment models using the LTPP measured deflection data along the various SMP test sections. The analyses were based on the measured pavement deflections and surface temperature, the data most commonly collected by State transportation departments. Please note that at the time of this report, most State transportation departments did not collect network level deflection data, however, that may change in the near future as more efficient deflection measuring techniques are developed. Nevertheless, the temperature adjustment analyses were based on network level assessments, in conjunction with the RFP/RSP concepts developed in this study. After the analyses of the temperature adjustment models, backcalculation of layer moduli were performed at the project level.

The AI method for temperature adjustment of the measured deflection data could be relatively easily applied to network-level deflection data. As stated earlier, the AI method (see figure 67) provided TAFs to be multiplied by the measured deflection basin based on the mean pavement temperature and the thickness of the untreated base layer.



1 inch = 25.4 mm.
°F = 1.8 × °C + 32.

Figure 67. Graph. AI TAF.(85)

To assess the accuracy and applicability of the AI TAFs, the measured LTPP deflection data along several SMP test sections were analyzed to determine the TAFs at each site using the following steps:

- **Step 1:** For each of the SMP test sections listed in table 108, the FWD measured deflection data, treatment history, and the inventory data were downloaded from the LTPP Standard Data Release 28.0. The corresponding measured pavement surface temperature at the time of each test was paired with the deflection data for analyses.

- **Step 2:** The measured deflection data were first analyzed and their variability along the test sections and linearity with the load magnitude were scrutinized to produce the following findings:

  - A linear relationship existed between the drop loads and the measured deflections.

  - The measured deflections along a given test section were variable.

  - The percent variability of the deflection data was more or less equal across the seven FWD sensors and in each climatic region.

- **Step 3:** The data were sorted based on the applied load and the various test locations along the test section. Drop height 2, which simulated the application of a 9,000-lbf (40-kN) half single axle load, was selected for analyses. The tests performed at the mid-lane (F1) and outer wheelpath (F3) locations were selected for individual analyses. All data from other test loads and locations were not included in the analyses.

- **Step 4:** On any particular test day, a series of 4 tests were conducted at drop height 2 and at about 10 locations along the test section. Note that tests conducted when the temperature was below freezing were not included in the analyses because the response to load is significantly different for frozen soils. On some days, the test sections were subjected to more than one series of tests. The various deflection tests conducted on a test section within each specific testing day were averaged along with the measured pavement surface temperatures to simulate the data that would be collected by a State transportation department (only one test per section). Note that the data averaging generally produced a less than 1-percent error and was dependent on the variability of the measured data.

- **Step 5:** The average peak deflection measured at sensors 1 through 7, located at 0, 8, 12, 18, 24, 36, and 60 inches (0, 203, 305, 457, 610, 914, and 1,524 mm), respectively, from the load, were plotted against the measured pavement surface temperatures. The deflection measured beneath the load (d_1) was composed of the response from all pavement layers; at 8 inches (203 mm) from the load (d_2), it had slightly less influence from the asphalt layer; and at 60 inches (1,524 mm) from the load (d_7), the pavement response was mainly from the roadbed soil. The asphalt layer was significantly more affected by temperature than the other layers, and as such, it was anticipated that the correlation between deflection and temperature would be significant at sensor 1 and would decrease with increasing distance from the load plate.

- **Step 6:** The measured deflection data for each sensor were then plotted and modeled as a function of temperature using the linear function shown in figure 68.

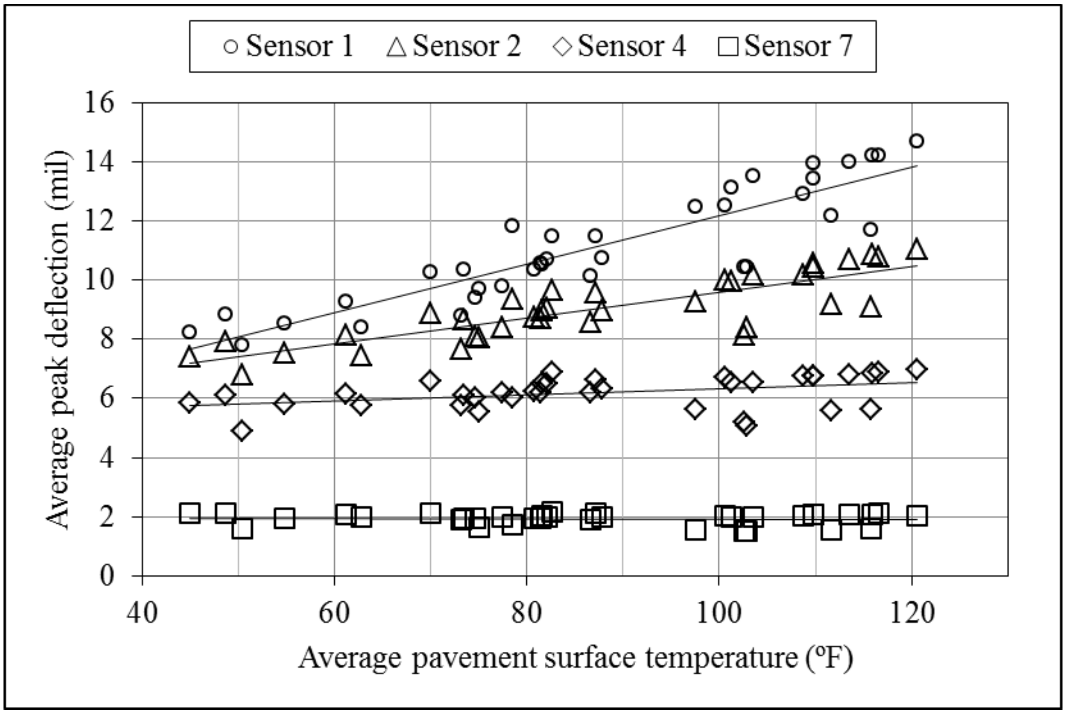
$$APD_i = \omega_i + C_i$$

Figure 68. Equation. APD_i.

Where:

APD_i = Average peak pavement deflection at sensor i.
ω_i and C_i are regression constants for sensor i.

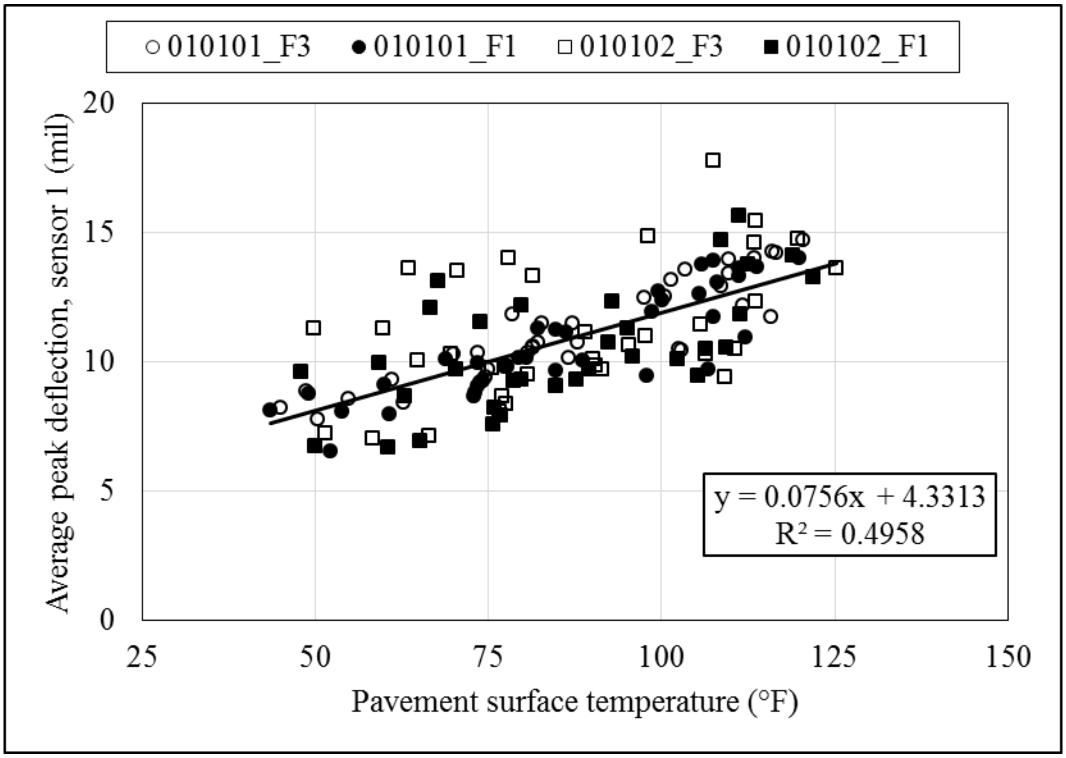
Figure 69 depicts the measured pavement deflections at sensors 1, 2, 4, and 7 as a linear function of the pavement surface temperature for SHRP test section 010101. The data in the figure indicate that, as it was expected, increases in the pavement temperatures caused increases in the pavement deflections at sensors 1 and 2, while the deflection data at sensors 4 and 7 were minimally affected by temperature. The data also indicate that the rate of change of deflection with respect to temperature (the slope of the lines) decreased as the distance from the load to the sensor increased. Indeed, the slope at sensor 7 was zero.



1 mil = 25.4 microns.
ºF = 1.8 × ºC + 32.

Figure 69. Graph. Peak measured pavement deflection at sensors 1, 2, 4, and 7 versus pavement surface temperature for SHRP test section 010101, F3.

In addition, the measured deflection data at sensor 1 along the outer wheelpath (F3) and mid lane (F1) along SHRP test sections 010101 and 010102 are depicted in figure 70 as a function of the measured pavement temperature. It can be seen from the figure that all the data could be modeled as a function of temperature using the same form as in figure 68.



ºF = 1.8 × ºC + 32.
1 mil = 25.4 micron.

Figure 70. Graph. Average measured peak deflection at sensor 1 versus the measured pavement surface temperature along the outer wheelpath and mid lane of SHRP test sections 010101 and 010102.

- **Step 7:** For each deflection sensor, figure 68 was then used to calculate the TAFs based on a standard temperature of 70 ºF (21 ºC). The calculated TAFs were then modeled as a function of temperature using figure 71. For each deflection sensor, the values of the regression constants of figure 71 were obtained and are listed in table 109.

$$TAF_{d_i} = \alpha_i T^2 + \beta_i T + \gamma_i$$

Figure 71. Equation. TAF_{di}.

Where:

TAF_{di} = Temperature adjustment factor for sensor d_i (multiply the measured pavement deflection at temperature T and sensor d_i by TAF_{d_i} to adjust the deflection to 70 ºF (21 ºC)).
d_i = Deflection sensor at the ith distance from the FWD load.
α_i, β_i, γ_i = regression constants (see table 109).
T = Measured pavement surface temperature at the time of FWD testing (ºF).

- **Step 8:** The parameters of figure 71, listed in table 109, were then plotted as a function of the lateral distance of the deflection sensors from the center of the load. The plot was used to model the regression parameters of figure 71 as a function of the lateral distance.

This yielded figure 72, which is a global temperature adjustment equation that can be applied to the measured deflections at all deflection sensors.

Table 109. The regression parameters of figure 71 for each deflection sensor.

Deflection Sensor	Distance from FWD load (inches)	Regression Terms and Values		
		α_i (E−05)	β_i (E−03)	γ_i
d_1	0	7.09	−18.24	1.930
d_2	8	3.72	−10.62	1.562
d_3	12	2.45	−7.33	1.394
d_4	18	1.59	−4.44	1.233
d_5	24	1.37	−3.00	1.143
d_6	36	1.65	−2.04	1.062
d_7	60	1.83	−2.18	1.062

1 inch = 25.4 mm.

$$TAF = AT^2 + BT + C$$

Figure 72. Equation. *TAF*.

Where:

TAF = Temperature adjustment factor (multiply the measured pavement deflection by this factor to obtain the temperature-adjusted deflection).
A, *B*, and *C* = Global regression parameters of the global temperature adjustment model.
T = Pavement surface temperature measured at the time of FWD testing (°F).

The equations in figure 73 through figure 75 define the global regression parameters in figure 72.

$$A = \alpha_1 d^3 + \beta_1 d^2 + \gamma_1 d + \delta_1$$

Figure 73. Equation. *A*.

$$B = \alpha_2 d^3 + \beta_2 d^2 + \gamma_2 d + \delta_2$$

Figure 74. Equation. *B*.

$$C = \alpha_3 d^2 + \beta_3 d + \gamma_3$$

Figure 75. Equation. *C*.

Where:

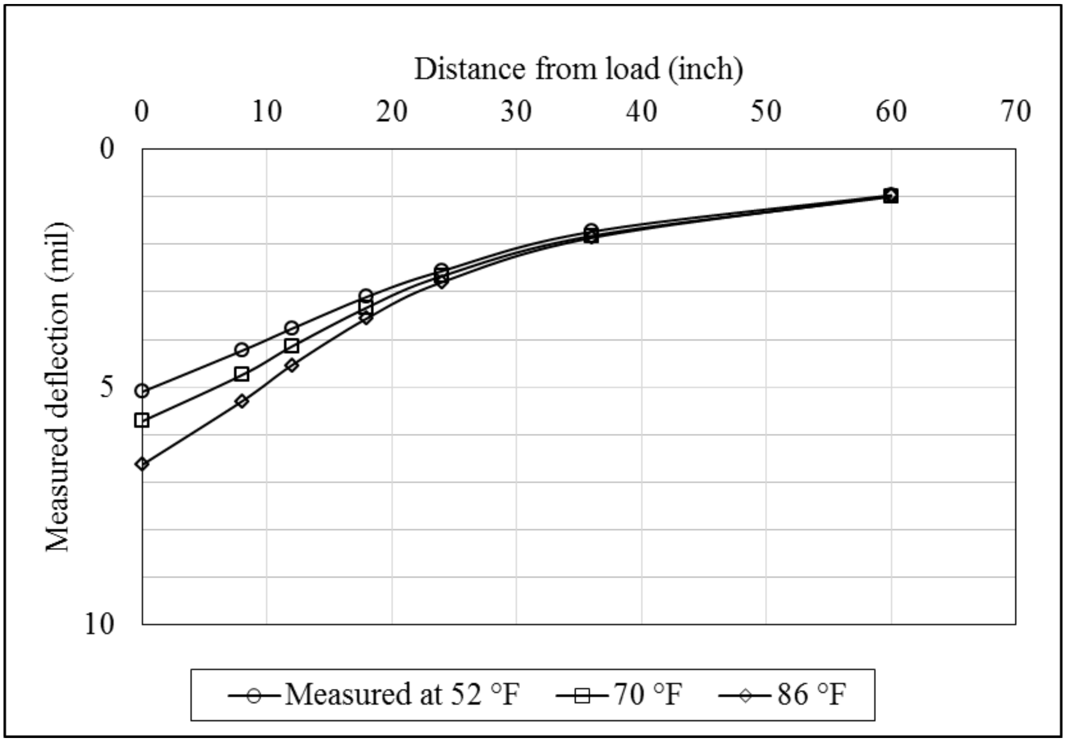
d = Lateral distance from the center of the load to the sensor (inch).
$\alpha_{1,2,3}$, $\beta_{1,2,3}$, $\gamma_{1,2,3}$, $\delta_{1,2}$ = Regression values (see table 110).

Table 110. Regression values for Calculation of Parameters *A*, *B*, and *C* in figure 73 through figure 75.

Regression Parameter	Regression Value		
	A	***B***	***C***
α_1	−1.429E−09	—	—
α_2	—	2.293E−07	—
α_3	—	—	4.839E−04
β_1	1.631E−07	—	—
β_2	—	−2.964E−05	—
β_3	—	—	−4.232E−02
γ_1	−5.516E−06	—	—
γ_2	—	1.221E−03	—
γ_3	—	—	1.881
δ_1	7.086E−05	—	—
δ_2	—	−1.831E−02	—

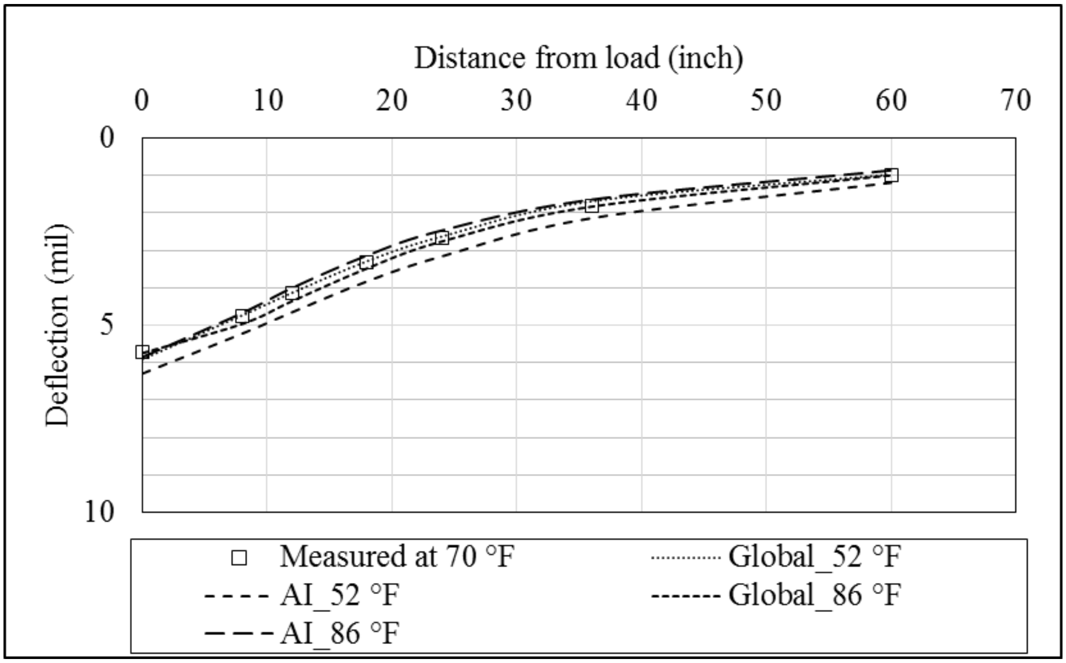
— Indicates not applicable.

- **Step 9:** The accuracy of the TAF equation (figure 72) and the AI temperature adjustment procedure were checked using the measured deflection data on April 25, 1995, along the mid-lane of the SHRP test section 351112. The measured deflection data at 52, 70, and 86 ºF (11, 21, and 30 ºC) are shown in figure 76. The data at 52 and 86 ºF (11 and 30 ºC) were adjusted to the standard temperature of 70 ºF (21ºC) using TAF and the AI procedures. Both results are depicted in figure 77. The square symbols in the figure represent the measured deflection data at 70 ºF (21 ºC) while the various dotted and dashed curves are the temperature adjusted deflection data measured at 52 and 86 ºF (11 and 30 ºC) for the TAF and AI methods, respectively. The differences between the square symbols and the dotted and dashed curves in the figure are indicative of the accuracy of the TAF and the AI temperature adjustment procedures. It can be seen from the figure that the differences between the curves obtained using TAF and the square symbols were much smaller than the differences between the curves obtained using the AI procedure and the square symbols. These differences were then calculated as a percent of the measured deflection data at 70 ºF (21 ºC). These percentages (see figure 78) represent the errors between the temperature adjusted deflection data and the measured deflection data at 70 ºF (21 ºC). It can be seen from figure 78 that the TAF (figure 72) generally produced much smaller errors than the AI procedure.



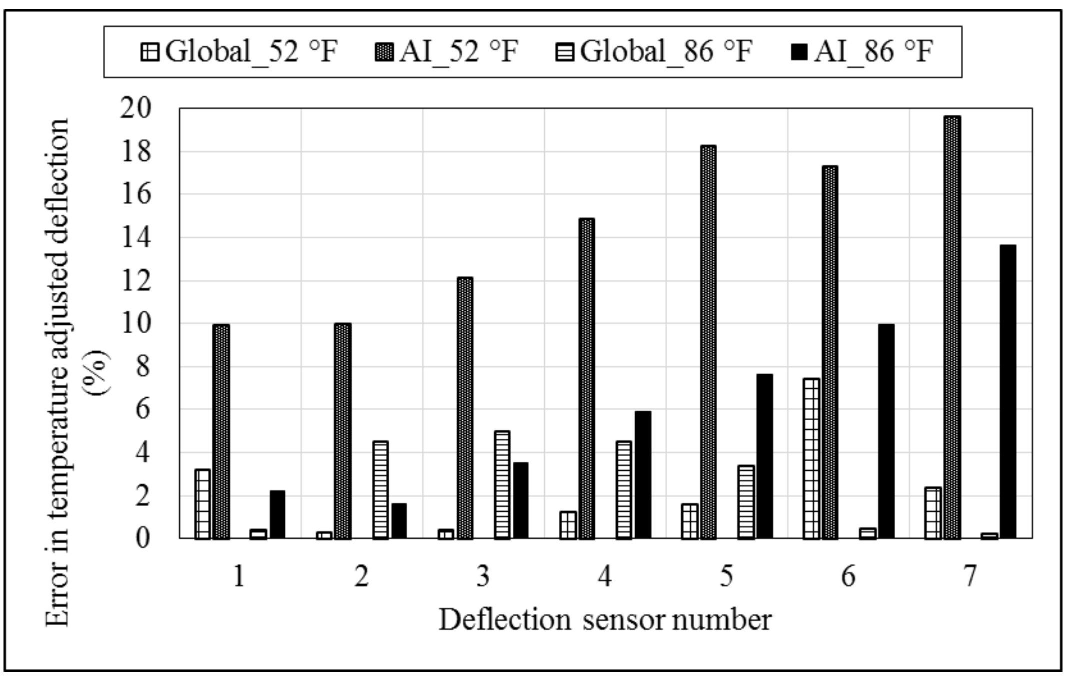
ºF = 1.8 × ºC + 32.
1 mil = 25.4 micron.
1 inch = 25.4 mm.

Figure 76. Graph. Measured deflection basins at three pavement surface temperatures 52, 70, and 86 ºF (11, 21, and 30 ºC) along the mid-lane of the SHRP test section 351112 on April 25, 1995.



°F = 1.8 × °C + 32.
1 mil = 25.4 micron.
1 inch = 25.4 mm.

Figure 77. Graph. Error from TAF and AI adjusted deflection basin at two pavement surface temperatures (52 and 86 °F (11 and 30 °C)), along the mid-lane of the SHRP test section 351112 on April 25, 1995.



°F = 1.8 × °C + 32.

Figure 78. Graph. Percent error of the temperature-adjusted deflection data using the equation in figure 72 and the AI procedure.

- **Step 10:** The previous step was repeated using the measured deflection data and pavement temperature of test sections located in the four climatic regions. The average regression parameters for figure 72 and the overall averages were then calculated and are listed in table 111. The differences in the TAF between climatic regions were generally minimal, especially above about 60 ºF (16 ºC).

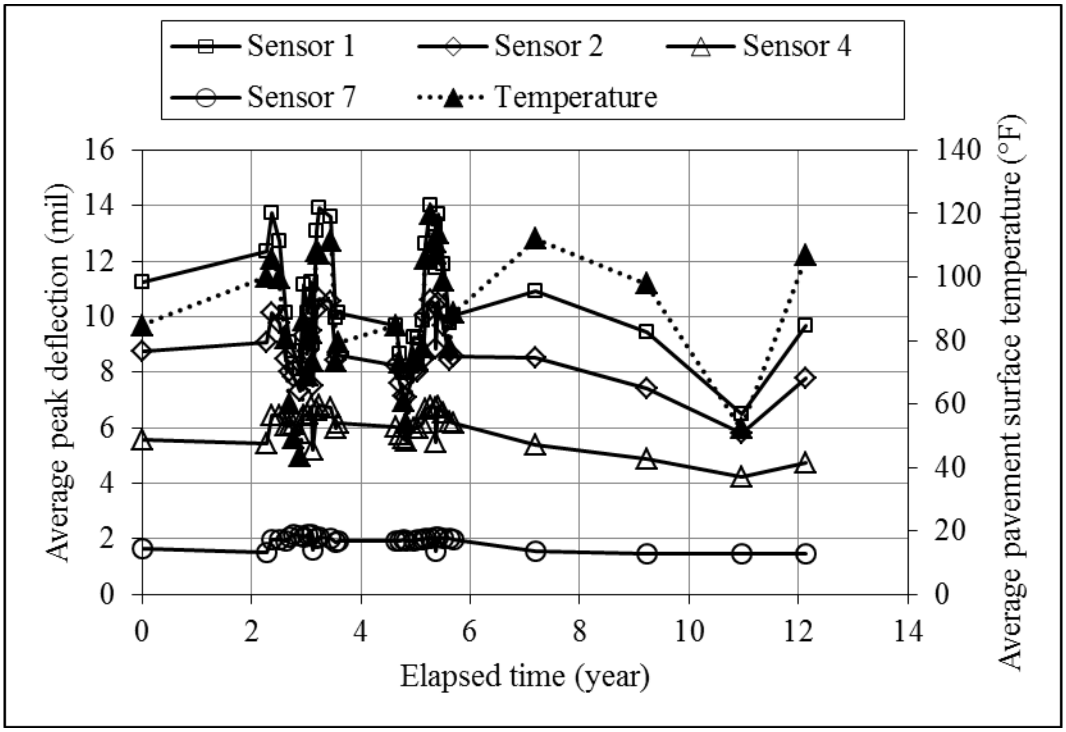
Table 111. Averages and overall average of the regression parameters of TAF (figure 72) for each climatic region.

Regression Term	Average Regression Value for Each Climatic Region				Overall Average
	WF	WNF	DF	DNF	
α_1	−1.94E−09	−1.27E−09	−8.70E−10	−1.63E−09	−1.43E−09
α_2	2.82E−07	2.18E−07	1.62E−07	2.49E−07	2.29E−07
α_3	6.73E−04	4.04E−04	3.23E−04	5.58E−04	4.84E−04
β_1	2.34E−07	1.39E−07	9.97E−08	1.75E−07	1.63E−07
β_2	−3.88E−05	−2.67E−05	−2.05E−05	−3.21E−05	−2.96E−05
β_3	−5.74E−02	−3.55E−02	−3.00E−02	−4.88E−02	−4.23E−02
γ_1	−8.35E−06	−4.24E−06	−3.71E−06	−5.71E−06	−5.52E−06
γ_2	1.67E−03	1.03E−03	8.69E−04	1.34E−03	1.22E−03
γ_3	2.19	1.65	1.90	1.85	1.88
δ_1	1.07E−04	4.61E−05	6.81E−05	6.73E−05	7.09E−05
δ_2	−2.54E−02	−1.32E−02	−1.82E−02	−1.77E−02	−1.83E−02

Flexible Pavement Deflection Versus Time

The average peak deflections, after applying temperature adjustment, were used to study the changes in deflection over time. It was envisioned that structural distresses, such as fatigue cracking, might be correlated to the pavement deflection. For example, fatigue cracking could propagate toward the pavement surface at a rate proportional to the rate of increase in pavement deflection. The pavement deflection was envisioned to decrease following construction as the pavement was further compacted by traffic, and then eventually began to increase as the pavement began to deteriorate. Note that this relationship was only anticipated for SPS sections, because GPS sections were in service before the commencement of data collection. The relationships between pavement deflection and time were studied using the following steps:

- **Step 1:** For each deflection sensor, the averages of the peak deflections measured before application of the first treatment were plotted as a function of time as shown in figure 79 (sensors 1, 2, 4, and 7 are shown). Note that for a few test sections, minor treatments such as skin patching were ignored when necessary to increase the available number of data points. The data in figure 79 indicates that the average measured peak deflections at each of sensors 1, 2, 4, and 7 were more or less consistent over time and fluctuated owing to pavement temperature and the moisture conditions at the time of testing. Note that fluctuation in the deflection generally mirrored fluctuation in pavement temperatures; lower temperature corresponded to lower deflection. The trend generally stayed consistent over time, and the deflection magnitude decreased with increasing distance from the load and decreasing temperatures.



°F = 1.8 × °C + 32.
1 mil = 25.4 microns.

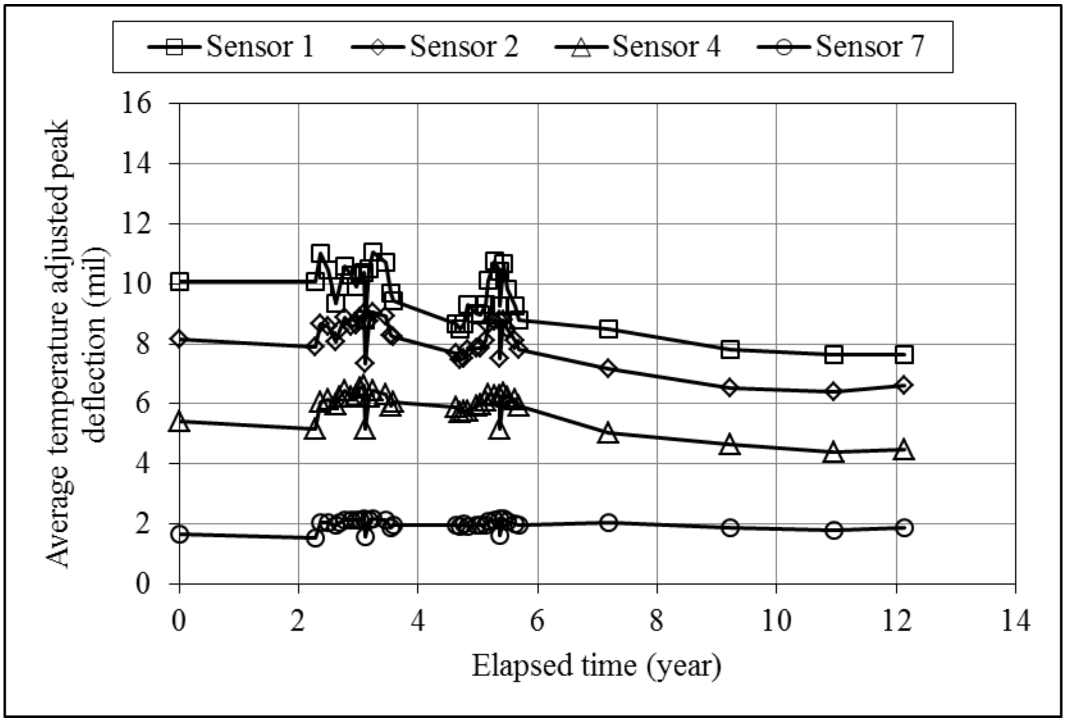
Figure 79. Graph. Average measured peak pavement deflection at sensors 1, 2, 4, and 7 versus time for SHRP test section 010101, F1.

- **Step 2:** Each deflection versus time plot (d_1) was visually inspected and assigned a trend description. The deflection was consistent over time, increasing over time, decreasing over time, or decreasing and then increasing over time. A summary of the results of this inspection is provided in table 112. Note that the majority of the SMP test sections exhibited more-or-less consistent deflection over time at sensor 7.

Table 112. Summary of deflection for sensor 1 versus time trend descriptions.

Trend Description	Measured Data		Temperature-Adjusted Data	
	No. of Test Sections	Percentage	No. of Test Sections	Percentage
Consistent	44	56	46	59
Increasing	24	31	25	32
Decreasing	3	4	3	4
Decreasing and then increasing	7	9	4	5

- **Step 3:** The measured deflection data were then temperature adjusted using the equation in figure 72 and plotted against time as shown in figure 80 for sensors 1, 2, 4, and 7, respectively. The data in figure 80 indicate that, during the 12-year period, the measured sensor 1 deflection decreased from about 9.84 to 7.87 mil (250 to about 200 microns) with some minor localized fluctuations. These fluctuations are the result of the pavement temperature and moisture conditions at the time of testing. The trend over time generally stayed consistent with distance from the applied load, while the deflection magnitude and variability decreased.

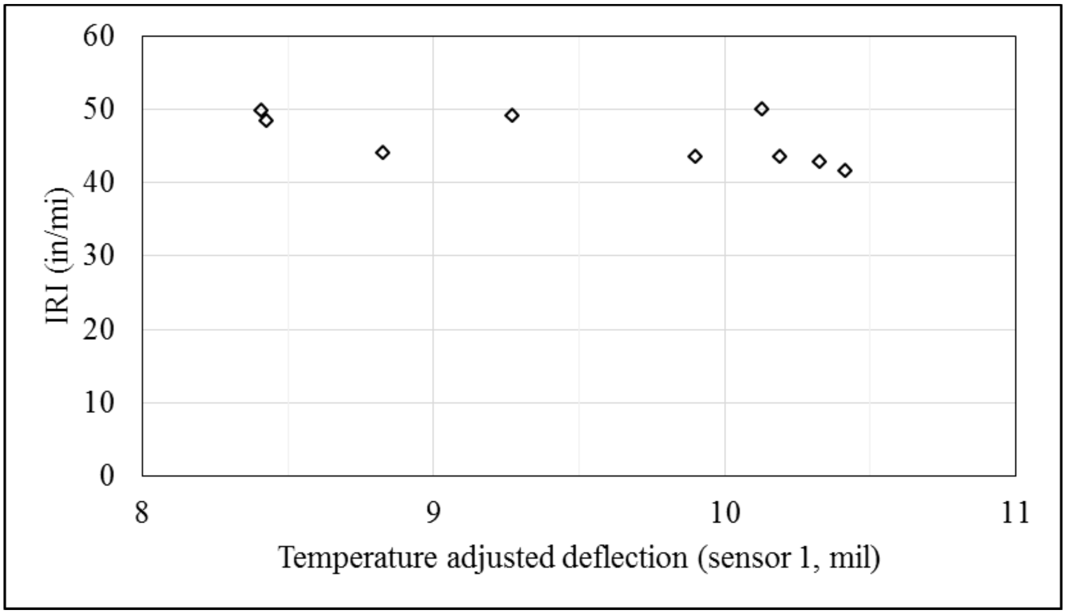


1 mil = 25.4 micron.

Figure 80. Graph. Temperature-adjusted peak deflection at sensors 1, 2, 4, and 7 versus time for SHRP test section 010101, F1.

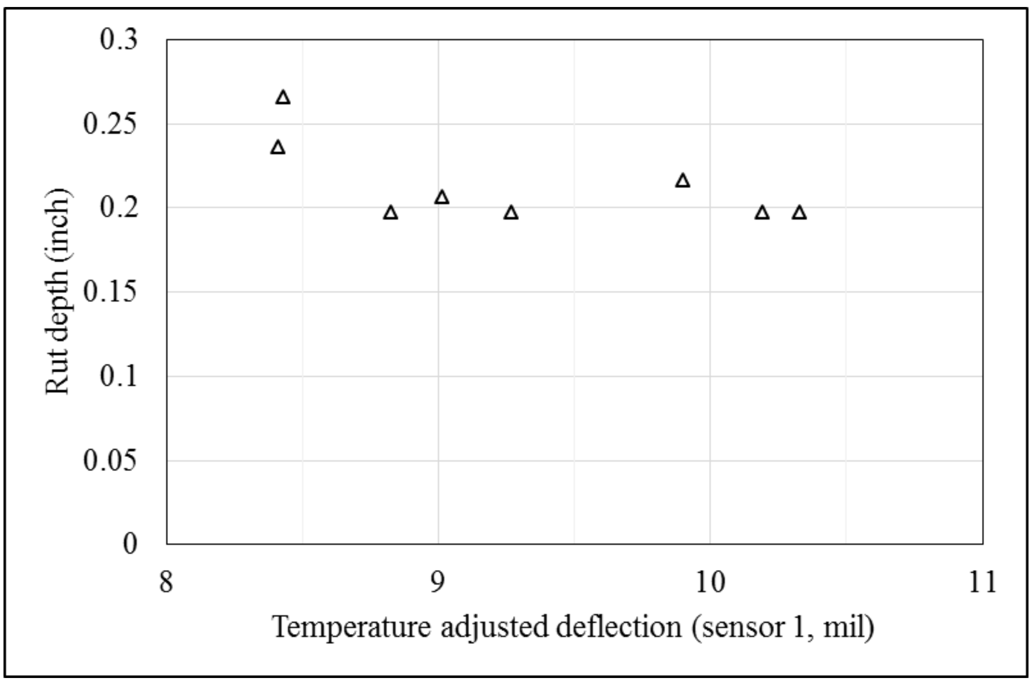
- **Step 4:** The temperature-adjusted deflection versus time plots were visually evaluated as in step 2, and the results summarized in table 112. The data in the table indicate that the majority of the time the deflection was more or less consistent over time, which indicates that no deterioration was occurring or that no relationship existed between deflection and distress. Only 37 percent of the data indicate increasing or decreasing and then increasing deflection over time.

- **Step 5:** Although the deflection data do not show a general trend over time, the data were further investigated by comparing deflection with the condition and distress data measured over the same period. Since deflection, IRI, rut, and cracking data were not collected at the same time, the data were grouped and taken as a calendar year average for comparison. Example of such comparison is shown in figure 81 through figure 85. The data in the figures indicate that no trend exists between deflection and pavement condition or distress.



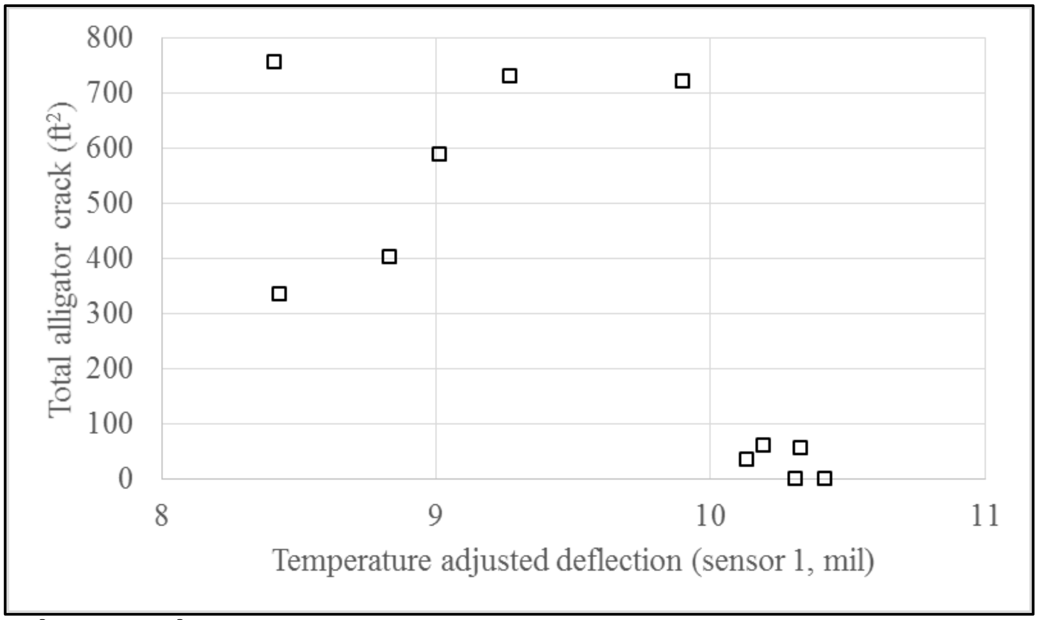
1 inch/mi = 0.0158 m/km.
1 mil = 25.4 microns.

Figure 81. Graph. Temperature-adjusted peak deflection at sensor 1 versus IRI for SHRP test section 010101, F1.



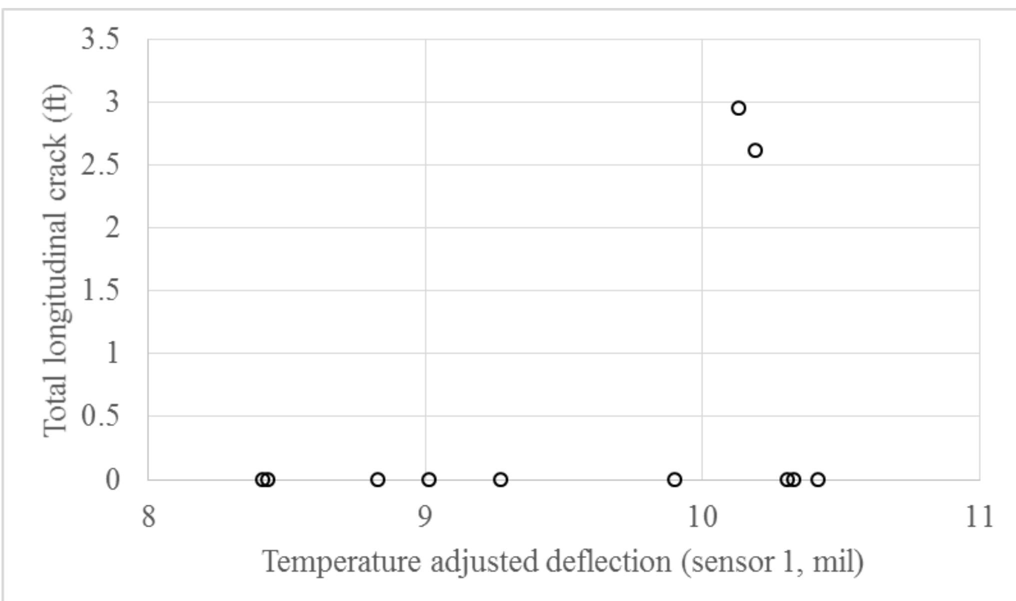
1 inch = 25.4 mm.
1 mil = 25.4 microns.

Figure 82. Graph. Temperature-adjusted peak deflection at sensor 1 versus rut depth for SHRP test section 010101, F1.



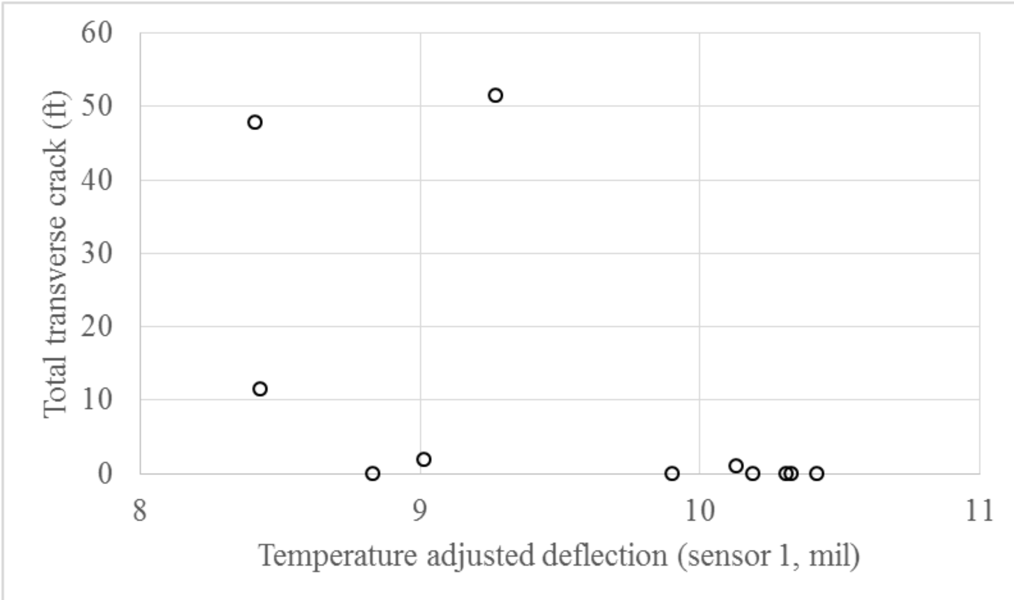
1 ft^2 = 0.0929 m^2.
1 mil = 25.4 microns.

Figure 83. Graph. Temperature-adjusted peak deflection at sensor 1 versus alligator cracking for SHRP test section 010101, F1.



1 ft = 0.305 m.
1 mil = 25.4 microns.

Figure 84. Graph. Temperature-adjusted peak deflection at sensor 1 versus longitudinal cracking for SHRP test section 010101, F1.



1 ft = 0.305 m.
1 mil = 25.4 microns.

Figure 85. Graph. Temperature-adjusted peak deflection at sensor 1 versus transverse cracking for SHRP test section 010101, F1.

Backcalculation of Pavement Layer Resilient Moduli Values

The measured and/or temperature-adjusted pavement deflection data, along with layer thickness and Poisson's ratio, can be used to backcalculate the pavement layer moduli (M_R) values. Such values are useful in the design of pavement rehabilitation treatments. To further develop and verify the temperature adjustment procedures described previously, the deflection data from a few test sections were used to backcalculate the layer M_R values. The data used were from SHRP test sections 010101, 081053, 271028, and 351112. A partial record of the inventory data for the test sections used in this analysis are listed in table 108.

The MICHBACK software package, developed at Michigan State University, was used in the analyses to backcalculate pavement layer M_R values. MICHBACK uses the Chevronx computer program (a five-layer elastic program) as the forward engine to calculate the pavement deflections for a given set of layer moduli, Poisson ratios, layer thicknesses, and load magnitude. The MICHBACK program uses a modified Newtonian algorithm to calculate a gradient matrix by incrementing the estimated layer modulus values and calculating the differences between the measured and the calculated pavement deflection in three consecutive cycles. When the convergence criteria (specified by the program user) are satisfied, the iteration process stops, and the final set of backcalculated layer moduli are recorded.[83] In this study, the following convergence criteria were used:

- **Modulus tolerance**: Maximum modulus tolerance (the difference between two successive backcalculated modulus values) of 1 percent.

- **Root mean square (RMS) error**: Maximum RMS error tolerance (the square root of the sum of squared errors between measured and calculated deflections) of 5 percent.

The pavement layers were assigned the following Poisson's ratio values:

- AC—0.35.
- Granular base—0.40.
- Roadbed soil—0.45.

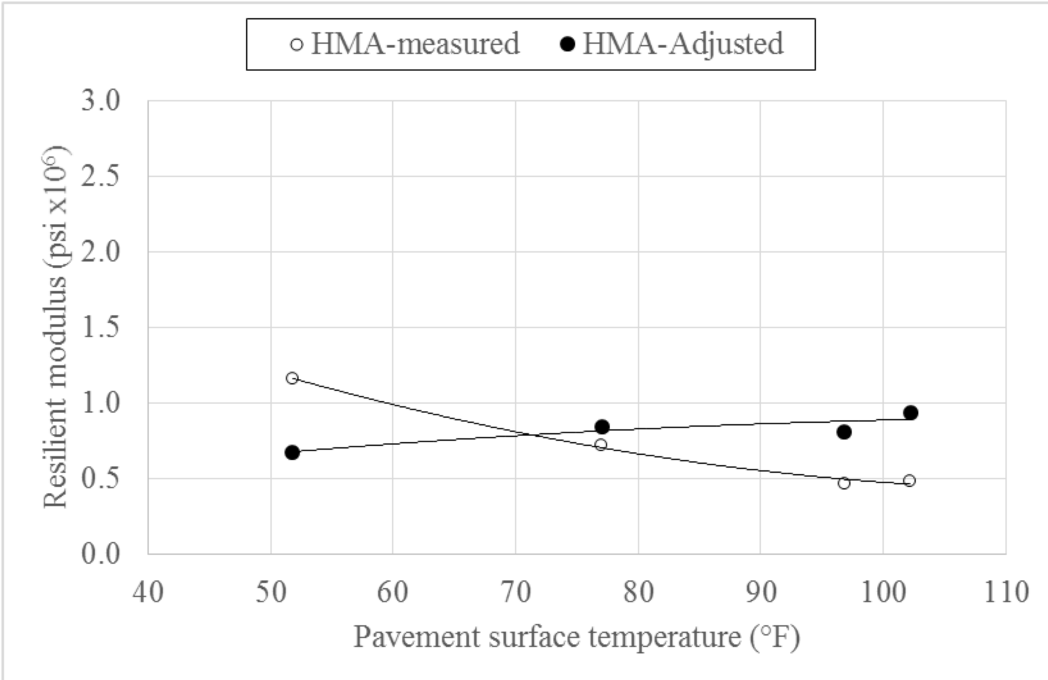
Note that the aggregate base and sand subbase layers were combined into a singular base layer. This was done to improve the accuracy of the backcalculation.

The results of the backcalculation are listed in table 113. Note that for each location, three or four tests were performed during a 1-day period. Also note that the first set of four tests was conducted at point location 114.3, the next four at point location 91.4, and the third four was the average for the entire test section. The first set of three tests was conducted at point location 30.5, the next three at point location 7.6, and the final three at point location 7.6. The pavement surface temperature varied throughout each day of testing by as much as 82 ºF (28 ºC). Hence, the AC layer M_R value varied throughout the day. Higher temperature corresponded to softer asphalt binder and lower M_R. There were also some changes in the base layer M_R values and minimal changes in the roadbed soil M_R values. The changes in AC layer M_R are shown as a function of pavement surface temperature for the analysis using both the measured and temperature-adjusted deflection data in figure 86 through figure 89. The data in the figures indicate that in general the AC layer M_R values decreased with increasing temperature based on the measured deflection, while the values based on the temperature-adjusted deflection were opposite. Note that the two curves generally cross near the reference temperature of 70 °F (21 ºC). This finding supports the use of the equation in figure 72 and indicates that the AC layer modulus values could be determined from the model of the backcalculated moduli values based on the measured or temperature-adjusted data by selecting the value at 70 °F (21 ºC).

Table 113. Backcalculated resilient modulus values.

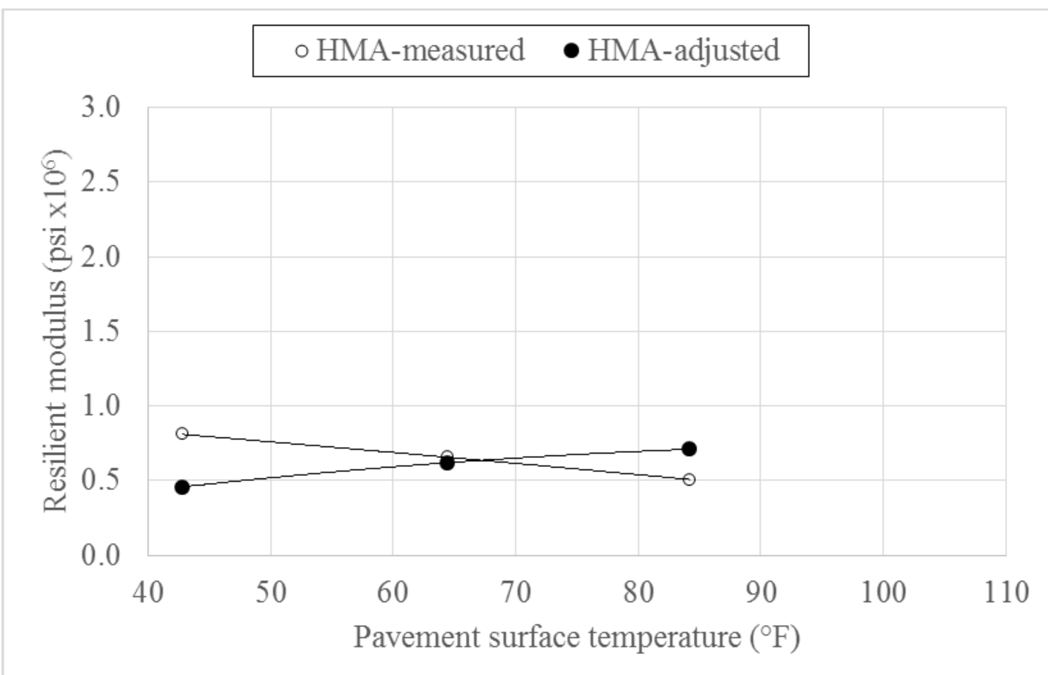
State Code	SHRP ID	FWD Test Date	FWD Test Time	Pavement Surface Temperature (°F)	Backcalculated Layer Resilient Modulus—Measured Deflection (psi)			Backcalculated Layer Resilient Modulus—Adjusted Deflection (psi)		
					Asphalt	Base	Roadbed Soil	Asphalt	Base	Roadbed Soil
1	0101	4/16/96	0832	52	1,162,303	9,176	19,771	674,661	16,317	18,823
1	0101	4/16/96	1121	77	718,996	9,819	18,931	844,193	8,525	19,248
1	0101	4/16/96	1402	102	487,634	11,556	18,635	941,488	7,592	19,340
1	0101	4/16/96	1639	97	466,884	12,809	19,059	812,734	9,834	19,532
1	0101	4/16/96	0821	50	1,191,564	53,493	22,261	656,820	56,902	22,102
1	0101	4/16/96	1112	79	641,012	65,940	20,930	906,343	54,878	21,040
1	0101	4/16/96	1350	104	455,836	58,919	21,264	1,132,931	49,501	21,107
1	0101	4/16/96	1630	99	379,298	82,785	21,602	895,810	78,689	21,423
1	0101	4/16/96	0836	53	1,098,468	22,168	19,160	639,126	30,920	18,815
1	0101	4/16/96	1126	78	698,266	21,903	18,821	889,003	17,587	19,068
1	0101	4/16/96	1407	102	439,646	27,870	18,610	954,010	19,785	18,789
1	0101	4/16/96	1643	96	407,830	32,042	18,916	811,745	24,301	19,119
8	1053	3/7/97	0954	43	809,038	25,551	20,518	457,612	21,733	20,496
8	1053	3/7/97	1121	64	656,069	25,579	21,976	619,150	24,196	22,198
8	1053	3/7/97	1246	84	506,896	26,722	22,196	709,939	28,053	22,219
27	1028	10/8/96	0822	50	2,092,102	—	25,441	1,343,882	—	25,956
27	1028	10/8/96	1031	64	1,908,807	—	25,231	1,689,933	—	25,415
27	1028	10/8/96	1219	73	1,862,257	—	25,621	1,984,829	—	25,539
35	1112	4/25/95	0754	52	2,626,520	102,303	42,397	1,668,134	84,612	42,021
35	1112	4/25/95	1017	70	2,045,288	81,978	40,578	2,266,073	63,815	40,914
35	1112	4/25/95	1219	86	1,647,345	47,891	40,152	2,571,180	38,301	40,846

— Indicates not applicable.
°F = 1.8 × °C + 32.
1 psi = 6.895 kPa.



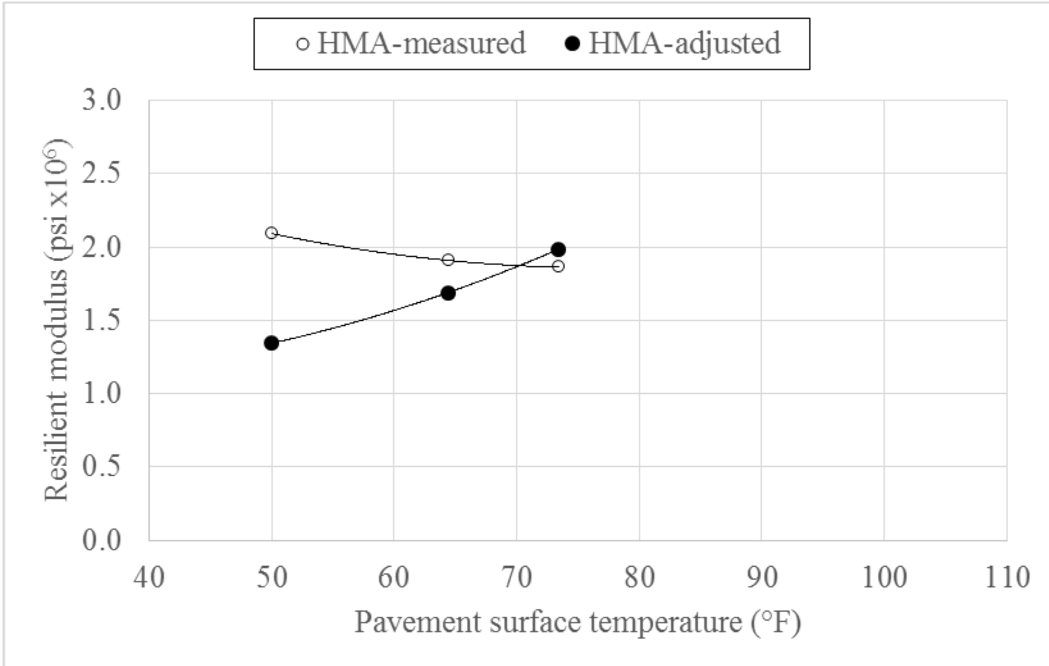
ºF = 1.8 × ºC + 32.
1 psi = 6.895 kPa.

Figure 86. Graph. AC layer M_R values versus pavement surface temperature for SHRP test section 010101.



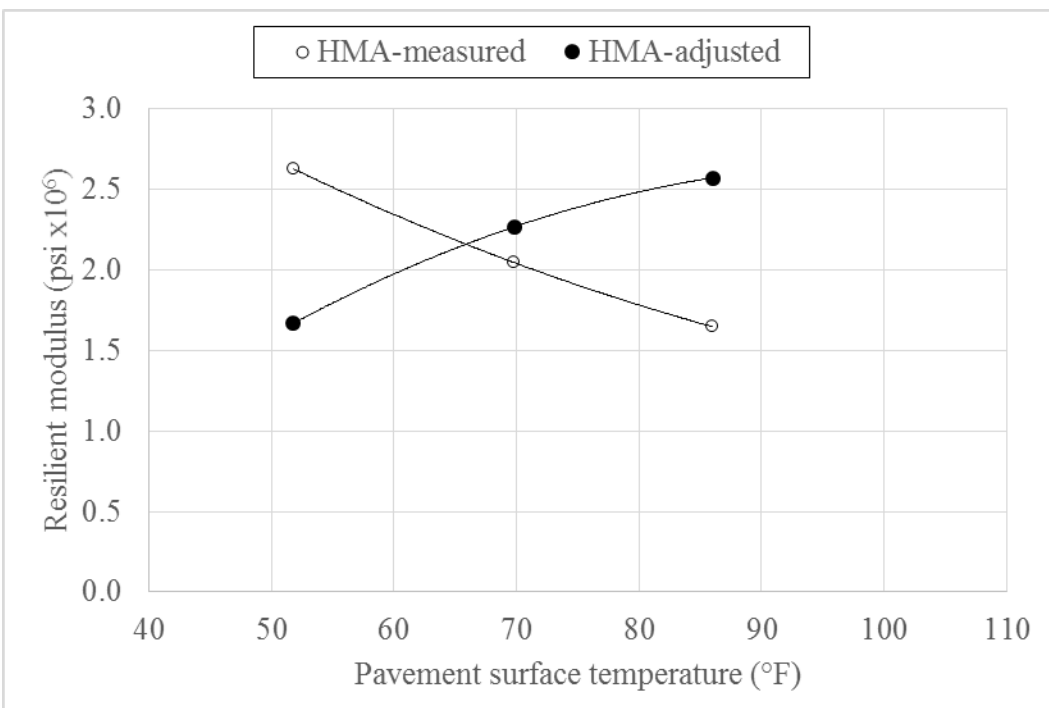
ºF = 1.8 × ºC + 32.
1 psi = 6.895 kPa

Figure 87. Graph. AC layer M_R values versus pavement surface temperature for SHRP test section 081053.



°F = 1.8 × °C + 32.
1 psi = 6.895 kPa.

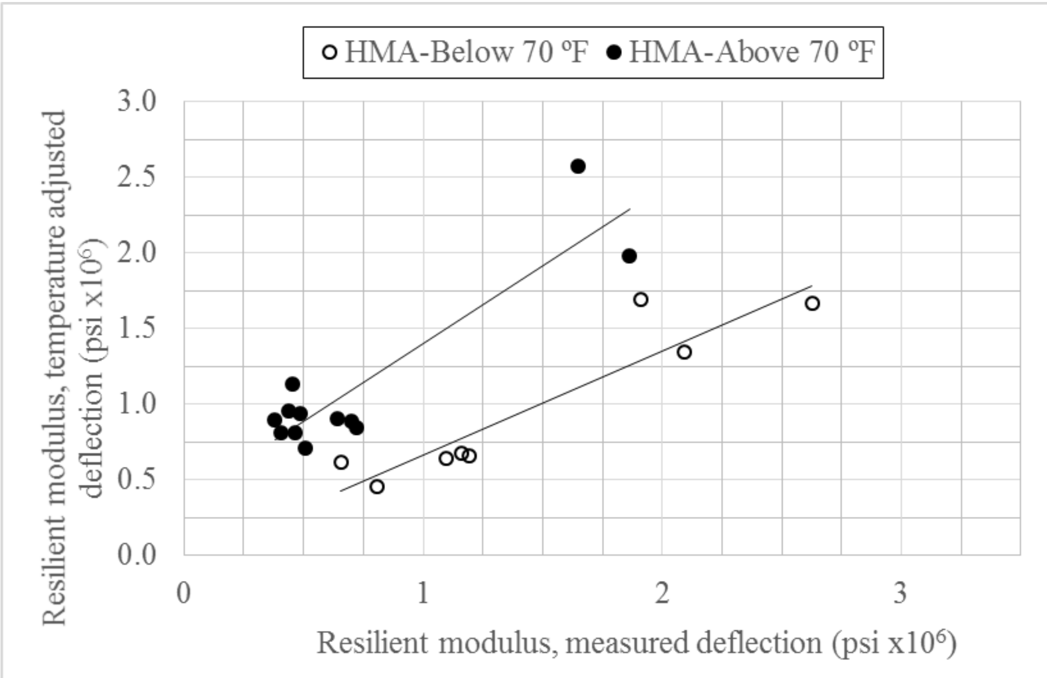
Figure 88. Graph. AC layer M_R values versus pavement surface temperature for SHRP test section 271028.



°F = 1.8 × °C + 32.
1 psi = 6.895 kPa.

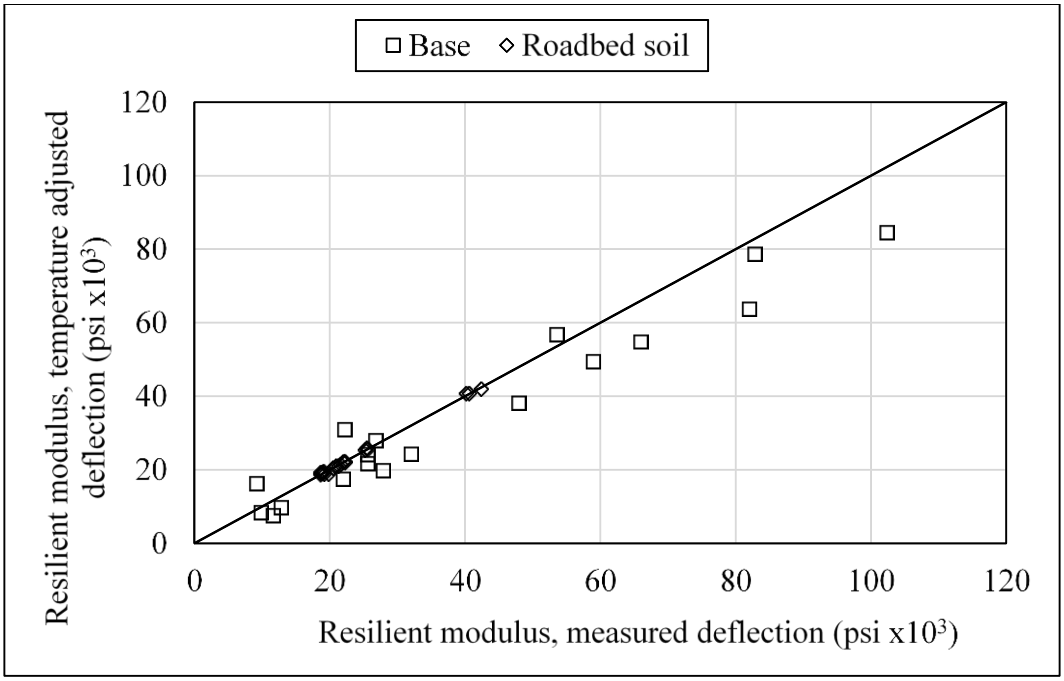
Figure 89. Graph. AC layer M_R values versus pavement surface temperature for SHRP test section 351112.

The backcalculated M_R values were further evaluated by plotting the results based on measured data versus those based on temperature-adjusted data, as shown in figure 90 and figure 91. The data in the figures indicate that minimal bias existed in the procedure for temperature adjustment across the various testing temperatures. Some data were reduced, and some data were increased following adjustment based on temperature (i.e., temperature adjustment did not consistently lead to increases or decreases in backcalculated M_R values).



ºF = 1.8 × ºC + 32.
1 psi = 6.895 kPa.

Figure 90. Graph. AC layer M_R values from measured and temperature-adjusted deflection data.



1 psi = 6.895 kPa.

Figure 91. Graph. Base and roadbed soil layer M_R values from measured and temperature-adjusted deflection data.

Although the backcalculation of pavement layer M_R values using temperature-adjusted pavement deflection was shown to work, many pavement engineers recommend performing backcalculation on measured data and then adjusting the results. This is because temperature adjustment in general is not exact, and it can introduce error. Such error can be magnified when the backcalculation routine is performed. Hence, making the adjustment at the end is preferred by many.

Rigid Pavement Deflection Versus Time

The measured rigid pavement deflections were evaluated over time using the following steps:

- **Step 1:** The deflection, treatment, and inventory data were downloaded from the LTPP Standard Data Release 28.0 for each of the SPS-2 test sections listed in table 114.

  The data were sorted based on the applied load and the various test locations within the lane. Drop height 2 simulated the application of a 9,000-lbf (40-kN) half single axle load and was selected for analyses. The tests performed at the middle lane at mid-panel location (J1) were selected for individual analyses. All data from other test loads and locations were not included in the analyses.

Table 114. LTPP SPS-2 partial inventory data.

SHRP ID	PCC Thickness (Inches)	Base Thickness (Inches)	Subbase Thickness (Inches)	Roadbed Soil Type	Most Recent Traffic (ESAL/Year)	Climatic Region
040213	7.9	5.8	—	Coarse grained soil: silty sand with gravel	1,444,000	DNF
040214	8.3	6.1	—	Coarse grained soil: silty sand with gravel	1,444,000	DNF
040215	11.0	6.3	—	Coarse grained soil: silty sand with gravel	1,454,000	DNF
040216	11.2	6.3	—	Coarse grained soil: silty sand with gravel	1,454,000	DNF
040217	8.1	6.1†	—	Coarse grained soil: silty sand with gravel	1,444,000	DNF
040218	8.3	6.2†	—	Coarse grained soil: clayey sand with gravel	1,444,000	DNF
040219	10.8	6.2†	—	Coarse grained soil: silty sand with gravel	1,453,000	DNF
040220	11.2	6.2†	—	Coarse grained soil: clayey sand with gravel	1,454,000	DNF
040221	8.1	8.4*	—	Coarse grained soil: silty sand with gravel	1,444,000	DNF
040222	8.6	8.2*	—	Coarse grained soil: clayey sand with gravel	1,445,000	DNF
040223	11.1	7.6*	—	Coarse grained soil: silty sand with gravel	1,453,000	DNF
040224	10.6	8.2*	—	Coarse grained soil: clayey sand with gravel	1,452,000	DNF
040262	8.1	6.1	—	Coarse grained soil: silty sand with gravel	1,444,000	DNF
040263	8.2	8.3*	—	Coarse grained soil: clayey sand with gravel	1,444,000	DNF
040264	11.5	8.2*	—	Coarse grained soil: clayey sand with gravel	1,454,000	DNF
040265	10.8	6.8	—	Coarse grained soil: clayey sand with gravel	1,453,000	DNF
040266	12.3	3.9†	—	Coarse grained soil: clayey sand with gravel	1,455,000	DNF
040267	11.3	3.9†	—	Coarse grained soil: clayey sand with gravel	1,454,000	DNF
040268	8.5	3.8†	—	Coarse grained soil: clayey sand with gravel	1,445,000	DNF
080213	8.6	5.9	—	Fine-grained soil: clay with sand	463,000	DF
080214	8.4	5.9	—	Coarse-grained soil: clayey sand with gravel	462,000	DF
080215	11.5	6.0	—	Fine-grained soil: sandy clay	470,000	DF
080216	11.9	5.9	—	Coarse-grained soil: poorly graded sand with silt	Not reported	DF
080217	8.6	6.7	—	Fine-grained soil: sandy lean clay	463,000	DF
080218	7.6	6.2†	—	Fine-grained soil: clay	459,000	DF

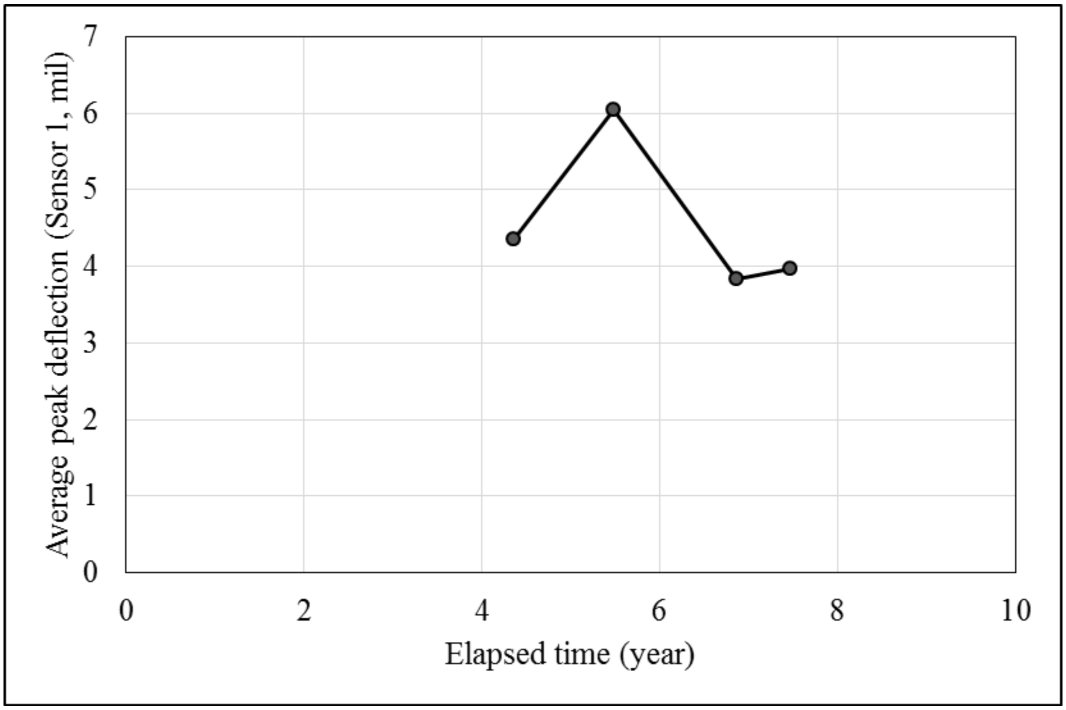
SHRP ID	PCC Thickness (Inches)	Base Thickness (Inches)	Subbase Thickness (Inches)	Roadbed Soil Type	Most Recent Traffic (ESAL/Year)	Climatic Region
080219	9.9	6.1†	—	Coarse-grained soil: clayey sand with gravel	467,000	DF
080220	11.2	6.2†	—	Fine-grained soil: clay	469,000	DF
080221	8.3	8.0*	—	Fine-grained soil: sandy lean clay	461,000	DF
080222	8.5	8.5*	—	Fine-grained soil: sandy clay	462,000	DF
080223	11.7	8.9*	—	Coarse-grained soil: well graded sand with silt	470,000	DF
080224	11.6	7.7*	—	Coarse-grained soil: clayey sand	470,000	DF
080259	11.9	—	—	Coarse-grained soil: clayey sand	470,000	DF
100201	8.3	6.2	34.0	Coarse-grained soil: silty sand	368,000	WNF
100202	8.8	6.2	22.0	Coarse-grained soil: well graded sand with silt	374,000	WNF
100203	11.7	6.1	14.0	Coarse-grained soil: silty sand	396,000	WNF
100204	11.0	6.3	12.0	Coarse-grained soil: well graded sand with silt	392,000	WNF
100205	9.2	5.5†	30.0	Coarse-grained soil: silty sand	378,000	WNF
100206	8.9	6.1†	42.0	Coarse-grained soil: well graded sand with silt	375,000	WNF
100207	11.3	6.9†	14.1	Coarse-grained soil: silty sand	394,000	WNF
100208	12.1	6.0†	30.0	Coarse-grained soil: sand	397,000	WNF
100209	8.2	8.0*	24.0	Coarse grained soil: clayey sand	367,000	WNF
100210	8.3	8.2*	36.0	Coarse-grained soil: silty sand	368,000	WNF
100211	11.8	7.5*	30.0	Coarse-grained soil: clayey sand	396,000	WNF
100212	12.4	7.4	30.0	Coarse-grained soil: silty sand	398,000	WNF
100259	10.2	7.9	42.0	Coarse-grained soil: well graded sand with silt	387,000	WNF
100260	10.2	7.8	42.0	Coarse-grained soil: silty sand	387,000	WNF
260213	8.3	6.1	18.5	Fine-grained soil: silty clay	2,042,000	WF
260214	8.8	5.8	18.5	Fine-grained soil: silty clay	1,481,000	WF
260215	11.1	6.2	15.5	Fine-grained soil: silty clay	2,129,000	WF
260216	11.3	5.9	15.5	Fine-grained soil: silty clay	1,550,000	WF
260217	8.4	6.1†	18.5	Fine-grained soil: silty clay	2,047,000	WF
260218	7.3	6.3†	18.5	Fine-grained soil: silty clay	1,895,000	WF

SHRP ID	PCC Thickness (Inches)	Base Thickness (Inches)	Subbase Thickness (Inches)	Roadbed Soil Type	Most Recent Traffic (ESAL/Year)	Climatic Region
260219	11.3	5.9†	15.5	Fine-grained soil: silty clay	1,550,000	WF
260220	11.2	5.8†	15.5	Fine-grained soil: silty clay	1,549,000	WF
260221	8.1	8.6*	16.5	Fine-grained soil: silty clay	1,513,000	WF
260222	8.3	8.4*	16.5	Fine-grained soil: silty clay	1,615,000	WF
260223	11.0	8.4*	13.5	Fine-grained soil: sandy clay	1,548,000	WF
260224	11.1	8.3*	13.5	Fine-grained soil: sandy clay	1,549,000	WF
260259	11.3	8.0*	18.0	Fine-grained soil: silty clay	1,550,000	WF

— Indicates no base layer was present.
*Upper portion of base was treated.
†Treated base.
1 inch = 25.4 mm.

- **Step 2:** On any particular test day, a series of four tests were conducted at drop height 2 for each of about 10 locations along the test section. The deflection tests conducted on a test section in each specific testing day were averaged to simulate the data that would be collected by a State transportation department (one test per section). Deflection data collected after the application of a treatment were not included in the analyses.

- **Step 3:** The measured deflection data were plotted against time as shown in figure 92 for sensor 1. The data in the figure indicate that the deflection increased and then decreased, and overall decreased over the 4-year period of data collection, with values of about 4.33 to 5.91 to 3.94 inches (110 to 150 to 100 microns). Localized fluctuations occurred due to moisture conditions at the time of testing and instrument and measurement variability.



1 mil = 25.4 microns.

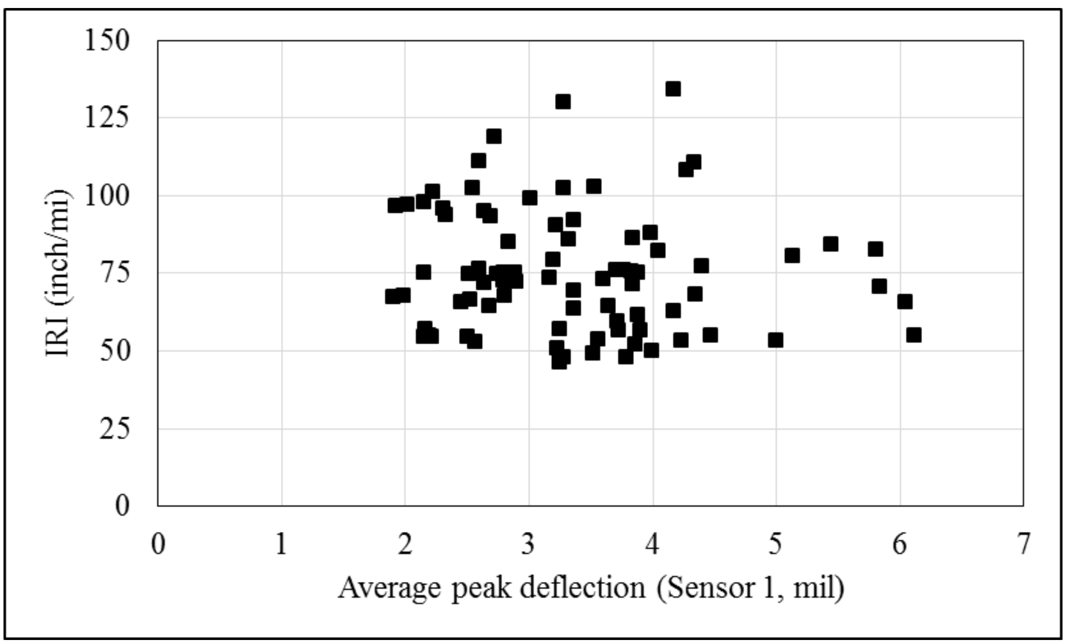
Figure 92. Graph. Measured peak deflection at sensor 1 versus time for SHRP test section 100201, J1.

- **Step 4:** The measured deflection versus time plots were visually evaluated, and the results are summarized in table 115. The data in the table indicate that 45 percent of the test sections showed increasing deflection over time, 27 percent showed more or less consistent deflection, and 27 percent showed decreasing deflection over time. Consequently, the time-dependent deflection measured over a 4-year period cannot be used as an indication of pavement deterioration or pavement distress.

Table 115. Summary of deflection for sensor 1 versus time trend descriptions for LTPP SPS-2.

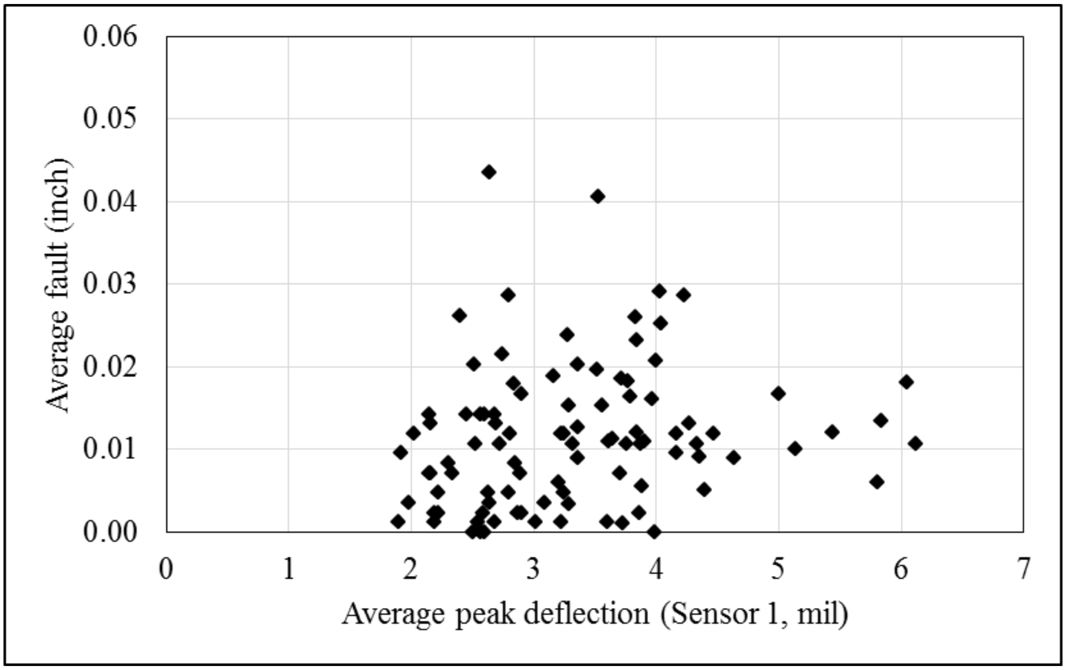
State	Trend Description					
	Increasing		Consistent		Decreasing	
	No. of Test Sections	Percentage	No. of Test Sections	Percentage	No. of Test Sections	Percentage
Arizona	10	53	5	26	4	21
Colorado	2	15	6	46	5	38
Delaware	5	42	5	42	2	17
Michigan	8	57	2	14	4	29
Total sections and average percent	25	45	15	27	15	27

- **Step 5:** Although the deflection data did not show a consistent trend over time, the data were further investigated by comparing the deflection with the pavement condition and distress data measured over the same time period. Because deflection, IRI, faulting, and cracking data were not collected at the same time, the data were grouped and taken as a calendar year average for comparison. Examples of such comparisons are shown in figure 93 through figure 96. The data in the figures indicate there was no trend in relationship between deflection and pavement condition or distress. Similar results were found for the other test sections.



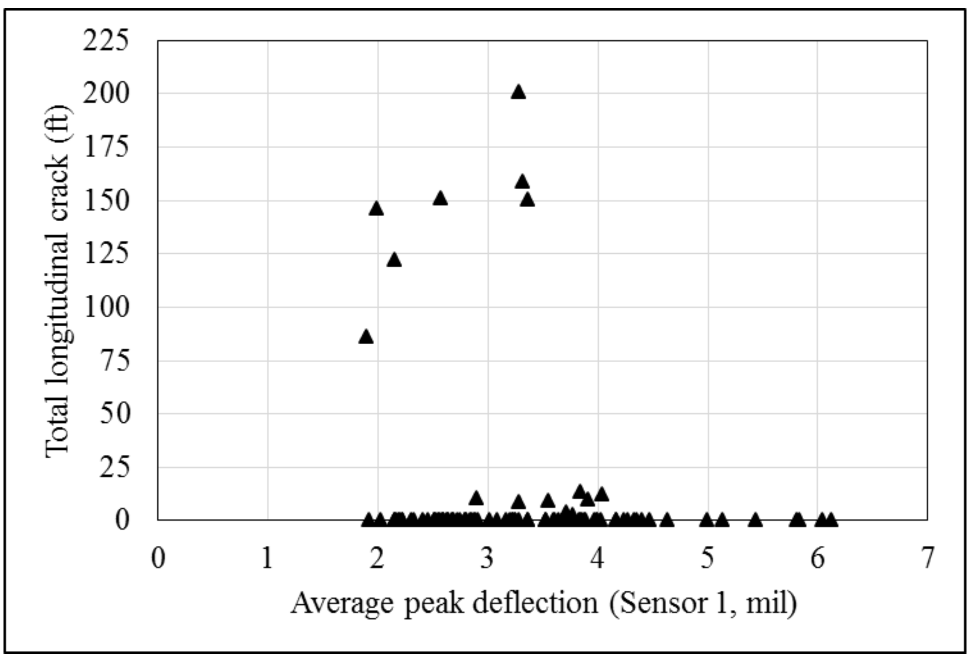
1 inch/mi = 0.0158 m/km.
1 mil = 25.4 microns.

Figure 93. Graph. Measured peak deflection at sensor 1 versus IRI for SHRP test section 100201, J1.



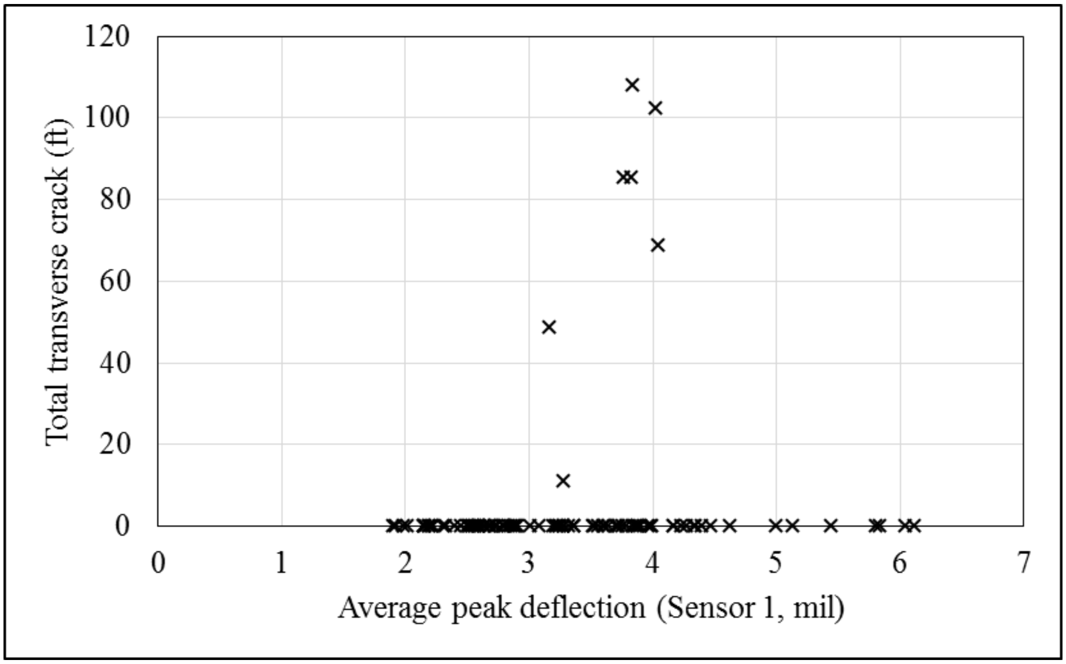
1 inch = 25.4 mm.
1 mil = 25.4 microns.

Figure 94. Graph. Measured peak deflection at sensor 1 versus faulting for SHRP test section 100201, J1.



1 ft = 0.305 m.
1 mil = 25.4 microns.

Figure 95. Graph. Measured peak deflection at sensor 1 versus longitudinal cracking for SHRP test section 100201, J1.



1 ft = 0.305 m.
1 mil = 25.4 microns.

Figure 96. Graph. Measured peak deflection at sensor 1 versus transverse cracking for SHRP test section 100201, J1.

The pavement deflection data could be used to estimate the pavement response to load and for the design of pavement treatments, etc. The LTPP rigid pavement deflection data did not support the use of deflection as an indicator of impending surface distress. Therefore, deflection could not be used in this study as a component of RFP or RSP.

Rigid Pavement LTE Versus Time

The LTEs calculated based on FWD testing were evaluated over time using the following steps:

- **Step 1:** The LTE, treatment, and inventory data were downloaded from the LTPP Standard Data Release 28.0 for each of the SPS-2 test sections listed in table 114.

- **Step 2:** The data were sorted based on the applied load and the various test locations within the lane. Drop height 2 simulated the application of a 9,000-lbf (40-kN) half single axle load and was selected for analyses. The tests performed in the outer wheelpath at joint approach locations (J4) were selected for individual analyses. All data from other test loads and locations were not included in the analyses.

- **Step 3:** On any particular test day, a series of four tests were conducted at drop height 2 for each of about 10 locations along the test section. The various LTE tests conducted on a test section in each specific testing day were averaged to simulate the data that would be collected by a State transportation department (only one test per section). LTE data collected after the application of a treatment were not included in the analyses.

- **Step 4:** The LTE data were then plotted as a function of time. An example of such a plot is provided in figure 97. The data in the figure indicate that the LTE generally increased over the 4-year period of data collection, ranging from about 88 to 98 percent. Some localized fluctuations occurred as a result of the moisture conditions at the time of testing along with instrument and measurement variability.

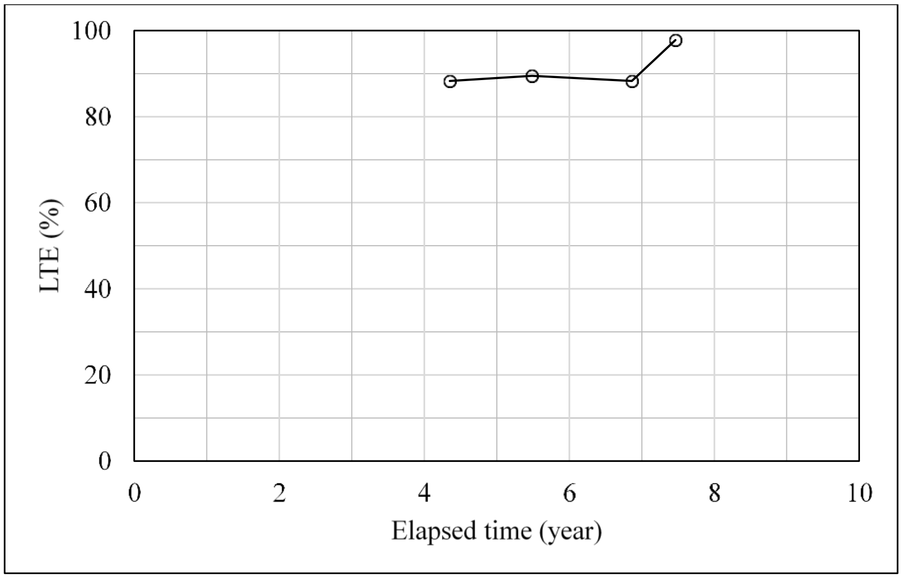


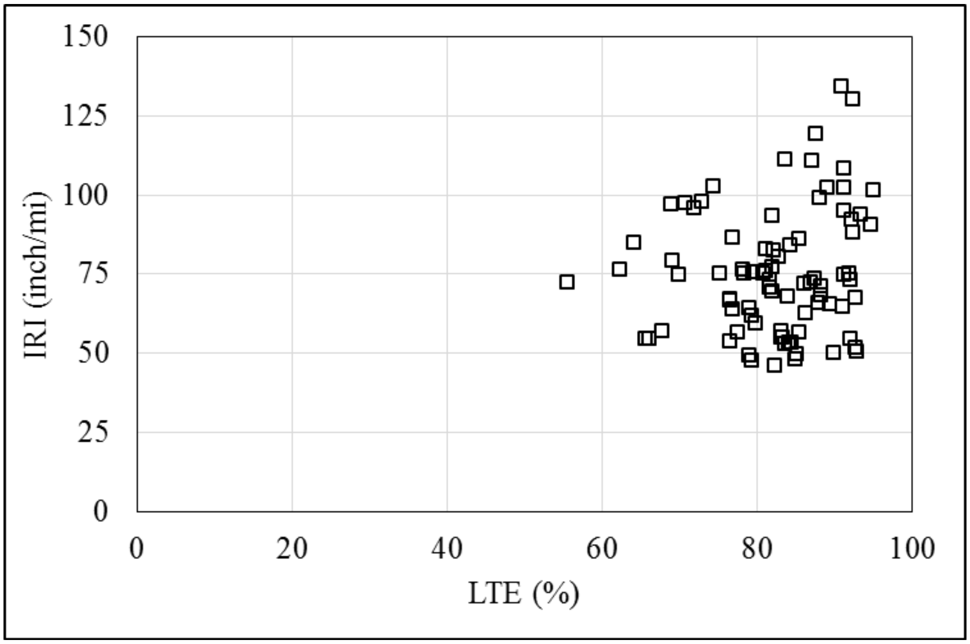
Figure 97. Graph. LTE versus time for SHRP test section 100201, J4.

- **Step 5:** The LTE versus time plots were visually evaluated and the results are summarized in table 116. The data in the table indicate that 21 percent of the test sections showed increases in the LTE over time, 30 percent showed consistent LTE, and 48 percent showed decreases in the LTE over time. Once again, statistically speaking, the LTPP LTE data could not be used as an indicator of pavement distress or deterioration.

Table 116. Summary of LTE versus time trend descriptions.

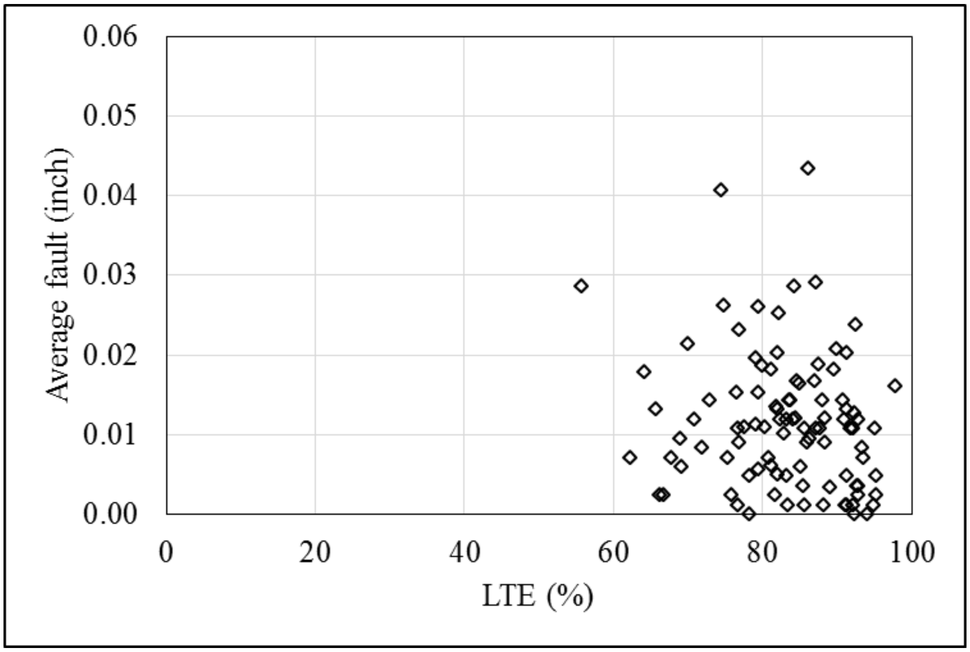
State	Trend Description					
	Increasing		Consistent		Decreasing	
	No. of Test Sections	Percentage	No. of Test Sections	Percentage	No. of Test Sections	Percentage
Arizona	2	11	3	17	13	72
Colorado	3	23	7	54	3	23
Delaware	3	25	4	33	5	42
Michigan	4	31	3	23	6	46
Total sections and average percent	12	21	17	30	27	48

- **Step 6:** Although the LTE data did not show a consistent trend over time, the data were further investigated by comparing LTE with the condition and distress data measured over the same time period. Because the LTE, IRI, faulting, and cracking data were not collected at the same time, the data were grouped and taken as a calendar year average for comparison purposes. Examples of such comparisons are shown in figure 98 through figure 101. The data in the figures indicate that there was no trend in the relationship between LTE and pavement condition or distress. Similar results were found for the other test sections.



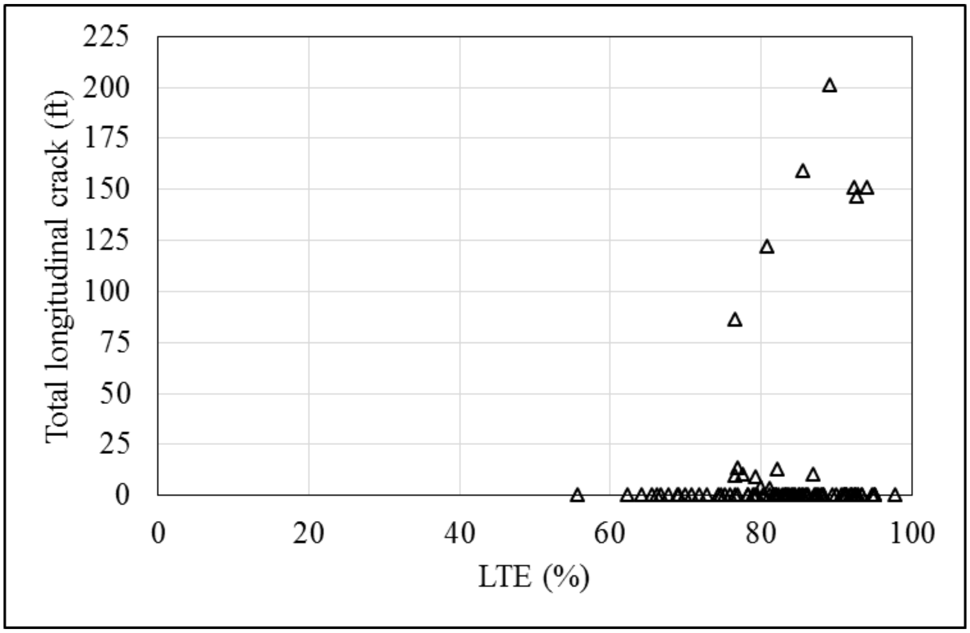
1 inch/mi = 0.0158 m/km.

Figure 98. Graph. LTE versus IRI for SHRP test section 100201, J4.



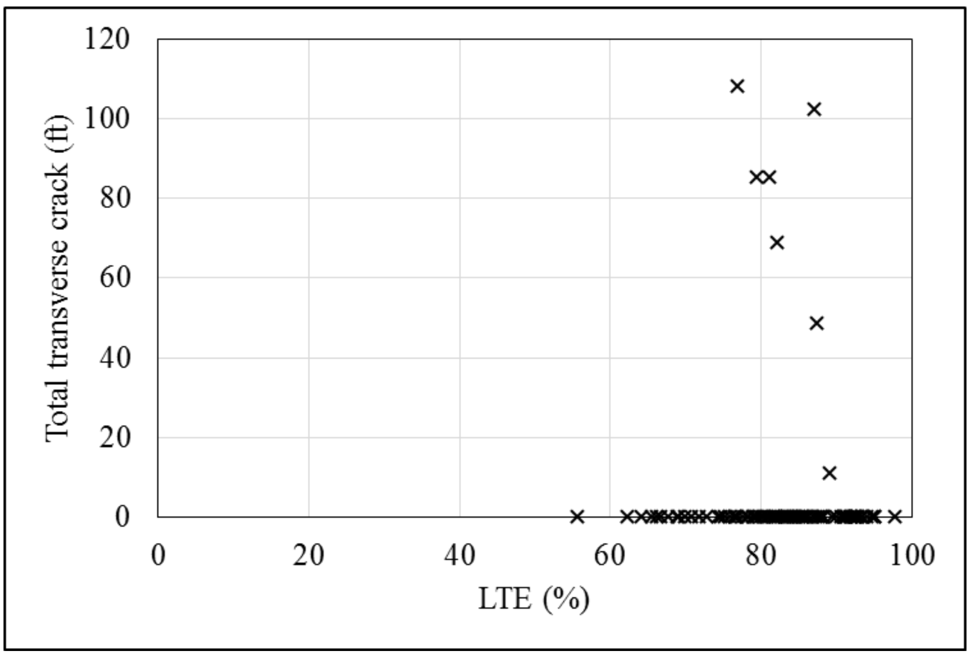
1 inch = 25.4 mm.

Figure 99. Graph. LTE versus faulting for SHRP test section 100201, J4.



1 ft = 0.305 m.

Figure 100. Graph. LTE versus longitudinal cracking for SHRP test section 100201, J4.



1 ft = 0.305 m.

Figure 101. Graph. LTE versus transverse cracking for SHRP test section 100201, J4.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This chapter presented the results of the analyses of the measured flexible and rigid pavement deflections and the LTE data. The main objective of the analyses was to determine whether pavement deflections and LTE data could be incorporated into the RFP and RSP algorithms.

It was envisioned that measured pavement deflections and/or LTE data would be correlated with pavement condition or distress, and hence, they could be used as an early warning of pending deterioration before it became visible on the pavement surface.

For flexible pavements, the measured pavement deflections were a function of the pavement temperatures. Therefore, the accuracy of the existing temperature adjustment methods were reviewed and scrutinized using the measured LTPP deflection data. Based on the results, a new global temperature adjustment methodology was developed that could be applied to all deflection sensors and all climatic regions.

Based on the results of the analyses, the following conclusions and recommendations were drawn:

- The LTPP deflection data collected for the SMP test sections did not support the use of the AI deflection temperature adjustment procedure. The equation in figure 72 was developed to more accurately adjust the measured deflection based on the asphalt surface temperature. It is recommended that the equation in figure 72 be used to adjust the measured flexible pavement deflections based on the measured pavement surface temperature.

- The LTPP deflection data collected for the SMP test sections did not support the inclusion of the deflection data in the RFP and RSP algorithms. It is recommended that the deflection data for flexible pavement be excluded from the RFP and RSP algorithms for the LTPP data.

- The LTPP deflection data collected for the SMP test sections did not correlate with the measured pavement condition and distress data. It is recommended that the flexible pavement deflection data not be used as an indication of impending pavement distress.

- The backcalculated AC layer modulus values of the selected LTPP test sections were dependent on the pavement surface temperature. To estimate the AC modulus at the standard temperature of 21 ºC, two procedures are recommended:

    - Adjust the measured deflection data to 21 ºC using the equation in figure 72 and then backcalculate the layer moduli.

    - During 1 day, conduct a minimum of three FWD tests and measure the pavement deflection at various times. Two FWD tests should be conducted at pavement temperatures below or above 70 °F (21 ºC) and the third test on the other side of 70 °F (21 ºC). Backcalculate the AC modulus value for each of the three FWD tests. Plot the modulus as a function of the measured pavement temperature. From the graph, estimate the AC modulus at 70 °F (21 ºC).

- The LTPP deflection and LTE data collected for the SPS-2 test sections in Arizona, Colorado, Delaware, and Michigan did not support modeling of the data as a function of time. It is recommended that the deflection and LTE data for rigid pavement be excluded from the RFP and RSP algorithms.

- The LTPP deflection and LTE data collected for the SPS-2 test sections in Arizona, Colorado, Delaware, and Michigan did not support the correlation of deflection or LTE data with the measured pavement condition and distress data. It is recommended that the deflection and LTE data for rigid pavement not be used as indication of impending distress.

- It is further recommended that, for new LTPP experiments, the deflection data should be collected over a longer period of time (such as 10 to 20 years), and deflection data should be measured before and after treatments are applied.

- It is recommended that further research should be conducted to allow the incorporation of deflection data into the RFP and RSP algorithms. An early indicator of impending distress could prove to be very helpful to State transportation departments to allow them to treat the pavement before surface distress manifestation. In this study, although no relationships could be established based on the LTPP deflection data, future research could uncover such relationships.

CHAPTER 8. STATE DATA ANALYSES

BACKGROUND

Results of the analyses of the LTPP time-series pavement condition and distress data measured along flexible and rigid pavement test sections were presented and discussed in previous chapters. The results are presented based on treatment type, climatic regions, and various other factors. This chapter addresses the similarities and differences between the LTPP data and the pavement condition and distress data measured by Colorado, Louisiana, and Washington along various pavement segments of their respective pavement networks. The following differences between the LTPP and the State data were observed:

- The LTPP data were measured along a 500-ft (152.4-m)-long test and control sections located throughout the United States and Canada, whereas the State data were measured along 0.5-mi (0.8-km)-long to 8-mi (13-km)-long pavement projects and stored in the databases for each 0.1-mi (0.16-km)-long pavement segment along the project. Thus, the data for an 8-mi (13-km)-long pavement project were stored in 80 different fields, 1 field per 0.1 mi (0.16 km).

- The units of measurement used by the LTPP Program were not the same as those used by the State transportation departments. For example, the LTPP unit of measurement for IRI was m/km, while it was inches/mi for the States. Therefore, before the analyses of the State data, the data were converted to the same units as the LTPP data.

- The pavement condition and distress along one single LTPP test section represented one single data point in time, whereas for each pavement project, the State data contained as many data points at one time as the number of 0.1-mi (0.16-km)-long pavement segments along the project. For example, the data from one survey of an 8-mi (13-km)-long pavement project was equivalent to 80 LTPP test sections.

- The State transportation departments' databases were based on their distress identification definitions and procedures, which may or may not have been compatible with the *Distress Identification Manual for the Long-Term Pavement Performance Program*.[80]

- The State transportation department's databases lacked details of the types, classifications, and properties of the pavement layers and roadbed soils.

The LTPP data contained the pavement conditions and distresses of flexible, rigid and composite pavements test and control sections. Although the data from CDOT, LADOTD, and WSDOT contained the same data, the number of rigid and composite pavement projects that received treatments for which the database contained three or more data points was very limited. Hence, it was decided to limit the comparison to flexible pavement sections only.

The research team developed a step-by-step procedure to mine, handle, organize, and analyze the data based on one of the objectives of this study. This objective was to compare the results of the analyses of the LTPP and State data and to determine whether the methodologies used in the analyses of the LTPP data applied equally to the State data. Therefore, the pavement condition and distress data for the pavement networks of CDOT, LADOTD, and WSDOT were requested, received, organized, and subjected to the same types of analyses as the LTPP data using the step-by-step procedure detailed in the next section.

ANALYSIS STEPS

This section presents the steps of the procedure used in the analyses of the State. These steps are similar to those used in the analyses of the LTPP data. The difference is that the LTPP test sections were analyzed individually. On the other hand, a State pavement project consisted of many 0.1-mi (0.16-km)-long pavement segments where the pavement condition and distress varied substantially along the project. Although each 0.1-mi (0.16-km)-long pavement segment along a given pavement project was analyzed individually, results of the analyses of each project in a State transportation department that received the same treatment were grouped into the five-CS system based on the RFPs and RSPs of each 0.1-mi (0.16-km) segment. The system was developed in this study and presented in chapter 3 (Pavement Condition Classification) of this report. Finally, the benefits of a given treatment type were calculated as the weighted average benefits of each 0.1-mi (0.16-km)-long pavement segment in each State using RFP or RSP, CFP or CSP, and FCROP or SCROP. For the LTPP data, the benefits were calculated using the same parameters based on the weighted average benefits of each test section that received the same treatment type and located in any of the four climatic regions. The analyses procedure used the following steps:

- **Step 1**: The pavement condition and distress data were converted into Microsoft® Excel spreadsheet format and separated per pavement type.

- **Step 2**: For each pavement network, the treatment data were searched, and each 0.1-mi (0.16-km)-long segment of several pavement projects that received one of the following treatments were identified, copied, and stored in a separate Microsoft® Excel datasheet:

    - Thin overlay (≤ 2.5 inches (63.6 mm)).

    - Thick overlay (> 2.5 inches (63.6 mm)).

    - Thin mill and fill (≤ 2.5 inches (63.6 mm)).

    - Thick mill and fill (> 2.5 inches (63.6 mm)).

    - Single chip seal.

- **Step 3**: The pavement condition and distress data for each identified 0.1-mi (0.16-km)-long pavement segment were examined to determine whether the segment had a minimum of three time-series data that could be modeled using the proper mathematical function. Those segments that did not pass the test were not included in the analyses. Table 117 summarizes the number of 0.1-mi (0.16-km)-long pavement segments that were accepted for analyses for each treatment type in each State. The table also lists the number of LTPP test sections that received similar treatment type and were analyzed. The results of these LTPP test sections were compared with those of the 0.1-mi (0.16-km)-long) pavement segments.

Table 117. Number of 0.1-mi (0.16-km)-long pavement segments and LTPP test sections for each treatment type available for analyses.

Treatment Type	Data Source	Number of 0.1-Mi (0.16-Km)-long Pavement Segments and LTPP Test Sections Available for Analyses: IRI	Rut Depth	Alligator Cracking	Longitudinal Cracking	Transverse Cracking
Thin overlay	Washington	349	709	1,746	1,000	1,538
	Colorado	94	126	128	129	70
	Louisiana	219	224	202	71	134
	SPS-3 and -5	36	35	34	40	37
	GPS-5	25	19	7	7	13
Thick overlay	Washington	10	122	403	310	220
	Colorado	ND	ND	ND	ND	ND
	Louisiana	1,416	1,242	1,199	595	984
	GPS-5	14	15	10	15	13
	GPS-6	15	13	5	2	6
Thin mill and fill	Washington	123	701	886	357	633
	Colorado	28	74	49	38	24
	Louisiana	163	191	146	80	135
	GPS-5	13	13	13	17	16
	GPS-6	27	33	9	22	6
Thick mill and fill	Washington	ND	ND	ND	ND	ND
	Colorado	ND	ND	ND	ND	ND
	Louisiana	735	957	605	286	396
	GPS-5	14	14	13	15	14
	GPS-6	12	13	3	3	4
Chip seal	Washington	52	38	156	111	194
	Colorado	50	12	43	35	52
	Louisiana	1,089	574	1,605	772	819
	SPS-3	21	22	18	21	17

ND = No data.

- **Step 4:** The data for each 0.1-mi (0.16-km)-long pavement segment of each project were analyzed, and RFPs and RSPs before treatment were calculated.

- **Step 5**: For each 0.1-mi (0.16-km)-long pavement segment, the treatment benefits were calculated in terms of RFP or RSP after treatment, CFPs or CSPs, and FCROPs or SCROPs. The treatment benefits were then compared with the treatment benefits obtained from the LTPP test sections. One issue that should be noted is that the history of the 0.1-mi (0.16-km)-long pavement segments and the treatment dates were different from one pavement project to another and from one LTPP test section to another. To compare the benefits using an equivalent reference, RFP and RSP were calculated from the treatment time to the time when the pavement reached the prespecified threshold value. Stated differently, the calculated RFP and RSP of each 0.1-mi (0.16-km)-long pavement segment and of the LTPP test sections represented the time in years from the treatment date to the time when the prespecified threshold value was reached. Further, the same threshold values were used in the analyses of the State and the LTPP data.

- **Step 6:** The results of the analyses of the 0.1-mi (0.16-km)-long pavement segments were grouped into six T^2Ms, which were submitted to FHWA and are available from the LTPP Customer Support Services.[(79)] The T^2Ms list the before and after treatment CSs for all 0.1-mi (0.16-km)-long pavement segments that were analyzed. The T^2Ms also list the benefits of the treatments in term of RFP/RSP, CFP/CSP, and FCROP/SCROP and their averages.

- **Step 7:** Results of the analyses of all LTPP test sections (the numbers are also listed in table 117) that were subjected to one of the previously listed treatments and located in any climatic region were grouped for each pavement condition and distress type. The weighted average benefits in terms of RFP/RSP, CFP/CSP, and FCROP/SCROP were then calculated.

- **Step 8**: The two sets of benefits were then compared per pavement condition and distress type, as detailed in the following sections.

IRI

Table 118 summarizes the calculated benefits (in terms of IRI) for all 0.1-mi (0.16-km)-long pavement segments in each State transportation department that received the indicated treatment type and the comparable LTPP test sections. The table also lists the number of 0.1-mi (0.16-km)-long segments and the number of LTPP test sections involved in the analyses. For ease of visual comparison of the benefits, they were plotted in bar graph format as shown in figure 102 through figure 104. Examination of the three figures indicated that the benefits, for IRI in terms of the RFP, CFP, and FCROP of each treatment type of the LTPP SPS and GPS test sections and of the 0.1-mi (0.16-km)-long pavement segments in each of the cited State transportation departments, were very similar. The conclusions and recommendations based on these observations are included in a separate section at the end of this chapter.

Table 118. Comparison of the weighted average treatment benefits based on IRI of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

Treatment Type	Data Source	Number of 0.1-Mi (0.16-Km)-Long Pavement Segments	Treatment Benefits (Year)		
			RFP	CFP	FCROP
Thin overlay	Washington	349	19	9	13
	Colorado	94	11	1	3
	Louisiana	219	18	14	17
	SPS-3 and -5	36	18	11	11
	GPS-6	25	19	10	13
Thick overlay	Washington	10	20	4	10
	Louisiana	1,416	19	14	18
	SPS-5	14	20	7	18
	GPS-6	15	20	14	16
Thin mill and fill	Washington	123	19	7	14
	Colorado	28	14	8	10
	Louisiana	163	18	11	15
	SPS-5	13	20	8	19
	GPS-6	27	18	7	6
Thick mill and fill	Louisiana	735	18	12	16
	SPS-5	14	20	8	19
	GPS-6	12	20	12	15
Chip seal	Washington	52	12	4	2
	Colorado	50	16	4	0
	Louisiana	1,089	12	2	1
	SPS-3	21	15	2	4

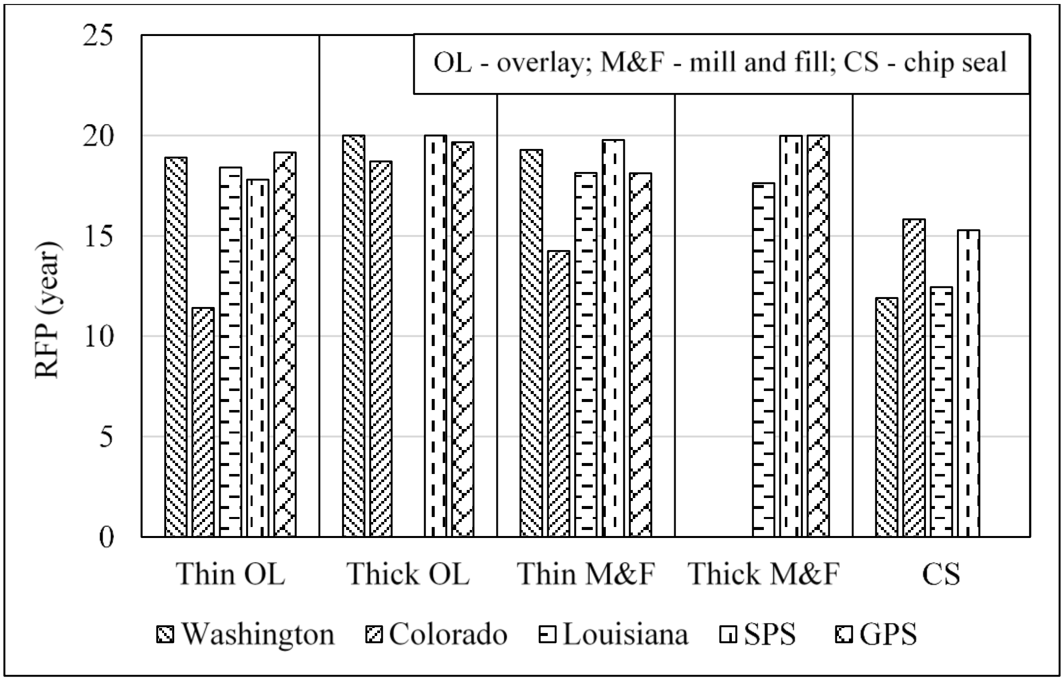


Figure 102. Graph. Comparison of the weighted average RFP based on IRI of five treatment types performed on LTPP test sections and on various pavement projects of CDOT, LADOTD, and WSDOT.

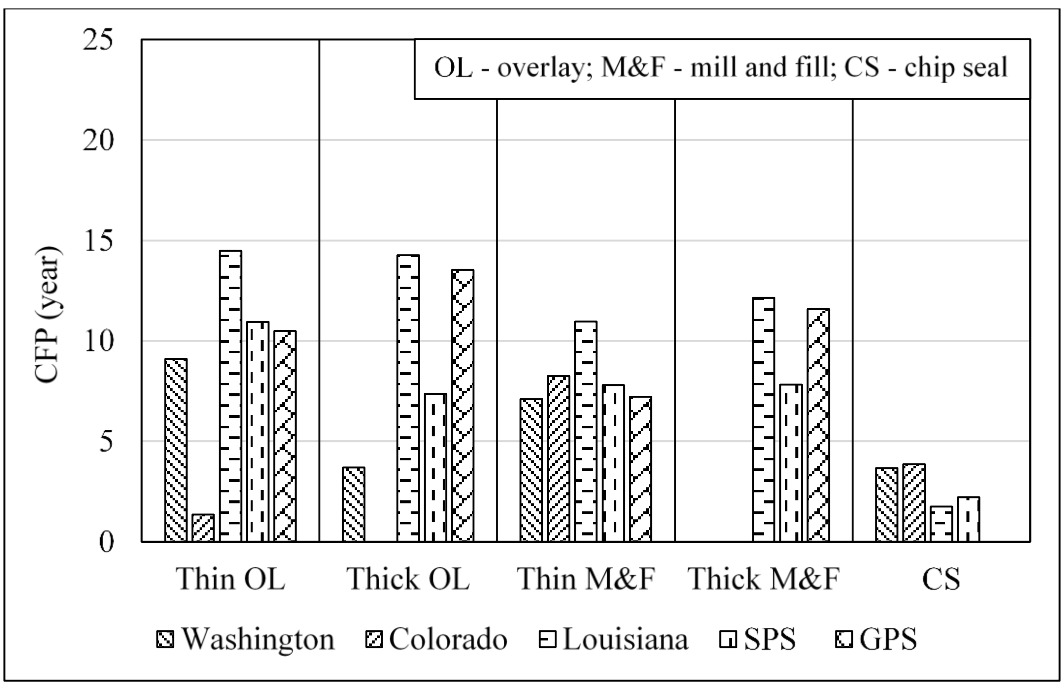


Figure 103. Graph. Comparison of the weighted average CFP based on IRI of five treatment types performed on LTPP test sections and on various pavement projects of CDOT, LADOTD, and WSDOT.

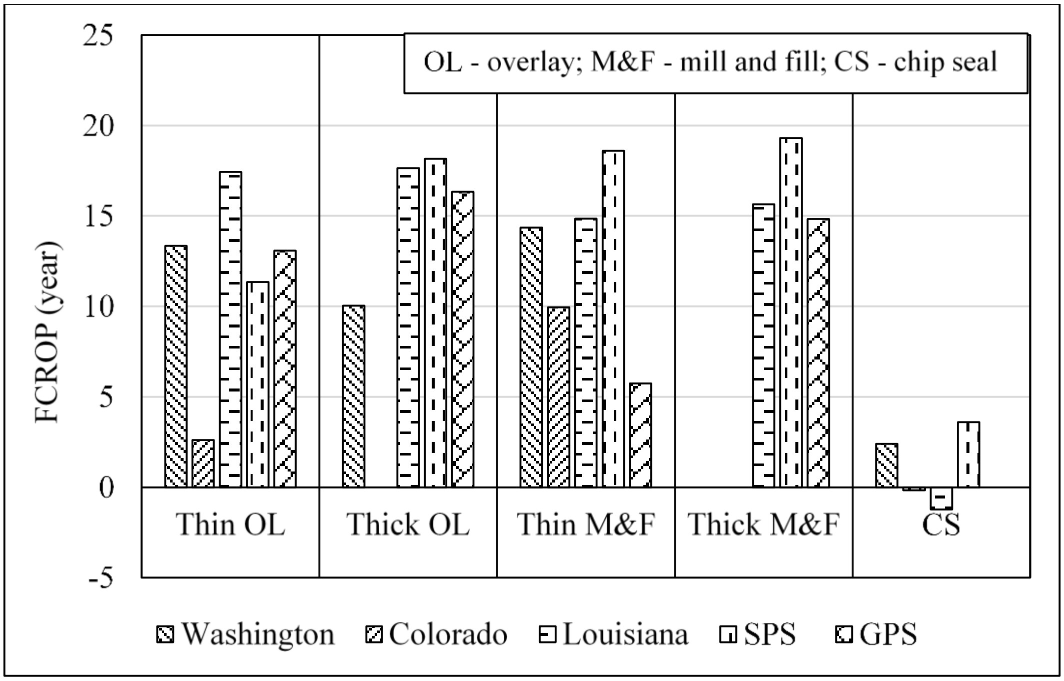


Figure 104. Graph. Comparison of the weighted average FCROP based on IRI of five treatment types performed on LTPP test sections and on various pavement projects of CDOT, LADOTD, and WSDOT.

RUT DEPTH

Table 119 summarizes the calculated benefits (in terms of rut depth) for all 0.1-mi (0.16-km)-long pavement segments in each cited State transportation department that received the indicated treatment type and the comparable LTPP test sections. The table also lists the number of 0.1-mi (0.16-km)-long segments and the number of LTPP test sections involved in the analyses. For ease of visual comparison of the benefits, they were plotted in bar graph format as shown in figure 105 through figure 107. Examination of the three figures indicated that the benefits, in terms of the RFP/RSP, CFP/CSP, and FCROP/SCROP of each treatment type of the LTPP test sections and of the 0.1-mi (0.16-km)-long pavement segments in each of the cited State transportation departments were very similar. Once again, the conclusions and recommendations based on these observations are included in a separate section at the end of this chapter.

Table 119. Comparison of the weighted average treatment benefits based on rut depth of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

Treatment Type	Data Source	Rut Depth: Number of 0.1-Mi (0.16-Km)-Long Pavement Segments	Rut Depth: RFP/RSP (Year)	Rut Depth: CFP/CSP (Year)	Rut Depth: FCROP/SCROP (Year)
Thin overlay	Washington	709	20	6	15
	Colorado	126	14	4	7
	Louisiana	224	20	4	18
	SPS-3 and -5	35	19	10	16
	GPS-6	19	20	11	18
Thick overlay	Washington	122	20	8	15
	Louisiana	1,242	20	6	10
	SPS-5	15	18	7	14
	GPS-6	13	19	11	18
Thin mill and fill	Washington	701	19	8	16
	Colorado	74	20	6	14
	Louisiana	191	20	14	19
	SPS-5	13	17	8	17
	GPS-6	33	19	14	17
Thick mill and fill	Louisiana	957	18	9	14
	SPS-5	14	18	7	17
	GPS-6	13	20	12	18
Chip seal	Washington	38	20	1	9
	Colorado	12	19	3	0
	Louisiana	574	19	6	8
	SPS-3	22	15	4	11

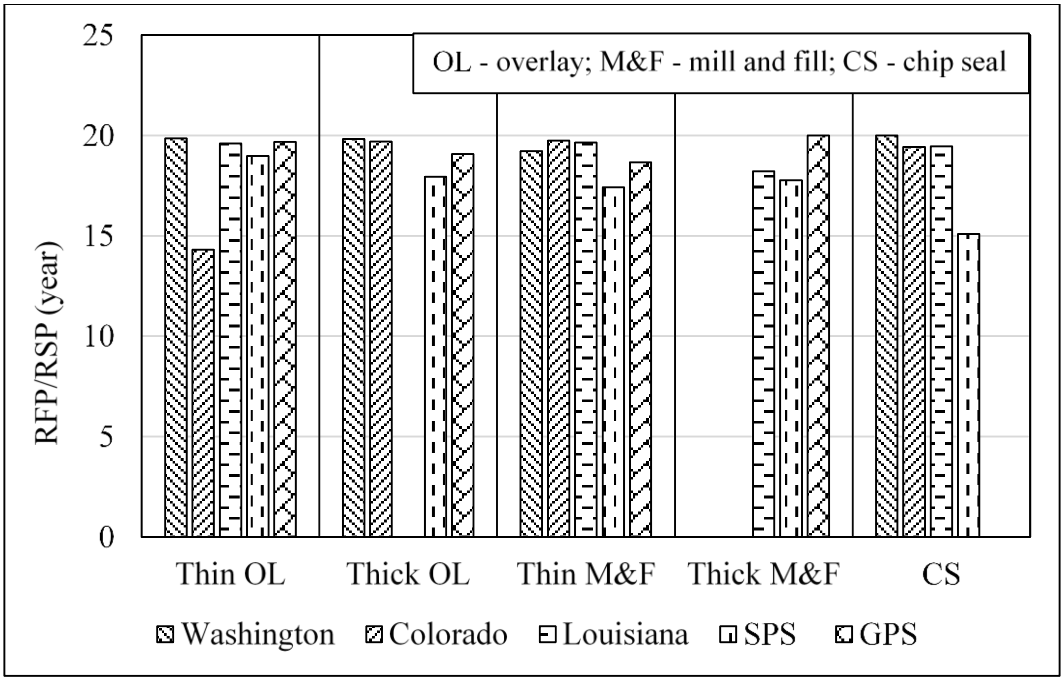


Figure 105. Graph. Comparison of the weighted average RFP/RSP based on rut depth of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

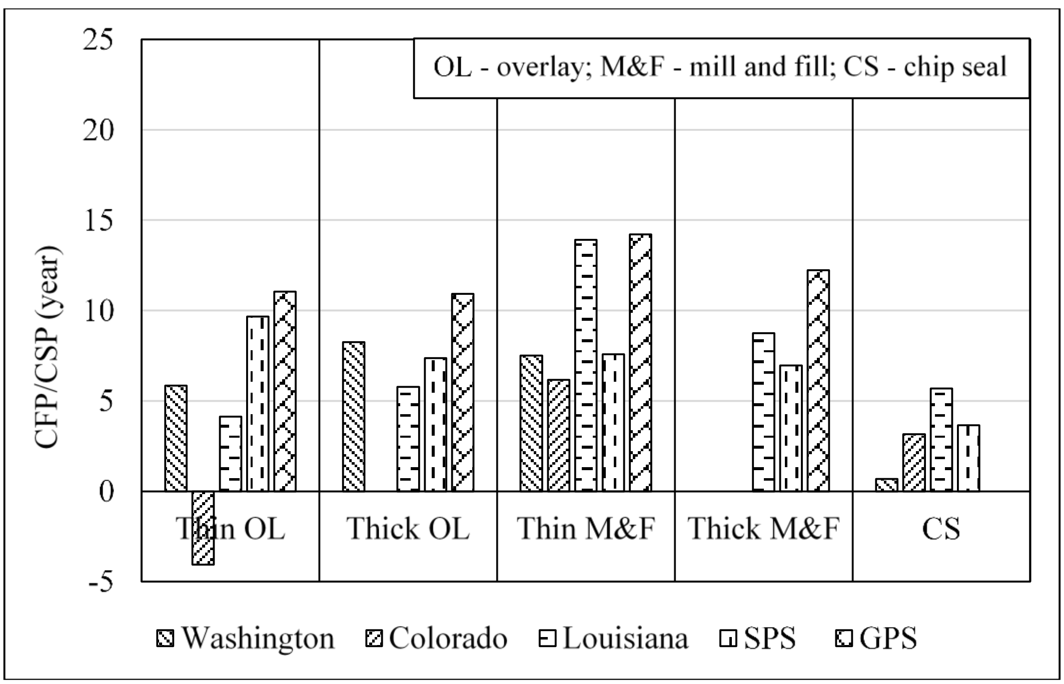


Figure 106. Graph. Comparison of the weighted average CFP/CSP based on rut depth of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

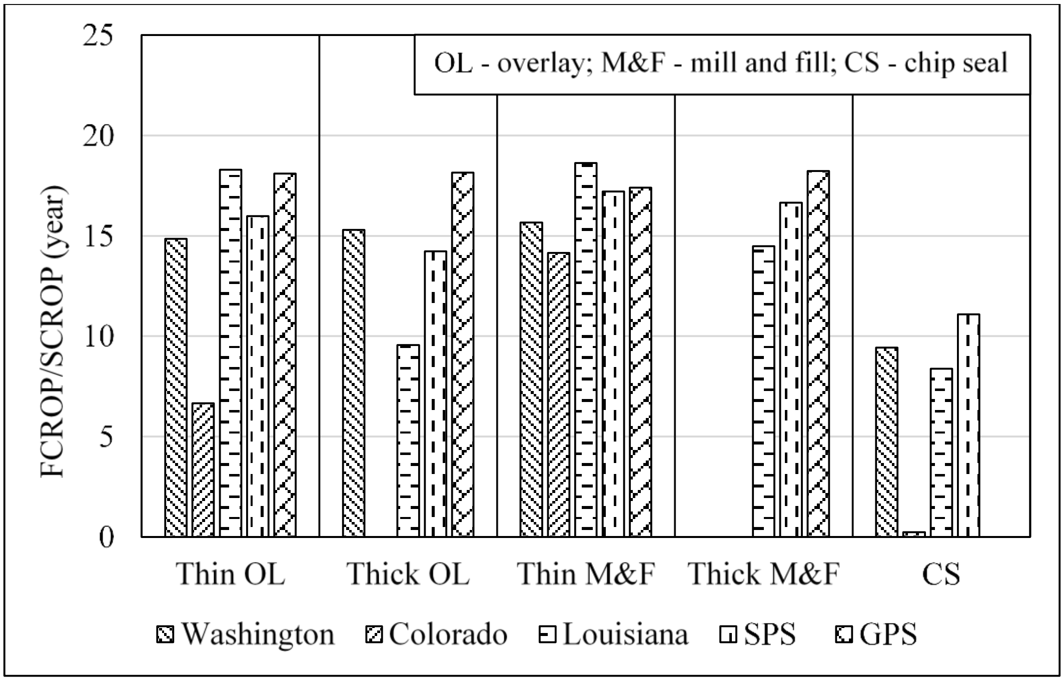


Figure 107. Graph. Comparison of the weighted average FCROP/SCROP based on rut depth of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

ALLIGATOR CRACKING

Table 120 summarizes the average calculated benefits (relative to alligator cracking) for all 0.1-mi (0.16-km)-long pavement segments in each cited State transportation department that received the indicated treatment type and the comparable LTPP test sections. The table also lists the number of 0.1-mi (0.16-km)-long segments and the number of LTPP test sections involved in the analyses. For ease of visual comparison of the benefits, they were plotted in bar graph format as shown in figure 108 through figure 110. Examination of the three figures indicated that the benefits of each treatment type of the LTPP test sections and of the 0.1-mi (0.16-km)-long pavement segments in each of the cited State transportation departments were very similar. The conclusions and recommendations based on these observations are included in a separate section at the end of this chapter.

Table 120. Comparison of the weighted average treatment benefits based on alligator cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

Treatment Type	Data Source	Alligator Cracking			
		Number of 0.1-Mi (0.16-km)-Long Pavement Segments	RSP (Years)	CSP (Years)	SCROP (Years)
Thin overlay	Washington	1,746	18	6	13
	Colorado	128	9	6	0
	Louisiana	202	11	9	10
	SPS-3 and -5	34	12	5	6
	GPS-6	7	16	14	15
Thick overlay	Washington	403	18	5	14
	Louisiana	1,199	15	13	15
	SPS-5	10	14	7	10
	GPS-6	5	14	9	12
Thin mill and fill	Washington	886	18	2	10
	Colorado	49	11	3	7
	Louisiana	146	18	9	13
	SPS-5	13	15	7	14
	GPS-6	9	9	7	11
Thick mill and fill	Louisiana	605	17	15	17
	SPS-5	13	15	7	11
	GPS-6	3	12	11	13
Chip seal	Washington	156	19	2	7
	Colorado	43	10	7	5
	Louisiana	1,605	10	8	9
	SPS-3	18	11	1	6

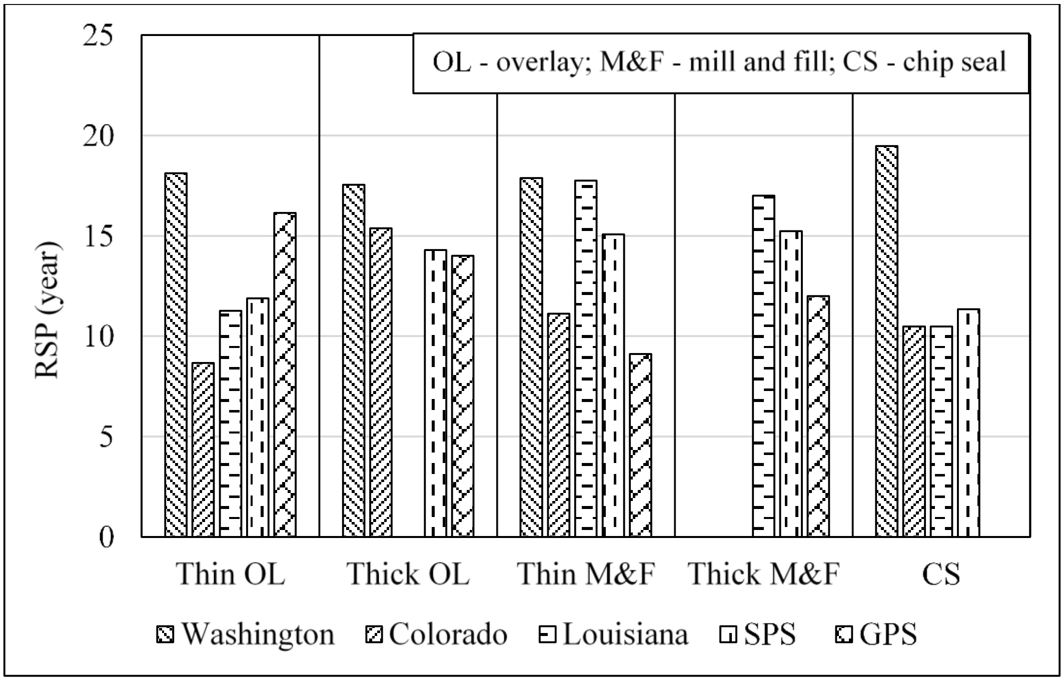


Figure 108. Graph. Comparison of the weighted average RSP based on alligator cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

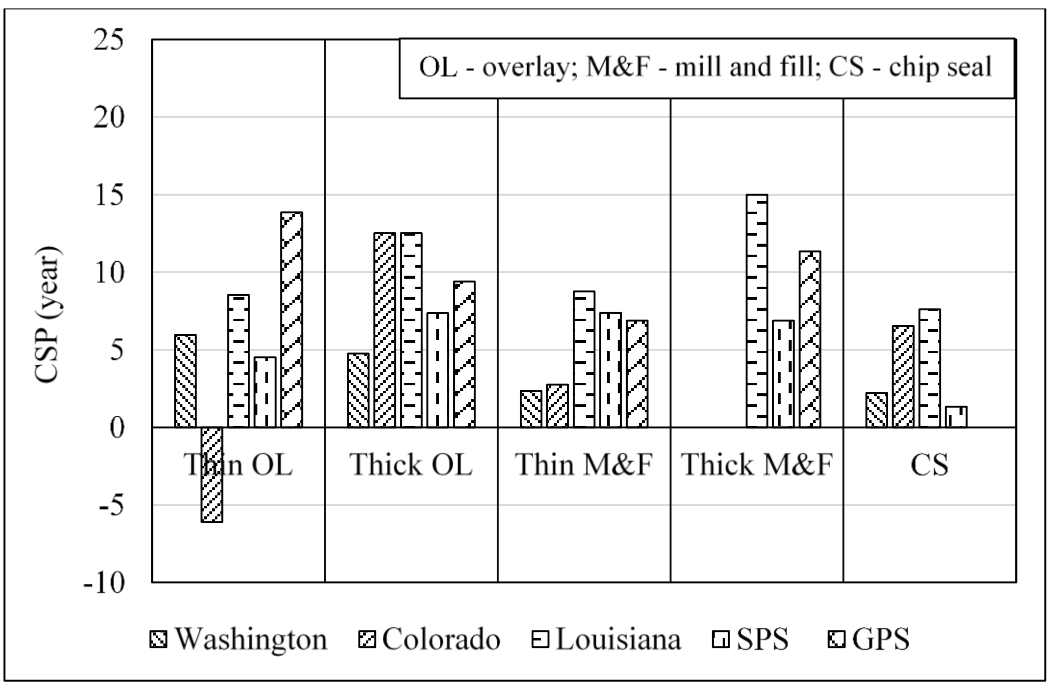


Figure 109. Graph. Comparison of the weighted average CSP based on alligator cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

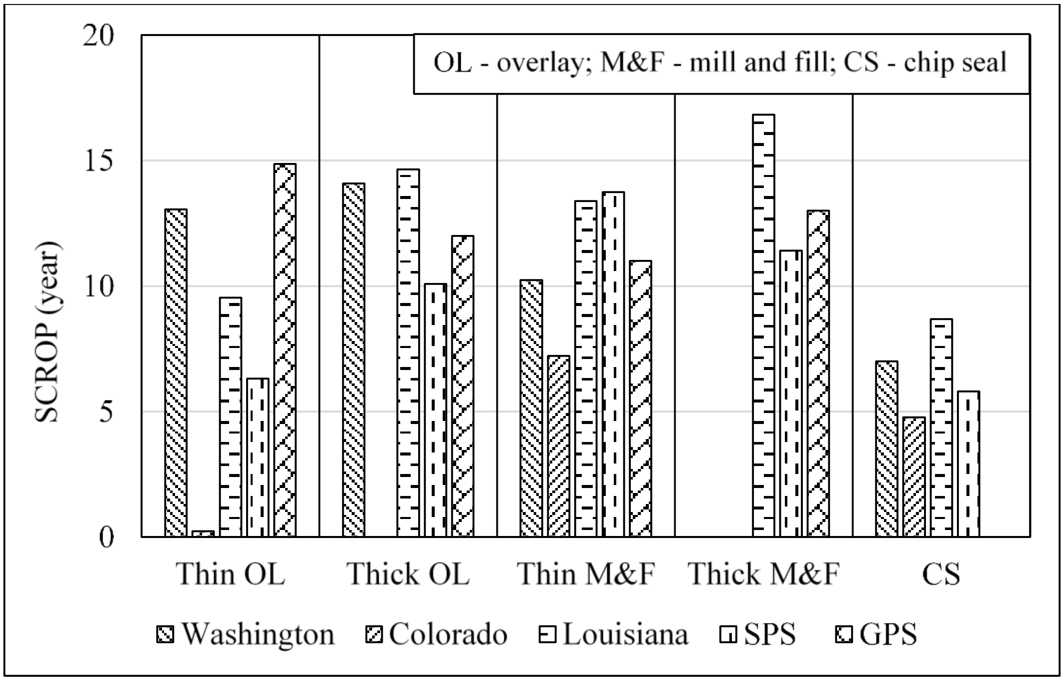


Figure 110. Graph. Comparison of the weighted average SCROP based on alligator cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

LONGITUDINAL CRACKING

Table 121 summarizes the average calculated benefits (in terms of longitudinal cracking) for all 0.1-mi (0.16-km)-long pavement segments in each cited State transportation department that received the indicated treatment. For ease of visual comparison of the benefits, they were plotted in bar graph format as shown in figure 111 through figure 113. Examination of the three figures indicated that the benefits of each treatment type of the LTPP test sections and of the 0.1-mi (0.16-km)-long pavement segments in each of the cited State transportation departments were very similar. The conclusions and recommendations based on these observations are included in a separate section at the end of this chapter.

Table 121. Comparison of the weighted average treatment benefits based on longitudinal cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

Treatment Type	Data Source	Longitudinal Cracking			
		Number of 0.1-Mi (0.16-Km)-Long Pavement Segments	RSP (Years)	CSP (Years)	SCROP (Years)
Thin overlay	Washington	1,000	18	4	13
	Colorado	129	11	2	1
	Louisiana	71	17	8	11
	SPS-3 and -5	40	13	1	5
	GPS-6	7	10	8	10
Thick overlay	Washington	310	19	0	14
	Louisiana	595	17	7	11
	SPS-5	15	14	3	4
	GPS-6	2	8	7	4
Thin mill and fill	Washington	357	18	4	9
	Colorado	38	9	2	4
	Louisiana	80	18	11	12
	SPS-5	17	14	2	6
	GPS-6	22	7	5	4
Thick mill and fill	Louisiana	286	16	7	13
	SPS-5	15	15	4	6
	GPS-6	3	15	9	10
Chip seal	Washington	111	19	4	10
	Colorado	35	13	10	5
	Louisiana	772	17	9	11
	SPS-3	21	18	1	8

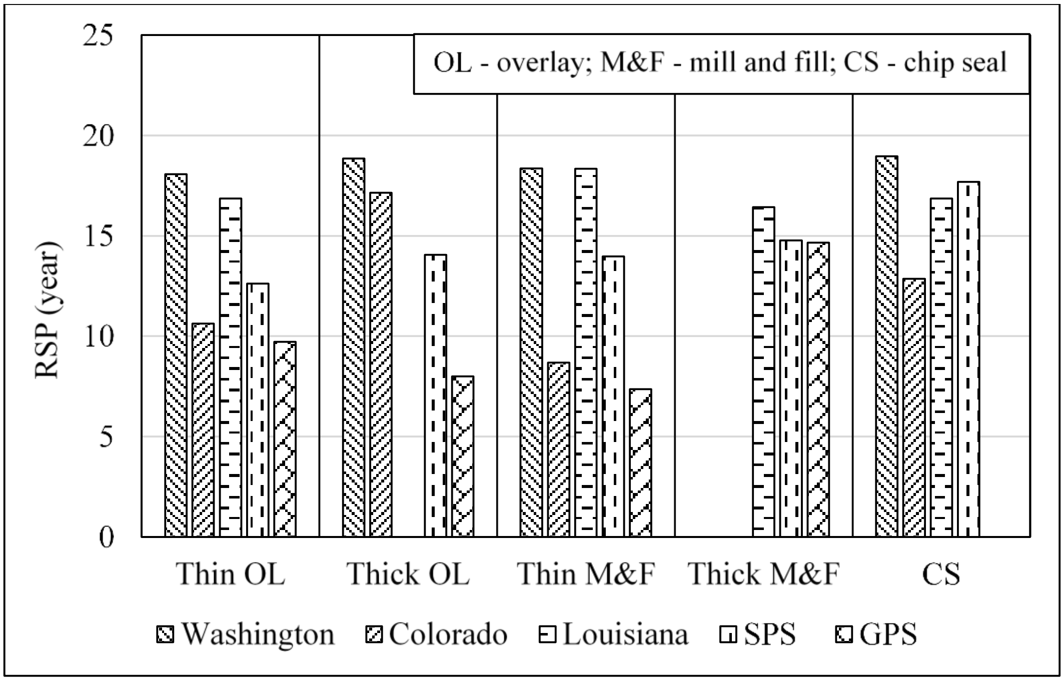


Figure 111. Graph. Comparison of the weighted average RSP based on longitudinal cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

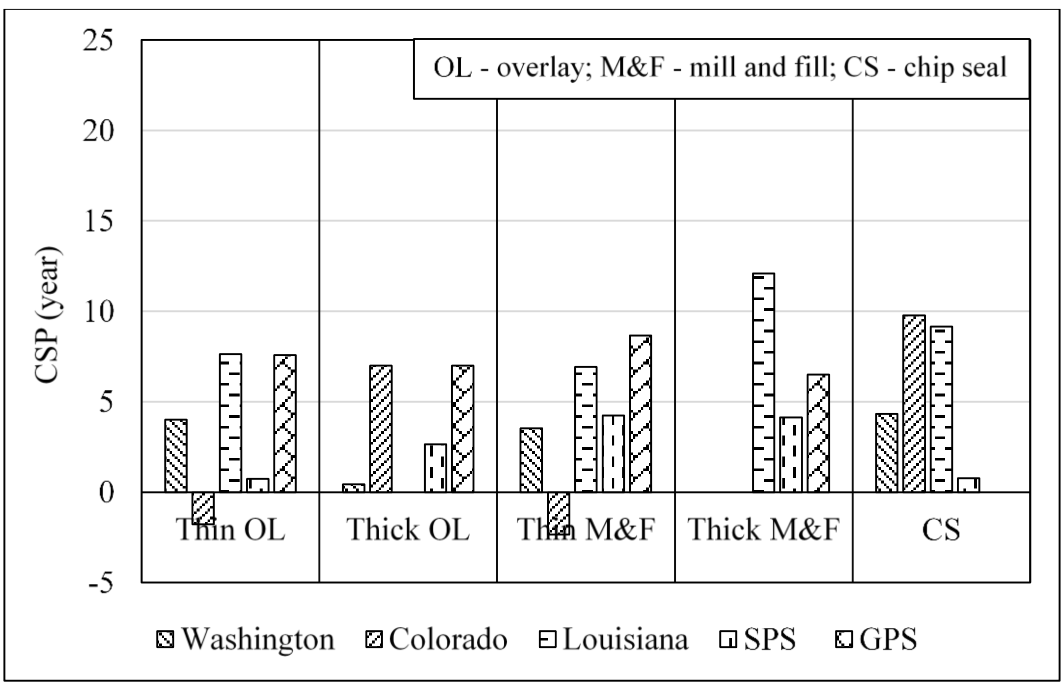


Figure 112. Graph. Comparison of the weighted average CSP based on longitudinal cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

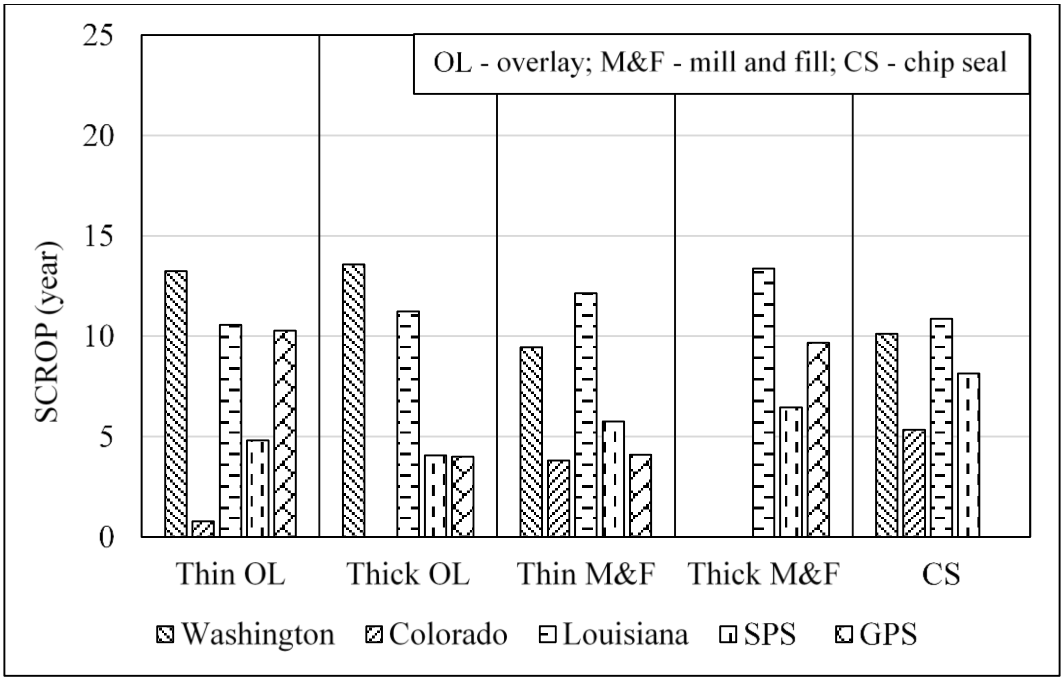


Figure 113. Graph. Comparison of the weighted average SCROP based on longitudinal cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

TRANSVERSE CRACKING

Table 122 summarizes the average calculated benefits (relative to transverse cracking) for all 0.1-mi (0.16-km)-long pavement segments in each cited State transportation department that received the indicated treatment type and the comparable LTPP test sections. The table also lists the number of 0.1-mi (0.16-km)-long segments and the number of LTPP test sections involved in the analyses. For ease of visual comparison of the benefits, they were plotted in bar graph format as shown in figure 114 through figure 116. Examination of the three figures indicated that the benefits, in terms of RSP, CSP, and SCROP of each treatment type of the LTPP test sections and of the 0.1-mi (0.16-km)-long pavement segments in each of the cited State transportation departments were very similar. The conclusions and recommendations based on these observations are included in a separate section at the end of this chapter.

Table 122. Comparison of the weighted average treatment benefits based on transverse cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

Treatment Type	Data Source	Transverse Cracking			
		Number of 0.1-Mi (0.16-Km)-Long Pavement Segments	RSP (Year)	CSP (Year)	SCROP (Year)
Thin overlay	Washington	1,538	19	2	12
	Colorado	70	13	4	4
	Louisiana	134	11	3	7
	SPS-3 and -5	37	13	0	7
	GPS-6	13	12	6	8
Thick overlay	Washington	220	20	2	17
	Louisiana	984	14	8	10
	SPS-5	13	16	3	11
	GPS-6	6	14	11	12
Thin mill and fill	Washington	633	19	2	11
	Colorado	24	8	4	1
	Louisiana	135	15	9	12
	SPS-5	16	16	2	9
	GPS-6	6	11	5	5
Thick mill and fill	Louisiana	396	16	12	15
	SPS-5	14	17	4	11
	GPS-6	4	16	7	14
Chip seal	Washington	194	20	0	5
	Colorado	52	7	3	3
	Louisiana	819	13	9	9
	SPS-3	17	12	2	5

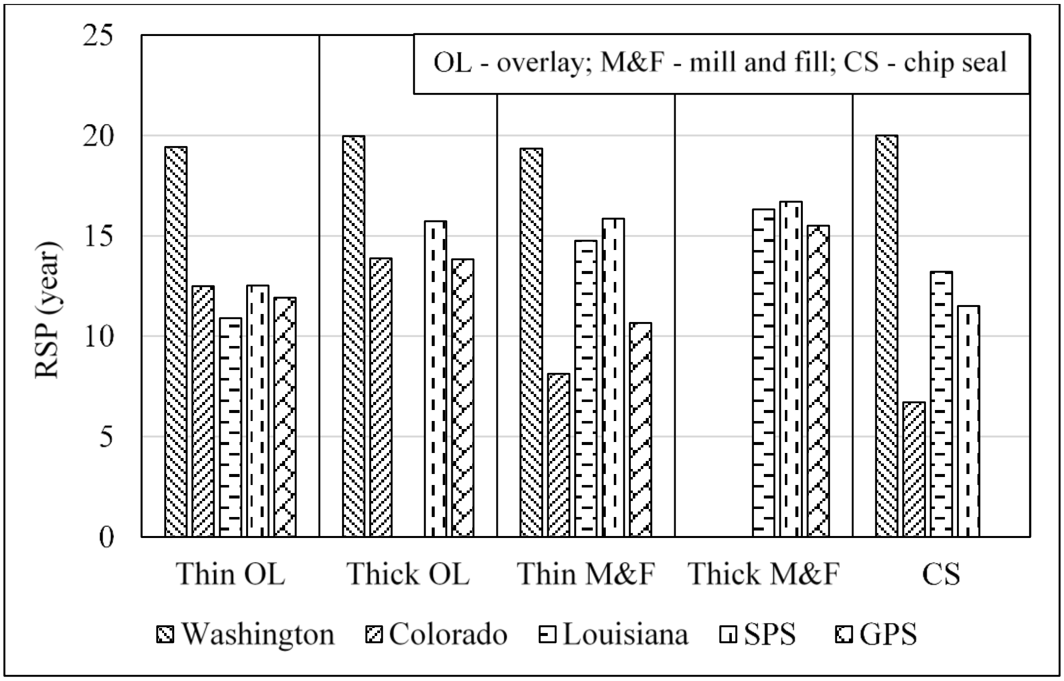


Figure 114. Graph. Comparison of the weighted average RSP based on transverse cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

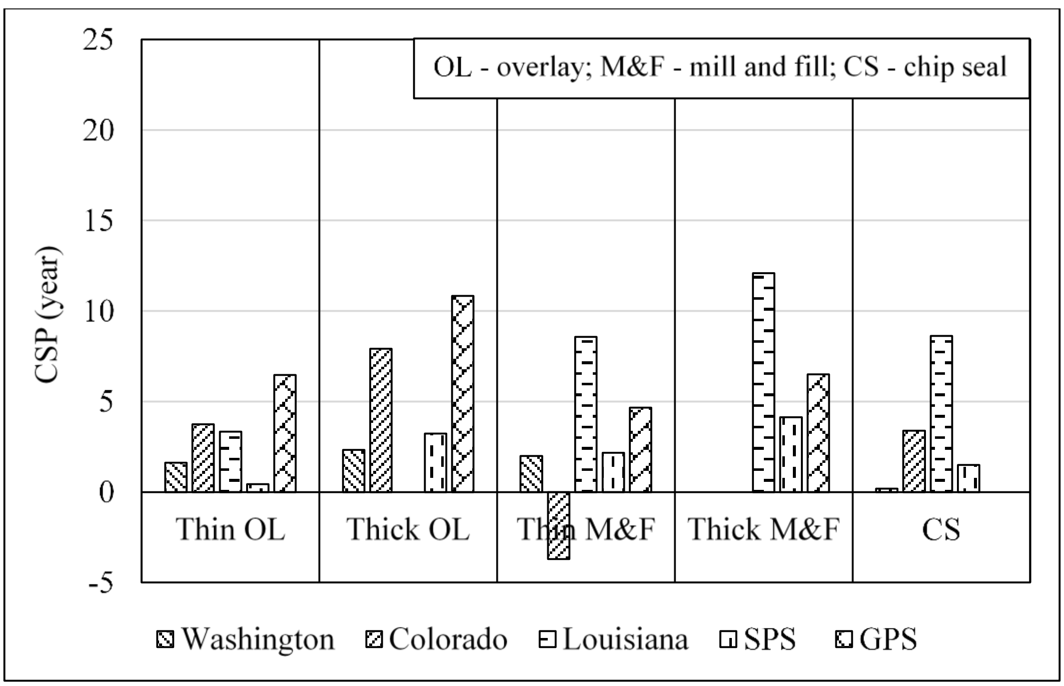


Figure 115. Graph. Comparison of the weighted average CSP based on transverse cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

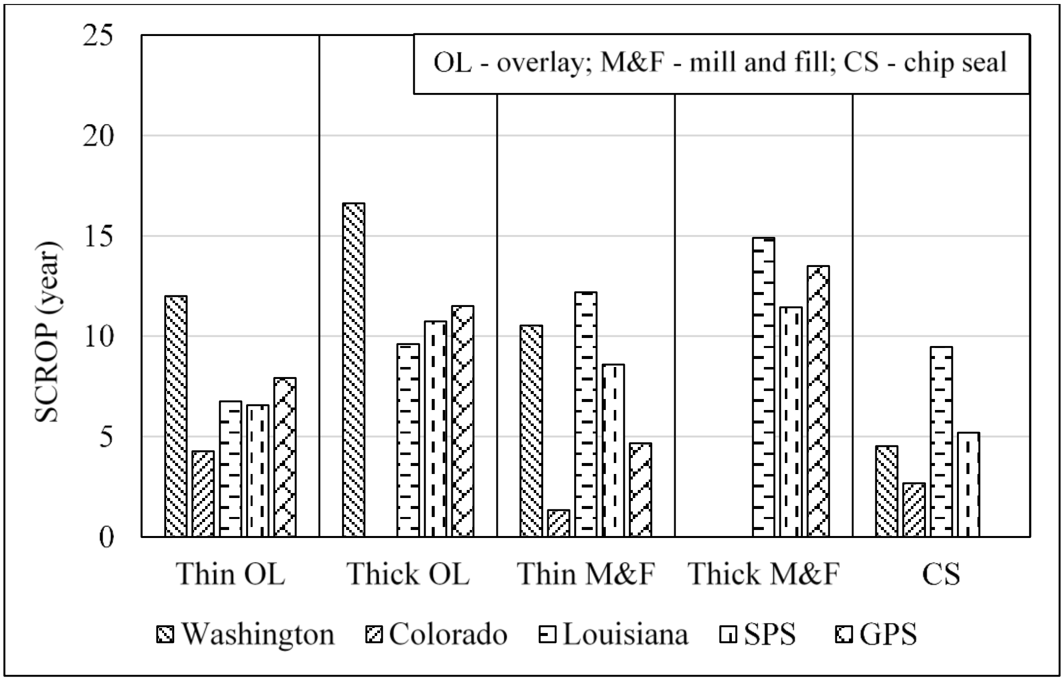


Figure 116. Graph. Comparison of the weighted average SCROP based on transverse cracking of five treatment types performed on LTPP test sections and on pavement projects of CDOT, LADOTD, and WSDOT.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Pavement condition and distress databases of three pavement networks were requested and received from three State transportation departments—CDOT, LADOTD, and WSDOT. Each database was searched and pavement projects that received one of the following five treatment types were identified.

- Thin overlay (≤ 2.5 inches (63.5 mm)).
- Thick overlay (> 2.5 inches (63.5 mm)).
- Thin mill and fill (≤ 2.5 inches (63.5 mm)).
- Thick mill and fill (> 2.5 inches (63.5 mm)).
- Single chip seal.

The pavement condition and distress data for each of the 0.1-mi (0.16-km)-long pavement segments along each selected pavement project that was treated using one of these five treatments was analyzed. Results of the analyses included RFPs and RSPs before and after treatment, CFPs and CSPs after treatment, and FCROPs and SCROPs after treatment. The pavement segments of all pavement projects in one State transportation department that received the same treatment type were grouped based on their RFPs or RSPs into the proper CSs before treatment. Each of the 0.1-mi (1.6-km)-long pavement segments in each CS group before treatment was listed in the after treatment CS based on its after treatment RFP or RSP. For each treatment type, the weighted average treatment benefits, in terms of each pavement condition and distress type, were then calculated. The results were submitted to FHWA and are available from the LTPP Customer Support Services.[79] These weighted average treatment benefits were then compared with the weighted average treatment benefits of the LTPP test sections. The results are listed in table 118 through table 122 and shown in figure 102 through figure 116. The data in the 15 figures indicated the following:

- The weighted average benefits of each of the five treatment types, in terms of each pavement condition and distress type, obtained from the analyses of the LTPP data were similar to the benefits obtained from the State data. The implication of this is that the treatment benefits described in this report that were found using the LTPP data, could be used as benchmark values for the national practice. State transportation departments could use such data to do the following:
    - Gauge the effectiveness of their current practices using similar analyses.
    - Conduct lifecycle cost analyses of various treatment alternatives to optimize the pavement network rehabilitation and treatment strategy.
- The methodologies described in this report for the analyses of the LTPP pavement condition and distress data applied to the State data.
- The three-level (poor, fair, and good) and five-level (very poor, poor, fair, good, and very good) pavement rating systems developed in this study, and presented in chapter 4 based on the time-series pavement condition and distress, were equally applicable to the LTPP and State data.
- The average variability in the measured pavement condition and distress data over time for the LTPP test sections was very similar to the variability of the State-measured data along most pavement projects.
- The percent of the LTPP test sections that were excluded from the analyses because of an inadequate number of data points or because of improving pavement condition and/or distress over time without the application of treatments was equivalent to the percent of the 0.1-mi (0.16-km)-long segments of a pavement project that was excluded from the analyses for the same reasons.

Based on the results of the analyses, the following is strongly recommended:

- The dual pavement condition rating systems should be submitted for approval and adoption by FHWA, AASHTO, and the State transportation departments.
- The algorithms developed in this study should be standardized and used on future research studies.
- The benchmark values regarding the benefits of the five treatment types included in this study should be expanded to include additional pavement treatments.

CHAPTER 9. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This chapter presents a summary of the studies described in chapters 2 through 8, followed by significant conclusions reached organized by topic, and finally, significant recommendations also organized by topic.

SUMMARY

For the benefit of the reader, this section is divided into several paragraphs. Each paragraph includes the summary of a particular chapter of the final report.

At the outset, the research team conducted a comprehensive review of the state of the practice of various State transportation departments with regard to several aspects of pavement condition measures, pavement condition and distress data analyses, and treatment selection. The review also included previous related studies that were conducted using the LTPP database. The detailed literature review can be found in chapter 2. The topics covered include the following:

- Pavement distress severity levels.
- Pavement condition and distress descriptions.
- Pavement performance modeling and treatment benefit calculations.
- Treatment type and time selection.
- Preservation costs and LCCA.
- Effectiveness of pavement treatments at the project and network levels.
- LTPP Program, its objectives, and the SPS and GPS test sections.
- Previous findings regarding the impacts of pavement treatments and various design factors on pavement performance.

At the same time, the research team downloaded from the six data volumes housed in the LTPP database Standard Data Release 28.0 the data elements of the more than 2,500 test sections included in the LTPP Program. The data were organized in a special format and readied for analyses. In addition, the pavement management databases from three State transportation departments (CDOT, WSDOT, and LADOTD) were requested and received. From each database, several pavement projects that were subjected to certain treatments in the past were identified, and their data were downloaded from the respective databases and formatted for analyses. The details of the LTPP and State data mining are in chapter 4.

Based on the literature review, and general review of the LTPP data, dual pavement condition rating systems were developed based on the pavement function and its structural integrity. One system was based on three CSs, and the other was based on five CSs. For each pavement section, the functional CS was based on ride quality in terms of IR) and safety in terms of rut depth. The functional CS was expressed in term of RFP in years for the pavement to reach the prespecified threshold value for IRI or rut depth. The structural CS was based on RSP in years for the pavement section to reach the threshold values in terms of alligator, transverse, or longitudinal cracking or rut depth. The rating system for each CS consisted of numerical classification, color coding, range of RFP and RSP, and the average cost per 0.1 mi (0.16 km) of preserving the pavement. Further, based on the literature review and common engineering practice, individual threshold values for IRI, rut depth, alligator, transverse, and longitudinal cracking were recommended and used in the analyses of the LTPP and State data. Details of the two pavement rating systems and the threshold values used in the analyses are in chapter 3 of this report.

The performance of each of the LTPP flexible pavement test sections included in the SPS-1, -3, and -5, and GPS-6 experiments was analyzed. In the analyses, the available before and after treatments time-series pavement condition (IRI and rut depth) and distress (rut depth, alligator, longitudinal, and transverse cracking) data were used. The data were modeled as a function of time using the proper mathematical function form (power function for rut depth, exponential for IRI, and logistic for cracking). Results of the analyses were expressed in terms of RFP for IRI, RFP/RSP for rut depth, and RSP for each cracking type. Thus, for each test section, two RFPs and four RSPs were calculated. These values were used to assess the impacts of regional climatic and design factors on pavement performance. In addition, the LTPP data on the flexible pavement test sections were used to develop a new and novel method to estimate the pavement performance based on a single data point. The ORCSE method was developed to be applied to pavement sections that had experienced less frequent data collection and/or pavement sections that might not yet have sufficient data records for modeling owing to age. The analyses and the results of the analyses of the SPS-1, -3, and -5, and GPS-6 test sections and the development and validation of the ORCSE method are detailed in chapter 5.

The performance of each of the LTPP rigid pavement test sections included in the SPS-2, -4, -6, and -7, and GPS-7 experiments was also analyzed. In the analyses, the available before and after treatments time-series pavement condition (IRI and rut depth) and distress (rut depth, alligator, longitudinal, and transverse cracking) data were used. The intent was to study the impact of each design variable on pavement performance. When the data were divided into various groups based on separation of variables, the number of test sections under each design variable was statistically insignificant (i.e., for some variables, there was only one or no test section). Therefore, the impacts of the design variables on pavement performance were not analyzed or discussed any further. Rather, the data were used to study the impacts of the conditions in climatic regions on pavement performance. Results of those analyses are detailed in chapter 6.

The measured flexible and rigid pavement deflection data were organized and analyzed. The analyses were conducted to accomplish the following two objectives:

- Determine whether the measured flexible pavement deflection data could be used to estimate the time to initiate bottom-up cracking before those cracks appeared on the pavement surface and whether those data could be used in the RFP and RSP algorithms.

- Study the LTE using the measured rigid pavement deflection data and its impact on ride quality in terms of IRI.

At the onset, it was envisioned that measured pavement deflections and/or LTE data would be correlated with pavement condition or distress, and hence, they could be used as an early warning of pending deterioration before it became visible on the pavement surface. For flexible pavements, the measured pavement deflections are a function of the pavement temperatures. Therefore, the accuracy of the existing temperature adjustment methods were reviewed and scrutinized using the measured LTPP deflection data. Based on the results, a new global temperature correction methodology was developed that could be applied to all deflection sensors and all climatic regions. Results of the analyses of the measured deflection data and the methodology and algorithms of the newly developed temperature correction method are detailed in chapter 7.

The pavement condition and distress databases of three pavement networks were requested and received from CDOT, LADOTD, and WSDOT. Each database was searched and pavement projects that received one of the following five treatment types were identified:

- Thin overlay (≤ 2.5 inches (63.5 mm)).
- Thick overlay (> 2.5 inches (63.5 mm)).
- Thin mill and fill (≤ 2.5 inches (63.5 mm)).
- Thick mill and fill (> 2.5 inches (63.5 mm)).
- Single chip seal.

The research team analyzed the pavement condition and distress data measured before and after treatment of each 0.1-mi (0.16-km)-long pavement segment along each selected pavement project that was treated using one of these five treatments. The main objective of the analyses was to calculate the treatment benefits in terms of the following:

- RFPs and RSPs before and after treatment.
- CFP and CSP resulting from the treatment.
- FCROP and SCROP.

For each treatment type, the weighted average treatment benefits, in terms of each pavement condition and distress type, were then calculated. The results were submitted to FHWA and are available from the LTPP Customer Support Services.[79] The weighted average treatment benefits were then compared with the weighted average treatment benefits of the LTPP test sections. Results of the comparison are detailed in chapter 8.

CONCLUSIONS

Based on the review of various national and international papers published in various journals and mainly on the results of the analyses, various conclusions (detailed in each chapter) were drawn that cover various study-related topics. For convenience, only significant conclusions were selected and listed by topic in the follow subsections.

Pavement Performance Measures

The following significant conclusions were drawn regarding pavement performance measures:

- The pavement cracking data were typically collected and stored based on three severity levels (low, medium, and high). For most cases, the problem was that the data could not be analyzed per severity level because of their excessive variability from one year to the next. Analyses of the cracking data based on the sum of all severity levels were proven to overcome the problem.

- For pavement projects received the same treatment type, T^2Ms were developed to display the distribution of the pavement conditions along the project before and after treatment. The data in the T^2Ms were used to estimate the benefits of the various treatments.

- Pavement condition rating should be based on current conditions and distresses as well as the pavement's rates of deterioration.

- The three- and five-level dual pavement condition rating systems developed in this study were useful and were equally applied to both State and LTPP data. The systems were flexible and could be easily tailored to fit the needs and constraints of any road agency.

- The estimated average cost of pavement preservation for each level of the dual pavement rating system could be used in the lifecycle cost analyses and in strategy optimization.

- Threshold values were provided for calculation of RFP and RSP. The values were based on minimum level of service to the user (functional), and loss of structural integrity (structural).

Flexible Pavements

The following significant conclusions were drawn regarding flexible pavements:

- The conditions in WF regions had significant adverse impacts on pavement performance in terms of IRI, rut depth, and cracking.

- Drainable bases decreased the impacts of the conditions in WF regions on pavement performance. The pavement performance owing to drainable bases was slightly better than that owing to increasing the AC thickness from 4 to 7 inches (102 to 178 mm). Therefore, the use of drainable bases in the WF region was cost effective.

- The inclusion of drainable bases in the DF and DNF regions did not affect pavement performance in terms of RFP or RSP. This was expected because the volume and frequency of available water was low. Furthermore, most rainfall occurred over short periods of time with most water running off the surface and not penetrating the pavement layers.

- Increasing the thickness of the AC layer from 4 to 7 inches (102 to 178 mm) increased the frost protection of the lower layers, and hence decreased the impacts of the climatic conditions in the WF region on pavement performance. However, this option was not a cost-effective one.

- The climatic conditions in the WNF, DF, and (DNF) regions did not affect the pavement performance in terms of rutting potential and IRI.

- The climatic conditions in the DF region had more adverse effects on cracking potential than those in the DNF region. This was mainly attributed to the higher oxidation (aging) potential of the AC layer in the DF region.

- The thin overlay treatment improved the pavement performance of the SPS-3 test sections in terms of IRI and rut depth in all climatic regions except the DNF region. This could be related to construction issues and, perhaps, the relatively high solar radiation (accelerated oxidation/aging) in the DNF region.

- In general, the thin overlay treatment did not improve the pavement performance of the SPS-3 test sections in terms of alligator, longitudinal, and transverse cracking. This is mainly because of the high rate of reflective cracking. Immediately after treatment, all cracks were hidden by the thin overlay. However, one or few years later, most cracks were reflected through the overlay, which implied relatively high rate of deterioration and hence short RSP. The exception was in the DNF region where the two test sections showed an increase of 12 years in the average RSP compared with the one control section. This oddity was mainly attributable to the limited number of sections. That is, the conclusion was not reliable because of the limited number of test sections and control sections.

- The slurry seal treatment improved the pavement performance of the SPS-3 test sections in terms of IRI and rut depth but did not have much impact on alligator, longitudinal, and transverse cracking.

- Crack sealing appeared to improve pavement performance of the SPS-3 test sections in terms of rutting. However, it did not improve the pavement performance in terms of cracking.

- Aggregate seal coats appeared to improve the pavement performance of the SPS-3 test sections in all climatic regions in terms of IRI, rut depth, and cracking.

- In general, the worse the pavement conditions were before treatment, the shorter the benefits of treatments were in terms of RFP and/or RSP.

- On average, the impact of 2- and 4-inch (51- and 102-mm)-thick virgin or recycled AC overlays on pavement performance of the SPS-5 test sections was almost the same.

- The 2-inch (51-mm)-thick AC overlay (virgin or recycled mix) did not provide long-term remediation of transverse cracking. The cracks in the lower pavement structure typically reflect through the overlay in few years.

- On average, SLE in terms of alligator crack for a flexible pavement structure as a result of application of a 4-inch (102-mm) AC overlay was slightly better than that which resulted from application of a 2-inch (51-mm) overlay.

- In each climatic region, the impacts of the thin and thick overlay or thin and thick mill-and-fill treatments on IRI and rut depths were almost the same. This was expected because good quality construction can decrease the pavement surface roughness substantially regardless of the overlay thickness and because most pavement rutting occurs early in the pavement life and can be removed during the treatment.

ORCSE Method

The following significant conclusions were drawn regarding the ORCSE method:

- The ORCSE method was successful in predicting the CSs of the test sections based on RFP and RSP. For older test sections, the average estimation error for the most critical CSs was less than 1 percent.

- The ORCSE method would have significant potential benefit for local roadway owners as well as State transportation department managers when only two or fewer data points were available.

Rigid and Composite Pavements

Significant conclusions regarding rigid and composite pavements were the following:

- The conditions in the WF region had more damaging impacts on the performance of the SPS-4 test sections in terms of transverse cracking than on test sections located in the WNF, DF, and DNF regions. This was expected owing to the combined effects of subfreezing temperatures and moisture.

- On average, the majority of the SPS-2 test sections located in the WNF region performed worse relative to longitudinal cracking than those in the DNF region. This was mainly due to the impact of excessive moisture on pavement performance.

- On average, in terms of IRI, joint and crack sealing treatment had a positive impact on the performance of the SPS-4 test sections located in the WNF region, no impact in the WF region, and negative impact in the DF and DNF regions.

- Joint and crack sealing was effective in the WF region and not effective in the other three climatic regions, and joint undersealing was not effective in any region.

- The performance of treated SPS-6 test sections in terms of IRI was independent of the climatic region and pavement type. It was also independent of treatment type in terms of rut depth.

- The alligator cracking data in the SPS-6 database were highly likely an advanced form of top-down fatigue cracking. (Top-down cracks are fatigue cracks that initiate at the pavement surface and, over time, propagate downward.) The short transverse and longitudinal cracks resembled the traditional bottom-up alligator cracking pattern.

- The performance of the test sections in terms of longitudinal cracking was worse after subjecting the section to any of the seven analyzed treatment types.
- Minimum and maximum pavement restoration with no AC overlay treatments did not improve the performance of the JRCP test sections.
- The IRI-based performance of the treated CRCPs (SPS-7) test sections was independent of the eight treatment types and the two climatic regions (WF and WNF).
- The performance of the JPCP test sections subjected to a 3-inch (76-mm) concrete overlay with milling (703) or with shot blasting (704) treatments was lower than the performance of the other JPCP test sections subjected to the other six treatments.
- None of the eight applied treatments were effective in treating transverse cracking problems of the CRCP test sections.

Deflection

The following significant conclusions were drawn regarding deflection:

- The LTPP deflection data collected for the SMP test sections did not support the use of the AI temperature adjustment procedure for the measured deflection data.
- The newly developed global flexible pavement deflection temperature adjustment algorithm was applicable to all deflection sensors and in all climatic regions.
- The LTPP deflection data collected for the SMP test sections did not correlate with the measured pavement condition and distress data. Therefore, the data were not included in the RFP and RSP algorithms.
- The LTPP deflection and LTE data collected for the SPS-2 test sections in Arizona, Colorado, Delaware, and Michigan did not support modeling of the data as a function of time.

State Data

The following significant conclusions were drawn regarding State data:

- The weighted average benefits of each of five treatment types (thin overlay (≤ 2.5 inches (63.5 mm)), thick overlay (> 2.5 inches (63.5 mm)), thin mill and fill (≤ 2.5 inches (63.5 mm)), thick mill and fill (> 2.5 inches (63.5 mm)), and single chip seal) with regard to each pavement condition and distress type, obtained from the analyses of the LTPP data, were similar to the benefits obtained from the three sets of State data.

- The methodologies described in this report for the analyses of the LTPP pavement condition and distress data applied to the State data.

- The three-level (poor, fair, and good) and five-level (very poor, poor, fair, good, and very good) pavement rating systems developed in this study, and presented in chapter 4 based on the time-series pavement condition and distress, were equally applicable to the LTPP and the State data.

- The average variability in the pavement condition and distress data over time for the LTPP test sections was almost the same as the average variability of the State-measured data.

- The percent of the LTPP test sections that were excluded from the analyses because of an inadequate number of data points or because of improving pavement condition and/or distress over time without the application of treatments was similar to the percent of the 0.1-mi (0.16-km)-long pavement segments of a given pavement project that was excluded from the analyses for the same reasons.

RECOMMENDATIONS

Based on the results of the LTPP and State data analyses and the conclusions listed in the previous section, various recommendations were developed. For convenience, the significant recommendations are listed by topic in the following subsections.

Performance Measures

The following recommendations were developed regarding performance measures:

- The sum of crack lengths or crack areas of all severity levels should be used to model the data as a function of time.

- Accurate pavement planning and management decisions should be based on the pavement conditions and rates of deterioration.

- The three and/or the five-level rating systems should be adopted by FHWA and submitted to AASHTO for approval.

- The threshold values used in this study should be adopted or similar ones should be developed by highway owners to estimate RFP and RSP of the various pavement sections.

- Each highway agency should develop the average cost of pavement preservation for each RFP and RSP level of the dual pavement rating systems using their own cost records.

- LCCA should be performed at the project level and strategy optimization at the network level to improve the overall cost effectiveness of the pavement management application.

- The T^2M procedure should be adopted and used by road owners to assess treatment effectiveness and to select the optimum treatment time.

- The dual pavement rating system described in chapter 3 should be used in future analyses and assessments of the benefits of pavement rehabilitation and/or maintenance treatments.

Flexible Pavements

The following recommendations were developed regarding flexible pavements:

- Drainable bases should be constructed to enhance the performance of pavement sections located in the WF region.

- For future study, the control or linked test sections should be selected to border the regular test sections in question, and their history should be included in the database. This would eliminate unnecessary variability.

- The pavement condition and distress data should be measured before and after treatments. The quality control data for project acceptance should be included in the PMS database.

- The frequency of pavement condition and distress data collection should be a function of treatment type. Treatments having short TL should be surveyed more often than long-life treatments.

ORCSE Method

The following recommendation was developed regarding the ORCSE method:

- The ORCSE method should be expanded to address other conditions and distresses as well as be applied to a wider range of LTPP and State transportation department data to further verify its successful prediction of the pavement CSs.

Deflection

The following recommendations were developed regarding deflection:

- The new global flexible pavement deflection temperature adjustment algorithm should be adopted and used to adjust the measured pavement deflection to a standard temperature of 70 ºF (21 ºC).

- The estimation of the AC modulus at the standard temperature of 70 ºF (21 ºC)could be accomplished using one of the following two procedures:

    - Adjust the measured deflection data to 70 ºF (21 ºC) using the newly developed global model and then backcalculate the modulus.

    - Conduct a minimum of three FWD tests at temperatures above and below the standard temperature of 70 ºF (21 ºC) and measure the pavement deflections. Backcalculate the AC modulus value for each of the three FWD tests. Plot the modulus as a function of the measured pavement temperature and estimate the AC modulus at 70 ºF (21 ºC) from the graph.

- For new LTPP experiments or road tests, FWD tests should be conducted before and after treatment.

State Data

The following recommendations were developed regarding State data:

- The dual pavement condition rating systems should be adopted by FHWA, AASHTO, and the State transportation departments. This would unify the analyses of pavement performance nationwide.

- The benchmark benefit values of the five treatment types included in this study should be expanded to include additional pavement treatments.

- The treatment benefits described in this report that were found using the LTPP data, could be used as benchmark values for the national practice. State transportation departments could use such data to do the following:

    - Gauge the effectiveness of their current practices using similar analyses.

    - Conduct lifecycle cost analyses of various treatment alternatives to optimize the pavement rehabilitation and treatment strategy at the network level.

Future Studies

The following actions are recommended regarding future studies:

- Initiate studies to produce a national catalog regarding the service life of various treatment types as a function of pretreatment pavement conditions and distress. The studies should be based mainly on the LTPP data and applied to some State data.

- Initiate studies to establish automated data collection processes, quality control procedures, and data storage to minimize the impact of subjective factors and human errors on pavement conditions and distress.

- Initiate studies to scrutinize the newly developed self-powered wireless Pico sensors (10^{12} mm) that can be embedded in pavement and transportation infrastructures to measure their performance in terms of cracking and induced stresses and strains.

- Explore the accuracy of the newly developed chemical sensors that can be included in concrete and asphalt mixes to measure future pavement conditions and distresses. This would eliminate the need for traditional data collection.

- Develop short course materials with examples to train engineers and staff of State transportation departments to use the MATLAB® computer program for the analyses of pavement condition and distress and to emphasize the benefits of the LTPP Program.

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