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THE USE OF LONG TERM PAVEMENT PERFORMANCE DATA FOR QUANTIFYING THE RELATIVE EFFECTS OF STRUCTURAL AND ENVIRONMENTAL FACTORS ON THE RESPONSE AND PERFORMANCE OF NEW FLEXIBLE PAVEMENTS

presented by

Syed Waqar Haider

has been accepted towards fulfillment
of the requirements for the

Ph.D. degree in Civil Engineering

Major Professor's Signature

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THE USE OF LONG TERM PAVEMENT PERFORMANCE DATA FOR QUANTIFYING THE RELATIVE EFFECTS OF STRUCTURAL AND ENVIRONMENTAL FACTORS ON THE RESPONSE AND PERFORMANCE OF NEW FLEXIBLE PAVEMENTS

By

Syed Waqar Haider

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ABSTRACT

THE USE OF LONG TERM PAVEMENT PERFORMANCE DATA FOR QUANTIFYING THE RELATIVE EFFECTS OF STRUCTURAL AND ENVIRONMENTAL FACTORS ON THE RESPONSE AND PERFORMANCE OF NEW FLEXIBLE PAVEMENTS

By

Syed Waqar Haider

Considerable progress has been made over the past 50 years in the field of pavement engineering. While much has been learnt about designing and maintaining pavements economically, still there exists a need for improving existing procedures to meet increasing infrastructure needs with limited resources. At present, highway agencies lack adequate information on the influence of pavement drainage on the performance of flexible pavements. There is very limited experimental field data to quantify its influence or effect on the pavement performance. The interactions of the structural factors with other variables such as climate are not well understood and therefore, still needs to be explored in order to improve relationships between pavement performance and response. As a result, controlled field experiments are necessary to answer questions which are currently not explained by the theoretical modeling.

This research documents and presents the results of relative influence of design and construction features on the response and performance of in-service new flexible pavements, included in SPS-1 and SPS-8 experiments of the Long Term Performance Pavement (LTPP) program.

In summary, for pavements in the SPS-1 experiment, base type seems to be the most critical design factor for fatigue cracking, roughness (IRI), and longitudinal cracking-WP. This is not to say that the effect of HMA surface thickness is not

significant. In fact, the effect of base type should be interpreted in light of the fact that a dense graded asphalt treated base effectively means thicker HMA layer. Drainage and base type, when combined also play an important role in improving flexible pavement performance, especially in terms of fatigue and longitudinal cracking. Base thickness has secondary effects on performance, particularly in the case of roughness and rutting.

Subgrade soil type seems to be playing an important role in flexible pavement performance. In general, pavements built on fine-grained soils have shown worst performance, especially in the case of roughness. Also, climate is a critical factor in determining flexible pavement performance. Longitudinal cracking-WP and transverse cracking seems to be associated with Wet Freeze environment, while longitudinal cracking-NWP seems to be the dominant in "freeze" climate.

On average, for pavements in the SPS-8 experiment, those located in WF zone have more fatigue cracking, longitudinal cracking-NWP, and roughness than pavements in other climates. Also, in general, pavements constructed on "active" subgrade (frost susceptible or expansive) soils have more longitudinal cracking-NWP, transverse cracking, and fatigue cracking than pavements on "non-active" soils. Pavements located in "wet" climate, on average, have higher change in IRI than those in "dry" climate. Furthermore, pavements located in WF zone and those built on active soils have higher change in IRI.

Although most of the findings from this research support the existing understanding of pavement performance, the methodology in this study provides a systematic outline of the interactions between design and site factors as well as new insights on various design options.

DEDICATION

This work is dedicated to my parents, late younger brother Syed Ali Haider, late uncle Syed Nazir Hussain Mashadi and my family, without whom this work would not have been achievable. Their emotional support and prayers consistently provided the motivation and inspiration to achieve this goal.

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CHAPTER 1 - INTRODUCTION

1.1 INTRODUCTION

Considerable progress has been made over the past 50 years in the field of pavement engineering towards providing economical pavements. Pavement design and analysis methods, construction practices, and material specifications have been developed and refined as tools to facilitate engineers in the design and management of highway infrastructure. While much has been learned in recent years, still there is a demand from highway agencies to improve existing procedures and tools for effective management of pavement networks. These needs from the highway agencies to explore more innovative design procedures will help them in meeting the ever increasing highway infrastructure demands within limited resources.

1.2 PROBLEM STATEMENT

At present, highway agencies lack adequate information on the influence of pavement drainage on the performance of flexible pavements. There is very limited experimental field data to quantify its influence or effect on pavement performance. Currently, the influence of drainage is incorporated into the AASHTO Design Guide through a modified structural number concept using subjective judgment for the selection of the layer modified coefficients. As stated in Volume II, Appendix DD of the AASHTO Design Guide (1986), these coefficients were developed through a theoretical approach because very limited field data existed at that time [1].

The modeling of the pavement structure as an elastic layer system has been the main focus of the research in the past two decades. The pavement layers are characterized by their thickness, stiffness and Poisson's ratio with stiffness (modulus) being the most important parameter. The information on interactions of these key

factors with other variables such as climate is not well understood and therefore, still needs to be explored in order to improve relationships between pavement performance and response. Therefore, controlled field experiments are necessary to answer the following questions [1]:

- "To what extent does the influence of pavement design on pavement performance vary from wet to dry climatic zone and what is the relative importance in each zone?"
- "To what extent does asphalt concrete thickness, type or thickness of the base layer influence pavement performance and what is the relative importance of each? How do environmental factors influence the relative importance?"

To address the above questions, the Long Term Pavement Performance (LTPP) Specific Pavement Studies (SPS) 1 and 8 experiments were designed to provide information on the relative merits of different design features in newly constructed flexible pavements for achieving different levels of performance under heavy traffic (SPS-1) as well as in the absence of heavy traffic (SPS-8). Typical features include hot mix asphalt (HMA) surface layer thickness, base type and thickness, and drainage (presence or absence thereof). In addition to this, instrumented sections were included in the SPS monitoring sites located in Ohio (SPS-1).

For specific site conditions (e.g., traffic level, climatic conditions, and subgrade type), the response and performance of flexible pavements will depend not only on pavement layer thicknesses and material properties, but also on other design and construction features (e.g., presence of in-pavement drainage and base type, etc.).

Recent research based on limited analyses has documented the effects of these features on pavement response (as measured by deflection and strain) and the contributions of these features to achieving different levels of performance (as measured by type and extent of distress or smoothness).

The importance of this experiment is highlighted by its ability to evaluate the interaction of drainage, structural parameters, subgrade type and climatic factors on pavement performance in a controlled manner. The data available from the LTPP studies, including instrumented SPS-1 test sections in Ohio are expected to provide the information needed for a more rigorous analysis to enhance understanding of the effects of these features on flexible pavement response and performance and to develop well-supported conclusions regarding their influence.

There is therefore a need to determine the effects of design and construction features on flexible pavement performance and response, and to establish their relative importance. This research should provide guidance for identifying appropriate features for different pavement types, preliminary information on the relationship between pavement response and performance, and recommendations for improving data collection activities.

1.3 RESERACH OBJECTIVES

The purpose of this research is to more precisely determine the relative influence of specific factors that affect the performance of flexible pavements. The factors addressed in this study include drainage, base type and thickness, and asphalt surface thickness. The study goals also include a determination of the influence of environmental region and soil type on these factors. Accomplishing these objectives will provide substantial improved tools for use in the design and construction of new and reconstructed flexible pavements. The specific objectives for this research are:

1) To determine, for specific site conditions (climate and subgrade type) the contributions of design and construction features to achieving different levels of performance.
2) To determine, for specific site conditions, the effects of design and construction features on pavement response (deflections and deflection basin parameters), and
3) To provide information on the apparent relationship between pavement response and performance.

The research is limited to new (i.e., non-rehabilitated) flexible pavements. The research is based on the data available from the LTPP experiments SPS-1 (strategic study of structural factors for flexible pavements), and SPS-8 (a study of environmental factors in absence of heavy loads). The analysis is limited to using the data available in the LTPP Information Management System (IMS) database classified as "Level E" (Release 17) and response data available from LTPP instrumented test sections.

1.4 SCOPE OF STUDY

The scope of the study included the review and analysis of LTPP data (DataPave) pertaining to the SPS-1and SPS-8 experiments. All relevant data for these experiments were obtained and reviewed from Release 17.0 (January 2004) of the IMS database. After the data were obtained, a relational database was prepared for analyses of the study. Based on the availability of the data and the extent (and occurrence) of distresses, appropriate analyses (site-level and overall analyses) were conducted to fulfill the objectives of the study. At the site-level analysis each site is considered separately and the consistency of the effects across the sites is studied. The

overall analyses are conducted, using the wealth of data from all the experiment sections in order to draw broad conclusions. Attempts are also made to verify apparent relationships between response (Falling Weight Deflectometer data and Dynamic Load Response data) and performance of the test sections in the SPS-1 experiment. Based on all analyses (site-level and overall), the effects of design and construction features on pavement performance and response are studied. Finally, recommendations for future research and data collection are given.

1.5 REPORT ORGANIZATION

The report is divided into eight chapters including this introductory chapter. Chapter 2 highlights the summary of findings from the LTPP related studies, which can be useful for this research. The detailed descriptions of the experiment designs and the current status of SPS-1 and SPS-8 experiments are presented in Chapter 3. A summary of data availability and extent and occurrence of distresses is presented for the two experiments in Chapter 4. Based on the extent and occurrence of distresses, different methods of analysis were employed. A brief description of each of these analytical methods and methodology used for the research are detailed in Chapter 5. Chapters 6 and 7 are summaries of results from all analyses conducted on SPS-1 and SPS-8 data, respectively. A synopsis of the salient findings from all the analyses, for each experiment, is presented in Chapter 8. Finally, based on the experience with LTPP data gained from this research, recommendations for future data collection and research are presented.

CHAPTER 2 - LITERATURE REVIW

2.1 GENERAL

Pavements are designed to fail after sustaining the anticipated design traffic loads. The damage due to the repetitions of heavy axle loads accumulates over time and eventually the pavement structure needs to be rehabilitated after a certain threshold for performance is exceeded. The performance of a particular pavement section depends on the interaction of many factors including structural design parameters, material properties, the operating environment and most importantly the amount and magnitude of traffic loads. Thus a pavement structure should be considered as a system, where different components play their individual roles.

Traditionally, the performance of various pavement materials can be investigated through the use of small-scale laboratory tests, observing field performance or both. Small-scale laboratory investigations may help in measuring basic material properties and understand the material behavior under controlled conditions; however, this might not be sufficient to capture the actual in-situ material behavior because of different boundary conditions in the field and the interaction of many factors at the same time. Therefore, researchers have also used full-scale accelerated testing to simulate the field boundary conditions and estimate the long term performance of pavements in a short period of time. However, the results of accelerated full-scale tests are also dependent on how close this simulation is to the actual field conditions in terms of materials, environment and loading conditions. On the other hand observing field test results from in-service pavements provide a good indication of the short and long term performance potential

[1]. This approach is more realistic, but demands long term monitoring and may become very expensive.

Many field tests [2] have been conducted in the past to monitor long term pavement performance to help develop a pavement design procedure. For example the design procedure recommended by the American Association of State Highway and Transportation Officials (AASHTO) is based upon the results of the extensive AASHO Road Test conducted in Ottawa, Illinois, in the late 1950's and early 1960's [3]. However, most of these field tests were conducted several decades ago and had various limitations such as short-term monitoring of pavements (1 to 2 years), limited climatic conditions, limited materials and most importantly limited truck loads and axle configurations. In order to address all these limitations the US Congress approved the Strategic Highway Research Program (SHRP) in 1987 [2]. This included the Long Term Pavement Performance (LTPP) program, described next.

2.2 BACKGROUND

Understanding "why" some pavements perform better than others is key to building and maintaining a cost-effective highway system. In 1987, the Long-Term Pavement Performance (LTPP) program — a comprehensive 20-year study of in-service pavements — began a series of rigorous long-term field experiments monitoring more than 2,400 asphalt and portland cement concrete pavement test sections across the U.S. and Canada [4]. Established as part of the Strategic Highway Research Program (SHRP) and now managed by the Federal Highway Administration (FHWA), LTPP was designed as a partnership between FHWA and the States and Provinces. LTPP's goal is to help the

States and Provinces make decisions that will lead to better performing and more cost-effective pavements.

In the LTPP program various experiment designs were planned to conduct research on a variety of factors affecting pavement performance. The experiment designs were divided into two main categories: general pavement studies (GPS) — focusing on the most commonly used pavement designs (existing pavements), and specific pavement studies (SPS) — to study certain pavement engineering factors (new pavements). Within these two categories, further experiment designs were planned to study different types of pavements, etc. Two of the experiments in the SPS categories for flexible pavements will be the focus of this research. These are the SPS-1 and SPS-8 experiments, which are described in Chapter 3. In this Chapter, the previous research conducted using LTPP data, especially SPS-1 and SPS-8 experiments is briefly presented.

2.3 SUMMARY OF RELATED LTPP STUDIES

This section summarizes the findings from the literature review of research reports that deal with pavement performance in the field. The review included FHWA/LTPP reports, NCHRP reports as well as additional literature, and was focused on research that has identified factors affecting pavement response and performance including roughness. The most relevant reports were found to be those from studies addressing the Long Term Pavement Performance (LTPP) experiments.

The information obtained from the literature review has been used to identify various factors that have been shown in past research as having an effect on response and performance progression. The brief findings from these studies are presented next.

2.3.1 Factors Affecting Flexible Pavement Performance

A study [5] entitled "Structural Factors for Flexible Pavements" was conducted using SPS-1 data. The summary of findings from this preliminary study is given below:

Layer Thickness

- The SPS-1 test sections with thick (178-mm (7-inch)) AC surface layers appear to be smoother and develop less fatigue cracking than those sections with thin (102-mm (4-inch)) surface layers. This confirms a similar finding from earlier studies.
- In the SPS-1 experiment, AC surface thickness and the age of the project appear to influence the amount of fatigue cracking that occurs. The test sections that are younger and have thicker AC surface layers have the least fatigue cracking.

Base Layer

- Hot-mix asphalt (HMA) pavements with unbound aggregate base layers show greater rut depths than those sections with asphalt-treated base layers. This suggests that a portion of the rutting measured at the surface is a result of permanent deformations in the unbound aggregate base layer, which is consistent with a previous finding from analysis of the GPS test sections.
- The HMA pavements with unbound aggregate layers have slightly more fatigue cracking and higher IRI values than those sections with asphalt-treated base layers.
- The test sections with coarse-grained soils, asphalt-treated base layers, permeable base layers, thicker bases, and thicker HMA layers were found to be smoother.
- The test sections with permeable asphalt-treated base layers exhibit more fatigue cracking than those without permeable base layers.

Subgrade

- HMA pavements built over coarse-grained subgrade soils are smoother than pavements built over fine-grained subgrade soils. This is consistent with the finding in the SPS-2 JPCP: A stiffer foundation contributes to smoother pavements.

- HMA pavements built over coarse-grained subgrade soils and in a no-freeze climate are smoother and stay smoother over a longer period of time than do those built over fine-grained subgrade soils in a freeze climate. HMA pavements built over fine-grained sub-grades and in a wet-freeze climate are substantially rougher than those built in other climates.

- HMA pavements built over fine-grained subgrade soils have more fatigue cracking than those projects built over coarse-grained subgrade soils.

- Subgrade soil type and, to a lesser degree, age are important to the amount of transverse cracking measured at each site. More transverse cracking has occurred on the HMA pavements built on fine-grained soils than on pavements built on coarse-grained soils.

Two studies[6, 7] were conducted to investigate the effects of sub-drainage on the performance of asphalt and concrete pavements. The following is a summary of their findings for asphalt pavements:

- Based on 7 years (on average) of SPS-1 data, those HMA sections built on permeable bases without edge drains were found to perform better than those with edge drains.

- The ranking of performance in terms of IRI and cracking for various base types with all other design features matched is from poor to good performance: un-drained dense-graded aggregate bases, drained permeable asphalt-treated bases, and un-drained dense-graded asphalt-treated bases.
- The results in terms of rutting for the above three sub-drainage designs were inconclusive.

A fourth study was conducted on the environmental effects in the absence of heavy loads using SPS-8 data [8], however, the final report of this study is not published. The SPS-8 experiment can be considered as an extension of SPS-1 (new flexible pavements) and SPS-2 (new rigid pavements) with limited traffic effects. The preliminary conclusions from this research are summarized below:

Temperature and Precipitation

- For SPS-8, asphalt concrete (AC) sections, the most prevalent early distress is longitudinal cracking outside the wheel path. The distress is most commonly observed for sections built in the wet-freeze climates and for sections on an active subgrade (frost-susceptible or swelling soils due to freeze-thaw cycles). Fatigue, longitudinal cracking in the wheel path, and transverse cracking are present on just a few sections. The mean rut depths for all AC sections are below 6 millimeters (mm) (0.24 inches).

Subgrade

- Pavements (flexible or rigid) constructed on active subgrade have the highest mean initial International Roughness Index (IRI) values and slopes (the smoothness rate of change over time), followed by pavements constructed on fine

subgrade and then pavements constructed on coarse sub-grade. The data support a similar finding from previous studies that a good working platform (specifically, stabilized base and granular subgrade or embankment) contributed to a smoother pavement construction.

- Initial IRI values for the SPS-8 test sections show that flexible pavements were constructed to be smoother than the rigid pavements. The analysis of IRI slopes indicates that the subgrade is the most important factor for flexible sections, while precipitation appears to be the most important factor for rigid sections.

Pavement Type and Layer Thickness

- The SPS-8 flexible pavements with thin (102-mm (4-inch)) AC surface layers were found to be smoother than the sections with thick (178-mm (7-inch)) AC layers.

Comparisons of SPS-1, -2, and -8 Test Sections

- As expected, traffic loading is much heavier on SPS-1 and SPS-2 than on SPS-8 sites. As of June 2001, the estimated accumulated ESALs on SPS-1 sites was about 1.46 million, compared to 0.043 million ESALs on SPS-8 AC sites.

- The average IRI slopes (the smoothness rate of change over time) for both SPS-1 and SPS-2 sections are much higher than for the corresponding SPS-8 sections. The variability of mean IRI slopes is higher for PCC than for AC sections.

- Overall, the much more heavily loaded SPS-1 and SPS-2 sections exhibit higher amounts of load-related distresses. Such distresses include AC rutting, AC fatigue cracking, JPCP joint faulting, and JPCP transverse cracking. However, the non-

load-related distresses including AC transverse cracking and non-wheel path longitudinal cracking are similar for SPS-1, SPS-2, and SPS-8.

Another study [9]was conducted using SPS-1 data to identify the factors affecting pavement smoothness. A summary of the main findings from this research is given below:

- A significant difference between early age IRI of pavements placed on DGAB and ATB was observed. No significant difference in early age IRI was obtained on pavements placed on PATB when compared to the other two base types.
- The SPS-1 projects that showed the highest increase in IRI were located in Kansas, Iowa and Ohio, reasons being fatigue and transverse cracking for KS (20), transverse and longitudinal cracking in the wheel path (WP) for IA (19), and rutting for OH (39) site. Some of the test sections in TX (48) are showing higher increase in IRI of over 10% within an approximate 6-month period, which is attributed to rutting.
- Although the pavements in IA (19), KS (20), and OH (39) achieved a smooth pavement initially, many sections, including very thick sections had high increases in roughness during the initial life of the pavement. Achieving a smooth pavement initially does not guarantee that it will remain smooth even during the initial life.
- Mix design problems in the AC, inadequate preparation of the subgrade prior to placing the pavement, or other construction problems can cause smoothly built pavements to have higher increase in roughness within a short time period.

Table 2-1 summarizes the effects of the structural and site factors on flexible pavement performance from other relevant research conducted using the LTPP data.

Table 2-1 Summary of Factors Affecting the Response and Performance of Flexible Pavements

Factor	Variable	Effect on Pavement Performance and Response	References
Site Conditions	Traffic Loading	Higher number of heavy axles repetitions (cumulative ESALs) was found to be significant in increasing (i) rutting, (ii) fatigue cracking and (iii) roughness (IRI). However, cumulative ESALs were not found to be significantly different between good and poorly performing AC pavements. Nevertheless, this does not mean that ESALs do not cause fatigue cracking, but it means that additional other variables can be responsible for the big differences in fatigue performance, so that a given pavement with the same or lower ESALs can have significantly more fatigue cracking due to design and construction factors. Additionally, increased ESALs may contribute to increased transverse and reflection cracking.	[9, 10]
	Climatic Zones	Major climatic variables include moisture and temperature. Moisture in LTPP is characterized by the number of wet days and location of site (wet or dry). While temperature in LTPP is characterized by location of site (non-freeze versus freeze), number of freeze-thaw cycles, annual mean temperature and number of days above 32°C. High temperatures are expected to favor rutting, while all cracking are less expected in warm climates. Colder climates experience more transverse cracking and have a higher growth rate of roughness. Wet climates tend to experience more rutting, fatigue, transverse cracking and perhaps roughness. Generally, higher roughness and fatigue cracking were found in areas of higher precipitation. In freezing environments, pavements in areas that had a higher freeze index or a high number of freeze-thaw cycles had higher IRI values. Sections in very hot environments generally had higher IRI value, however, better performance was indicated for sections that were subjected to higher precipitation as well.	[9-11]
	Subgrade Properties	The effect of clayey sub-grades are not clear for rutting, fatigue cracking and transverse cracking, but may be expected to increase the potential for roughness. Lower subgrade compaction percentage values result in more fatigue cracking. IRI growth in GPS 1 sections over fine grained soils was strongly related to Atterberg Limits and percentage of subgrade passing #200 sieve.	[9-11]

Table 2-1 Summary of Factors Affecting the Response and Performance of Flexible Pavements (continued …)

Factor	Variable	Effect on Pavement Performance and Response	References
Site Conditions	Age	Age was found to be the most significant factor for transverse cracking.	[10]
Design Features	Asphalt Thickness	Using thicker asphalt concrete layers or increasing the overall pavement structural stiffness may be expected to decrease rutting, fatigue cracking and roughness and would probably help in decreasing transverse cracking assuming that mixture design and construction are adequate.	[10]
	Base Thickness	Increasing the base thickness may lead to a decrease in rutting, as long as the material properties and placement are appropriate. Effects on other distress are unclear. The existence of treated base in the GPS 2 sections may have been responsible for their superior performance.	[12] [10]
	Pavement Thickness (SN)	In SPS 1 projects, sections that had lower SNs had higher rate of increase of IRI when compared with sections that had higher SNs.	[11]
Construction Features	Compaction	It has generally been believed from past experience that increasing compaction for granular base material was generally good for performance, as long as sufficient drainage was not precluded. However, for GPS sections increased compaction was found to increase roughness and some distresses. There is no engineering explanation for this trend.	[10]
	Drainage	The benefits of positive drainage features were not substantiated in GPS 1 and 2 experiments. This may be related to the assumption that the positive drainage system is functioning properly. Therefore, sites with positive drainage features should be inspected to confirm that they are, in fact, functioning.	[13]
Material Properties	Air Voids in AC Mix	Air voids must be controlled through mixture design and proper compaction. Too low air voids after compaction by traffic often result in deeper ruts.	[10]
	Asphalt Viscosity	There are indications from studies of fatigue and transverse cracking that use of high-viscosity asphalt will increase these distresses. Viscosity was not found to be significant for rutting. However, this may be the result of using softer asphalts in the colder climates.	[10]

Table 2-1 Summary of Factors Affecting the Response and Performance of Flexible Pavements (continued...)

Factor	Variable	Effect on Pavement Performance and Response	References
Material Properties	AC Modulus	Higher resilient modulus of the AC layer would result in lower strains in the granular base layer, and thus reducing the rutting potential. Sections with higher resilient modulus tend to have less fatigue cracking.	[10]
	Aggregate gradation	Pavements with good performance in rutting had more % passing sieves 9.52mm, 4.75mm, 0.425mm, 0.18mm and 0.075mm than those with poor rutting performance. However, this good performance might be due to the influence of other factors such as higher resilient modulus. Similar results were found for fatigue cracking.	[10]
	Base Modulus	Thick weak bases will exhibit more fatigue cracking than thinner but stronger bases. Effect of the subbase resilient modulus on the rutting potential was not found to be significant in GPS sections.	[10]
	Base moisture content	The moisture content of the granular base was found to be proportional to rutting. Sections with lower base moisture content had a better performance.	[10]
	Base type	Pavements with cement treated base had lower rut depths than those on unbound granular bases. Similar results were found for fatigue cracking.	[10]

2.3.2 Relationship between Pavement Response and Performance

In the NCHRP Report No. 10-48, "Assessing Pavement Layer Condition using Deflection Data"[14], the performance assessment of the pavement layers using deflection data has been investigated. Some results of this study are enumerated below: "

- For full depth and aggregate base pavements, E_{ac}[1] can be used as an indicator to detect cracking and stripping in the AC layer. BDI[2] and ε_{abc}[3] were found to be good condition indicators for base layer condition, while BCI[4], ε_{sg}[5], SSR[6] and E_{sg}[7] appeared to be good condition indicators for the subgrade. For intact pavements, the pavement overall fatigue cracking and rutting potentials are mainly controlled by ε_{ac}, ε_{abc} and ε_{sg}.
- Temperature adjustment is an important procedure in assessing the condition of asphalt-surfaced pavements. Temperature adjusted indicators were found to be able to predict fatigue cracking and rutting potentials of full depth and aggregate base pavements.
- For full depth and aggregate base pavements, the analysis from both synthetic data and field data showed that SCI[8] can be used to predict the AC modulus. Also, high correlation was found between BDI and ε_{ac}.
- The deflection basin parameter BDI can be used to assess upper layer condition in cement treated base (CTB) pavements.

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[1] AC modulus
[2] $BDI=D_{12}-D_{24}$ (Base Damage Index)
[3] Compressive strain at top of base layer
[4] $BCI=D_{24}-D_{36}$ (Base Curvature Index)
[5] Compressive strain at top of subgrade
[6] $SSR=\sigma_d/q_u$ (Stress Parameter for rutting potential of subgrade)
[7] Resilient modulus for subgrade
[8] $SCI=D_0-D_{12}$ (Surface Curvature Index)

- Deflection values from the sensor four feet from the FWD load center (D48) can be used to estimate subgrade condition in CTB pavements.

A recent study [15] on evaluating the feasibility of using FWD deflection data to characterize the pavement construction quality using SPS-1 data concluded that "FWD can be effectively used to assist in the characterization of pavement quality for new or reconstructed pavements during construction". Furthermore, "quality" can be directly equated to stiffness or modulus, and to other deflection-based parameters. It is therefore expected that these parameters should correlate to the future performance of pavements.

Equation (2-1) can be used to calculate the area under the deflection basin using the first four sensors equally spaced at 12 inches from each other [15, 16].

$$Area = 6*\left[1+2\left(\frac{d_1}{d_o}\right)+2\left(\frac{d_2}{d_o}\right)+\left(\frac{d_3}{d_o}\right)\right] \qquad (2\text{-}1)$$

Where;
Area = *The "Area" beneath the first three 3 feet (914 mm) of the deflection basin;*
d_0 = *FWD deflection measured at the center of load plate;*
d_1 = *FWD deflection measured at 12 inches from the center of plate;*
d_2 = *FWD deflection measured at 24 inches from the center of plate;*
d_3 = *FWD deflection measured at 36 inches from the center of plate.*

Equation (2-1) has been used for a load plate diameter of 12 inches. A maximum value of 36 is possible for "Area" when all the deflections are the same in Equation (2-1). This will represent a very stiff pavement (extreme value). It was found using the forward analysis[15] that when all the layers in a flexible pavement were assumed to have the same stiffness/modulus, the "Area" term is always equal to 11.037. Therefore, it was concluded that if FWD deflection measurements carried out in the field result in an "Area" value close to 11.037, then the effective modulus or stiffness of the upper layer is

identical to that of the underlying layer(s). Hence, this particular value of "Area" can be used as a cutoff in order to ascertain whether the upper layer has a higher stiffness than the underlying layer(s).

The surface deflection at the center line of the circular plate may be calculated by using Boussinesq's solution for semi-infinite half space by using equation (2-2) [17, 18].

$$E_o = \frac{2\left(1-\nu^2\right)\sigma_o * a}{d_o} \tag{2-2}$$

Where;

E_o = *The surface modulus (psi);*
σ_o = *Peak pressure of FWD impact load under loading plate;*
d_o = *Peak surface deflection at center of load plate (inches);*
a = *radius of FWD loading plate;*
ν = *Poisson's ratio of the semi-infinite half space*
"2" in above equation is a factor for uniform stress distribution [18]

The equation (2-2) can be simplified to equation (2-3) by assuming a Poisson's ratio of 0.5 for the half space.

$$E_o = \frac{(1.5 * a * \sigma_o)}{d_o} \tag{2-3}$$

The problem arising from an unknown stress distribution for equation (2-2) and (2-3) may be solved by measuring the deflections at different distances from the center of the load [17]. Measuring the deflections at several distances (as is the case with FWD) also serves as a check on the assumption that the subgrade is a linear elastic semi-infinite space. In that case the moduli calculated at different distances must be identical, otherwise the subgrade is either non-linear or it consists of different layers.

$$E_i = \frac{\left(1-\nu^2\right)\sigma_o * a^2}{r_i * d_i} \tag{2-4}$$

Where;

E_i = *The surface modulus (psi);*

σ_o = *Peak pressure of FWD impact load under loading plate;*

d_i =*Surface deflection at distance "i" from center of load plate (inches);*

a =*radius of FWD loading plate;*

ν =*Poisson's ration of the semi-infinite half space*

r_i =*Distance from the center of the load where d_i is measured.*

Equation (2-4) can be simplified to Equation (2-5).

$$E_i = \frac{\left(0.84 * \sigma_o * a^2\right)}{\left(r_i * d_i\right)} \tag{2-5}$$

The E_o value calculated using Equation (2-3) represents a composite or effective stiffness of *all* the layers under the FWD loading plate. Based on Equations (2-1), (2-3) and (2-5), the following equations were derived [15].

$$AF = \left[\frac{\left(k_2 - 1\right)}{\left(k_2 - \left(\frac{Area}{k_1}\right)\right)}\right]^2 \tag{2-6}$$

AF = *Area Factor, i.e. the "improvement" in AREA from 11.037 squared;*

k_1 = *11.037 (the Area when the stiffness of the upper layers is the same as that of the lower layers;*

k_2 = *3.262 (Maximum possible improvement in Area = 36/11.037).*

Equation (7) can effectively be used to approximate the relative stiffness of the upper (bound) layers in the pavement cross section.

$$ES = \left[E_O * AF * (k_3)^{\left\{\left(\frac{1}{AF}\right)-1\right\}} \right] \quad (2\text{-}7)$$

Where;

ES = *Effective Stiffness of upper (bound) layers;*

E_o = *as defined by Equations (3);*

AF = *as defined by Equation (6);*

k_3 = *thickness of upper layer/load plate diameter* = $\frac{h_1}{2 * a}$

2.4 REVIEW OF STATISTICAL METHODS

Previous studies that dealt with LTPP data (GPS and SPS experiments) quantified performance based on engineering criteria and expert opinion. For example, the engineering criteria may include the rate of growth, severity levels and impact of distress on the functionality of the pavement.

Several statistical methods can and have been employed to establish performance criteria, to study the effect of design and construction features on pavement response and performance. The statistical methods range from trend plotting to complex multivariate analysis. A detailed statistical methodology for LTPP data analysis for prediction modeling (performance models) has been laid out in SHRP-P-393 report [19].

The simpler statistical methods include *Univariate and Bivariate analysis* of data. These methods include the determination of data statistics such as mean, standard deviation and data frequencies. Simple histograms and box plots can also be generated to determine data distribution. These methods also allow for determining the degree of dependence between variables. The results of such an analysis can be graphically illustrated. Such an analysis can also provide summary statistics such as the coefficient of correlation. Bivariate analysis can also assist in identifying outliers.

Hypothesis testing is a tool that allows one to determine if a specific numerical value is equal to, greater or less than a specified number, or if the means of two sets of data are equal or significantly different. The mean response values for each group can be determined and then compared using hypothesis testing for a certain level of confidence. If this relationship is significant then the impact of the given factor on the response should be further investigated.

Some of the *multivariate* statistical methods include:

- Analysis of Variance (ANOVA)
- Regression Analysis
- Principal Component Analysis
- Discriminant Analysis

The *ANOVA* is a tool that allows for better understanding of how the independent variables influence the dependent variable. For example, the effects of different base types or asphalt thickness on the amount of cracking (dependent variable) are required candidates for such an analysis.

Regression analyses attempt to explain some dependent variable, y, in terms of many independent (explanatory) variables, x's. The model (equation) can be either linear or non-linear, and with actual, transformed, or interaction clusters of variables. The model coefficients can be estimated using best (least squares) fitting techniques.

Principal component analysis is a multivariate procedure which allows for simultaneously examining the data for both influential observations and collinearity. In essence, it is a diagnostic/explanatory data analysis tool used in modeling to identify the relationships between variables, and can help in deciding the reduction of the number of

variables (independent) so that these may be lumped into a fewer number of factors/variables which explain the greater part of the variance in the data.

Discriminate analysis allows one to distinguish between two or more groups of data. This is done by identifying which variables are significant in classifying the data into various groups. The procedure for predicting membership is to initially analyze pertinent variables where group membership is already known. For example, groups of observations can include one group of pavements with poor performance and the second group with good performance. The method allows for determining which variables discriminate between poorly and good performing pavements.

Cluster analysis is in some way similar to factor analysis as both of these statistical techniques take a larger number of cases or variables and reduce them to a smaller number of factors (or clusters) based on some sort of similarity those members within a group share. The factor analysis is used to reduce a larger number of variables to a smaller number of factors that describe these variables. On the other hand cluster analysis is more typically used to combine cases into groups. However, the statistical procedures underlying each type of analysis and interpretation of the output are different.

Logistic regression is used often in the case where the outcome variable is discrete (dichotomous or multi-nominal). The difference between logistic and linear regression is reflected both in the choice of a parametric model and in the assumptions. This method is based on the maximum likelihood method for determining the parameters of interest. The interpretation of effects for various levels of the categorical variables (independent) is very convenient in terms of the odds ratio when this type of model is

used. Logistic regression models are also very useful for discrimination analysis (of various groups) when categorical variables are used as independent variables.

Survival Analysis can be used to study longitudinal data when survival time data that measures the time to a certain event (such as failure, death or some threshold value of distress) is the focus of interest. These times are subjected to random variations and like any random variable, can be represented by a distribution. The distribution of survival times is usually characterized by three functions: the survivorship function, probability density function and hazard function. These three functions are interdependent on each other, i.e., if one of them is known the other two can be derived. The non-parametric methods developed for survival analysis (Cox's regression) are capable of handling discrete and continuous independent variables effects in the multivariate survival function. Thus this technique has the potential for analyzing pavement time to failure given various design and construction factors.

The detailed discussion and description of the statistical methods used in this research is presented in Chapter 5 under the research methodology.

CHAPTER 3 - DESCRIPTION OF SPS-1 AND SPS-8 EXPERIMENTS

3.1 INTRODUCTION

This chapter describes Specific Pavement Studies (SPS) experiments 1 and 8, in terms of their respective goals, experimental designs, and associated factors (design and construction). A separate section is included for the Dynamic Load Response (DLR) experiment which constitutes a subset of the SPS-1 and -2 experiments.

3.2 STRATEGIC STUDY OF STRUCTURAL FACTORS FOR FLEXIBLE PAVEMENTS—SPS-1

The effects of drainage, pavement structural parameters (thickness and stiffness of asphalt concrete surface, and strength and thickness of base layers), and climatic factors could be rationally accounted for in current design procedures for flexible pavements. However, some of these effects were developed through theoretical approaches because of the limited available field data. The in-service tests in this experiment will help quantify the influence of these parameters on pavement performance and improve current design procedures. Consequently, the experiment will help highway agencies in selecting more economical design options for new and reconstructed flexible pavements.

The SPS-1 experiment is focused on the strategic study of structural factors for flexible pavements, and was intended to study the effect of specific design and construction features on pavement response and performance. As the test sections in the experiment are monitored since inception, the experiment provides an opportunity to determine the relative influence of the key pavement design and construction elements that affect pavement performance.

3.2.1 Experiment Design

Generally, a scientific study can be categorized as either an experimental study or an observational study. This distinction is important because experimental studies provide a much firmer basis for the establishment of cause-and-effect relationships one or more explanatory factors and a response variable than do observational studies. With the later one can establish only associations between the explanatory factors and response variables, but not the causation. The experimental studies allow the researcher to control effects of various treatments and difference in response between the treatments is only attributed to the factors in the study.

Randomization of treatments to the experimental units is employed to neutralize the effects of any unknown factors between the experimental units. However, if the randomization is not possible then the difference in the experimental units can be handled by using co-variates (variables effecting response other than experimental factors) in order to detect the pure effects of factors under study.

The SPS-1 trial is an experimental study, which aimed at finding the effects of strategic structural and site factors on various pavement performances (cracking, rutting and roughness) and response (deflections and strains). The site factors include: climatic region (wet-freeze and wet-no-freeze, dry-freeze and dry-no-freeze), subgrade soil (fine- and coarse-grained), and traffic rate (as a covariate) on pavement sections incorporating different levels of structural factors. The structural factors include:

- drainage (presence or lack of it),
- asphalt concrete (AC) surface thickness – 4" (102 mm) and 7" (178 mm),
- base type – dense-graded untreated aggregate base (DGAB), dense-graded asphalt-treated base (ATB) and a combination of both,
- base thickness – 8" (203 mm) and 12" (305 mm) for un-drained sections; and 8" (203 mm), 12" (305 mm) and 16" (406 mm) for drained sections.

The study design stipulates a traffic load level in excess of 100,000 Equivalent Single Axle Loads (ESALs) per year for the study lane [1].

The experiment includes four structural and two site factors with 2 (Drainage), 2 (AC surface thickness), 3 (Base type), 3 (Base thickness) and 4 (Climatic zones), 2 (Subgrade type) levels respectively. These levels for all six factors will render a total of 288 (2 x 2 x 3 x 3 x 2 x 4) runs. The number of runs indicates the possible treatment combinations between the levels of all the factors considered in the experiment. This full factorial design is shown in Table 3-1 for a single replication. However, due to economic considerations, the total number of runs were reduced to 192 [288- (16+16+16+48)]. This drop in the number of runs was accomplished by dropping some of the levels within factors (16-inch base thickness in non-drained pavements and ATB/DGAB combination for drained pavements).

Table 3-2 shows the modified experiment design. The reduction in the number of pavement sections seems to be the only advantage by dropping some of the treatment levels. However, this fractional factorial design will restrict the calculation of the higher order interactions and certain effects will be confounded (can not be separated) within each other. Table 3-2 also indicates that a total of 24 treatment combinations for structural factors are required in each subgrade-climate combination. These 24 treatments represent different pavement design options. Furthermore, 12 designs were assigned to two sites in the same subgrade-climate combination with an intention to reduce the number of sections for each site and to maximize the participation of states in the experiment. This partitioning of 24 designs (12 designs at two sites) will also cause an increase in the variability in the observed performance of the pavement sections because of the site related factors (materials, construction quality and measurement).

The fractional factorial design for the SPS-1 experiment is shown in Table 3-3. The overall experiment consists of 192 factor level combinations which consist of 8 site-related (subgrade soil and climate) and 24 pavement structure combinations. The experiment design requires that "48 test sections representing all structural factor and subgrade type combinations in the experiment are to be constructed in each of the climatic zones, with 24 test sections to be constructed on fine-grained soil and 24 test sections to be constructed on coarse-grained soil"[2].

According to the experiment design, twelve test sections were constructed at a given project location (site). Each section is represented by either XX-0101 to XX-0112 or XX-0113 to XX-0124, where XX denotes the state ID. Six sections have a target HMA surface thickness of 4-inch (102 mm) and the remaining six have a target HMA surface thickness of 7-inch (178 mm). Out of 12 sections, 5 have 8-inch (203 mm) base layer, 5 have a 12-inch (305 mm) base layer and the remaining 2 have a 16-inch (406 mm) base layer. Also 2 test sections have dense-graded aggregate base (DGAB), 2 sections have asphalt treated base (ATB), 2 sections have a combination of ATB/DGAB, 3 sections have permeable asphalt treated base (PATB) over DGAB, and 3 sections have ATB over PATB. In-pavement drainage is provided only for sections with PATB as the base.

Table 3-1 Full Factorial Experiment Design Matrix

Drainage	Base Type	Base Thickness	HMA Thickness	WF		WNF		DF		DNF		Row Total
				F	C	F	C	F	C	F	C	
No	DGAB	8"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		12"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		16"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
	ATB	8"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		12"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		16"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
	ATB/DGAB	8"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		12"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		16"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
Yes	DGAB	8"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		12"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		16"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
	ATB	8"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		12"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		16"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
	ATB/DGAB	8"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		12"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
		16"	4"	1	1	1	1	1	1	1	1	8
			7"	1	1	1	1	1	1	1	1	8
Column Total				36	36	36	36	36	36	36	36	288

Table 3-2 Fractional Factorial Experiment Design Matrix and Replications

Drainage	Base Type	Base Thickness	HMA Thickness	WF				WNF				DF				DNF				Row Total
				F		C		F		C		F		C		F		C		
No	DGAB	8"	4"		1		1		1		1		1		1		1		1	8
			7"	1		1		1		1		1		1		1		1		8
		12"	4"	1		1		1		1		1		1		1		1		8
			7"		1		1		1		1		1		1		1		1	8
	ATB	8"	4"	1		1		1		1		1		1		1		1		8
			7"		1		1		1		1		1		1		1		1	8
		12"	4"		1		1		1		1		1		1		1		1	8
			7"	1		1		1		1		1		1		1		1		8
	ATB/DGAB	8"	4"	1		1		1		1		1		1		1		1		8
			7"		1		1		1		1		1		1		1		1	8
		12"	4"		1		1		1		1		1		1		1		1	8
			7"	1		1		1		1		1		1		1		1		8
Yes	PATB/DGAB	8"	4"	1		1		1		1		1		1		1		1		8
			7"		1		1		1		1		1		1		1		1	8
		12"	4"		1		1		1		1		1		1		1		1	8
			7"	1		1		1		1		1		1		1		1		8
		16"	4"		1		1		1		1		1		1		1		1	8
			7"	1		1		1		1		1		1		1		1		8
	ATB/PATB	8"	4"		1		1		1		1		1		1		1		1	8
			7"	1		1		1		1		1		1		1		1		8
		12"	4"	1		1		1		1		1		1		1		1		8
			7"		1		1		1		1		1		1		1		1	8
		16"	4"	1		1		1		1		1		1		1		1		8
			7"		1		1		1		1		1		1		1		1	8
Column Total				12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	192
				24		24		24		24		24		24		24		24		

Table 3-3 SPS-1 Experiment Design Matrix

Pavement Structure				Climatic Zones, Subgrade															
				WET								DRY							
Drainage	Base Type	Base Thickness	HMA Thickness	FREEZE				NO FREEZE				FREEZE				NO FREEZE			
				Fine		Coarse		Fine		Coarse		Fine		Coarse		Fine		Coarse	
				J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y
No	DGAB	8"	4"		113		113		113		113		113		113		113		113
			7"	101		101		101		101		101		101		101		101	
		12"	4"	102		102		102		102		102		102		102		102	
			7"		114		114		114		114		114		114		114		114
	ATB	8"	4"	103		103		103		103		103		103		103		103	
			7"		115		115		115		115		115		115		115		115
		12"	4"		116		116		116		116		116		116		116		116
			7"	104		104		104		104		104		104		104		104	
	ATB/4" DGAB	8"	4"	105		105		105		105		105		105		105		105	
			7"		117		117		117		117		117		117		117		117
		12"	4"		118		118		118		118		118		118		118		118
			7"	106		106		106		106		106		106		106		106	
Yes	PATB/DGAB	8"	4"	107		107		107		107		107		107		107		107	
			7"		119		119		119		119		119		119		119		119
		12"	4"		120		120		120		120		120		120		120		120
			7"	108		108		108		108		108		108		108		108	
		16"	4"		121		121		121		121		121		121		121		121
			7"	109		109		109		109		109		109		109		109	
	ATB/PATB	8"	4"		122		122		122		122		122		122		122		122
			7"	110		110		110		110		110		110		110		110	
		12"	4"	111		111		111		111		111		111		111		111	
			7"		123		123		123		123		123		123		123		123
		16"	4"	112		112		112		112		112		112		112		112	
			7"		124		124		124		124		124		124		124		124

3.2.2 Discussion on the SPS-1 Experiment Design

Fractional designs are effective for characterizing the joint effects of multiple factors. However, the number of treatments, which is a product of the number of levels for factor, grows rapidly with increasing factors. For example, an experiment involving three factors each with three levels, will involve $3^3 = 27$ treatment combinations. One way of economizing will be to limit each factor to two levels, which will reduce the treatment combinations to $2^3 = 8$. These two-level designs are extremely useful in exploratory or screening studies where the objective is to identify the most important factors from a larger set of potential factors. However, if many factors are to be studied in a single experiment, even two-level designs with single replicate may become overwhelming. For example, an experiment with 10 factors will involve $2^{10}=1024$ treatment combinations. Therefore, in such cases, a subset of the treatment combinations can be chosen so that little or no information is lost concerning crucial main effects and low-order interactions. This chosen subset of the treatment combination is termed as "fractional factorial design".

In light of above discussion on the aspect of "how to reduce the number of treatment combinations", the SPS-1 experiment design could have been designed as a half (144 treatment combinations required) or quarter (72 treatment combinations required) fractional factorial design. The detailed methodology for designing these 2^{k-f} experiments can be found in references [3, 4], where k denotes the number of factors and f the fraction.

The SPS-1 experiment has some inherent concerns, because of the reduction of levels for some factors. For example, the accurate effect of 16-inch base thickness can

not be studied as this level was dropped for un-drained pavements. Also, the pure effect of ATB/DGAB base type can not be evaluated because there are no combinations of this base type for drained pavements. It is important to note that the effect of drainage is also confounded with PATB base type as these two effects are not separable within the SPS-1 experiment design.

It would have been more logical and useful to keep all the levels of base thickness and base type in the experiment design and a systematical approach would have been used for designing a fractional factorial designs. This approach would have rendered a more economical and simple design with all main and two-way interaction effects. In order to establish a clear cause-and-effect, the inherent variability between sites is another concern mainly because of difference in traffic, materials and construction quality. To resolve this issue, it would have been more useful to repeat 24 design combinations in representative sites with four (4) climate or eight (8) subgrade-climate combinations.

3.2.3 Current Status of the Experiment (Release 17 of DataPave)

The SPS-1 experiment includes eighteen sites with twelve sections each, for a total of 216 sections located in all four LTPP climatic regions. The Wet-Freeze (WF) and the Wet-No-Freeze (WNF) zones contain the majority of the sections. This is in line with the common wisdom that WF and WNF conditions critically affect flexible pavement performance.

The geographical distribution of sites within the SPS-1 experiment is presented in Figure 3-1 . The full factorial design for the SPS-1 experiment design requires that a total of thirty six (36) similar designs be replicated across eight (8) soil-climate

combinations. The 36 designs were reduced to 24 designs in each soil-climate combination making the experiment design a fractional factorial. However, it was considered that the construction of 24 test sections at each site would require a greater effort on the part of the participating agencies [2]. Therefore, to reduce the cost of construction the experiment was developed so that only 50% of the possible combinations of factors (i.e. 12 test sections) will be built at each site. The experiment, designed in a factorial manner to enhance implementation practicality, permits the construction of 12 test sections (0101 through 0112 or 0113 through 0124) at one site with the complementary 12 test sections to be constructed at another site within the same climatic region on a similar subgrade type [1].

The LTPP IMS data (DataPave 3.0) shows that the site populations within the SPS-1 experiment design are not equally distributed. This deviation is partly because of the cutoff values of precipitation and freeze index used for categorizing the "wet/dry" and "freeze/non-freeze" climates. The current status of the factorial design, along with the current distribution of sites in each climatic zone, is shown in Table 3-4. As these deviations are expected to seriously affect the results of the analysis (the experimental design is unbalanced), this issue will be further discussed in Chapter 4 under design versus actual construction. It can also be seen from Table 3-4, that there is no replication available for sites in DF zone for different subgrade types. Therefore, the subgrade effects in DF zone can not be estimated. Similarly, the results may be seriously hampered due to the small number of sites in Dry zones. A discussion on the current status of the experiment for each site can be found in Appendix A1 of reference [5].

Uniform construction guidelines were established for test sections within SPS-1 experiment designs. These guidelines are presented in the next section.

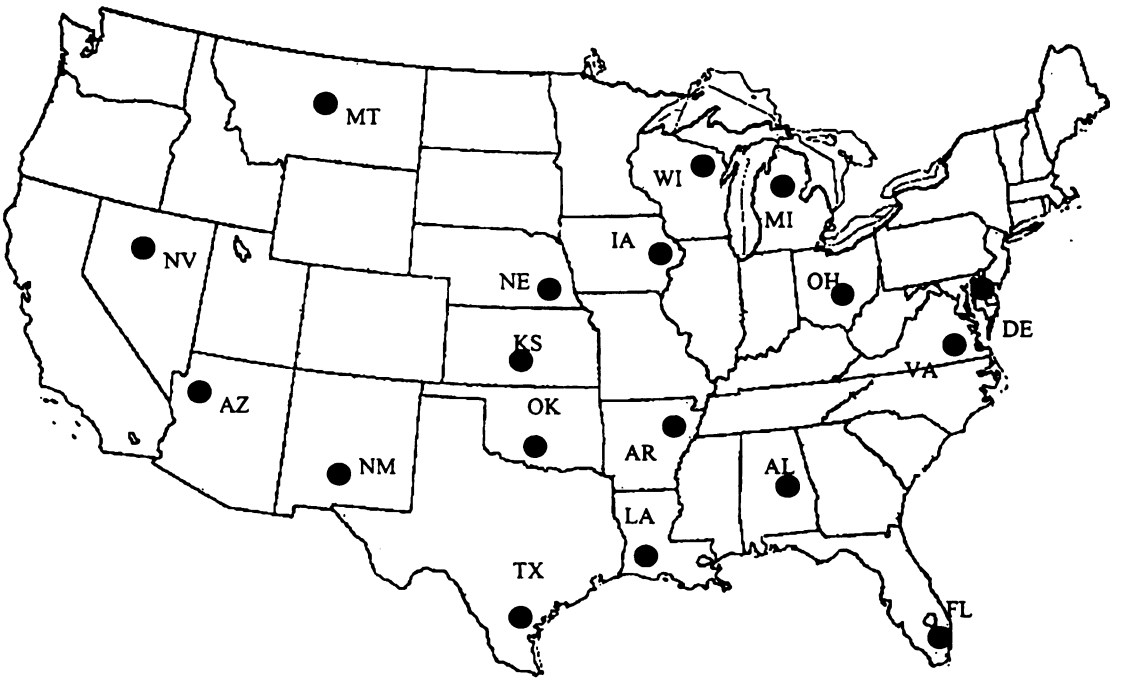


Figure 3-1 Geographical location of SPS-1 sites

Table 3-4 SPS-1 site factorial — From DataPave 3.0

Subgrade Type	Designs	Wet[a]		Dry[b]		Total
		Freeze[c]	Non-Freeze[d]	Freeze	Non-Freeze	
Fine	0101-0112	IA (19) OH (39) KS (20)	AL (1)	-	NM (35)	9
	0113-0124	MI (26) NE (31)	LA (22) VA (51)	-	-	
Coarse	0101-0112	AR (5) DE (10)	FL (12)	NV (32)	-	9
	0113-0124	WI (55)	OK (40) TX (48)	MT (30)	AZ (4)	
Total		8	6	2	2	18

Note:

a. Wet Regions — Average Annual Rainfall > 20 inches (508 mm)
b. Dry Regions — Average Annual Rainfall < 20 inches (508 mm)
c. Freeze Regions — Average Annual Freezing Index > 83.3 °C-day (150 °F-day)
d. Non-Freeze Regions — Average Annual Freezing Index < 83.3 °C-day (150 °F-day)

3.2.4 Construction Guidelines for SPS-1 Experiment

The study of the SPS-1 experiment has specific objectives; mainly the experiment was designed to study the influence of design and construction features on the response and performance of new flexible pavements. Therefore the focus of the experiment is on the main factors (HMA and base thicknesses, base type and presence or absence of drainage). The designs were repeated across 18 states in order to study the effect of different climates and subgrade soils. To study the specific objectives of the SPS-1 experiment, it is essential to control for other sources of variability which can mask the effects of the main factors. These factors may include differences in construction quality, material properties and traffic levels across sites. Therefore, each SPS-1 project had to meet certain construction criteria. To approach uniformity across projects, there were limitations on the methods and materials used in construction, as well as requirements for testing and continued monitoring. These guidelines are outlined below.

Construction Requirements

Construction requirements were provided in the "Construction Guidelines" section of the *SHRP-LTPP Specific Pavement Studies: Five-Year Report[1, 6]*. The overall length of each section was required to be 183 m (600 ft) with 152.4 m (500 ft) for monitoring and 15.25 m (50 ft) on each side for material sampling. The distance between each of these sections had to be long enough to allow sufficient space (transition) for changes in materials and thicknesses during construction. The suggested length for these transitions was 30.5 m (100 ft).

Subgrade Requirements

The finished subgrade elevations could not vary from the design by more than 12 mm (0.5 inches). This could be determined using rod and level readings taken on the lane edge, outer wheel path, mid lane, inner wheel path and inside lane edge at 15 m (50 ft) intervals throughout the length of the project. Surface irregularities could not exceed 6 mm (0.5 inches) between two points in any direction in a 3.05 m (15 ft) interval. Modifiers may be used to provide a stable working platform for construction but not to increase subgrade strength.

Base Layers

Two types of bases are included in each SPS-1 project — drained and un-drained. The drained bases include a permeable asphalt treated base with edge drains. The un-drained bases consist of dense graded materials. Two types of dense graded bases were specified for the sections without drainage. The un-drained bases were used in sections 101-106 and 113-118 and were defined as dense graded aggregate base (DGAB), asphalt treated base (ATB), or a combination of these two materials. The drained base was used in sections 107-112 and 119-124 with a combination with DGAB and ATB base types. The requirements for each base type are as follows:

Dense graded aggregate base (DGAB)

- Minimum 50% retained on the No. 4 sieve.
- Top-size aggregate was specified as 1.5 inches (38 mm).
- Less than 60% passing the No. 30 sieve and less than 10% passing the No. 200 sieve.
- Liquid limit less than 25 and plasticity index less than 4 for fraction passing No. 40 sieve.
- In L. A. Abrasion Test, the loss must not exceed 50% at 500 revolutions.
- The compacted lift thickness must not exceed 200 mm (8 inches).
- The DGAB must be compacted to at least 95% of maximum density.

- In-place density of DGAB should be determined prior to the application of an asphalt prime coat.
- The base surface must be primed with low-viscosity asphalt and allowed to cure prior to placement of the asphalt concrete surface.
- The finished DGAB elevations should not vary from design by more than 12 mm (0.5 inches).

Asphalt treated base (ATB)

- The aggregate used in the ATB layer must meet the same requirements as the aggregate for DGAB layer.
- Asphalt emulsions should not be used in ATB.
- Experimental modifiers were allowed only in the supplemental sections.
- No recycled HMA was allowed in ATB.
- For the Hveem mix design procedure, the following requirements were required for the ATB:

Swell	0.7 mm
Stabilometer Value	35 min
Moisture Vapor Susceptibility	25
Design Air Voids	3 to 5 percent

- For the Marshal mix design procedure, the following requirements were required for the ATB:

Compaction blows	50
Flow	3 to 5 mm
Stability	4.4 KN
Design Air Voids	3 to 5 percent

- The maximum compacted lift thickness for the ATB layer should be limited to a maximum of 8 and 4 inches (200 and 100 mm) for the first and subsequent lifts, respectively.
- The minimum compaction requirement was 90% of the maximum theoretical specific gravity for the first lift and 92% for subsequent lifts.
- The finished surface of the ATB base should not vary from the design more than 12 mm, as measured using rod and levels.
- The base layer thickness should not vary from design by more than 6 mm (0.25 inches).

Permeable asphalt treated base (PATB)

The drained base was used in sections 107-112 and 119-124 with a combination of DGAB and ATB base types. Each of these sections included a PATB layer with edge drains to permit water to drain out of the pavement structure. The requirements for the PATB layer were as follows:

- An asphalt emulsion was not allowed as binder for PATB base layer.
- The gradation for the PATB layer should be within the following ranges:

Sieve No.	**% Passing**
38 mm (1.5 inch)	100 %
25 mm (1 inch)	95 – 100 %
13 mm (0.5 inch)	25 – 60 %
No. 4	0 – 10 %
No. 8	0 – 5 %
No. 200	0 – 2 %

- More than 90% of the aggregate has at least one crushed face.
- No recycled HMAshould be used in PATB.
- Compaction should be performed by using static wheel roller applying 0.5 to 1.0 ton of force per foot of roller width.
- No portion of the PATB should be day-lighted.

HMA Layer

The HMA surface layers were required to meet the following minimum requirements.

- For the Hveen mix design procedure, the following requirements were required for the HMA mix:

Swell (maximum)	0.7 mm
Stabilometer Value (minimum)	37 min
Air Voids	3 - 5%

- For the Marshal mix design procedure, the following requirements were required for the HMA mix:

Compaction blows	75
Flow	2 to 4 mm
Stability	8 KN

- No recycled materials were permitted in HMA mixtures.
- The aggregates should have a minimum of 60% retained on the No. 4 sieve with at least two fractured faces, and a minimum sand equivalent of 45.
- The asphalt grade and characteristics should be selected based on normal agency practice.
- The use of modifiers should be discouraged in the main sections.
- Lift thickness could not exceed 102 mm and compacted thickness of any single layer had to be at least 51 mm.
- Longitudinal joints should be staggered between successive lifts to avoid vertical joints.
- All transverse joints should be placed outside the main sections.
- The thickness of the HMA layer (surface and binder) should be within 6 mm of the thickness specified by the experiment design.

- The as-constructed finished surface should have a profile index of less than 10 inches per mile (158 mm per km) as measured by the California-type profile-graph.

Shoulders

- The shoulders placed on these projects should have a minimum width of 1.2-m (4 ft) and have the full pavement structure across their width.
- If possible, the shoulders should be paved full-width with the surface course to eliminate longitudinal joints. If not, then the shoulders should be paved such that the longitudinal joint is at least 205 mm (12 inches) outside the travel lane.

Drainage Materials

Filter fabric (or geo-textile) was required on sections that included a PATB layer. This was specified to prevent the clogging of the PATB layer due to migration of fine material from the subgrade. The filter fabrics used should meet the American Association of State Highway and Transportation Officials-American Building Contractors-American Road and Transportation Builders Association (AASHTO-ABC-ARTBA) Task Force 25 recommendations, which include the following requirements for the geo-textiles:

- In order to separate the base layer from the subgrade non-woven and woven geo-textile materials that conform to Class A requirements should be used.
- The geo-textile material conforming to Class B requirements could be used in the edge drains.
- Geo-textiles should be overlapped a minimum of 2 ft (610 mm) at all longitudinal and transverse geo-textile joints.
- Filter fabrics should be installed in accordance with the manufacturer's specifications.
- For the sections where the PATB layer was placed on DGAB, the filter fabrics should extend around each edge drain and wrap around the outer edge of the PATB layer.
- Exposure time of the geo-textile to elements between lay down and cover should be limited to a maximum of 3 days.

Edge drains were to be installed on sections containing a PATB layer to collect water draining from the permeable base. The requirements on these drains were as follows:

- Inside and outside edge drains should be constructed for crowned pavements.
- Edge drains should be at least 3 ft (914 mm) away from the edge of the travel lane.
- The PATB was recommended for backfill around the edge drains; however, other open graded materials could be used as backfill materials if approved.
- Collector pipes (slotted) should be at least 3-in (76 mm) diameter.
- Outlet pipes (un-slotted) should be rigid plastic pipes with a minimum diameter of 3-in (76 mm).
- Drainage pipes should be sized for the expected discharge determined as part of design.
- Discharge outlet pipe should be placed at a maximum interval spacing of 76.2 m (250-ft).

Material Sampling and Testing

Sampling and testing were required for each of the material used for the construction of sections. The material characterization is necessary to evaluate the differences between the as-constructed sections within a site and between different sites within the experiment. These measured parameters are used mostly in the design procedures as well as to assess important performance characteristics of the materials.

A general sampling and testing plan was created for use as a guideline[6]. These guidelines were then used to develop the sampling and testing plan specific to each site. These plans were created prior to the construction of each project and the location of each sample was predetermined. The following types of samples should be taken from each project:

- Bulk samples from the upper 305 mm (12 inches) of the subgrade.
- Thin-walled tube samples of the subgrade to a depth of 1.2 m (4 ft).
- Jar samples for subgrade.
- Bulk samples for the DGAB.
- Jar samples for the DGAB.
- Bulk samples for PATB.
- Bulk samples for ATB.
- Bulk samples for the asphalt mixes used in the surface and binder course.
- Bulk sample of asphalt mixes used in all mixes.
- Cored samples for bound bases and surface asphalt layers.

In addition to these samples, bulk samples were to be taken for the asphalt cement, aggregates and un-compacted HMA mixes. These samples were to be stored for long term. A series of auger probes should be performed in the shoulder of each test section up to a depth of 6 m. This allows for the determination of the stiff layer depth. Finally, as part of the construction activity, nuclear density and moisture testing should be conducted at the location of the bulk sampling areas for the subgrade and on the top of each layer in every test section. The type and number of tests per layer are given elsewhere [6].

Monitoring Requirements

The monitoring of the sections at each site includes several types of data. These include distress surveys, deflection measurements, transverse profiles and longitudinal measurements. Each of these measurements has different frequency requirements which can be revised over time.

Distress Surveys

A distress survey was to be performed on each section within 6 months of construction. A manual distress survey should be performed on the sections biennially, with the exception of "weak" sections (2, 5, 7 and 13 in SPS-1 projects where distress surveys should be more frequent). The survey could be postponed by a year if necessary.

Deflection Surveys

Deflection measurements should be collected using a falling weight deflectometer (FWD) from 1 to 3 months after the construction of the project. The deflection survey of these projects is to be completed biennially except of the "weak" sections (2, 5, 7 and 13

in SPS-1 projects where distress surveys should be more frequent). This testing also could be postponed up to 1 year if necessary.

Transverse Profile

Transverse profile measurements should be taken at the same frequency and at the same time, as the distress surveys.

Longitudinal Profile

Longitudinal profiles should be taken on the sections within 3 months after construction. These measurements can be postponed up to 3 additional months. The "weak" sections (2, 5, 7 and 13 in SPS-1 projects) should be monitored every 6 months but monitoring can be postponed up to 6 additional months. The other sections should be monitored biennially and can be postponed by 1 year if necessary.

Traffic Data

Traffic data should be collected on each site. The current requirement states that weigh-in-motion (WIM) data should be continuously collected on all SPS-1 sections. Continuous data collection has been defined as the "use of a device that is intended to operate throughout the year and to which the SHA commits the resources necessary to both monitor the quality of the data being collected and to fix problems quickly upon determination of any fault" [1]. WIM devices are to be calibrated biannually. This level of data collection is necessary to assess accurate traffic loading measurements.

Climatic Data

Each SPS-1 site was required to install an automatic weather station (AWS). The AWS should be located close enough to each of the sites to provide weather data that is

representative of the climate on each site. The equipment installed at these locations should measure the following weather components:

- Rain
- Humidity
- Wind speed
- Temperature

All the data collected should be stored by a data-logger. The data should be downloaded from the data-logger at least every 6 months.

In addition to AWS used to collect weather data, weather data should also be obtained from the four or five closest National Oceanic and Atmospheric Association (NOAA) weather stations. The data should be averaged using the weighting procedure, with the weights based on the distance of the weather station from the particular site. The data collected from NOAA stations should include information about the temperatures, rainfall, wind and solar radiation levels.

3.3 STRATEGIC STUDY OF ENVIRONMENTAL FACTORS FOR FLEXIBLE PAVEMENTS— SPS-8 EXPERIMENT

The SPS-8 experiment evaluates environmental effects in the absence of heavy traffic loads. The study examines the effect of climatic factors and subgrade type (frost-susceptible, expansive, fine, and coarse) on pavement sections incorporating different designs of flexible and rigid pavements, which are subjected to very limited traffic as measured by ESAL accumulation. Pavement structure includes two levels of structural design for each class of pavements. Flexible pavement sections consist of 4" (102 mm) and 7" (178 mm) HMA surfaces on 8" (203 mm) and 12" (305 mm) layers of DGAB, respectively. Rigid pavement test sections consist of 8" (203 mm) and 11" (279 mm) doweled JPCP slabs on 6" (152 mm) DGAB. The study design stipulates the traffic volume in the study lane be at least 100 vehicles per day but not more than 10,000 ESALs in a year. The combination of study factors results in four possible section combinations, two flexible and two rigid. The flexible and rigid sections may be constructed at the same or at different sites. Table 3-5 shows the experiment design matrix for SPS-8.

For flexible pavements in SPS-8 experiment, the sections are identified as XX-0801 to XX-0806 while rigid pavements are identified as XX-0807 to XX-0812, where 'XX' is the state code and '08' stands for SPS-8 experiment. The sections with SHRP ID that ends with an odd number have target HMA thickness of 102 mm or PCC slab thickness of 203 mm, while the others have HMA thickness of 178 mm or PCC slab thickness of 279 mm thickness. In the section ID, an alphabet is introduced before the SHRP ID in case a second site is constructed in the same state.

3.3.1 Discussion on the SPS-8 Experiment Design

The SPS-8 experiment design for flexible pavements is not a full factorial design. Table 3-5 shows that there are 2 design factors (AC surface thickness and base thickness) and 2 site factors (Subgrade type and climatic zone). There are 2, 2, 3 and 4 levels for each of these 4 factors, respectively. Accordingly, a full factorial design involves 48 (2 x 2 x 3 x 4) treatment combinations (runs) for a single replicate. The experiment was reduced to only 24 runs for economic and practical reasons. Consequently, the pure effects of AC surface thickness and base thickness can not be studied from this study. However, this reduction in the runs in SPS-8 experiment design seems logical in light of the specific objectives of this study.

3.3.2 Current Status of the SPS-8 Flexible Pavements

Table 3-6 shows the presence of flexible pavement sites in the SPS-8 experiment. There are fifteen (15) sites available for SPS-8 flexible pavement sections with the largest number of sites (7) located in the WF climatic zone. The distribution of test sections for flexible pavements according to SPS-8 experiment design is given in Table 3-7. In total, 32 flexible pavement sections have been constructed in the 15 sites. There are a limited number of test sections in the Dry zones with no pavements on fine subgrade in DF or active subgrade in DNF zones.

3.3.3 Construction Guidelines for SPS-8 Flexible Pavements

Construction guidelines were provided to ensure uniformity and consistency among the test sites. The requirements for preparation and compaction of the subgrade for flexible sections are the same as for the SPS-1 experiment.

Table 3-5 SPS-8 Experiment Design Matrix

Pavement Type	AC/PCC Thickness	Base Thickness	WF			WNF			DF			DNF		
			A	F	C	A	F	C	A	F	C	A	F	C
Flexible	4	8	x	x	x	x	x	x	x	x	x	x	x	x
	7	12	x	x	x	x	x	x	x	x	x	x	x	x
Rigid	8	6	x	x	x	x	x	x	x	x	x	x	x	x
	11	6	x	x	x	x	x	x	x	x	x	x	x	x

A: Active subgrade soil (either frost susceptible or swelling type relative to climatic zone)
F: Fine-grained subgrade soil
C: Coarse-grained subgrade soil

Table 3-6 Distribution of SPS-8 flexible pavements sites by subgrade type and climatic zone

Subgrade Type	Wet[a]		Dry[b]		Total
	Freeze[c]	Non-Freeze[d]	Freeze	Non-Freeze	
Fine	AR (5) MO (29) NJ (34) OH (39)	MS (28) TX (48)	SD (46)	NM (35)	**8**
Coarse	NY (36) WA (53) WI (55)	NC (37)	MT (30) UT (49)	CA (6)	7
Total	7	3	3	2	15

Note: a. Wet Regions — Average Annual Rainfall > 20 inches (508 mm)
b. Dry Regions — Average Annual Rainfall < 20 inches (508 mm)
c. Freeze Regions — Average Annual Freezing Index > 83.3 °C-day (150 °F-day)
d. Non-Freeze Regions — Average Annual Freezing Index < 83.3 oC-day (150 oF-day)

Table 3-7 Distribution of SPS-8 flexible pavements sections by design, subgrade type-climatic zone

Pavement Structure			Moisture, Temperature and Subgrade Type																					
			Wet										Dry											
			Freeze						No-Freeze				Freeze						No-Freeze					
Pavement Type	Surface Thickness (inches)	DGAB Thickness (inches)	Active*		Fine		Coarse		Active		Fine		Coarse		Active		Fine		Coarse		Active		Fine		Coarse	
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Flexible	4	8	2		2		3		1		1		2		1		0		2		0		1		1	
	7	12	3		0		4		1		2		1		2		0		1		0		1		1	

Note: DGAB= Dense-graded aggregate base.

*Active soil can be either frost-susceptible or swelling (expansive) type. Each no. indicates presence of sections fulfilling the criteria of the cell

The construction guidelines stipulated the use of DGAB; the requirements for the materials and construction of the base layers are the same as those of the SPS-1 experiment. Similarly, the guidelines for the materials and construction of asphalt layers are similar to the un-drained sections included in the SPS-1 experiment, which require DGAB.

3.4 INSTRUMENTED SPS TEST SECTIONS

The Strategic Highway Research Program (SHRP) has included two projects with instrumented test sections as part of the SPS-1 and SPS-2 experiments. This subset of sections is referred to as the Dynamic Load Response (DLR) experiment. The sections are located in Ohio and North Carolina. The Ohio test sections include both flexible and rigid pavements while the North Carolina test sections include only rigid pavements. The objective of this experiment is to support the development of mechanistic-empirical design procedures for flexible and rigid pavement systems. More specifically, it can be used to investigate the relationship between pavement response and performance, and to validate pavement response and performance prediction models.

In addition to standard FWD, profile and distress measurements, pavement dynamic response parameters are being measured in these test sections including:

- Vertical deflections in the surface layer, base and subgrade;
- Horizontal strains in the pavement;
- Vertical pressure at layer interfaces; and
- Joint opening in PCC pavements.

The seasonal parameters being measured include:

- Temperature within the pavement layers including base and subgrade;
- Frost depth in base and subgrade;
- Soil suction in the subgrade;
- Water table elevation; and
- Moisture in the subgrade.

In addition to measurements within the pavement system, the loads being applied on the pavement are measured using weigh-in-motion (WIM) scales. Several series of controlled vehicle tests at different speeds and non-destructive load tests have been conducted to measure the pavement response.

3.4.1 SPS-1 Experiment Sections

The instrumented flexible pavement sections are located in the SPS-1 site Ohio (39). These sections were instrumented with strain gauges, pressure cells and multi-depth deflectometer (MDD) to conduct the controlled loading experiments.

The experiment targeted four core sections for the installation of sensors to monitor dynamic pavement response during controlled vehicle testing. These sections include 39-0102, 39-0104, 39-0108 and 39-0110. Tests were to be performed with a single axle and tandem axle dump truck. The rear axle on the single–axle truck was loaded to approximately 18 kips (40 kN) and 22 kips (49 kN) while the total load on the rear axles of the tandem-axle dump truck were 32 kips (142 kN) and 42 kips (187 kN), respectively. Both trucks ran over the instrumented sections at 50(30), 65(40) and 80(50) km/hr (mph) in the morning and in the afternoon.

Experiment Setup

The details of the instrumented flexible pavement sections are given in Table 3-6. Tests were conducted in the morning and in the afternoon to gather information on how temperature differences in the pavement layers affect response. Table 3-9 shows the instrumentation details of all the strain gauges and Linear Variable Differential Transformers (LVDT) for each instrumented section. This information is taken from Report No. FHWA/OH-94/019 by Ohio University, as this data was not available in the DataPave Release 17.0.

Table 3-8 Details of instrumented sections for flexible pavements

Section ID	HMA Thickness (inches)	Base Thickness (inches) / Base Type	Drainage	Comments
39-102	4	12 DGAB	No	Strain gauges at 4"
39-104	7	12 ATB	No	Strain gauges at 7" and 19"
39-108	7	4 PATB 8 DGAB	Yes	Strain gauges at 7"
39-110	7	4 ATB 4 PATB	Yes	Strain gauges at 7" and 11"

Note: DGAB – Dense graded aggregate base, ATB – Asphalt treated base, PATB – Permeable asphalt treated base

Table 3-9 Instrumentation details for all the SPS-1 sections in Ohio

Section ID	Strain Gauge Designation	Location	LVDT Designation
39-102	DYN7 – Transverse DYN8 – Longitudinal DYN9 – Transverse DYN10 – Longitudinal DYN11 – Transverse DYN12 – Longitudinal	All strain gauges are installed at the bottom of AC, 4" deep from the surface	LVDT1 – Deep[1] LVDT2 – Shallow[2] LVDT3 – Shallow LVDT4 – Deep
39-104	DYN10 – Transverse DYN11 – Longitudinal DYN12 – Transverse DYN13 – Longitudinal DYN14 – Transverse DYN15 – Longitudinal DYN16 – Longitudinal DYN17 – Longitudinal DYN18– Longitudinal	DYN10 to DYN15 are located at bottom of AC, 7" deep from the surface These three strain gauges are installed at the bottom of ATB, at 19" deep from the surface	LVDT1 – Deep LVDT2 – Shallow LVDT3 – Shallow LVDT4 – Deep
39-108	DYN10 – Transverse DYN11 – Longitudinal DYN12 – Transverse DYN13 – Longitudinal DYN14 – Transverse DYN15 – Longitudinal	All strain gauges are installed at the bottom of AC, 7" deep from the surface	LVDT1 – Deep LVDT2 – Shallow LVDT3 – Shallow LVDT4 – Deep
39-110	DYN10 – Transverse DYN11 – Longitudinal DYN12 – Transverse DYN13 – Longitudinal DYN14 – Transverse DYN15 – Longitudinal DYN16 – Longitudinal DYN17 – Longitudinal DYN18– Longitudinal	DYN10 to DYN15 are located at bottom of AC, 7" deep from the surface These three strain gauges are installed at the bottom of ATB, at 11" deep from the surface	LVDT1 – Deep LVDT2 – Shallow LVDT3 – Shallow LVDT4 – Deep

[1] The deep referenced LVDT is anchored at 10 ft depth from the surface. [2] The shallow referenced LVDT is anchored at bottom of base layer for each section from the surface, Source: Report No. FHWA/OH-94/019

CHAPTER 4 - DATA AVAIABILITY AND EXTENT

4.1 INTRODUCTION

This chapter presents a summary on the available data and extent of various performance measures for SPS-1 and SPS-8 experiments. For this study DataPave (Release 17, January 2004) was the primary data source. However, data from previous releases and other sources were used only to supplement the level E data from Release 17. The construction information of each site was extracted from the construction reports. The essential data required for this study can be broadly classified into following categories:

- Site Information: site information, construction issues, climatic and traffic data.
- Material Data: Material type and properties for various bound and un-bound pavement layers.
- Pavement Structure: Layer type and thickness information and other design features such as lane width, shoulder type and dowel bar diameter etc.
- Monitoring Data: Longitudinal and transverse profiles, distress and deflections.
- Dynamic Load Response Data: Response data for instrumented sections.

4.2 IDENTIFICATION OF DATA ELEMENTS

The relevant variables contained in the SPS-1and SPS-8 experiments can be divided into: (1) dependent variables, and (2) independent variables.

The dependent variables are those used to describe pavement response and performance. Measures of pavement response are those measures that do not cumulate with time. The bulk of pavement responses in these experiments are surface deflections from Falling Weight Deflectometer (FWD) testing. For flexible pavements, FWD testing is conducted in the wheel path and outside the wheel path. Other pavement responses

collected in the SPS-1 experiment include strain data and vertical deflections at various depths. These measurements are only available from the instrumented sections in the Dynamic Load Response (DLR) experiments.

Measures of pavement performance are those that cumulate with time (e.g., alligator cracking in flexible pavements). These are collected using both manual and automated surveys.

The independent variables are those that describe design and construction factors. These can be divided into: (1) main variables, and (2) exogenous (or confounding) variables. Main variables are those used to specify the design matrices of the respective SPS experiments (e.g., base type). Whereas, the variables that have potential impacts on pavement response and performance but are not controlled in the experiment design were considered as exogenous variables. Exogenous variables that are independent of the main experiment variables are the actual cumulative traffic (KESALs) and age. All other exogenous variables are associated with the main design and construction variables. These include: (1) material properties of the various pavement layers, which constitute the structural factors in the design matrix, and (2) climatic factors, which describe the four climatic zones in the matrix. Figure 4-1 shows the summary of main variables (independent) along with their levels considered for explaining the dependent variables (performance measures) for the SPS-1 experiment.

Table 4-1 lists the relevant independent and dependent variables identified for flexible (SPS-1 and SPS-8) pavements. After the identification of the relevant variables, a database was developed for this research. The development of database is briefly discussed next.

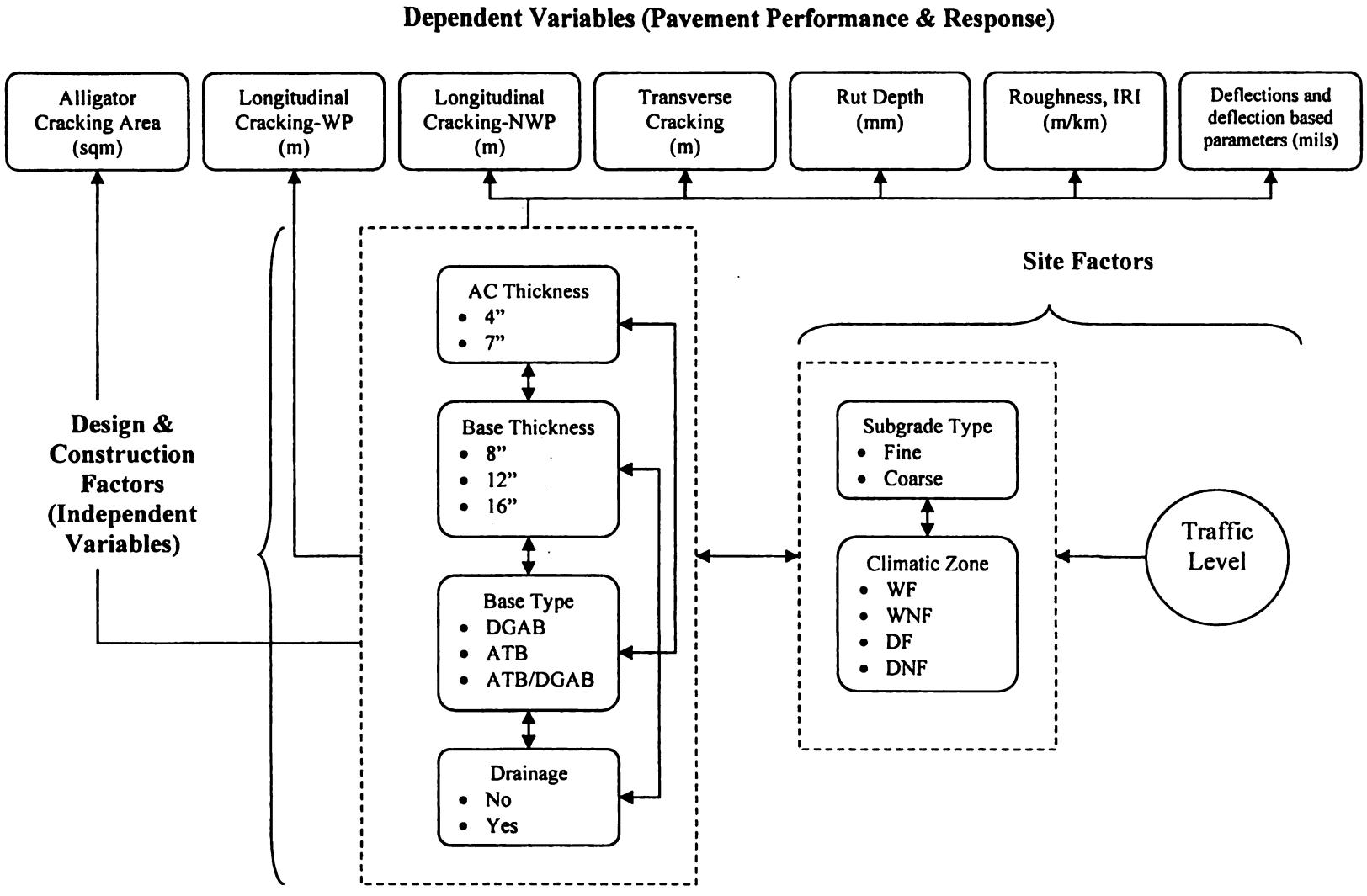


Figure 4-1 Summary of dependent and independent variables

Table 4-1 Categorized list of variables for flexible pavements (SPS-1 and SPS-8)

Factor	Factors
Environmental Factors	No. of days with Freezing Temperature No. of days with temperature>32°C Annual No. of days with precipitation Annual No. of days with high precipitation Avg. Annual No. of FT cycles **FI, Degrees-Days** **Avg. Annual Precipitation** **Environmental Zone** Avg. Max Temperature, °C, Avg. Min Temperature, °C, Avg. Temperature Range, °C
Asphalt Concrete Material Properties	AC Grade **Target AC Thickness, mm** ***Thickness deviations, mm*** AC Back calculated Resilient Modulus AC Indirect Tensile Strength after M_R Test, kpa AC Indirect Tensile Strength Prior to M_R Test, kpa AC Instantaneous Resilient Modulus at 5, 25 and 40 °C, MPa AC Total Resilient Modulus at 5, 25 and 40 °C, MPa Bulk Specific Gravity of AC Mix Water absorption for AC mix aggregate AV%, AC%, AC mix gradation (all sieves) and AC viscosity at 60 °C
Aggregate Base Material Properties	**Target base thickness, mm** ***Thickness deviations, mm*** **Type of base (GB, TB, PATB)** Granular base Compaction (Max. density and OMC) Base back calculated resilient modulus Avg. Lab based granular base resilient modulus Base gradation (all sieves) Atterberg Limits (LL, PL, PI)
Subgrade Material Properties	**Subgrade soil type** Subgrade Compaction (Max. density and OMC) Subgrade back calculated resilient modulus K_1 ,K_2 and K_5 parameters from the resilient modulus testing for subgrade Avg. Lab based granular base resilient modulus Subgrade gradation (all sieves) Atterberg Limits (LL, PL, PI) Embankment heights (cut or fill)
Traffic/Age	Cumulative Annual Traffic in KESALs ***Average Annual Traffic in KESALs*** ***Age, Years***
Performance	**Alligator Cracking (fatigue)** **Transverse Cracking** **Longitudinal Cracking in WP and NWP** Bleeding Raveling **Roughness (IRI)** **Rutting**
Response	**Deflections** **Various Deflection Basin Parameters** **Strains (DLR)**

Note: The variables in bold are the potential main factors (independent variables) and performance/response (dependent variables). The variables in italics were considered as exogenous factors.

The data used in this study are "Level E" data from the IMS database (Release 17.0) for SPS-1and SPS-8 experiments. All data were extracted from the Release 17.0 CD. The DLR data contained in the DataPave 3.0 database is insufficient and/or inadequate for the analysis.

For the purpose of this research, the data available along with its type is shown in Figure 4-2. The flowchart describing the process of data extraction from the DataPave Release 17.0 is shown in Figure 4-3. The database has been set up such that the linkage between different data elements is preserved. This was done using ACCESS™, EXCEL™ and SPSS™ software. This relational database allows for describing the data in different ways by combining various factors according to the specific objective of the particular analysis at hand. Tables and figures produced and presented in the data availability section for all experiment designs are example outcomes of this data structure.

For cases where multiple data values were available for a data element, the values were averaged to obtain a unique value. For example, IRI values were averaged over five runs for each section and for a particular date. Deflection measurements were averaged over the length of each section for a particular date.

To complement/cross-check the inventory data available in Release 17.0, construction reports (in the form of PDF files) for all sections within the SPS-1 and -8 experiments were obtained.

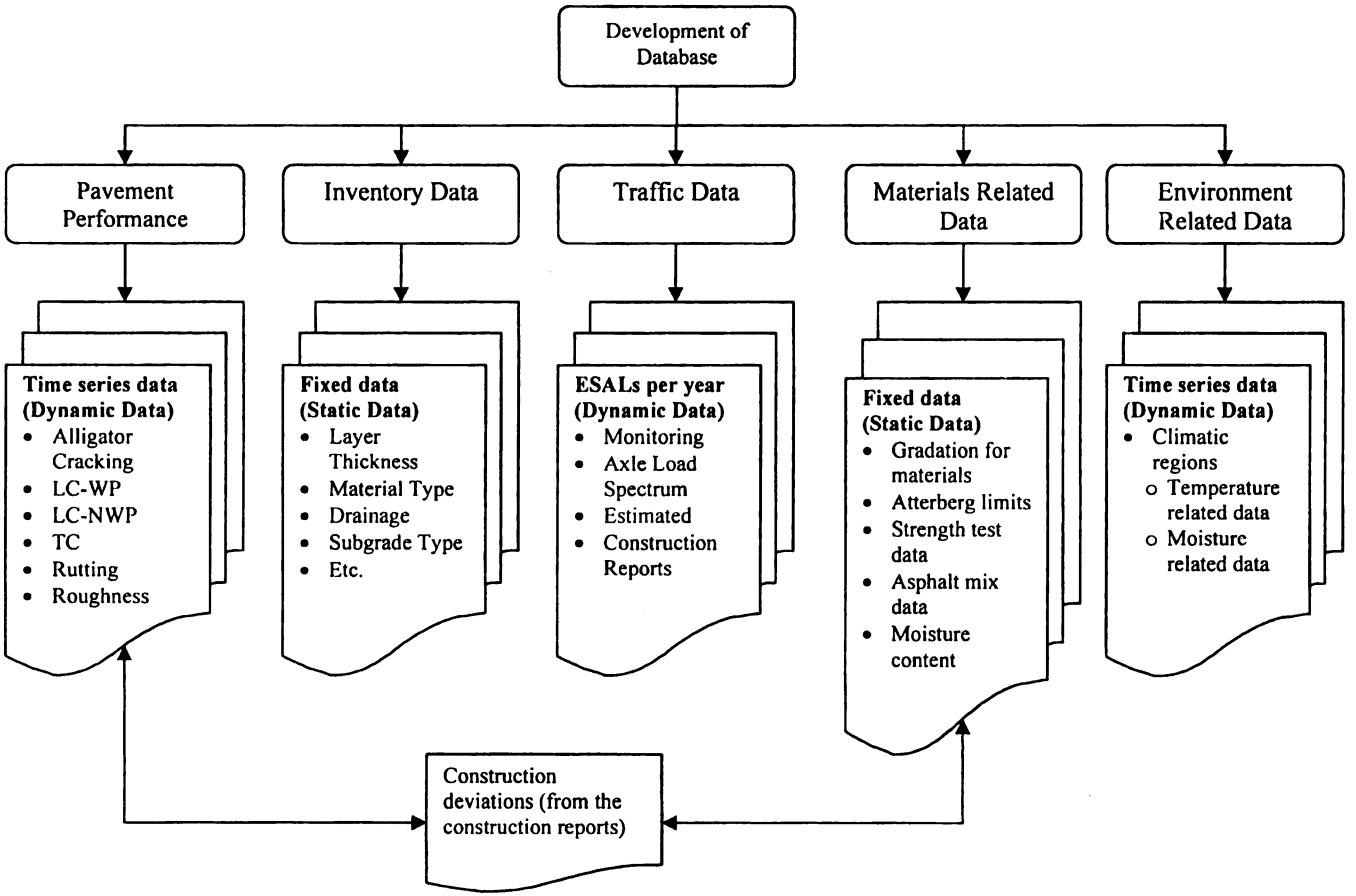


Figure 4-2 Type of available data in LTPP database

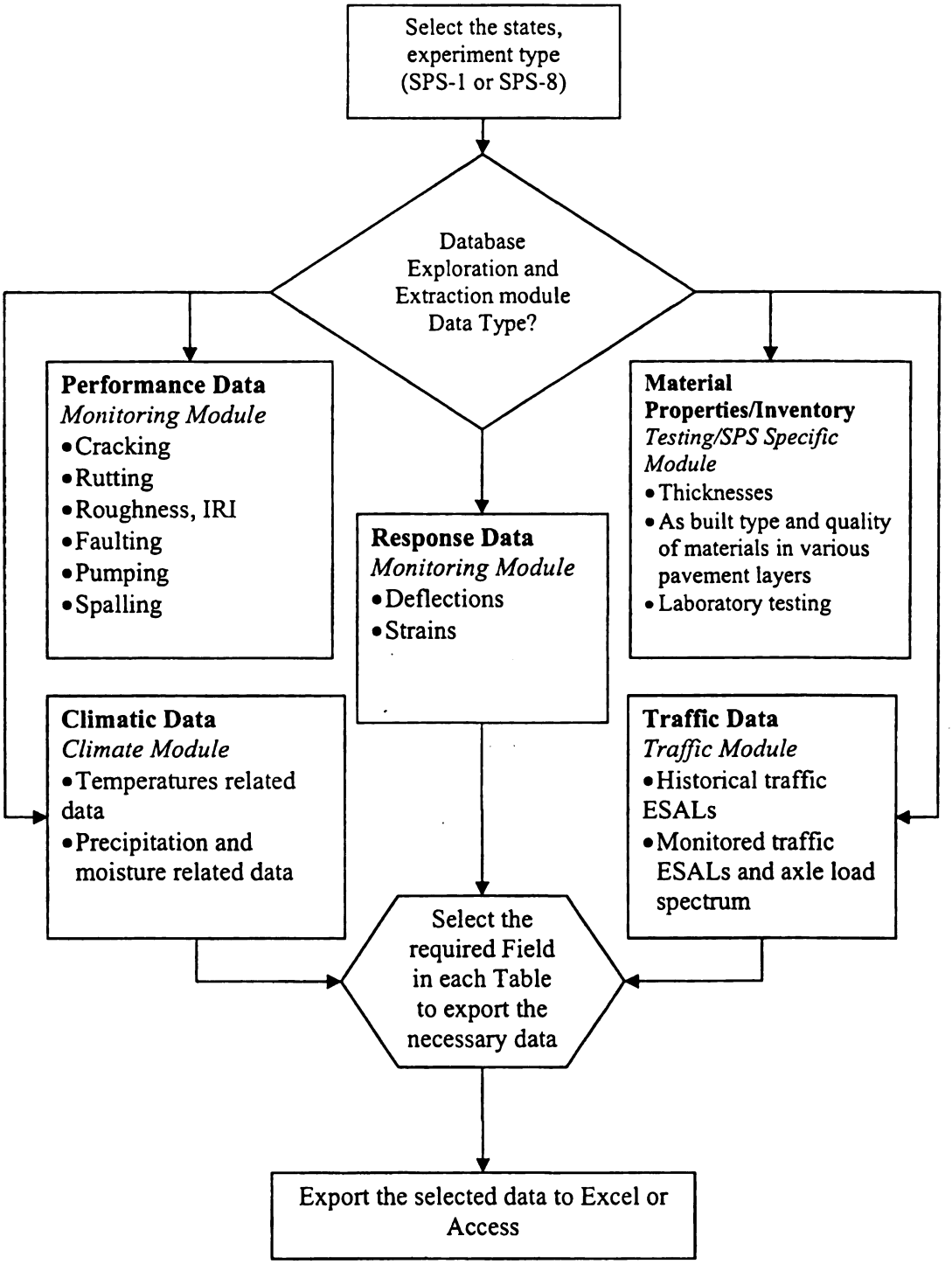


Figure 4-3 Data Extraction Process Flow Chart

The construction reports were reviewed for the purpose of obtaining additional detailed information on construction and design features. They also include problems encountered during construction of the SPS pavement sections. Some of these problems have been highlighted in this Chapter. The reports were useful in confirming/disproving conflicting information as well as identifying and explaining some anomalies in the database.

After extraction of all relevant data elements and building the analysis database, the data were reviewed to determine:

- the availability of main (design and construction) factors and exogenous (confounding) factors in the identified data element tables,
- the availability and extent of response and performance data,
- the variability of response and performance measures, and
- the variability of the main and exogenous factors.

The sections below present the details of the information on the availability and extent of these variables for SPS-1and SPS-8 experiments, respectively.

4.3 DATA AVAILABILITY IN SPS-1 EXPERIMENT

Table 4-2 presents the overall availability of the relevant data elements within the SPS-1 experiment in Release 17.0 of DataPave. This includes both the main design and construction factors as well as other exogenous factors such as traffic, material, and environmental data. The data is presented as a percentage of the total number of sections for the main factors, while for the exogenous factors, data is expressed as a percentage of the total number of sites (states) included in the experimental design matrix.

The table shows that the data availability for the main factors is high, while that of the exogenous factors is somewhat lower. In particular, the availability of traffic data

(61% for monitored data and 50% for estimated/historical data) is lower than expected and should be improved. It should be noted that traffic estimates from construction reports are available for all but one site [MI (26)]. The availability of relevant data elements is discussed in detail in the subsequent sections.

4.3.1 General Site Information

This section of the report presents the summary of the site identification and location, construction report availability and important dates associated with each of the SPS-1 projects. Also the details of other factors such as climate and traffic, which pertain to a particular site in the SPS-1 experiment, will be discussed in this section.

Construction Reports

The construction reports have been prepared for each site by the supervisory consultant on the project. These documents contain the details of the construction process from conception to the completion. In addition, these reports presents information on the geometric layout of various sections within a site, construction issues (deviations from the guidelines, if any), traffic, environmental conditions during the construction and material quality control data. These reports are available for all the sites in the SPS-1 experiment except MI (26). A summary of the construction issues at each of 18 sites is given in Appendix A1 of reference [1]. These construction issues can be helpful in explaining any poor performance at a particular site.

Table 4-2 Summary of SPS-1 data elements availability

Data Category	Data Type	Data Availability, %
Site Information	*Construction Reports*	94
	Climatic data	
	Virtual Weather Station	
	Annual Temperature	**100**
	Annual Precipitation	**100**
	Automatic Weather Station	
	Monthly Temperature	83
	Monthly Precipitation	83
	Traffic data	
	Traffic Open date	**100**
	Estimated ESALs	**50**
	Monitored ESALs	**61**
	Axle Load Spectrum	**72**
Material Data	*Asphalt Layer*	
	Core Examination	99
	Bulk Specific Gravity	89
	Max Specific Gravity	42
	Asphalt Content	56
	Asphalt Resilient Modulus	15
	Penetration	49
	Viscosity	48
	Asphalt Specific Gravity	47
	Aggregate Gradation	56
	Fine Aggregate Particle Shape	26
	Layer Thickness	**100**
	Unbound Base Gradation	20
	Subgrade	
	Subgrade Gradation	44
	Atterberg Limits	56
	Subgrade Modulus	51
Pavement Structure	*Layer details*	
	Type	**100**
	Representative thicknesses	**100**
	Constructed thicknesses	94
	Shoulder information	
	Type	86
	Width	86
	Thickness	86
Monitoring**	*FWD data*	
	Deflections	**100**
	Temperature at Testing	99
	Backcalculated Moduli	6
	Manual Distresses data	**100**
	Longitudinal Profile (IRI)	**100**
	Transverse Profile (Rut Depth)	**100**

Note: ** Data is said to be available for a section even if it is available for one survey.

Climate Data

The climate data were essentially used in defining boundaries between various climatic regions. The *average annual rainfall* for each site is considered as discriminating variable between "wet" and "dry" regions, whereas, *average annual freezing index* is used to locate each site in "freeze" and "no-freeze" regions. The climate data is available form two sources in LTPP database— Automatic Weather Stations (AWS) and Virtual Weather Stations (VWS). The AWS data are collected by a weather station installed at each of SPS-1 experiment site. AWS data for three sites; NV (32), OH (39) and WI (55) are not available in the Release 17.0 of the database. The VWS data are collected from the existing weather stations in the vicinity of a specific site in the SPS-1 experiment. Climate data for all the sites are available from VWS. Therefore, in DataPave, the climate data from VWS have been used to classify each site in a particular climate region/zone.

Traffic Data

Heavy truck traffic plays a vital role in determining the level of performance in flexible pavements. The traffic data in terms of ESALs per year was obtained from different sources. These sources can be summarized as;

- IMS Database: The LTPP IMS database contains traffic data in the following forms;
    - Monitored Data—data obtained from weigh-in-motion equipment installed at each SPS-1 site.
    - Estimated Data—data obtained from the DOT's based on their best estimates from the previous history of the highway section.

- Axle Load Spectrum—data obtained from the axle weight data; this is essentially similar to monitored data.

The lack of traffic data for various states in the LTPP database has given rise to the quest for reasonable traffic estimates for the missing states. Therefore, other sources were explored, including:

- Construction Reports—the estimated design ESALs were taken from construction reports for all the states in the SPS-1 experiment.
- FHWA VTRIS database—this was used for estimating the average truck factors for each site, once the ADTT is known from the construction reports, the ESALs per year were estimated for a particular site.
- Previous Studies—the available studies on SPS-1 [2, 3] were also used to extract traffic information.

Finally, the ESALs per year were estimated by combining the information from all the sources and confidence levels were assigned to the quality of available traffic data. Table 4-3 summarizes the traffic data availability for all the states within the SPS-1 experiment. The importance of traffic data can not be ignored in pavement design and analysis; however; in this research only the traffic estimate is required to neutralize its effects between different sites.

Traffic opening date is the date on which a newly constructed project was opened to traffic. This data is available in the database for all sites. The age of a section has been calculated using traffic opening date and corresponding last survey date.

Table 4-3 KESAL per year for SPS-1 Experiment

State	Code	KESALs per year						Summary Statistics				Proposed	Confidence Level	Remarks
		LTPP			Other Sources									
		Monitored	Estimated	Axle Spectrum	Const. Reports	FHWA-RD-01-166	NCHRP-499	Mean	Median	Std	CoV			
Alabama, AL	1	-	-	-	237	237	-	237	237	-	-	**237**	Low	Taken from Construction Report
Arizona, AZ	4	236	277	160	185	185	250	214	211	52	24%	**214**	High	Mean value of first four columns
Arkansas, AR	5	332	959*	438	170	170	420	475	385	341	72%	**385**	Med.	Median value is adopted by ignoring Const. Report and Estimated
Delaware, DE	10	-	414	-	203	203	440	309	309	149	48%	**309**	Low	Mean of Const. Report and Estimated
Florida, FL	12	464	-	448	530	1463*	460	481	464	44	9%	**464**	High	Median value of first four columns
Iowa, IA	19	29*	171	133	130	130	150	116	132	61	52%	**132**	Med.	Median value by ignoring monitored
Kansas, KS	20	203	241	200	268	-	250	228	222	33	14%	**228**	High	Mean value of first four columns
Louisiana, LA	22	-	-	-	524	524	-	524	524	-	-	**524**	Low	Only Const. Report
Michigan, MI	26	77	-	189	-	-	70	133	133	79	59%	**189**	Med.	Only from Axle Load Spectrum
Montana, MT	30	-	-	81	174	-	-	127	127	66	52%	**127**	Med.	Mean value of first four columns
Nebraska, NE	31	111	136	87	119	119	100	113	115	21	18%	**113**	High	Mean value of first four columns
Nevada, NV	32	525	492	323	560	799	540	475	509	105	22%	**475**	High	Mean value of first four columns
New Mexico, NM	35	147	150	125	393	393	150	204	149	127	62%	**149**	High	Ignore Const. Report
Ohio, OH	39	390	-	380	507	-	70	426	390	71	17%	**390**	High	Median value of first four columns
Oklahoma, OK	40	-	-	-	281	280	-	281	281	-	-	**281**	Low	Only Const. Report
Texas, TX	48	-	-	-	1000	10	-	1000	1000	-	-	**360**	Low	*Using construction report traffic data & TF from FHWA
Virginia, VA	51	257	917*	187		-	330	454	257	403	89%	**257**	High	Ignore Estimated value
Wisconsin, WI	55	-	-	134	189	-	-	161	161	39	24%	**161**	Med.	Mean value of first four columns

Note: * Data considered as outlier

4.3.2 Material Data

The data pertaining to various material related properties of various pavement layers in the construction of each pavement section have been categorized in material data. The data used in this research mainly include the material properties of the subgrade soil (passing #200 sieve and Atterberg Limits). These data were used to verify the subgrade soil classification (fine or coarse). However, in Release 17.0 of the DataPave this data is only available for 44% and 45% of the sections for soil gradation and plasticity index respectively. Therefore, the materials code available in the materials field was used to get the soil type for each section within SPS-1 experiment.

4.3.3 Design versus Actual Construction Review

The SPS-1 experiment is based on the fractional factorial design i.e., all the combinations between levels of various factors were not taken in the design factorial. However, the design matrix was populated with equal number of sites within each climatic zone. To ascertain the homogeneity of the planned experiment with actual sections in the field, in this section the site and structural factors will be compared between as-designed versus as-constructed. First a brief discussion on the construction issues will be presented, and then the deviations in the site and design features within the SPS-1 experiment will be presented.

Construction Issues

The construction guidelines as discussed in Chapter 2 were specified for each site within the experiment. However, there were some deviations and construction issues related to the some of the sites. This information was obtained from the construction reports. A brief site wise discussion on construction issues can be found in Appendix A1 of reference [1]. Some of major construction issues which may have adverse effects on

the pavement performance for some particular sites in SPS-1 experiment are summarized below.

For SPS-1 site in Kansas [KS (20)], it was mentioned in the construction report that:

- The contractor experienced several problems during construction, many of which were caused by the weather. The area experienced much higher than average precipitation during spring 1993, resulting in delays and a wet subgrade. To dry out the subgrade, the contractor was allowed to incorporate fly ash.
- During the FWD testing, high deflections were measured in the base in some areas.
- There was also segregation in the mix; these problems were "corrected" with adjustments in construction methods.

Similarly for Texas [TX (48)], it was found that most of the sections prematurely failed in rutting [4]. This rutting was attributed mainly to asphalt layers because of following reasons:

- Excessive asphalt content in the top layer.
- Change in the gradation of the aggregates without modifying the asphalt mix.

Site Factors

The SPS-1 experiment design stipulates that a total of twenty four (24) similar designs will be replicated across eighteen (18) sites in the US. The experiment, designed in a factorial manner to enhance implementation practicality, permits the construction of 12 test sections (0101-0112 or 0113-0124) at one site with the complementary 12 test sections to be constructed at another site within the same climatic region on a similar subgrade type[5]. Table 4-4 lists the intended sites in each subgrade type-climate zone combination within the SPS-1 experiment [2, 3].

However, the LTPP IMS data (DataPave 3.0) shows that the sites within the SPS-1 experiment design are not balanced. This deviation was found to be mainly due to: (i) different cutoff values used for categorizing the "wet/dry" and "freeze/non-freeze"

environments and, (ii) difference between geographical locations and particular climate at a specific site. The climatic data available for the sites were used to categorize sites into four (4) climatic zones according to LTPP definitions for the climatic zones. All the SPS-1 sites were appropriately classified. Figure 4-4 shows the scatter plot between rainfall and freezing Index (FI) for all sites in SPS-1 experiment. It can be observed from this plot that some of the sites are close to the threshold between wet-no-freeze and wet-freeze. However, as mentioned above, the cut off defined by the LTPP was used for location of the sites within different climatic regions.

The as-constructed location of the different sites is shown in Table 4-5. Further, it can be seen that there are more sites available in wet climate (8 and 6 in "freeze" and "no-freeze" respectively). There are only four sites in the dry climate (DF and DNF zones); the two sites present in DF zone are constructed on a coarse subgrade type. Therefore, the effect of subgrade type can not be determined in DF zone. These deviations are expected to affect the analysis (the experiment design will become unbalanced). Consequently, the analysis of the SPS-1 experiment design mainly focuses on the WF and WNF zones.

Table 4-4 Intended SPS-1 site factorial [2]

Subgrade Type	Wet		Dry		Total
	Freeze	**Non-Freeze**	**Freeze**	**Non-Freeze**	
Fine	IA (19) OH (39)	AL (1)	KS (20)	NM (35)	10
	VA (51) MI (26)	LA (22)	NE (31)	OK (40)	
Coarse	DE (10)	FL (12)	NV (32)	TX (48)	8
	WI (55)	AR (5)	MT (30)	AZ (4)	
Total	6	4	4	4	18

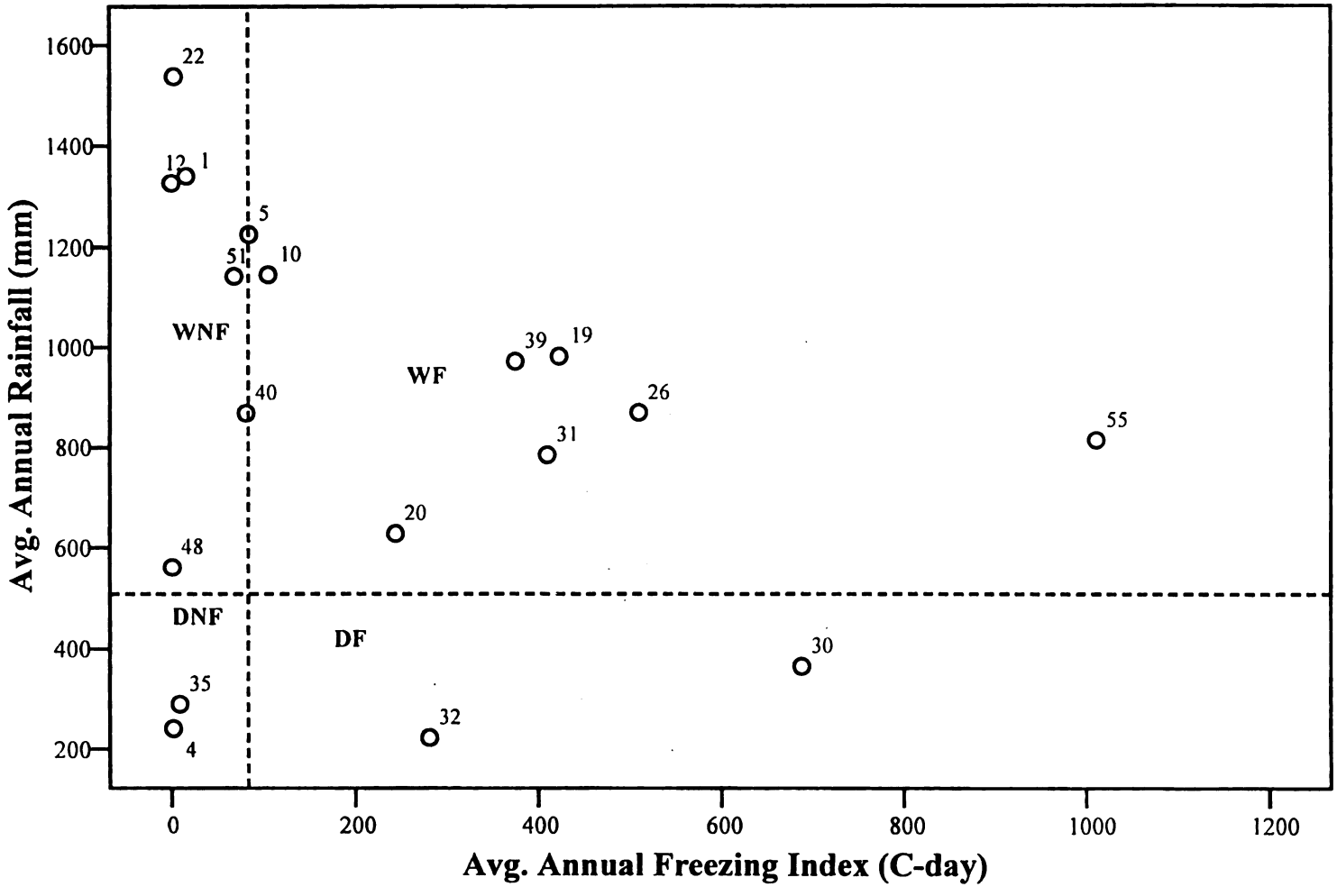


Figure 4-4 Scatter plot showing site distribution by climate

Table 4-5 SPS-1 site factorial — From DataPave 3.0

Subgrade Type	Wet[a]		Dry[b]		Total
	Freeze[c]	**Non-Freeze[d]**	**Freeze**	**Non-Freeze**	
Fine	IA (19) OH (39)	AL (1)	-	NM (35)	9
	KS (20) MI (26) NE (31)	LA (22) VA (51)	-	-	
Coarse	DE (10)	FL (12) TX (48)	NV (32)	-	9
	AR (5) WI (55)	OK (40)	MT (30)	AZ (4)	
Total	8	6	2	2	18

Note:

a. Wet Regions — Average Annual Rainfall > 20 inches (508 mm)
b. Dry Regions — Average Annual Rainfall < 20 inches (508 mm)
c. Freeze Regions — Average Annual Freezing Index > 83.3 °C-day (150 °F-day)
d. Non-Freeze Regions — Average Annual Freezing Index < 83.3 °C-day (150 °F-day)

Design Factors

The design or structural features which are considered to be the main experimental factors in the SPS-1 experiment are:

- AC Thickness (4 versus 7 inches)
- Base Thickness (8, 12 and 16 inches)
- Base Type (DGAB, ATB and ATB/DGAB)
- Drainage (No or Yes)

Each of the above features will be reviewed in this section to identify any deviation from the target values. The asphalt and base layers were targeted for 2 and 3 thickness levels respectively; however, the construction of these target values may contribute variability in these thicknesses. The amount of variability introduced by the construction and how this variability can affect the analysis will be discussed in this section.

Layer Thickness

The as-constructed asphalt and base thickness were compared with their respective target thickness. The results of this comparison are given below.

AC Thickness: The SPS-1 experiment has two levels of HMA surface thickness — 4-inch (102 mm) and 7-inch (178 mm). The allowable deviation from the target HMA surface thickness according to guidelines is 6.53 mm. Table 4-6 shows the summary statistics for each level of asphalt thickness. Among sections with target thickness of 102 mm, the as-constructed thicknesses between all 18 sites has a coefficient of variation (CoV) of 12.7% with about 43% of the sections within the allowable deviations and 49% sections having more asphalt thickness than the allowable upper limit. Only 7.5% of the sections have slightly less asphalt thickness than the allowable lower limit. Similarly, for the pavement designs which were targeted for 178 mm, the as-constructed asphalt thickness has a CoV of about 9% with about 78% of the sections meeting the tolerable limits or having higher asphalt thickness than the upper limit. The frequencies of as-constructed asphalt thickness are shown in Figure 4-5, whereas Figure 4-6 shows the scatter of asphalt thickness in different sites within the SPS-1 experiment. The overall low values of CoV for as-constructed asphalt thickness between all sites show that the asphalt thickness was quite well controlled during construction, especially for 7-inch (178 mm) target HMA surface thickness.

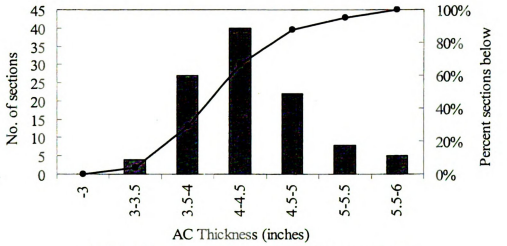
Base Thickness: The SPS-1 experiment has three levels of base thickness— 8-inch (203 mm), 12-inch (305 mm) and 16-inch (406 mm). The allowable deviation from the target base thickness according to guidelines is 12.7 mm. The summary statistics for as-constructed base thicknesses at each level are shown in Table 4-6. Among sections with

target thickness of 203 mm, the as-constructed thicknesses between all 18 sites has a coefficient of variation (CoV) of 10.1% with about 65% of the section within the allowable deviations and 25.3% sections having more base thickness than allowable higher limit. Only 9.2% of the sections have slightly less thickness than allowable lower limit. Similarly, the designs which were targeted for 305 mm, the as-constructed thickness has a CoV of 4.7% with about 79% of the sections meeting the tolerable limits or either have higher thickness than the higher limit. The designs with targeted 406 mm of base thickness have a CoV of 4.6% with about 22% of the section having slightly less thickness than the lower limit. The frequencies and scatter plots of as-constructed base thicknesses are shown in Figure 4-7. The overall low values of CoV for as-constructed base thickness for all levels between all sites show that the base thickness was also quite well controlled during the construction.

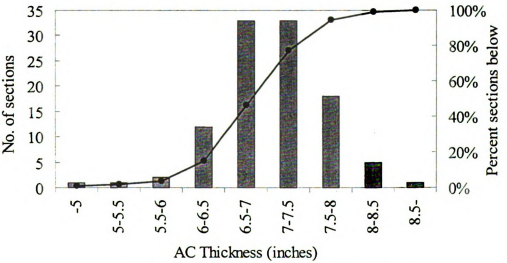
The variations between as-constructed and target thickness for asphalt and base within some sites have shown significant difference (see Figure 4-6 and Figure 4-7). This variation may affect the pavement performance for these sites; therefore, the deviations between target and actual thickness were taken as covariates in the analysis of variance.

Table 4-6 Summary of comparison between target and as-constructed layer thickness

Pavement Layer / Target thickness	Count	Mean (inches)	Std	CoV (%)	Comparison with allowable deviation		
					< Lower limit	With tolerable limit	> Upper limit
AC Layer							
4-inch (102 mm)	106	4.38	0.557	12.7	< 3.75=7.5%	3.75-4.25=43.4%	>4.25=49.1%
7-inch (178 mm)	106	7.12	0.654	9.2	< 6.75=21.7%	6.75-7.25=37.7%	>7.25=40.6%
Base Layer							
8-inch (203 mm)	87	8.26	0.84	10.1	<7.50=9.2%	7.50-8.50=65.5%	>8.50=25.3%
12-inch (305 mm)	89	11.9	0.56	04.7	<11.5=21.3%	11.5-12.5=66.3%	>12.5=12.4%
16-inch (406 mm)	36	15.9	0.74	04.6	<15.5=22.2	15.5-16.5=61.1%	>16.5=16.7%

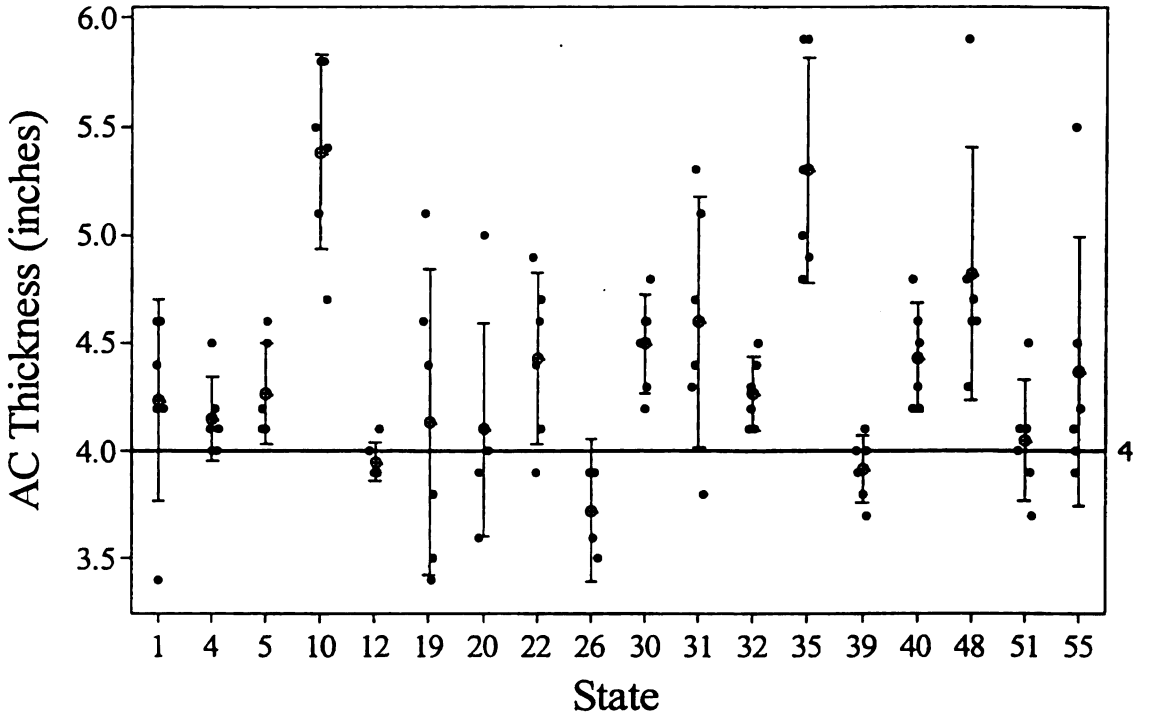


(a) Cumulative frequency for actual AC thickness— 4"target

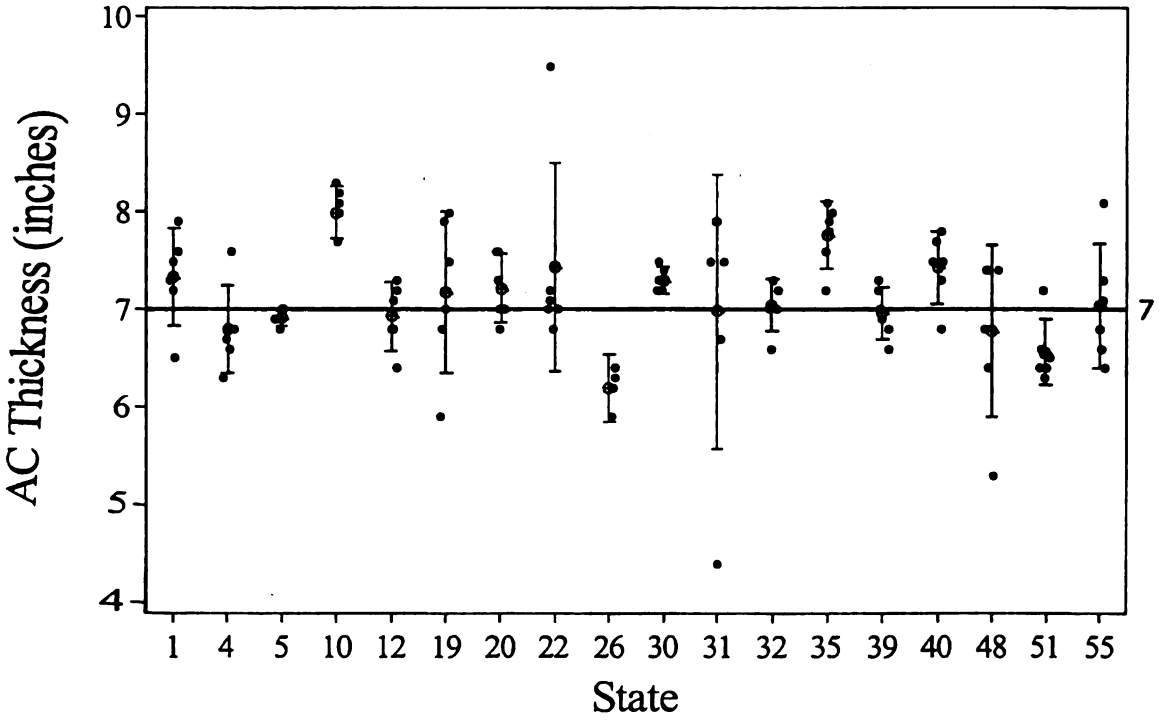


(b) Cumulative frequency for actual AC thickness— 7"target

Figure 4-5 Frequency plot for actual AC thickness

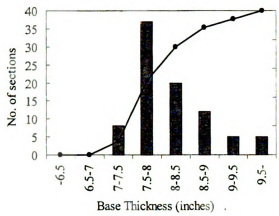


(a) Scatter plot of actual AC Thickness — 4"target

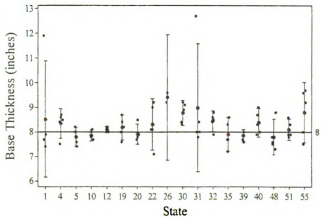


(b) Scatter plot of actual AC Thickness — 7"target

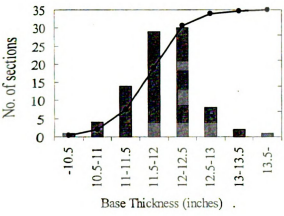
Figure 4-6 Scatter plot for actual AC thickness by site



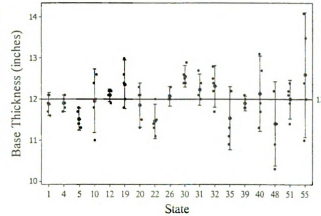
(a) Cumulative freq. for actual base thickness — 8"target



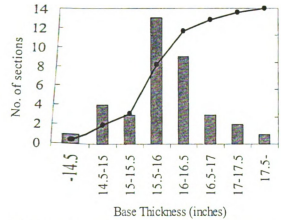
(b) Scatter plot of actual base Thickness — 8"target



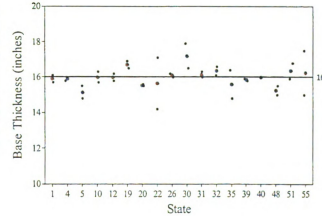
(c) Cumulative freq. for actual base thickness — 12"target



(d) Scatter plot of actual base Thickness — 12"target



(e) Cumulative freq. for actual base thickness — 16"target



(f) Scatter plot of actual base Thickness — 16"target

Figure 4-7 Frequency and scatter plots for actual base thickness

4.3.4 Extent and Occurrence of Distresses

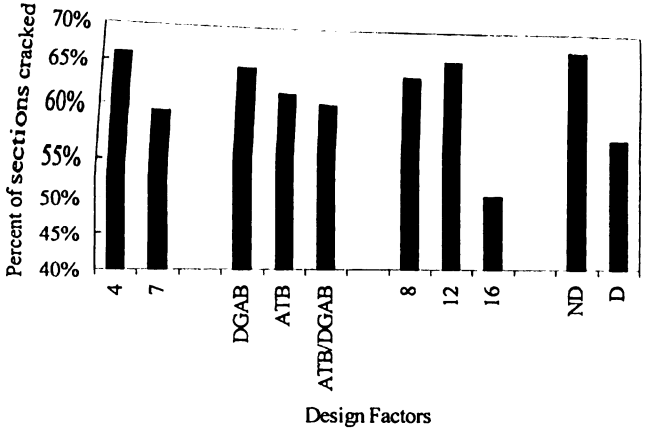
This section of the report presents the availability of the pavement performance data for all SPS-1 sites. The availability of the performance data will be discussed in terms of extent and occurrence of a particular performance measure. The pavement performance measures considered in this research include:

a. Fatigue cracking (total area, sq-m)
b. Longitudinal cracking-WP (length, m)
c. Longitudinal cracking-NWP (length, m)
d. Transverse cracking (length, m)
e. Rut depth (mm)
f. Roughness (IRI, m/km)

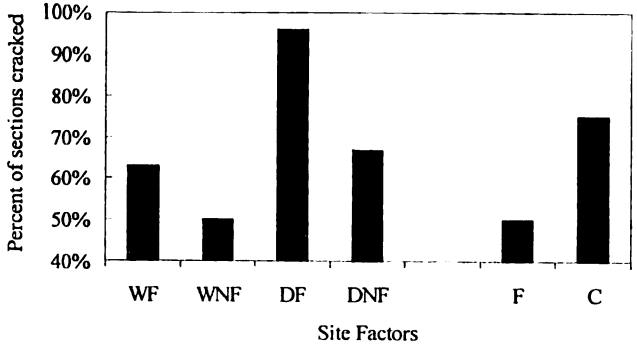
It should be noted that various severity levels of the first four distresses were simply added to calculate the total cracking area or length. The change in roughness ($\Delta IRI = IRI_{latest} - IRI_o$) was considered in the roughness analysis. The extent (mean distress) and occurrence (frequency of distress) are presented below for each performance measure based on data from DataPave (Release 17.0).

Fatigue Cracking

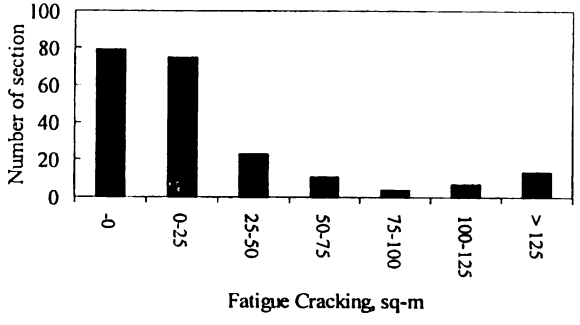
Figure 4-8 shows the occurrence of fatigue cracking in all SPS-1 sections by design and site factors. Based on the latest available data (Release 17.0), about 62% of the sections have exhibited some level of fatigue cracking whereas about 38% of the sections have not yet shown any signs of fatigue [see Figure 4-8 (d)]. Similarly, Figure 4-9 presents the extent of fatigue cracking by design and site factors. The distribution of latest age for all sections is presented in Figure 4-10. It shows that about 10% of the sections can be considered as young (< 3 years), while the overall average for latest age of all sections is 6.5 years. Figure 4-11 shows the variation of fatigue cracking within each site of the SPS-1 experiment.



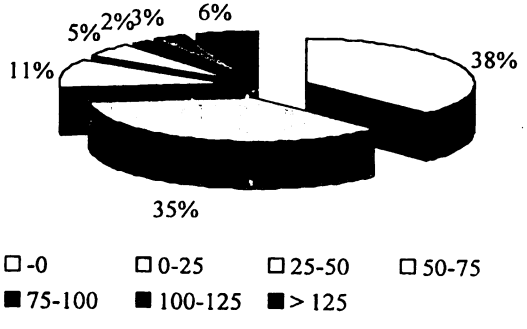
(a) Extent of occurrence of fatigue cracking by design factors



(b) Extent of occurrence of fatigue cracking by site factors

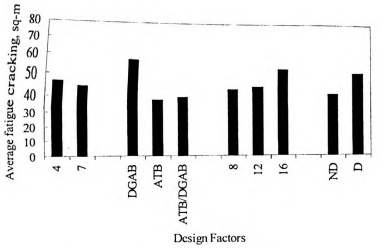


(c) Frequency of sections for fatigue cracking

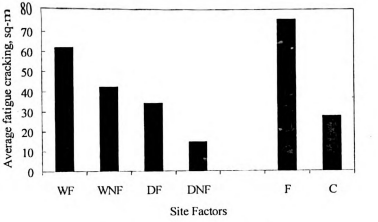


(d) Distribution of sections for fatigue cracking

Figure 4-8 Occurrence of fatigue cracking — SPS-1 experiment

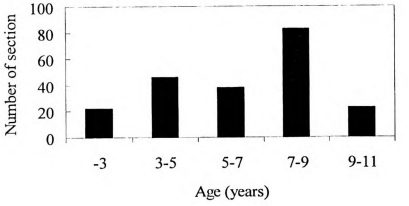


(a) Average fatigue cracking by design factors

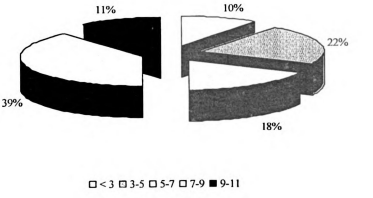


(b) Average fatigue cracking by site factors

Figure 4-9 Extent of fatigue cracking— SPS-1 experiment



(a) Frequency of sections for latest age



(b) Distribution of sections for latest age

Figure 4-10 Age distribution of all cracking distresses — SPS-1 experiment

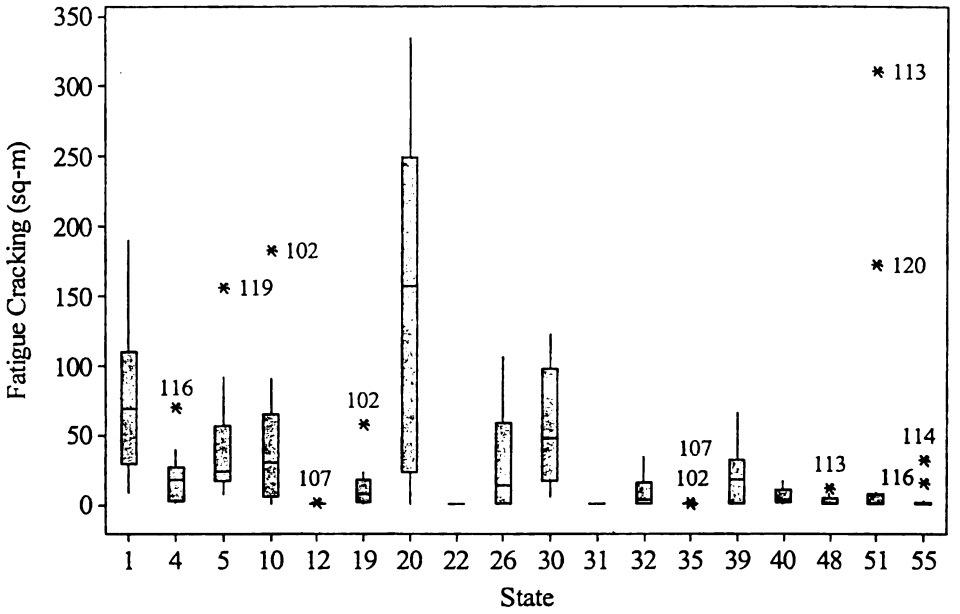


Figure 4-11 Fatigue cracking by site — SPS-1 experiment

Longitudinal Cracking-WP

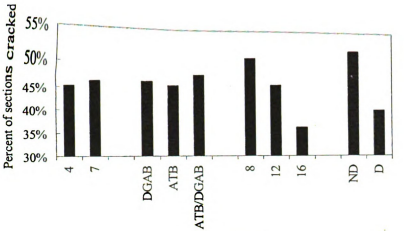
Figure 4-12 shows the occurrence of longitudinal cracking-WP in all SPS-1 sections by design and site factors. Based on the latest available data (Release 17.0), about 46% of the sections have exhibited some level of longitudinal cracking whereas about 54% of the sections have not yet shown any signs of cracking [see Figure 4-12 (d)]. Similarly, Figure 4-13 presents the extent of longitudinal-WP cracking by design and site factors. Figure 4-14 shows the variation of longitudinal cracking-WP within each site of the SPS-1 experiment.

Longitudinal Cracking-NWP

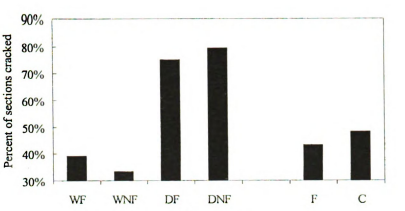
Figure 4-15 shows the occurrence of longitudinal cracking-NWP in all SPS-1 sections by design and site factors. Based on the latest available data (Release 17.0), about 68% of the sections have exhibited some level of cracking whereas about 32% of the sections have not yet shown any signs of cracking [see Figure 4-15(d)]. Similarly, Figure 4-16 presents the extent of longitudinal-NWP cracking by design and site factors. Figure 4-17 shows the variation of longitudinal cracking-NWP within each site of the SPS-1 experiment.

Transverse Cracking

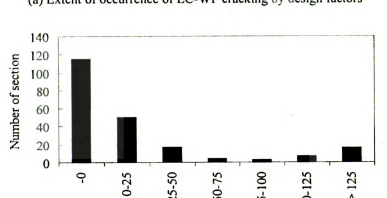
Figure 4-18 shows the occurrence of transverse cracking in all SPS-1 sections by design and site factors. Based on the latest available data (Release 17.0), only 35% of the sections have exhibited some level of transverse cracking whereas about 65% of the sections have not yet exhibited any transverse cracking [see Figure 4-18 (d)]. Similarly, Figure 4-19 presents the extent of transverse cracking by design and site factors. Figure 4-20 shows the variation of transverse cracking within each site of the SPS-1 experiment.



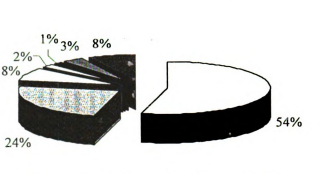
(a) Extent of occurrence of LC-WP cracking by design factors



(b) Extent of occurrence of LC-WP cracking by site factors

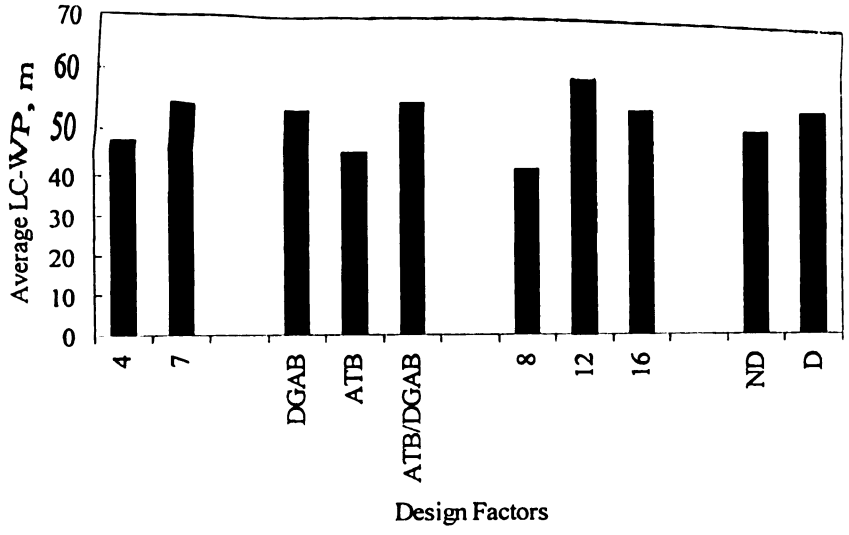


(c) Frequency of sections for LC-WP cracking

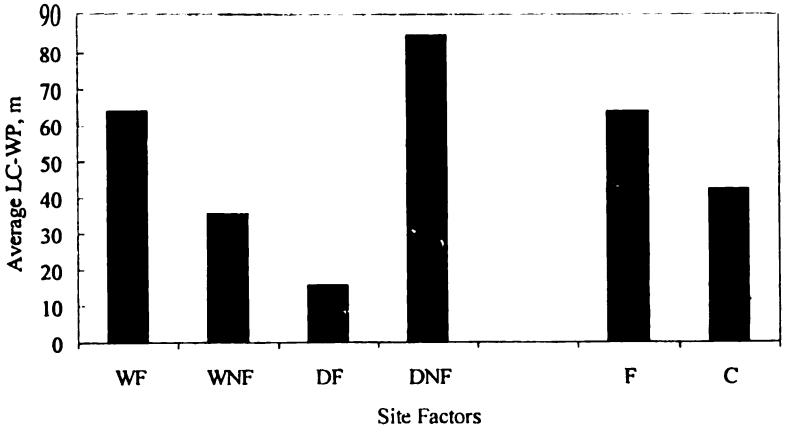


(d) Distribution of sections for LC-WP cracking

Figure 4-12 Occurrence of LC-WP — SPS-1 experiment



(a) Average LC-WP by design factors



(b) Average LC-WP by site factors

Figure 4-13 Extent of LC-WP — SPS-1 experiment

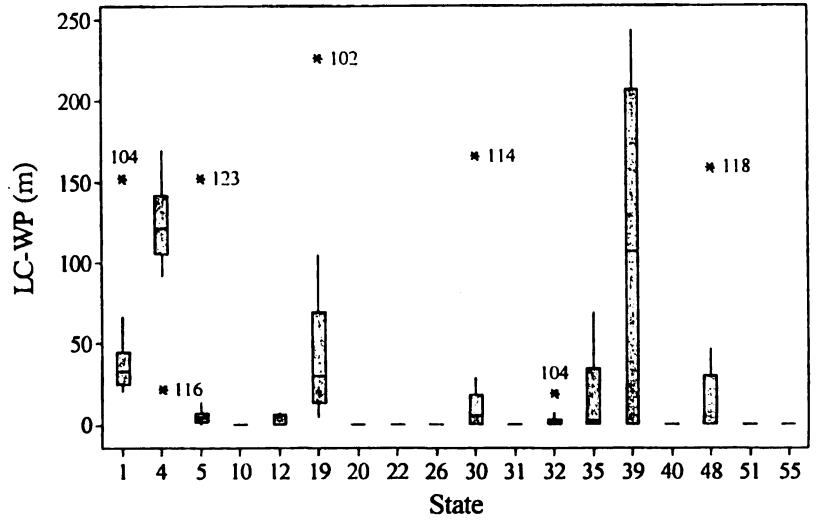
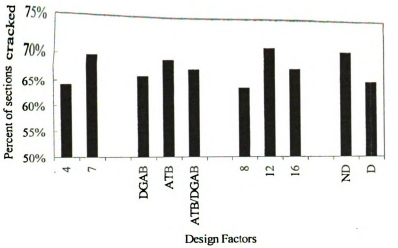
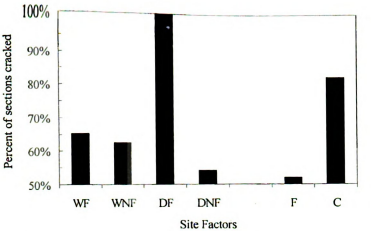


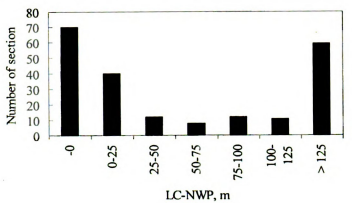
Figure 4-14 LC-WP by site — SPS-1 experiment



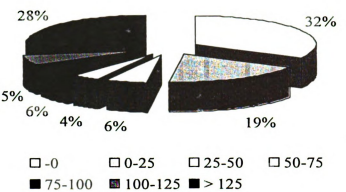
(a) Extent of occurrence of LC-NWP cracking by design factors



(b) Extent of occurrence of LC-NWP cracking by site factors

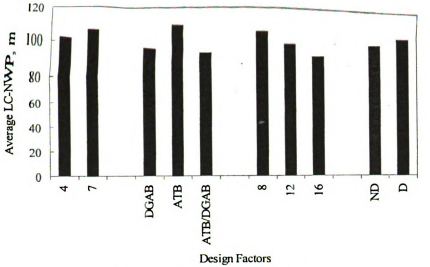


(c) Frequency of sections for LC-NWP cracking

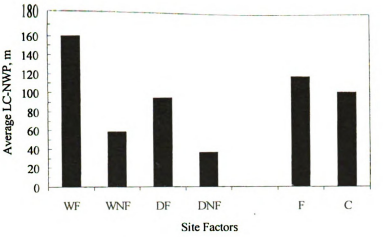


(d) Distribution of sections for LC-NWP cracking

Figure 4-15 Occurrence of LC-NWP — SPS-1 experiment



(a) Average LC-NWP by design factors



(b) Average LC-NWP by site factors

Figure 4-16 Extent of LC-NWP — SPS-1 experiment

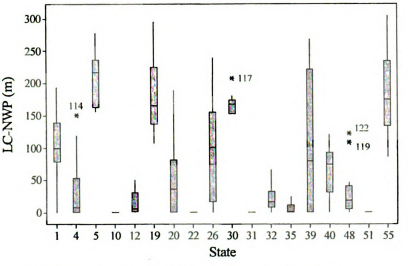
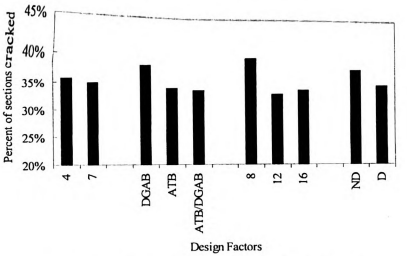
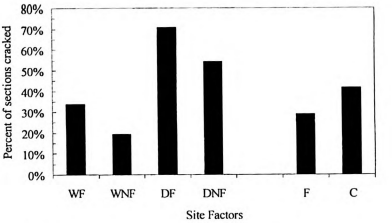


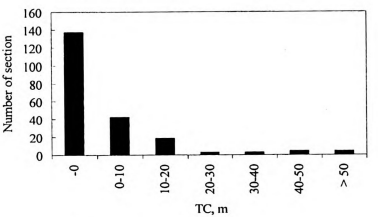
Figure 4-17 LC-NWP by site — SPS-1 experiment



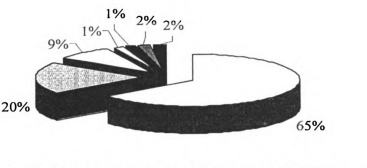
(a) Extent of occurrence of transverse cracking by design factors



(b) Extent of occurrence of transverse cracking by site factors

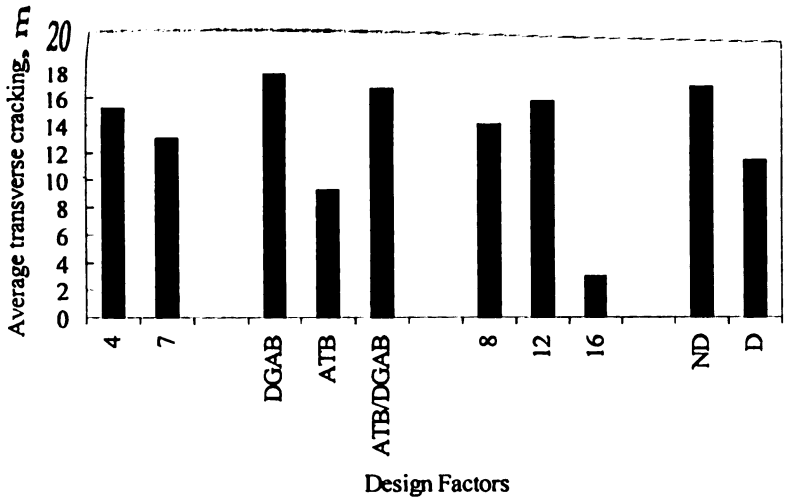


(c) Frequency of sections for transverse cracking

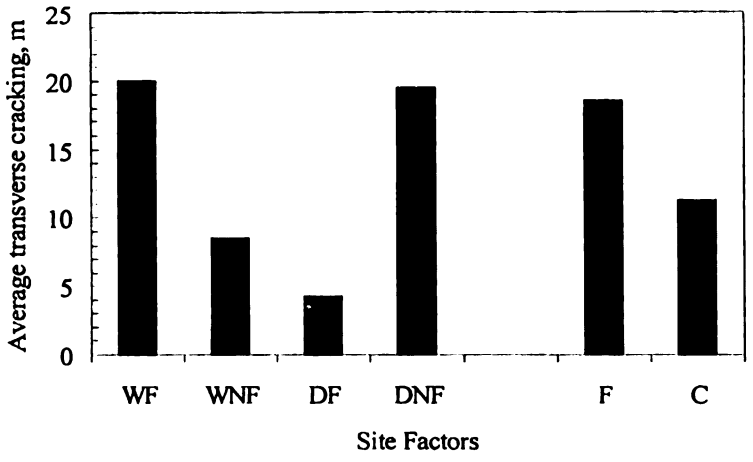


(d) Distribution of sections for transverse cracking

Figure 4-18 Occurrence of transverse cracking — SPS-1 experiment



(a) Average transverse cracking by design factors



(b) Average transverse cracking by site factors

Figure 4-19 Extent of transverse cracking — SPS-1 experiment

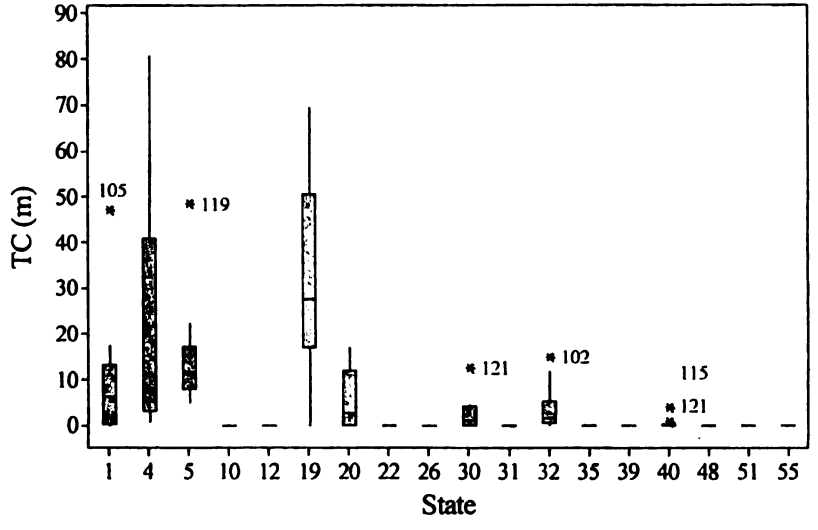


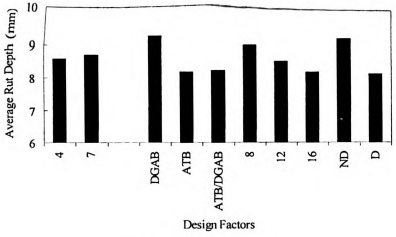
Figure 4-20 Transverse cracking by site — SPS-1 experiment

Rut Depth

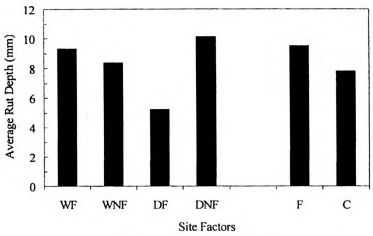
Figure 4-21 shows the extent of rut depth in all SPS-1 sections by design and site factors. Based on the latest available data (Release 17.0), about 29% of the sections have exhibited less than 5 mm of rut depth whereas about 71% of the sections have shown more than 5 mm rut depth, with 10% of the section showing more than 15 mm of rut depth [see Figure 4-21 (d)]. Figure 4-22 shows the variation of rut depth within each site of SPS-1 experiment. Figure 4-23 shows the age distribution of all the pavement sections for the latest rut depth measurement.

Roughness

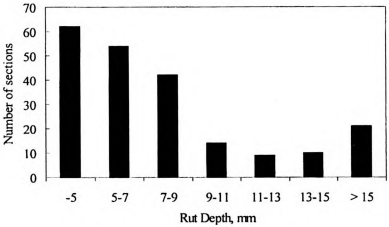
Figure 4-24 shows the extent of change in IRI (ΔIRI) for all SPS-1 sections by design and site factors. Based on the latest available data (Release 17.0), about 23% of the sections have exhibited a negligible change in IRI whereas about 77% of the sections have shown some level of change in IRI, with 10% of the section showing ΔIRI more than 0.4 m/km [see Figure 4-21 (d)]. The data is also summarized for the initial IRI (smoothness just after the construction). Figure 4-25 shows the extent of the initial IRI by design and site factors. It can be seen that about 84% of the sections were built with initial IRI of less than 1.0 m/km and about 16% of the sections with initial IRI more than 1.0 m/km [see Figure 4-25 (d)]. Figure 4-26 shows the variation of roughness within each site of the SPS-1 experiment. Figure 4-27 presents the age distribution of all the sections at the latest roughness profile measurement.



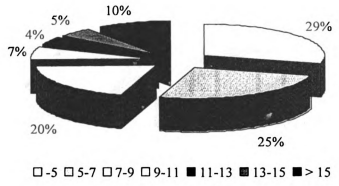
(a) Average rut depth by design factors



(b) Average rut depth by site factors



(c) Frequency of sections for rut depth



(d) Distribution of sections for rut depth

Figure 4-21 Extent of rut depth — SPS-1 experiment

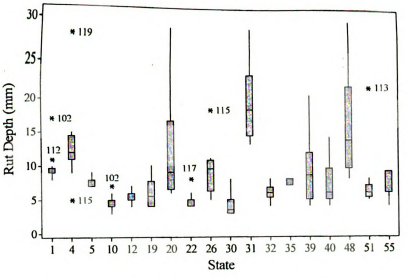
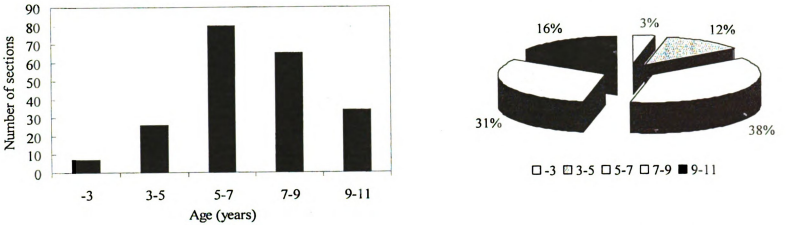


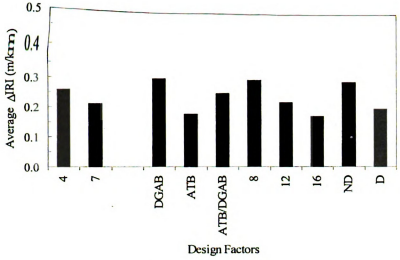
Figure 4-22 Rut depth by site — SPS-1 experiment



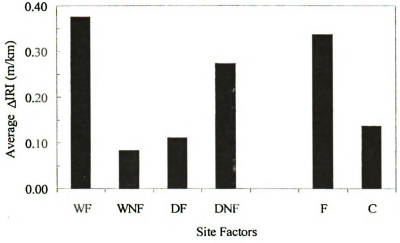
(a) Frequency of sections for latest age

(b) Distribution of sections for latest age

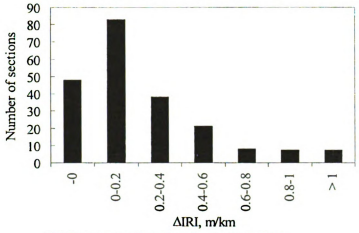
Figure 4-23 Age distribution of rut depth measurement — SPS-1 experiment



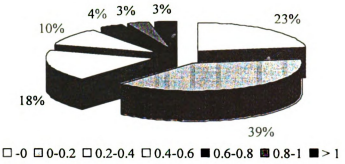
(a) Average change in IRI by design factors



(b) Average change in IRI by site factors

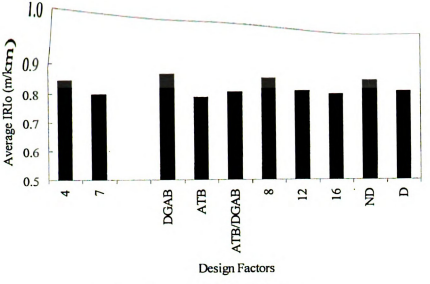


(c) Frequency of sections for change in IRI

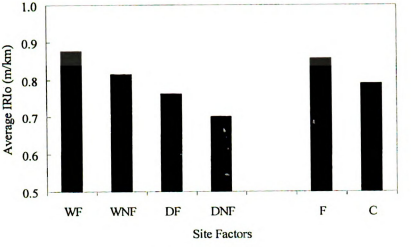


(d) Distribution of sections for change in IRI

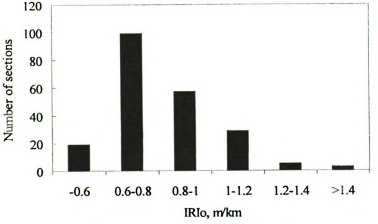
Figure 4-24 Extent of ΔIRI— SPS-1 experiment



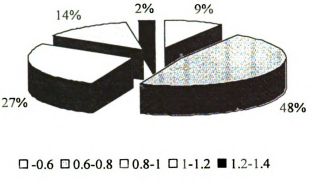
(a) Average initial IRI by design factors



(b) Average initial IRI by site factors

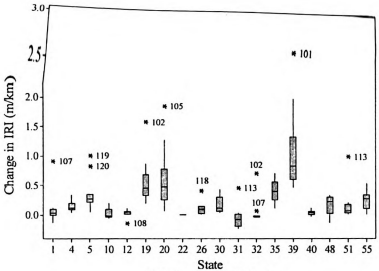


(c) Frequency of sections for initial IRI

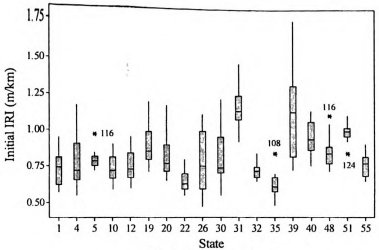


(d) Distribution of sections for initial IRI

Figure 4-25 Extent of IRI_o— SPS-1 experiment

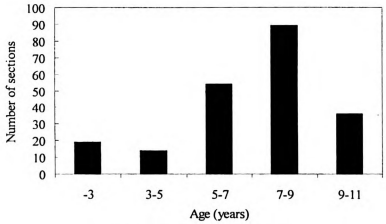


(a) Change in IRI by site

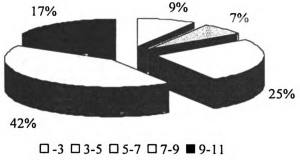


(b) Initial IRI by site

Figure 4-26 Roughness by site — SPS-1 experiment



(a) Frequency of sections for latest age



(b) Distribution of sections for latest age

Figure 4-27 Age distribution of roughness measurement — SPS-1 experiment

4.3.5 Dynamic Load Response Data (DLR) — Flexible Pavements

This section of the report summarizes the data availability for the instrumented flexible pavement sections in OH (39). According to Ohio University report [6], eight series of controlled truck tests had been completed on these instrumented pavement sections as shown in Table 4-7.

Each series of tests followed a similar pattern with regards to how the tests were setup and conducted. The general steps followed during each test are discussed in reference [6].

Only series II data and part of series IV data are available in DataPave (Release 17.0). Also data pertaining to instrumented SPS-8 sections in Ohio are not available in the database. The testing setup details have been obtained from DLR_TEST_MATRIX table. The locations of strain gauges and LVDTs data were obtained from DLR_STRAIN_CONFIG_AC and DLR_LVDT_CONFIG tables. The depth at which strain gauges were installed is not available in the DataPave; therefore this data was obtained from Ohio University report [6]. The peak strain, deflection and pressure data were extracted from DLR_STRAIN_TRACE_SUM_AC, DLR_LVDT_TRACE_SUM_AC and DLR_PRESSURE_TRACE_SUM_AC tables. Only data collected from these instrumented sections in 1996 and 1997 are available in DataPave. The specifics of the tests during series II (in 1996) are listed in Table 4-8 and Table 4-9. The test dates for which strain data are available in DataPave are shaded in grey. Table 4-10 details the series IV test sequence, which is available in the Release 17.0 version of DataPave.

Table 4-7 Controlled vehicle parameters

Test Dates	Test Series	Truck	No. Passes	Section Monitored	Dynamic Parameters					
					Load	Speed	No. Axles	Axle Spacing	Tires	Veh. Dyn.
12/95 3/96	I	CNRC	144	1	X	X	X	X		
8/96	II	Single Tandem	85 87	6	X	X				
6/97	III	CNRC Tandem	127 122	7 7	X X	X X	X	X	X	X
7/97 8/97	IV	Single Tandem	77 77	12	X	X				
10/98	V	Single Tandem	72 60	8	X	X				
9/99 10/99	VI	Single Tandem	86 86	8	X	X				
10/99	VII	Single Tandem FWD Dynaflect	30-60/sec. 30-60/sec 50 drops/sec. 20 read/sec	7	X	X				
4/01 5/01	VIII	Single Tandem	80 80	10	X	X				

Table 4-8 Series II Truck Parameters – ODOT Single-Axle Dump Truck

Date	Nominal Load (kips)	Rear Axle (kips)	Nominal Speed (mph)	Load I.D	Run No.
8/6/96	18	18.45	30,40,50	C	1-14
8/7/96	18	18.45	30,40,50	C	1-14
8/9/96	22	22.23	30,40,50	C	1-13

Table 4-9 Series II Truck Parameters – ODOT Tandem-Axle Dump Truck

Date	Nominal Load (kips)	Rear Axle Load (kips)		Nominal Speed (mph)	Load I.D	Run No.
		Lead	Rear			
8/2/96	32	16.62	16.23	30,40,50	A	1-17
8/3/96	32	16.62	16.23	30,40,50	A	1-15
8/5/96	42	21.14	21.38	30,40	B	1-11
8/6/96	42	21.14	21.38	30,40,50	B	1-16

Table 4-10 Series IV Truck Parameters – ODOT Single-Axle Dump Truck

Date	Nominal Load (kips)	Rear Axle (kips)	Nominal Speed (mph)	Load I.D	Run No.
7/2/97	18	17.35	30,40,50	K	1-20
7/3/97	22	24.95	30,40,50	L	1-18

Source: [6]

It was also observed that not all the runs conducted during each test series for a specific date and sections are available in the strain data (see details in Appendix A3 of reference [1]). Furthermore, strain data is not available for all gauges or all speed levels. For example, in section 39-102 data recorded for only 3 strain gauges are available in the database, whereas, the instrumentation plan for this section shows that there are 6 strain gauges located under the asphalt layer. Appendix A3 of reference [1] shows the average peak strain values of the data for different offset categories.

Data summaries were also prepared for surface deflection data (from LVDT) and pressure data (from pressure cells) within each section, and are attached in Appendix A3 of reference [1].

Discrepancies in Dynamic Load Response Data

The data availability for dynamic load response for the SPS-1 experiment in the current version of DataPave has highlighted several discrepancies. These deficiencies can seriously affect the usefulness of this data for any type of analysis; some of these shortcomings are highlighted here for future improvements:

- Keeping in consideration the amount of data collected for these instrumented sections; only limited data from series II and series IV are currently available (DataPave Release 17.0).
- The direction of strain gauges (Longitudinal or transverse) is not available in DataPave. These had to be obtained from the Ohio University report [6].
- Similarly, the depth of strain gauges from the surface is also currently missing from the database.
- In order to validate the dynamic load response mechanistically, the material properties for pavement layers have to be calculated at the time of testing. To facilitate this objective, the time of testing and temperature should be included as a part of dynamic load response data. Also it would have been useful to have data from FWD testing conducted at the locations at the strain gauges and pressure cells.

4.4 DATA AVAILABILITY IN SPS-8 EXPERIMENT – FLEXIBLE PAVEMENTS

This section of the report summarizes the data availability for flexible pavement sections in the SPS-8 experiment. The availability of all relevant data elements is summarized in Table 4-11. The table shows that availability of the main factors is high, while that of the exogenous factors is somewhat lower. In particular, the availability of traffic data is low; however, the impact of traffic data may be insignificant for the SPS-8 experiment if all the sites have very limited traffic. The availability of the relevant data elements in SPS-8 experiment is discussed in the following sections of the report.

4.4.1 General Site Information

Each site has unique characteristics, which can be mainly explained by the particular climatic and soil conditions at a particular location. The SPS-8 experiment mainly focuses on pavement performance based on the environmental aspects of sites in combination with different subgrade types. The particular site information can be further divided into construction, climate and traffic.

Construction Issues

The construction reports prepared by the supervisory consultants for each site were reviewed to identify the deviations/problems during the construction of each site. These deviations might be helpful in explaining the unusual trends in performances (premature failures) at a particular site.

The summary of deviations has been prepared for all 15 sites in the SPS-8 experiment and is given in Appendix C of reference [1].

Table 4-11 Summary of SPS-8 data element availability –Flexible pavements

Data Category	Data Type	Data Availability, %
Site Information	*Construction Reports*	93
	Climatic data	
	Virtual Weather Station	
	Annual Temperature	**93**
	Annual Precipitation	**93**
	Automatic Weather Station	
	Monthly Temperature	47
	Monthly Precipitation	47
	Traffic data	
	Traffic Open date	**93**
	Estimated ESALs	**60**
	Monitored ESALs	**33**
	Axle Load Spectrum	**33**
Material Data	*Asphalt Layer*	
	Core Examination	80
	Bulk Specific Gravity	75
	Max Specific Gravity	78
	Asphalt Content	78
	Asphalt Resilient Modulus	19
	Penetration	69
	Viscosity	65
	Asphalt Specific Gravity	69
	Aggregate Gradation	81
	Fine Aggregate Particle Shape	21
	Layer Thickness	**100**
	Unbound Base Gradation	78
	Subgrade	
	Subgrade Gradation	78
	Atterberg Limits	84
	Subgrade Modulus	44
Pavement Structure	*Layer details*	
	Type	**100**
	Representative thicknesses	**100**
	Constructed thicknesses	100
	Shoulder information	
	Type	93
	Width	93
	Thickness	93
Monitoring**	*FWD data*	
	Deflections	**100**
	Temperature at Testing	100
	Backcalculated Moduli	13
	Manual Distresses data	**100**
	Longitudinal Profile (IRI)	**100**
	Transverse Profile (Rut Depth)	**100**

Note: ** Data is said to be available for a section even if it is available for one survey.

Climatic Data

As explained before for the SPS-1 experiment, the average annual rainfall and average annual freezing index were used to classify each site into four climatic regions. The classification definitions for each zone were taken from the LTPP DataPave. The summary of the climatic data from the VWS for all sites in SPS-8 is given in Table 4-19. Only climatic data for CA (6) is not available in Release 17.0 of DataPave.

Traffic data

The SPS-8 experiment design stipulates that traffic volume in the study lane be at least 100 vehicles per day but not more than 10,000 ESAL per year. Therefore, it is important to check the traffic not exceeding the threshold specified for this experiment. The traffic data is only available for 8 out of 15 sites from estimate and monitoring modules of DataPave (Release 17.0). No traffic data is available for AR (5), CA (6), MO (29), NJ (34), NM (35), NC (37) and WI (55).

4.4.2 Material Data

The material properties of all the layers in a pavement system play a very significant role in its future performance. The SPS-8 Experiment was designed to study the specific effects of a range of environments on the pavement performance; therefore the material properties which are susceptible to climatic changes need to be investigated. In this experiment the subgrade type was a factor (fine or coarse), while the asphalt mix and base material properties were assumed to be uniform across all states. The subgrade material properties were investigated. The summary of soil gradation and Atterberg limit information required for classification is given in Table 4-13.

Table 4-12 Summary of Environmental data of the sections in SPS-8

State	Climatic Zone	AATP[1] (mm)	AIPD[2] (days)	WDPY[3] (days)	Avg. Days Above 32 °C	Avg. Days below 0 °C	AAT[4] (°C)	FI (deg days)	FT (cycles)
050800	WNF	1374	34	133	64	52	17	46	48
080800	DF	372	7	95	31	162	10	326	142
280800	WNF	1427	37	145	52	65	16	57	60
290800	WF	1079	27	144	37	105	13	167	92
29A800	WF	945	22	137	29	112	12	334	84
300800	DF	371	4	132	4	198	6	574	163
340800	WF	1071	27	119	8	68	13	127	56
350800	DNF	346	5	92	83	99	15	9	100
360800	WF	891	17	193	5	130	9	437	87
370800	WNF	1342	33	151	36	46	17	14	47
390800	WF	972	24	153	10	130	10	374	96
460800	DF	423	8	96	25	175	7	978	107
480800	WNF	1015	24	131	99	19	20	10	18
48A800	WNF	846	22	100	94	35	19	21	34
490800	DF	473	7	118	8	198	7	498	170
530800	WF	510	7	137	30	91	11	169	73
53A800	WF	386	3	135	33	88	11	163	71
550800	WF	814	17	151	4	175	6	1015	96

Note:
1-Average Annual Total Precipitation (mm), 2-Average Intense Precipitation Days in a year, 4-Wet Days per Year,
4-Average Annual Temperature

Table 4-13 Subgrade soil properties for SPS-8 flexible pavements

State	SHRP ID	# 200	HYDRO_02	HYDRO_002	HYDRO_001	COARSE_SAND	FINE_SAND	SILT	CLAY	COLLOIDS	LL	PL	PI	Expansive	SG	Zone	Frost
5	0803	77	34	16	-	0	24	60	16	*	29	17	12	N	F	WNF	Y
5	0804	58	34	18	*	3	30	40	18	-	34	15	10	N	F	WNF	Y
6	A805	11	5	2	-	28	61	9	2	-	-	-	0	N	C	DNF	N
6	A806	14	5	2	-	29	57	12	2	-	-	-	0	N	C	DNF	N
29	0801	63	49	25	22	0	10	38	25	22	44	19	26	Y	F	WF	Y
29	0802	59	57	43	37	2	6	22	43	37	68	26	42	Y	F	WF	Y
29	A801	92	77	41	38	2	2	36	41	38	57	22	35	Y	F	WF	Y
29	A802	87	70	36	30	3	3	35	36	30	58	19	40	Y	F	WF	Y
30	0805	9	6	2	66	11		6	2	-	-	-	0	N	C	DF	N
30	0806	8	6	2	-	14	11	6	2	-	-	-	0	N	C	DF	N
34	0801	8	3	1	-	16	67	7	1	-	-	-	-	N	C	WF	N
34	0802	7	4	1	-	19	67	7	1	-	-	-	-	N	C	WF	N
36	0801	27	13	7	7	5	54	21	7	7	8	5	2	N	C	WF	N
36	0802	6	6	4	-	32	57	3	4	-	0	0	0	N	C	WF	N
37	0801	8	7	2	-	6	84	8	2	-	-	-	0	N	C	WNF	N
37	0802	12	8	4	-	12	76	8	4	-	-	-	0	N	C	WNF	N
39	0804	71	55	28	-	8	15	43	28	-	30	17	13	N	F	WF	Y
46	0803		27	16	12	1	64	20	4	12	36	19	17	N	F	DF	Y
46	0804	35	30	19	10	26	28	35	4	17	39	18	21	Y	F	DF	Y
48	0801	54	17	10	7	23	38	40	10	-	16	9	7	N	F	WNF	N
48	0802	51	23	12	-	7	39	33	12	-	29	23	6	N	F	WNF	N
49	0803	35	19	10	-	9	15	22	10	-	29	15	14	N	C	DF	N
49	0804	34	21	10	-	7	12	23	10	-	37	18	19	Y	C	DF	N
53	0801	61	29	9	-	8	8	53	9	-	31	25	6	N	F	WF	N
53	0802	42	21	5	-	8	7	37	5	-	-	-	0	N	C	WF	N
55	0805	12	7	3	-	20	28	8	3	-	-	-	0	N	C	WF	N
55	0806	14	9	4	-	26	29	11	4	-	-	-	0	N	C	WF	N

Note: Colloidal Content >15% & PI>18, for expansive soils this criterion was adopted (source: Holtz (1959) and U.S.B.R (1974))
Silt, coarse clay having more than 15% material finer than 0.02 mm to be dangerous for frost heave (Source: Holtz & Kovacs, 1981)

In addition, two critical aspects of soil behavior were further investigated from the available soil data: expansion of clayey soils in dry zones and frost susceptibility in freeze zones. The active soils were identified by using the following criteria [7]:

- Expansive Soils— colloidal content >15% and PI>18
- Frost Susceptible Soils— silt, coarse clay having > 15% material finer than 0.02 mm.

By using the above criteria, the subgrade soils in States 29 (Missouri), 46 (South Dakota, section 0804) and 49 (Utah, section 0804) were classified as active (expansive) soils, while sections in States 5 (Arkansas), 29 (Missouri), 39 (Ohio, section 0804) and 46 (South Dakota) were identified as having subgrade soils with frost heave potential.

4.4.3 Design versus Actual Construction Review

According to the original experiment design as discussed in chapter 2, 12 sites were essential required with two different structural designs. These sites were selected based on the geographical location so that they may be located in different climatic regions. However, due to site specific climatic data the region identified at the design stage may be different. Similarly, the target layer thickness may have variability due to construction. The specific as-constructed site conditions are discussed in the section below.

Construction Issues

The construction guidelines for SPS-8 sections were discussed in chapter 2. The construction deviations for each site were taken from the construction reports and are summarized in Appendix C of reference [1].

Site Factors

The SPS-8 flexible experiment design required that two different structural designs should be repeated in at least 12 sites. However, the actual data on the site factors (climate and subgrade) showed that there are 15 sites in the SPS-8 flexible experiment and currently these are distributed according to Table 3-4. There are 7 sites in WF, and 3 sites each for WNF, DF and DNF zones, respectively. Almost half of the sites were constructed on coarse subgrade, and the others were built on fine subgrade soil.

Design Factors

The design or structural features which are considered to be the main experimental factors in SPS-8 flexible pavement experiment include:

- AC Thickness [4-inch (102 mm) versus 7-inches (178 mm)]
- Granular Base Thickness [(8-inch (203 mm) versus 12-inch (305 mm)]

The summary of the as-constructed and target thicknesses for all flexible pavements is given in Table 4-14.

4.4.4 Extent and Occurrence of Distress

The age of the section is a very important factor in the SPS-8 experiment, as a higher age of a particular section will translate in higher environment related distresses. Figure 4-28 shows the latest age for all flexible pavement sections in SPS-8. Further age distribution of flexible pavements among the SPS-8 sections is shown in Figure 4-29. The age data for SPS-8 sections shows that most of these sections are aged below seven years and are in the early stage on the performance curve.

The distress data for the SPS-8 sections was obtained from the files MON_DIS_AC_REV (cracking and non-load related distresses data), MON_T_PROF_INDEX_SECTION (rutting data) and MON_PROFILE_MASTER

(roughness). Figure 4-30 and Figure 4-31 show the occurrence and distribution of distresses in the SPS-8 Experiment flexible pavements. The available distress data in Data Pave (Release 17) has only shown five types of distresses in all SPS-8 flexible pavements.

Figure 4-31 (a) shows the distribution of rutting in SPS-8 flexible pavement sections. It can be observed that only 9% of the sections have shown more than 5 mm of rutting, where as in the majority of the sections (60%) rutting ranges from 3 to 5 mm. A low amount of rutting is expected in the SPS-8 pavements since load is the major cause of rutting in flexible pavements.

Figure 4-31 (b) shows the distribution of roughness data (IRI) based on its magnitude. The data suggests that the majority of sections did not exhibit high levels of roughness, with only 9% of the population with IRI greater than 2 m/km.

Table 4-14 Construction details of the flexible pavement sections in SPS-8

State	SHRP_ID	Subgrade Type	AC	GB	GS[1]	SS[2]	TS	Target AC	Target GB
5	0803	F	3.8	7.3				4	8
5	0804	F	7.2	12.7				7	12
6	A805	C	4.2	8.2				4	8
6	A806	C	6.6	12.2				7	12
28	0805	C	4	9				4	8
28	0806	F	7	12				7	12
29	0801	F	4.9	7.8				4	8
29	0802	F	7.5	11.5				7	12
29	A801	F	4.3	8.3				4	8
29	A802	F	6.9	12.3				7	12
30	0805	C	4.5	7.1				4	8
30	0806	C	6.9	11.8				7	12
34	0801	C	3.5	7.8				4	8
34	0802	C	6.8	11.6				7	12
35	0801	F	4.4	9.7				4	8
35	0802	F	7.3	12.6				7	12
36	0801	C	4.9	8.4		168		4	8
36	0802	C	7.6	10		156		7	12
37	0801	C	4	8.7				4	8
37	0802	C	7	11.5				7	12
39	0803	F	3.9	7.9	36			4	8
39	0804	F	6.6	11.9	30			7	12
46	0803	F	4.8	8				4	8
46	0804	F	7.2	12				7	12
48	0801	F	4	8.5			10	4	8
48	0802	F	5.5	10.7			10	7	12
49	0803	C	4.9	7.8	41.2			4	8
49	0804	C	6.9	12	41.2			7	12
53	0801	F	3.7	8	38.4			4	8
53	0802	C	6.8	11.7	38.4			7	12
55	0805	C	4.4	8				4	8
55	0806	C	7	12				7	12

Note: 1-Granular subbase, 2-SS represents subgrade layer, the thickness in this column is the fill

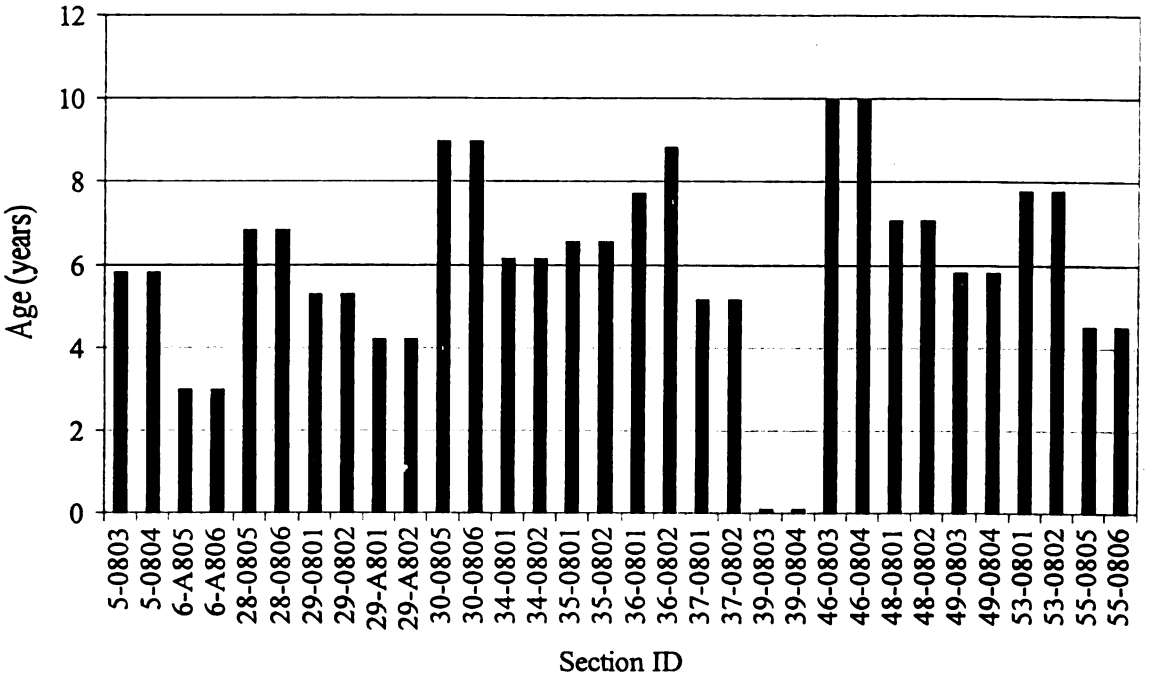
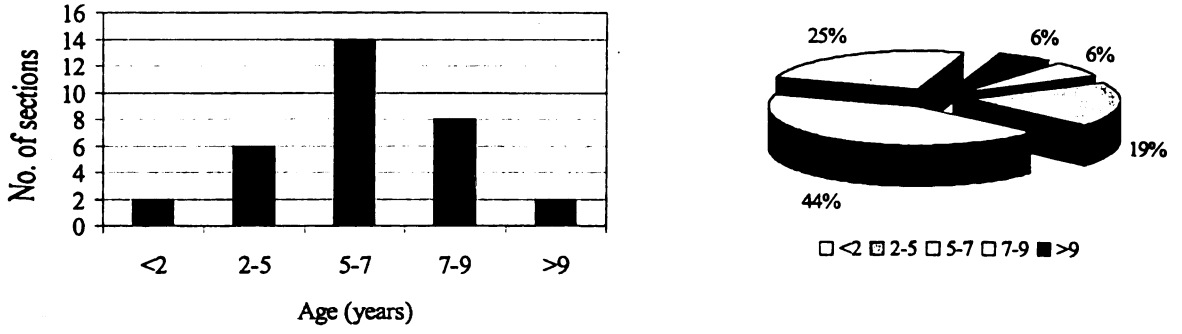


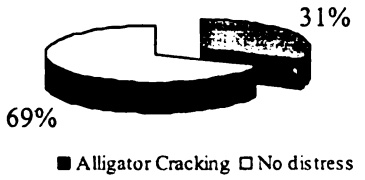
Figure 4-28 Age of the flexible pavements in SPS-8



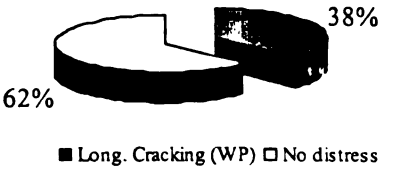
(a) Frequency of age

(b) Distribution of age

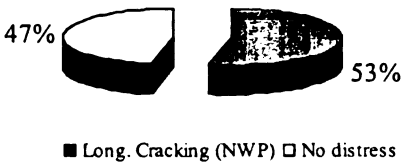
Figure 4-29 Age distribution in the SPS-8 sites — flexible pavements



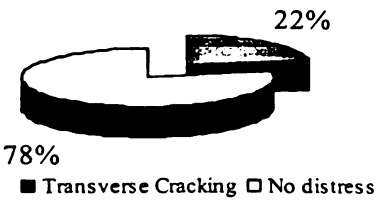
(a) Fatigue cracking



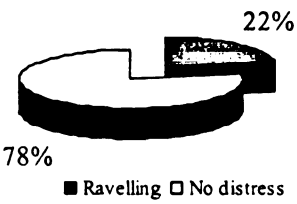
(b) Longitudinal cracking-WP



(c) Longitudinal cracking-NWP

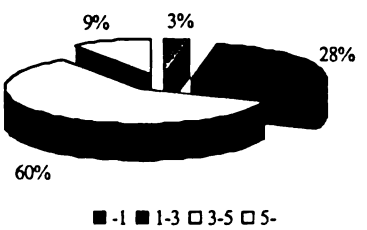


(d) Transverse cracking

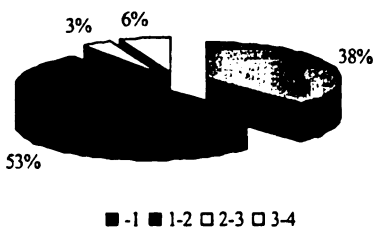


(e) Raveling

Figure 4-30 Distribution of distresses in SPS-8 flexible pavements sections



(a) Average Rut (mm)



(b) Average IRI (m/km)

Figure 4-31 Distribution of IRI and Rutting in SPS-8 flexible pavements

CHAPTER 5 - METHODOLOGY FOR ANALYSIS

5.1 INTRODUCTION

The purpose of this chapter is to provide a summary of analysis methods that were used to perform this research. Some of the previous studies analyzed LTPP data (GPS and SPS experiments) based on engineering criteria ("basic" statistics) and subjective judgment [1-4]. For example, the engineering criteria may include the rate of growth, severity levels and impact of distress on the functionality of the pavement. Several statistical methods were employed for establishing performance criteria to study the effect of design and construction features on pavement performance in this research. The statistical methods range from trend plotting to complex multivariate analysis.

This research focuses on evaluating the effects of specific design and constructions features on the response and performance of flexible pavements (SPS-1 Experiment). The selection of statistical methods was based on the specific objectives of this research and the extent and occurrence of performance data. These methods, as well as the concept of Performance Index (PI) developed and employed in the analysis, are explained within this chapter.

5.2 PERFORMANCE INDICATORS

The performance of a pavement is an accumulation of damage over time. All pavement sections within each SPS-1 site were monitored over time; however, the monitoring of these sections is staggered with age (i.e., the distress data were collected at different times for individual sections), and the performance measures (cracking, rutting and roughness) have shown a variable trend with time. Therefore, it was felt necessary to develop a measure that can quantify the overall performance of a pavement section over

time. Figure **5-1** through Figure 5-4 show various performance curves (cracking and rutting distresses) for twelve test sections within two sites [Alabama, AL (1) and Iowa, IA (19)] of the SPS-1 experiment. These figures show the measurement variability of distresses with time. The following discussion presents various options that were considered to transform the time series data of a section into a single performance indicator. The options considered are listed below:

- Maximum distress at the latest age/survey.
- Area under the performance curve.
- Area under the performance curve normalized to the latest age.
- Performance Index.

Maximum distress at the latest age is one of the options used for time series data analysis. This performance indicator only considers the maximum distress that was recorded for the test section in its monitored lifetime. Also, this performance indicator will not capture the performance trend over time and will provide only a snapshot of performance at a given time. In addition, the measurement variability over time is not taken into account.

Area under the curve represents the actual pavement performance for a distress; larger area indicates poorer performance. The area under the curve can be calculated with the trapezoidal rule by using Equation (5-1).

$$Area = \sum_{i}\left[\left(t_{i+1} - t_i\right)\left(\frac{y_i + y_{i+1}}{2}\right)\right] \qquad (5\text{-}1)$$

The shortcoming of "area under the curve" is that this indicator cannot discriminate the performance of two sections having the same area but with different times for distress occurrence. For example, the performance curves in Figure 5-5 and

Figure 5-6 may have similar "area under the curve" but the curve in Figure 5-6 shows better performance than that in Figure 5-5.

Area under the curve normalized to the latest age can be another alternative which can eliminate the discrepancy of using "area" alone (as mentioned above). This indicator can also be calculated based on the trapezoidal rule and can be represented mathematically by Equation (5-2).

$$\frac{Area}{L_{age}} = \frac{\sum_i \left[\left(t_{i+1} - t_i \right) \left(\frac{y_i + y_{i+1}}{2} \right) \right]}{L_{age}} \qquad (5\text{-}2)$$

Where; "L_{age}" is the latest age used to normalize the "area".

This indicator distributes the performance of a section (area) evenly over all years. However, performance curves can exhibit highly variable trends with time (see Figure 5-1) and may have gaps in the data for some years. Therefore, an alternative indicator was selected, where the performance is weighted with age.

The *Performance Index* (PI) is defined as:

$$PI = \frac{\sum_i y_i \cdot t_i}{\sum_i t_i} \qquad (5\text{-}3)$$

Where:

t_i = the age at distress measurement year i

y_i = distress measured at year i (for example alligator cracking in sq-m, rut depth in mm and IRI in m/km)

Note that only the ages at which distress measurements were taken are included in the calculation of PI. Equation (5-3) can be further simplified to the form of a series as shown by Equation (5-4).

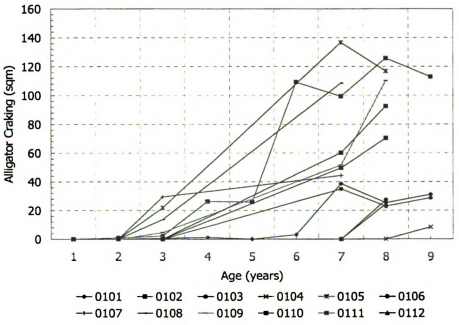


Figure 5-1 Fatigue cracking with age— AL (1)

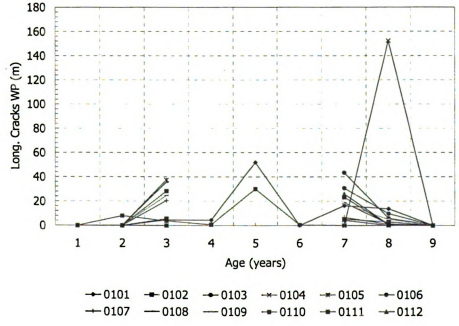


Figure 5-2 Longitudinal cracking-WP with age — AL (1)

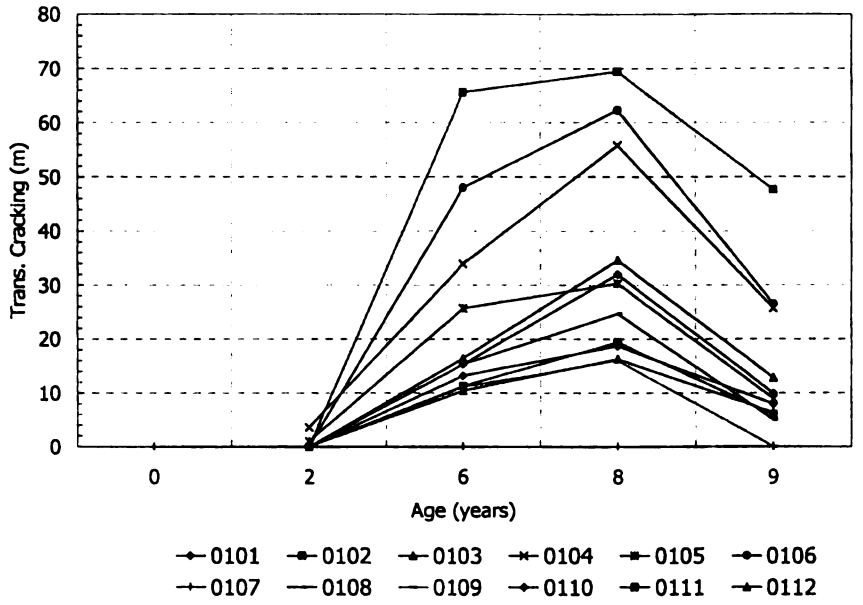


Figure 5-3 Transverse cracking with age — IA (19)

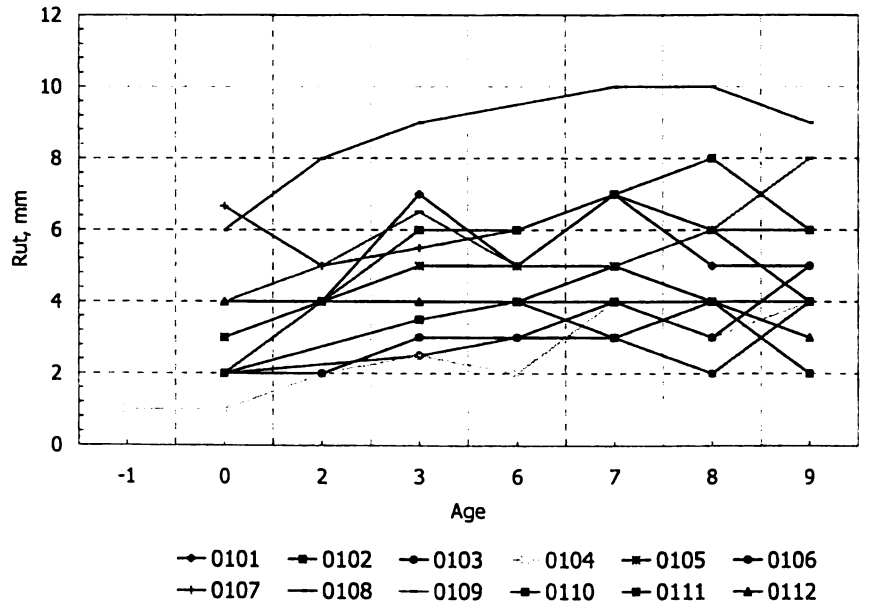


Figure 5-4 Rutting with age — IA (19)

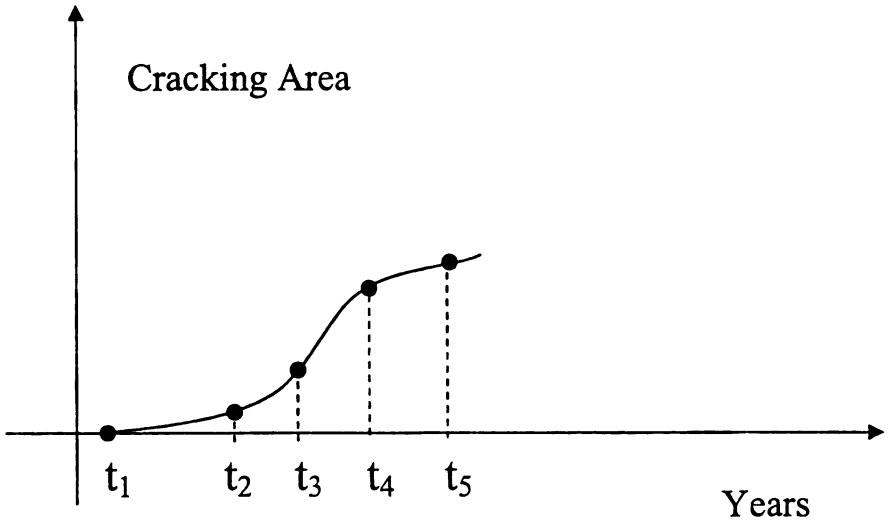


Figure 5-5 Poor Performance

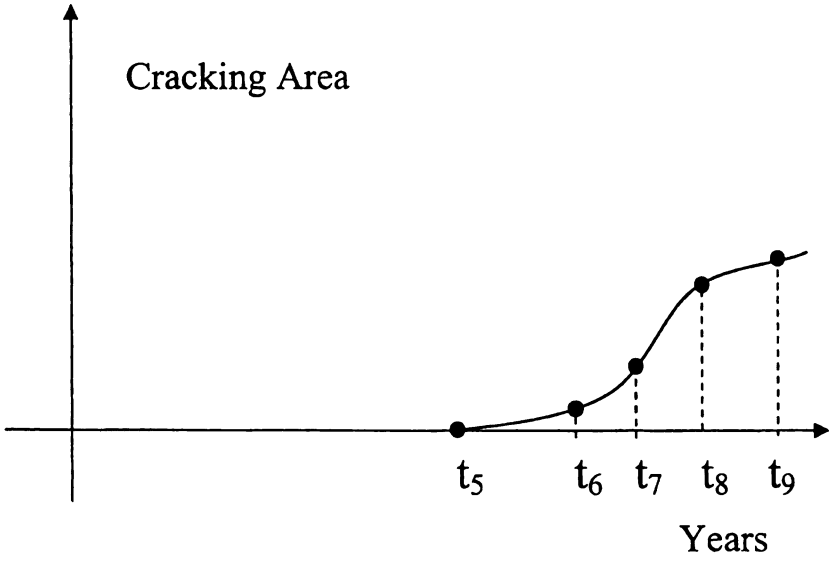


Figure 5-6 Good Performance

$$PI = \frac{y_1.t_1}{\sum_i t_i} + \frac{y_2.t_2}{\sum_i t_i} + \frac{y_3.t_3}{\sum_i t_i} + ... + \frac{y_i.t_i}{\sum_i t_i} \qquad (5\text{-}4)$$

It can be seen from Equation (5-4) that higher weights will be given to the performance measured at the later ages (as $t_{i+1} > t_i$ and $\Sigma\, t_i$ is constant for a given pavement section). This makes the performance index more applicable to the SPS-1 experiment, which stipulates that no maintenance or rehabilitation action should be taken during the life of the pavement sections. The following hypothetical example illustrates the comparisons between various performance indicators discussed above.

Figure 5-7 shows the performance curves for five different pavement sections. The best and the worst performing sections are to be identified from the time series data. Three of the performance indicators were calculated for all five sections and the results are summarized in Table 5-1. It is clear from the results that section D is best performing because the distress remains at the same level over the years and all indicators are capturing this well. The second best section according to "Area" and "Area/L_{age}" is section B; however according to "PI" section A is second best.

By visual comparison of the performance curves for sections A & B, it can be said that section B will deteriorate at a faster rate compared to section A, given the performance history of the sections (see Figure 5-7). As higher weights are given for later years in the calculation of the Performance Index (PI), it is expected that this indicator will be more suitable to capture the present and relative future performance of each section. Therefore, PI was selected from among various performance indicators for this study.

The performance indices (PIs) were calculated for each section and for the different performance measures such as cracking, rutting and roughness. This was calculated by summing the product of distress and age for all available surveys and dividing it by the sum of ages for available surveys, as shown by Equation (5-3). All analyses (overall and site level) were performed using PIs for test sections.

Although PI seems to be the best option among all the performance indicators considered, it has some inherent limitations. These limitations are mainly because PI is dependent on the number and timing of the distress surveys. For two pavement sections of the same age, with one monitored each year and the other monitored on alternate years, the PI for the former section will be slightly lower. For the same sections if the monitoring was not performed at a regular time interval, the section with more surveys towards the later age will have a slightly inflated PI. However, this limitation may not have considerable impact in the case of the SPS-1 experiment as all the pavement sections were monitored with a regular time interval of 1 to 2 years. In SPS-1 and SPS-8 experiments the pavement sections at different sites have different ages. Among pavement sections with different ages and similar performance, the PI of younger sections will be lower than that of older sections. To address this issue, the age of test sections was considered as a covariate in all statistical analyses of PI. This will adjust the PIs according to the age of pavement sections. The statistical methods used in the study are briefly explained next.

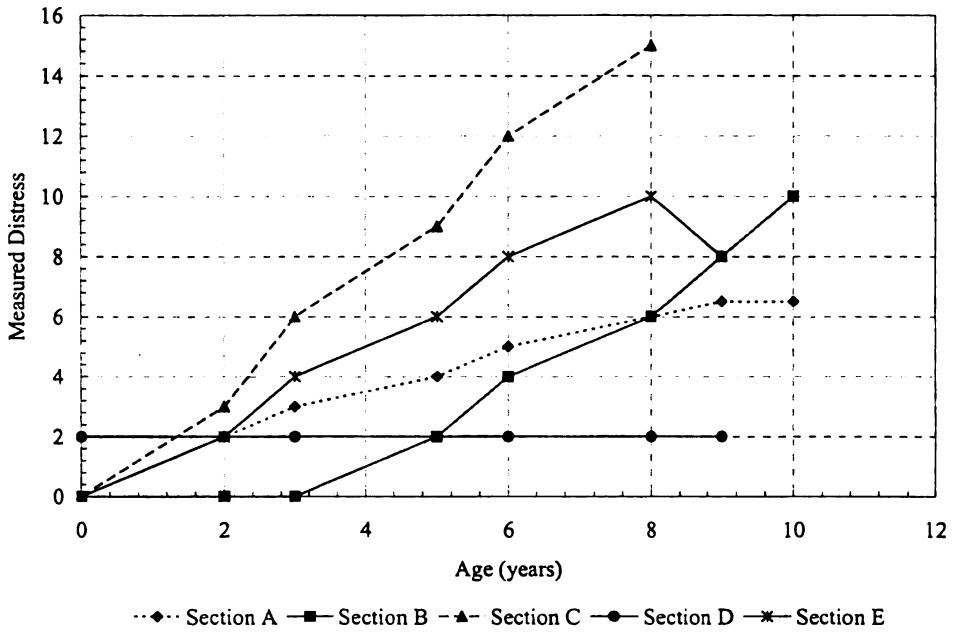


Figure 5-7 Comparing different performance curves — An Example

Table 5-1 Calculated performance indicators

Performance Indicator	Sections				
	A	B	C	D	E
Area	39.75	*31*	67.5	***19***	53
Area/L_{age}	4.0	*3.1*	8.4	***2.1***	5.9
PI	*5.5*	5.9	10.9	***2.0***	7.5

5.3 OVERALL STATISTICAL ANALYSIS METHODS

The type of statistical analysis depends on the objectives and nature of data collected for a particular research question or hypothesis. Generally, the vast majority of data likely to be encountered in research fall into the following seven designs[5]:

- One-sample data
- Two sample data
- K-sample data
- Paired data
- Randomized block data
- Regression data
- Categorical data

There are four basic questions needed to classify a set of data into one of the seven models listed above; these are:

- Are the observations (variables) quantitative or qualitative?
- Are the units similar or dissimilar for different variables?
- How many treatment levels are present in the data?
- Are the observations (variables) dependent or independent?

The dependent (target) variables (response and performance measures) in the SPS-1 experiment design are quantitative, whereas the independent (explanatory or attributes) variables are qualitative. Figure 5-8 is a flowchart that summarizes the model-identification process. It can be observed from this model identification process that the data from SPS-1 experiment falls into the category of randomized block design or *K*-sampled model. Figure 5-9 shows a summary of research tasks and respective hypotheses to achieve specific objectives within each task. Depending on the extent and availability of data specific statistical methods were employed to investigate the research questions. Figure 5-10 summarizes the data analysis methodology for overall analysis.

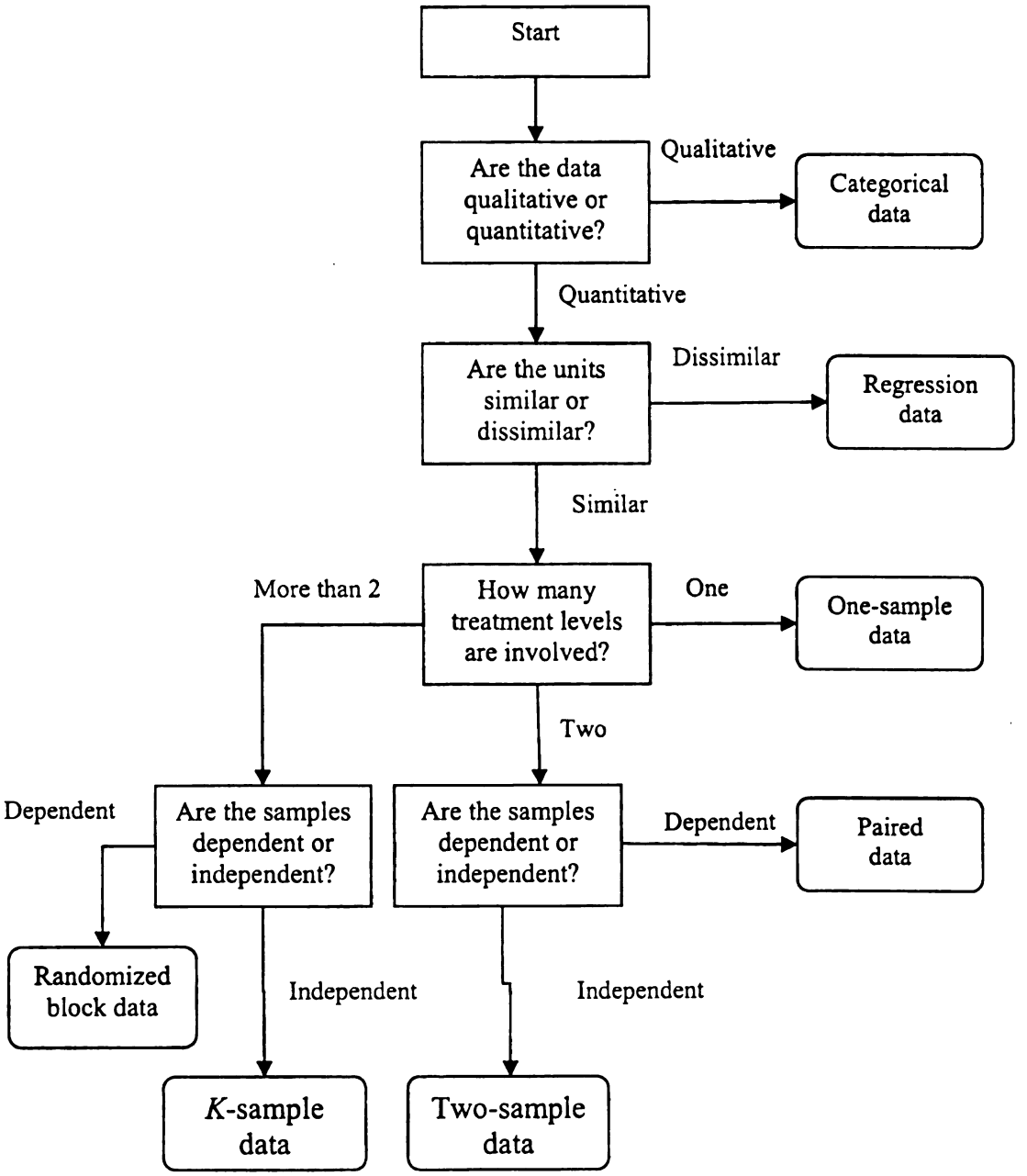


Figure 5-8 Summary of the model identification process [5]

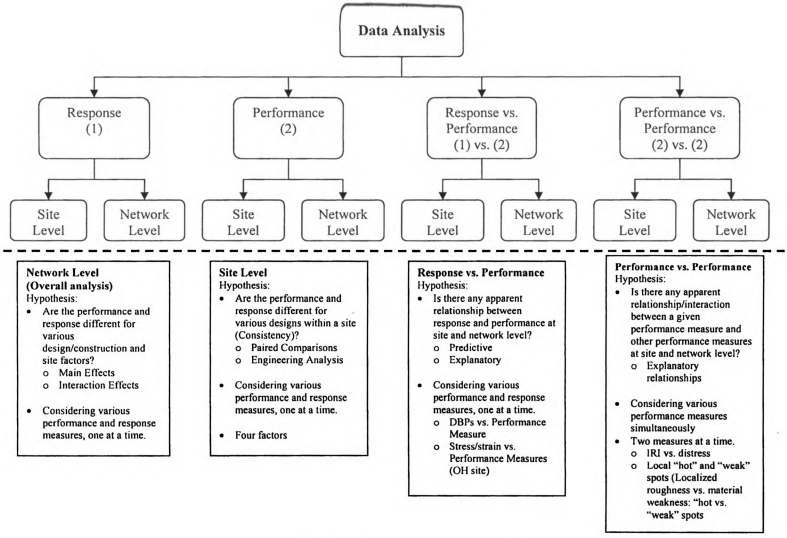


Figure 5-9 Summary of research tasks

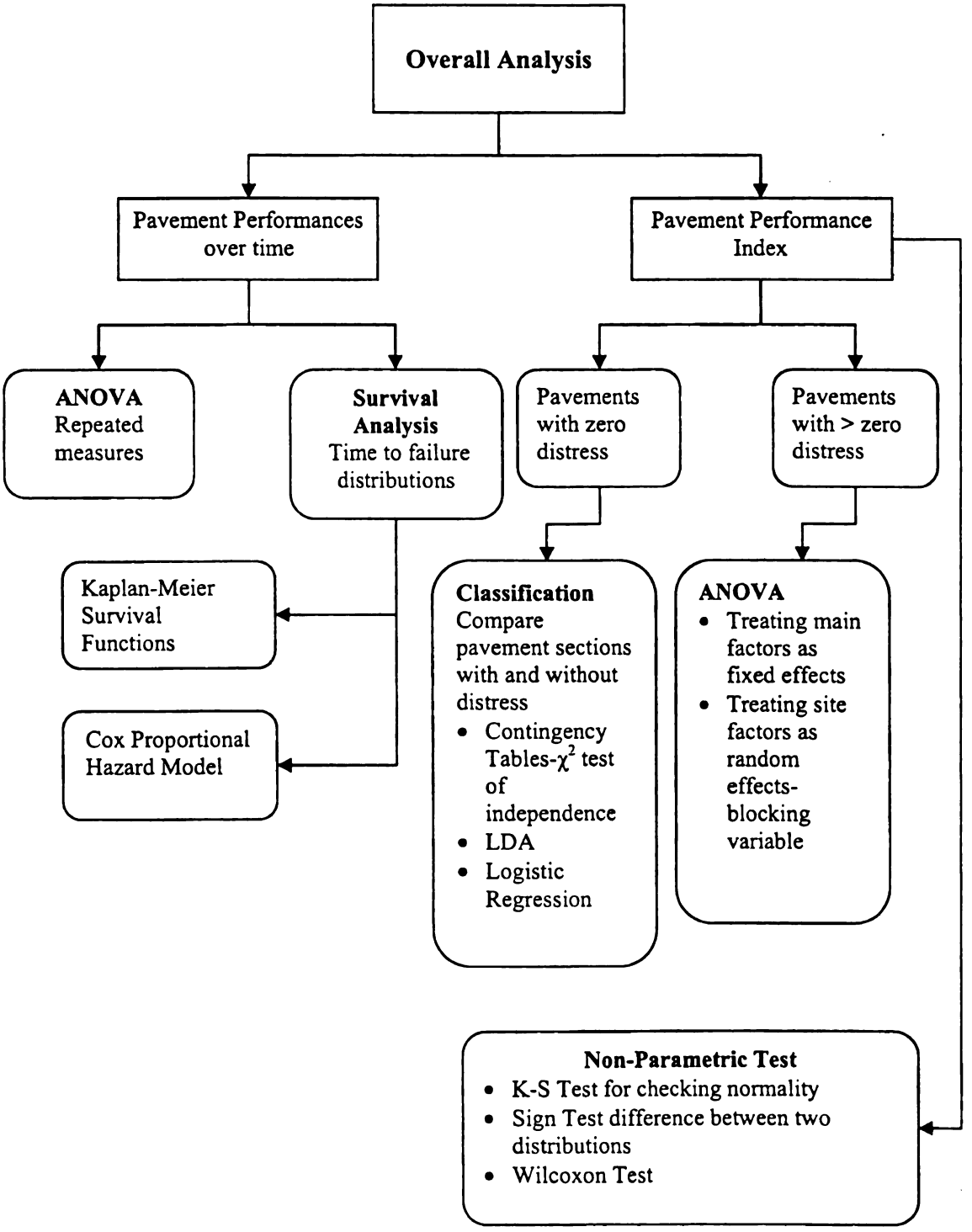


Figure 5-10 Methodology for overall analysis

As shown in the flowchart (Figure 5-10) for overall analysis, two types of methods were used for overall statistical analysis. One is based on the magnitude of the performance, i.e. comparison of mean performances between the levels of various factors. The other is based on the frequency of occurrence of distresses, i.e. probability of occurrence or non-occurrence. The ANOVA (one-way and multivariate) method belongs to the first type. The Linear Discriminant Analysis (LDA) and the Binary Logistic Regression (BLR) belong to the second type. These methods in the context of this research are discussed below.

5.3.1 Analysis of Variance (ANOVA)

The *ANOVA* is a tool that allows for better understanding of how the independent variables (categorical) influence the dependent variable (continuous). Using the General Linear Model (GLM) univariate procedure, various hypotheses can be tested about the mean of a single dependent variable when cases are classified into groups based on one or more factors (independent variables). For example, the effects of different base types or asphalt layer thickness (factors as independent variables) on the amount of cracking (dependent variable) are candidates for such an analysis. Moreover, some of these independent variables may be considered to be having a fixed or a random effect on the dependant variable. Also, any other continuous variables (independent) for which the dependent variable is to be adjusted can be included in the model as a covariate. Both balanced and unbalanced models can be tested by ANOVA. A design is considered as a balanced design if each cell in the model contains the same number of cases.

ANOVA can be performed by considering one factor at a time, or by considering more than one factor at a time. ANOVA is "one-way" when the effect of a single factor

on a dependant variable is studied, while it is "multivariate" when the effect of more than one factor on a dependant variable is studied. Also, multivariate ANOVA is more efficient as it adjusts for the effects of various factors at a time. Moreover, interaction effects, if any, between various factors can be studied by multivariate ANOVA.

ANOVA Diagnostics (Assumptions): It is not necessary, nor is it usually possible (when dealing with filed data) that an ANOVA model fits the data perfectly. ANOVA models are reasonably robust against certain types of departure from the model, such as the error terms not being exactly normally distributed [6]. The major purpose for the examination of the appropriateness of the model is therefore to detect serious departures from the conditions assumed by the model. To apply ANOVA models, the observations must be independent random samples from normal population with equal variances. The residuals can be used to check these assumptions in order to have confidence on the observed significance levels. The residual plots can be helpful in diagnosing the following departures from ANOVA model [6]:

- Non-constancy of error variance
- Non-independence of error terms
- Outliers
- Omission of important explanatory variables
- Non-normality of error terms

Generally, two common departures from ANOVA model— non-constancy of the error term and non-normality of the distribution of the error terms, are found in the data. The following frequently recommended remedial measures are found in the literature [6-8]:

- Often, non-constancy of the error variance is accompanied by non-normality of the error term. A standard remedial measure here is to transform the response variable (dependent variable). Two approaches are normally considered to find the appropriate transformation to accomplish normality and constant variance—

some simple guidelines (e.g., log, natural log or square root etc.,) and the Box-Cox procedure (an iterative procedure to find the best power for transformation).

- If the error terms are normally distributed but the variance of the error term is not constant, a standard remedial measure is to use weighted least squares.
- When there are major departures from the ANOVA model and even transformations are not successful in stabilizing the error variance and error normality, a non-parametric test for the equality of the factor level means may be used instead of the standard *F*-test.

All the assumptions of the ANOVA models used in this research were checked and appropriate remedial measures as discussed above were adopted where ever necessary. For example, ANOVA model used to identify structural factors affecting the change in IRI has residual scatter as shown in Figure 5-11. The residual scatter against predicted value shows a fan shaped structure, indicating violation of the constant variance of error terms. Similarly, as mentioned above this violation of constant variance of error terms is accompanied by the non-normality of the error terms as shown by Figure 5-12 and Figure 5-13. A natural log transformation of the dependent variable (change in IRI) was adopted as a remedial measure. The same ANOVA model was executed on the transformed variable and again the residual plots were examined. Figure 5-14 shows the residual plot from the model. It can be observed that no serious violation of the constant variance of the error terms is detected and residuals seem to be randomly distributed along the predicted values. Figure 5-15 and Figure 5-16 show the diagnostics for normality of the error terms. It can be seen from these plots that residuals are also normally distributed. Therefore, the conclusions from ANOVA model based on the transformed change in IRI were only considered in the research. The same methodology was employed for all the performance and response measures in ANOVA models. It should be noted that the number of data points may be reduced when log transformation

is adopted; as the sections having zero distress will be eliminated from the analysis. The consequence of this elimination is further discussed in Chapter 6.

Statistical Significance: Statistical significance of an effect of a factor implies that there exists a significant mean difference between the performances (in this study) of any two levels within the factor. For example, a statistically significant effect of HMA surface thickness on fatigue cracking implies that there is a significant (statistical) mean difference between fatigue cracking on sections with 4-inch (102 mm) HMA thickness and sections with 7-inch (178 mm) HMA thickness. Moreover, in simple terms, a statistical significance indicates that the effect is not a happenstance. Regardless of the type of analysis the p-value identifies the likelihood that a particular outcome occurs by chance. The smaller the p value, the greater the likelihood that the findings are valid. It is generally accepted that if the p value is less than 0.05, the result is considered statistically significant. Thus, when there is less than a 1 in 20 probability that a certain outcome occurred by chance, then that result is considered statistically significant. Another common convention is that when a significance level falls between 0.05 and 0.10, the result is considered "marginally significant". When the significance level falls far below 0.05 (e.g. 0.001 or 0.0001 etc.), one will have greater confidence on the research's findings. A significance level of 0.05 was used in all the statistical analyses performed in this research. However, it is important to confirm the practical or operational difference between the means of various levels of a factor, if a factor has a statistically significant effect. An attempt was thus made in this study to gauge the practical or operational significance of statistically significant differences in the analysis. The operational significance adopted for various performance measures is discussed next.

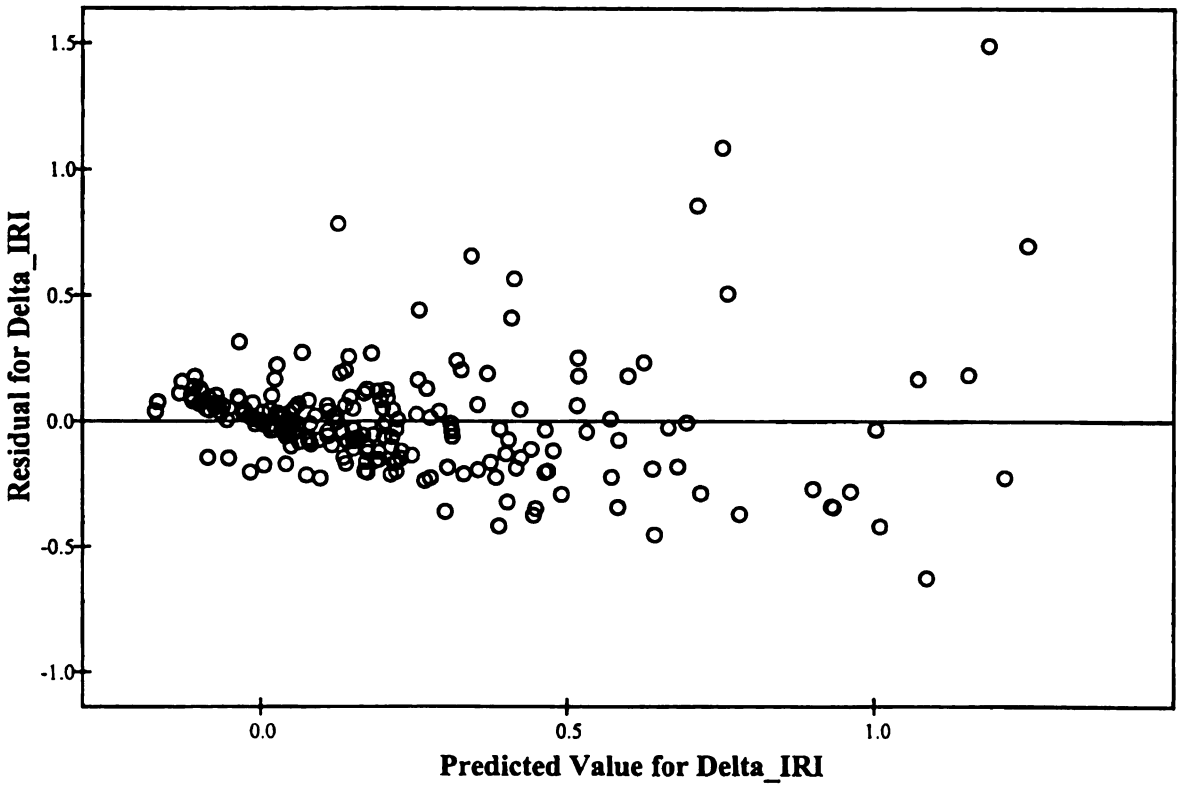


Figure 5-11 ANOVA diagnostics-Residual plot without transformation

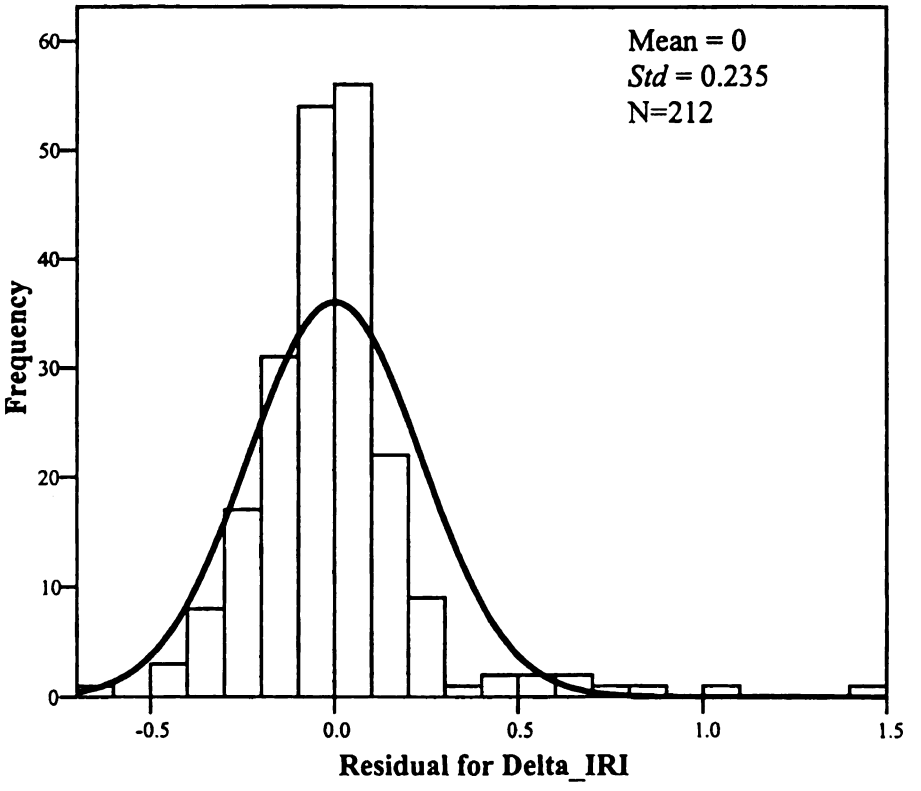


Figure 5-12 ANOVA diagnostics-Residual frequency without transformation

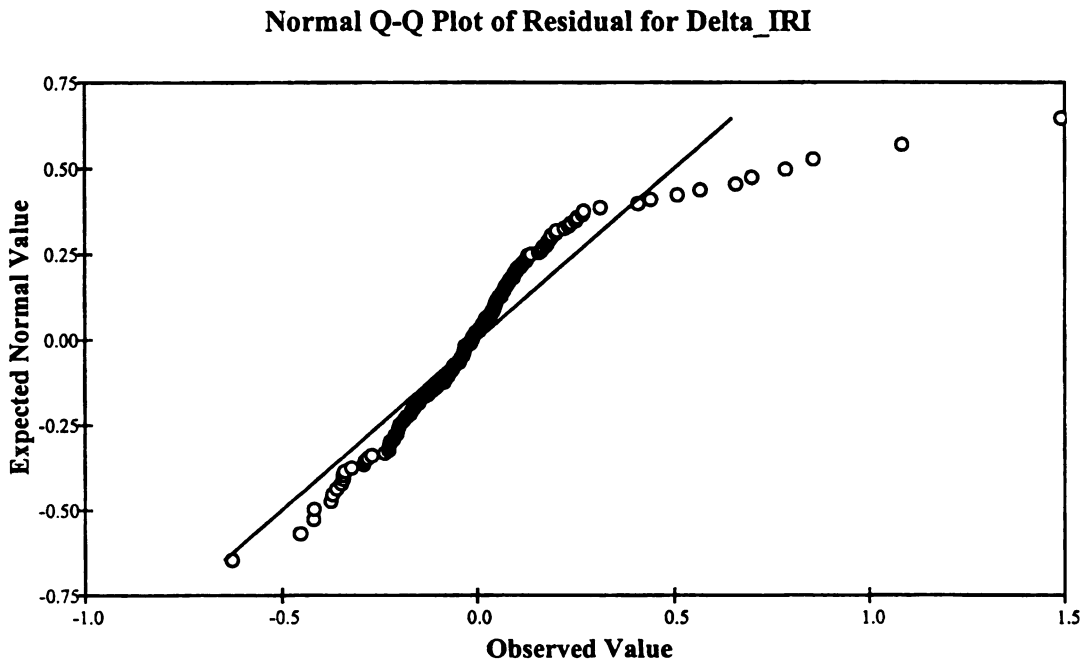


Figure 5-13 ANOVA diagnostics-Residual normality without transformation

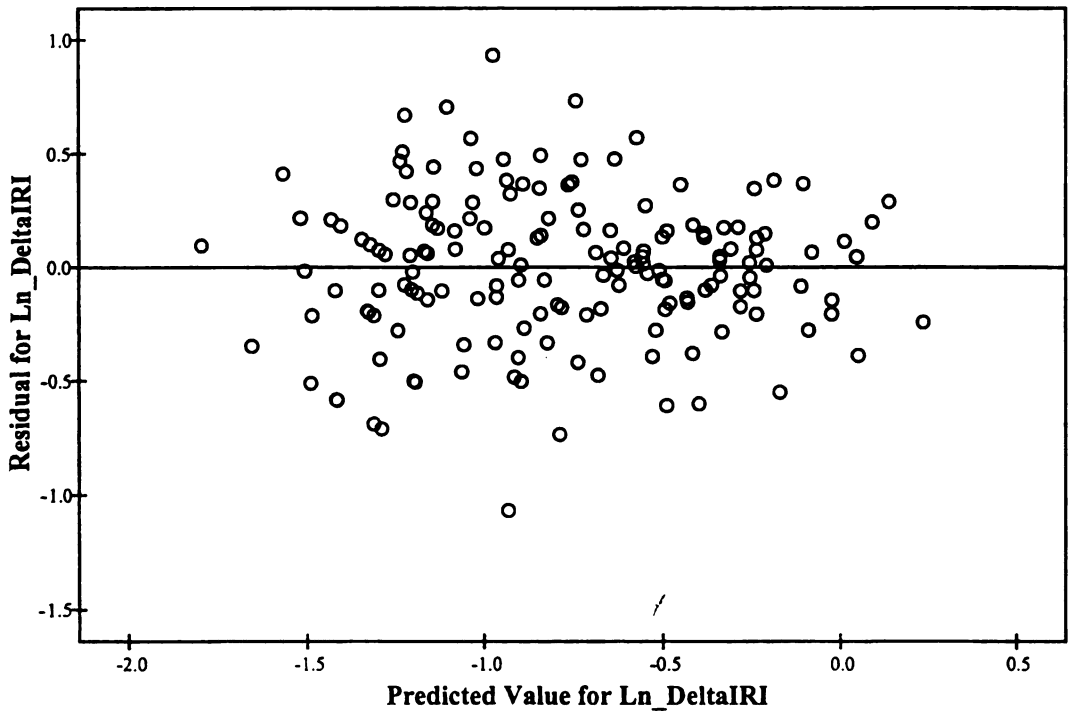


Figure 5-14 ANOVA diagnostics-Residual plot with log (natural) transformation

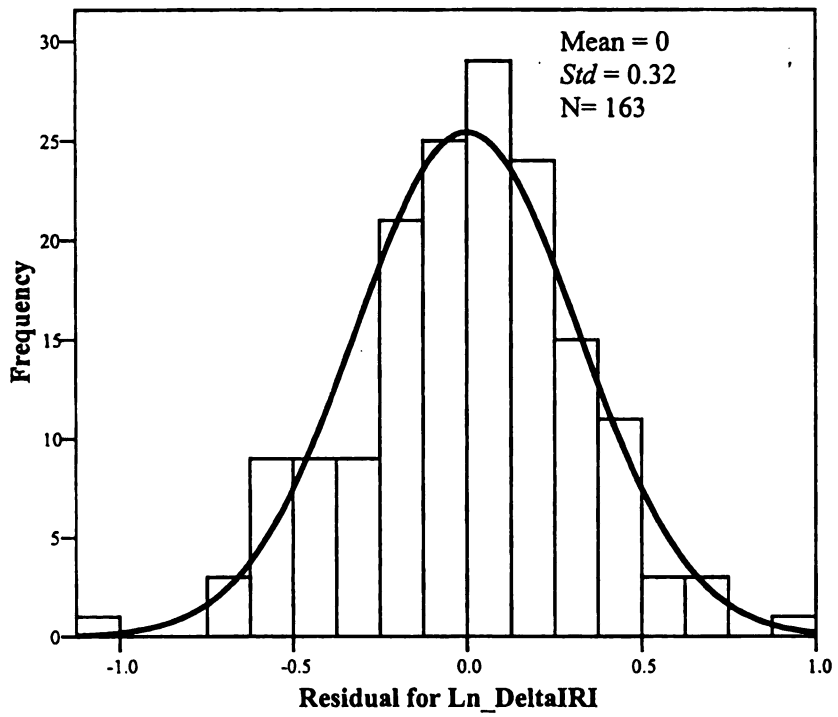


Figure 5-15 ANOVA diagnostics-Residual frequency with log (natural) transformation

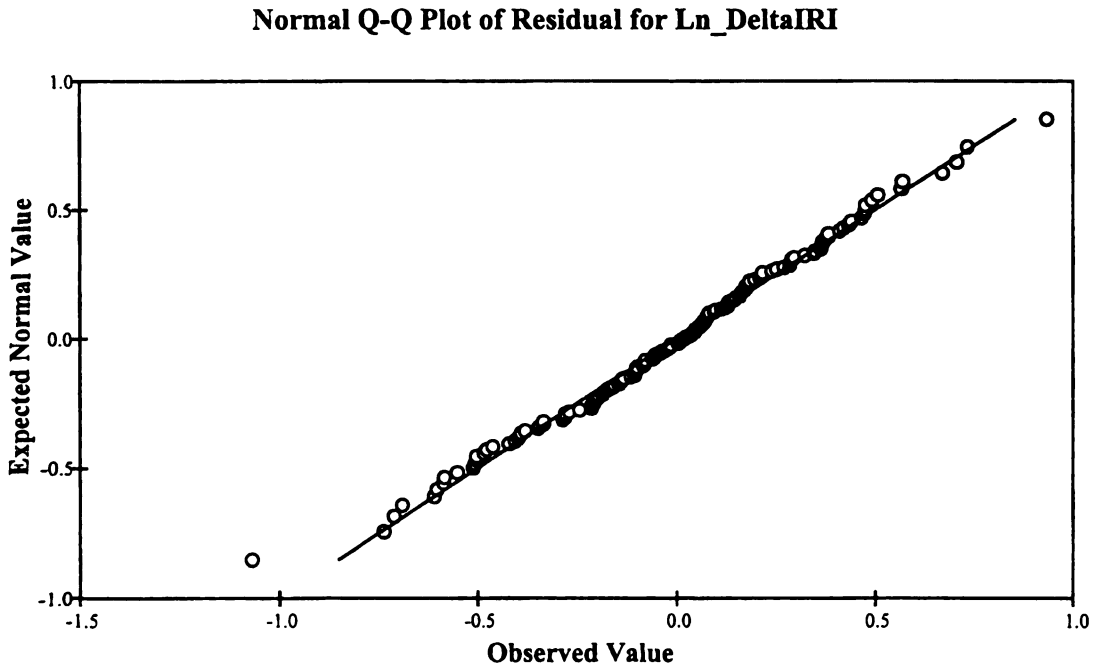


Figure 5-16 ANOVA diagnostics-Residual normality with log (natural) transformation

Practical Significance

The statistical significance of the difference between the marginal means for various levels of design and site factors needs to be judged from a practical (operational) point-of-view. This practical significance is dependent on the magnitude of the mean difference of levels for a particular factor and will vary for each performance measure. For example, if the means for alligator cracking are significantly (statistically) different for pavement sections constructed on DGAB and on ATB, one should check whether this difference has any practical or operational meaning from an engineering point-of-view. The practical significance therefore depends on the subjective judgment of actual pavement performance observed in the field.

To determine reasonable levels of practical significance for different distress types (fatigue cracking, rut depth, transverse cracking and roughness), the performance curves developed based on engineering judgment of an expert panels were used. These curves for various distress types were developed under a previous study [1, 9]. The criteria for fatigue cracking, rut depth, roughness and transverse cracking performances are shown in Figures 5-17, 5-19, 5-21 and 5-23, respectively.

As mentioned before, the ANOVA was conducted on PI for all performance measures except roughness. For roughness the change in IRI (Latest IRI- Initial IRI) was used as dependent variable in ANOVA. Because the marginal means from ANOVA are in terms of PI, the performance curves from the expert panel were converted to PI, assuming 1 year monitoring interval. These curves, in terms of PI, for fatigue cracking, rut depth and transverse cracking are shown in Figures 5-18, 5-20 and 5-22, respectively.

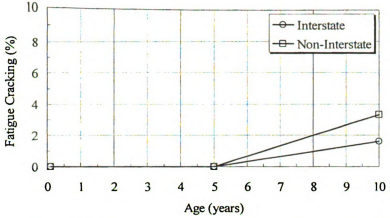


Figure 5-17 Performance criteria for fatigue cracking [1]

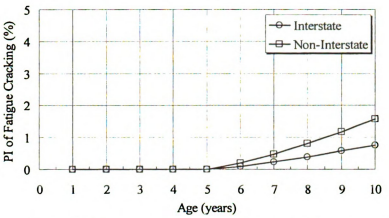


Figure 5-18 Performance criteria for PI of fatigue cracking

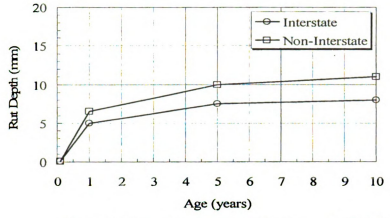


Figure 5-19 Performance criteria for rut depth [1]

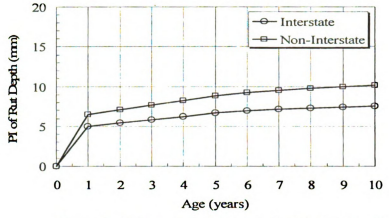


Figure 5-20 Performance criteria for PI of rut depth

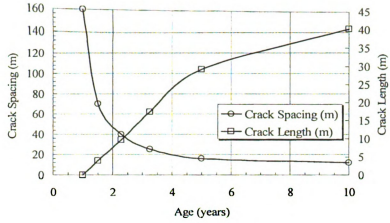


Figure 5-21 Performance criteria for transverse cracking [1]

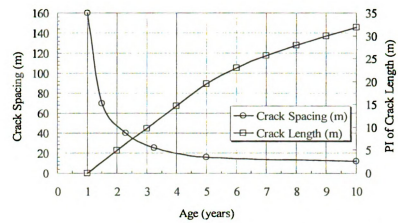


Figure 5-22 Performance criteria for PI of transverse cracking

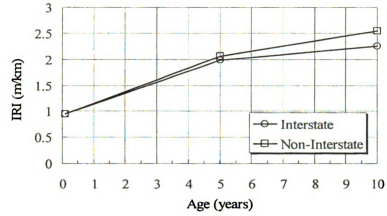


Figure 5-23 Performance criteria for roughness-Flexible [1]

It can be seen from these curves that the slopes of the individual performance curves vary with age. For example, the slope of the IRI curve is the same up to year 5 and later can be separated into two parts. From these two slopes, change in IRI per year can be calculated for the first five years and for the next five years (see Figure 5-23). The weighted average of these slopes was used to calculate the change in IRI per year.

Furthermore, the above described curves define the boundaries between good and poorly performing pavements for interstate and non-interstate highways, respectively. For the SPS-1 experiment, it was estimated that 80% of the designs corresponded to the interstate highway class, while the remaining 20% were non-interstate, based on the asphalt layer thicknesses. Therefore, the slope (change per year) was further weighted for the proportions of the pavement class within the SPS-1 experiment. Table 5-2 shows the threshold values for practical or operational significance for the various distress types.

Another important aspect of ANOVA models is the sample size and statistical power to detect the means difference between the levels of a factor. This issue of sample size and statistical power is discussed next in the context of this research.

Statistical Power and Sample Size (Replication)

Unfortunately, this aspect of the statistical analysis is ignored in most of the studies involving data from the actual field, mainly because of the economic and practical considerations. Sample size is usually determined from statistical considerations, resource or budget considerations or both. Generally, the larger the sample size, the greater will be the ability to detect any difference in the response due to different treatments.

Table 5-2 Operational significant differences for various performance measures

Performance measure	Weighted slope per year	Remarks
Fatigue cracking (%)	0.20	This will translate into 1.0 sq-m of area per year.
Rut depth (mm)	0.80	The operational significant difference for rut depth is 0.8 mm per year.
Transverse cracking (m)	3.50	This will translate into 75 m of crack spacing per year.
Longitudinal Cracking (m)	4.50	The weighted slope was calculated based on 5000 ft/mile failure criterion used in NCHRP 1-37A. The failure criterion is thus 144 m for a SPS-1 test section. The operational value is based on the slope of the performance curve between 0 and 10 years, assuming zero cracking up to 5 years and failure at 20 years.
Roughness ΔIRI(m/km)-Flexible	0.13	The change in IRI was calculated based on initial IRI and latest IRI

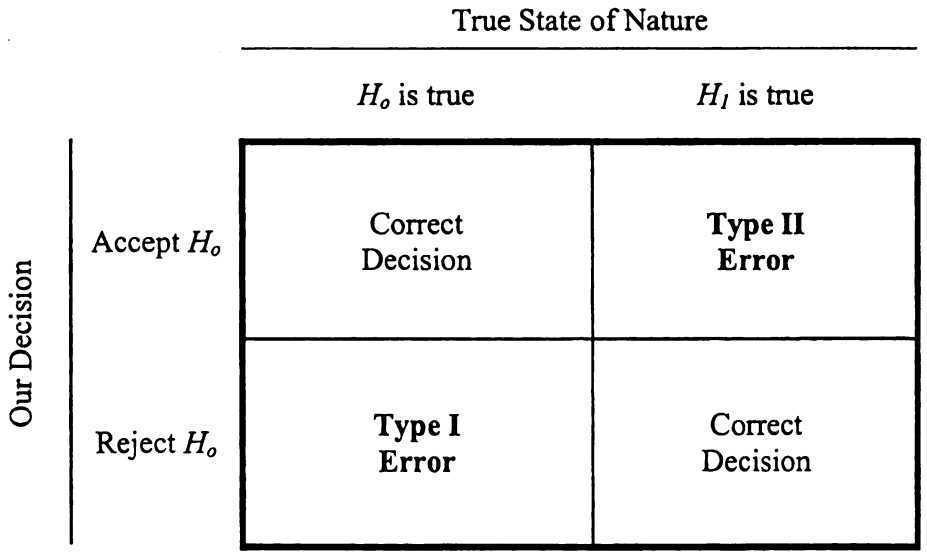


Figure 5-24 Type I and Type II Errors

Thus a key step in experimental design is to assess the power of the statistical test to be used in the analysis of the data or conversely, the precision of the estimates to be produced by the analysis, as a function of the sample size. Eventually, a trade-off must be made between the increase in power and precision resulting from the higher sample size and added cost or time required to complete the experiment.

Planning of sample size can be approached in terms of (1) controlling the risk of making Type I and Type II errors, (2) controlling the widths of the desired confidence intervals or (3) combination of both approaches. Type I error (α) occurs if the null hypothesis (H_o) is rejected when H_o is actually true. On the other hand, Type II error (β) occurs if one fails to reject H_o when the alternate hypothesis (H_1) is actually true. Figure 5-24 shows the four possible "decision-state of nature" combinations. Generally, researchers do not directly discuss the probability of Type II error (β); rather, they use power, which is equal to $1-\beta$. Since β is the probability of the event of incorrectly accepting (or failing to reject) the null hypothesis when it is not true then the probability of the complement of that event (correctly rejecting the null hypothesis when it is not true) is $1-\beta$, which is called the power of statistical testing. Thus "the power of a hypothesis test is the probability of correctly rejecting the null hypothesis when the null hypothesis is not true".

For any statistical analysis, one would like to have a test that gives low Type I error rates and high power, but one cannot have both with a limited sample size due to financial constraints. Therefore, a power of 70% to 80% is considered reasonable for most of the scientific studies. The power for any statistical analysis can be calculated given the sample size, an estimate of the standard deviation and expected mean

difference. Conversely, for an assumed power of the hypothesis, the sample size can be calculated. When a single population is involved with unknown variance, the power of hypothesis test can be calculated using Equation (5-5).

$$Power = \Pr\left(T > t_{\alpha/2,df} - \frac{|\mu_o - \mu_1|\sqrt{n}}{s} \right) \qquad (5\text{-}5)$$

Where;
H_o: $\mu=\mu_o$ versus H_1: $\mu\neq\mu_o=\mu_1$
df= n-1
n= sample size

Equation (5-5) can also be modified to Equation (5-6) to calculate the sample size (n) for a given power.

$$n \geq \frac{\left(t_{\alpha/2} - t_{1-\beta}\right)^2 s^2}{\left(\mu_o - \mu_1\right)^2} \qquad (5\text{-}6)$$

As mentioned above another approach based on the desired range of confidence interval is also used for calculating the sample size. Equation (5-7) can be used to calculate sample size for a single population with unknown variance.

$$n = \frac{t^2_{\alpha/2,df} s^2}{E^2} \qquad (5\text{-}7)$$

Where;
E = margin of error, which is equal to half width of confidence interval
df= n-1
n = sample size

When two population means are involved in the statistical analysis with unknown variances, Equation (5-8) can be used to calculate the power of test when sample sizes are unequal, whereas Equation (5-9) is for equal sample sizes.

$$Power = 1 - \beta = \Pr\left(t > t_{\alpha/2,df} - \frac{|\delta_o - \delta_1|}{\sqrt{MSE\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}} \right) \quad (5\text{-}8)$$

$$Power = \Pr\left(t > t_{\alpha/2,df} - \frac{|\delta_o - \delta_1|}{\sqrt{\frac{2MSE}{n}}} \right) \quad (5\text{-}9)$$

Where;
H_o: $\mu_1 - \mu_2 = \delta_o$ versus H_1: $\mu_1 - \mu_2 \neq \delta_o = \delta_1$
$df = n_1 + n_2 - 1$ for unequal sample size
$df = 2(n-1)$ for equal sample size
n= sample size

Equation (5-9) can also be modified to Equation (5-10) to calculate the sample size (n) for a given power.

$$n \geq \frac{\left(t_{\alpha/2} - t_{1-\beta}\right)^2 \left(2s_p^{\,2}\right)}{\left(\delta_o - \delta_1\right)^2} \quad (5\text{-}10)$$

Where:

$$s_p = pooled\ \text{variance} = \frac{df_1 \times s_1^{\,2} + df_2 \times s_2^{\,2}}{df_1 + df_2}$$

Note: Pooled variance is only calculated if both populations gave equal variance

Similar to the case of single population, the sample size can also be estimated using the margin of error approach, as shown in Equation (5-11).

$$n = \frac{t^2_{(\alpha/2,df)} 2\left(s_p\right)^2}{E^2} \quad (5\text{-}11)$$

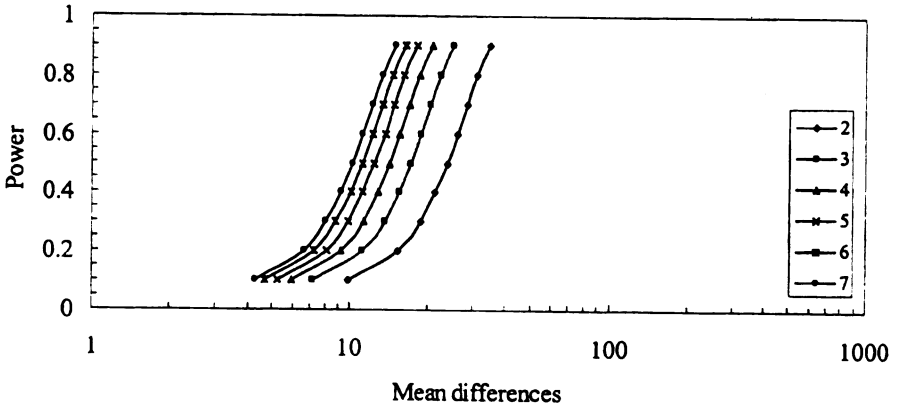
- *Factors affecting Power*

From the above mentioned equations for calculating statistical power and sample size, the following factors should be considered for achieving reasonable statistical power in any statistical analysis:

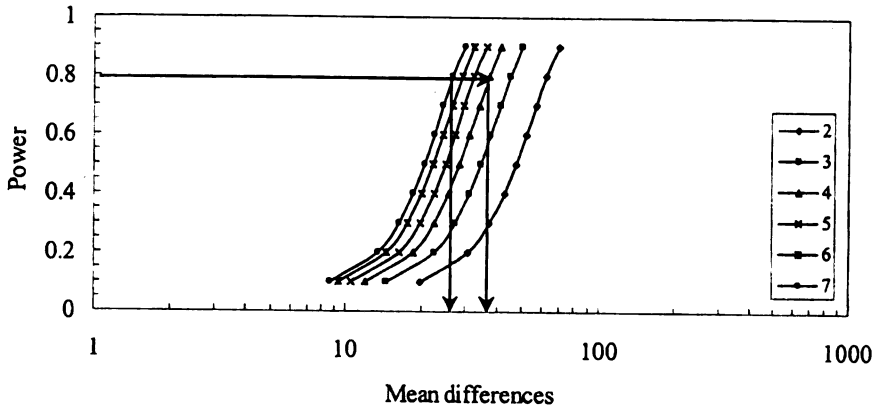
- A smaller value of α will decrease the power. This means that one will always trade-off Type I against Type II error.
- If the alternative mean is further away from the null mean the difference $(|\mu_o - \mu_1|)$ increases, then the power increases. Note that if the means are close, then it is less likely to reject the null hypothesis; in fact, failing to reject the null hypothesis may not have serious consequence if the difference is not practically significant.
- Power decreases with increasing variance (σ^2). The variance can be controlled by selecting homogeneous experimental units (pavement sections) and by obtaining replicate measures for each pavement section and using the average of several replicates for an individual section.

The power calculations are somewhat more involved for one-way ANOVA and multi-factorial ANOVA models. However, the interpretation of power concepts remains essentially the same as explained above for statistical analyses involving one and two sample populations. The methodology for power calculation using these ANOVA models can be found in references [6, 10]. Statistical Computer Software such as SAS™ and Minitab ™ can be used to calculate the sample size (replicates) for a given power of an experiment design involving many factors or a single factor with many levels.

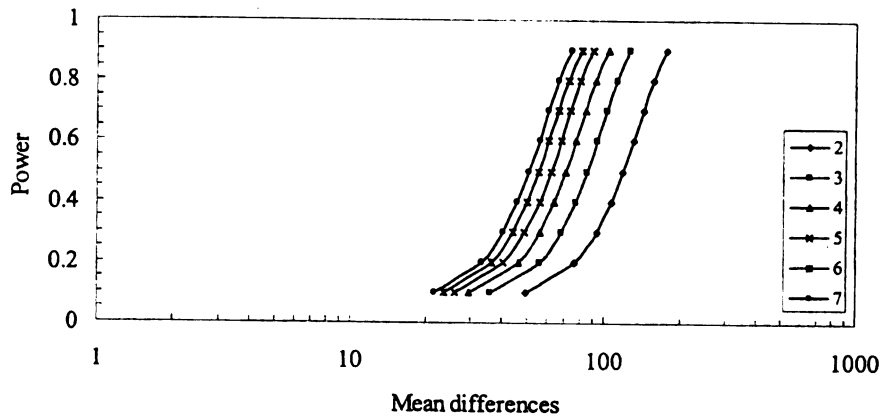
Figure 5-25 is an example of power plots for a one-way ANOVA, in which a single factor (designs) with 24 different levels was considered. These 24 treatments represent all possible combinations of the designs in the SPS-1 experiment.



(a) Error Variance = 25



(b) Error Variance = 100



(c) Error Variance = 625

Figure 5-25 Effect of replication and variance on statistical power—One-way ANOVA

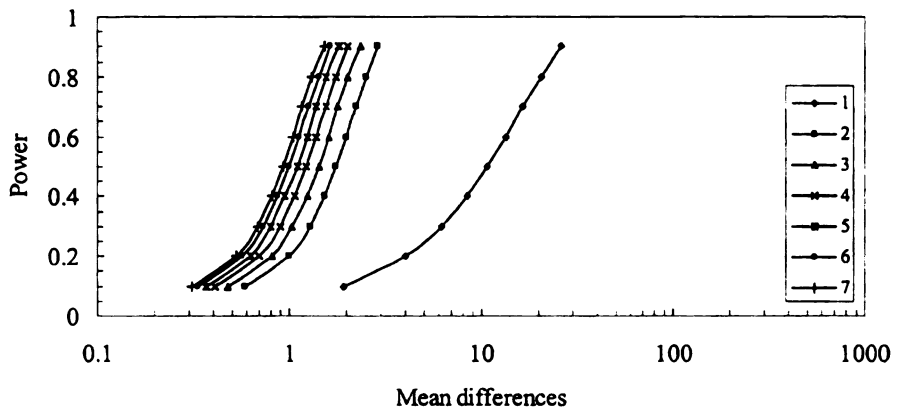
The power curves represent three levels of error variance (see Figure 5-25 a, b and c), which can be represented by the mean square error (MSE) — an estimate of error variance in the one-way model. The power curves were plotted for different levels of replicates in the model (n=2, 3 ...7). It can be seen from these plots that a lower mean difference to attain the same statistical power can be detected either for a low variance or for higher number of replicates. Also, higher mean differences for the same number of replicates are associated with higher statistical power. For example, to attain a power of 80% in this design with σ=10, about 4 replicates will be required for each of the 24 design to detect a mean difference of about 35 between any two of the designs. If the replicates are increased to 7 for each design, a mean difference of about 25 can be observed with a power of 80% [see Figure 5-25 (b)]. In order to detect a lower mean difference (less than 15) for 80% power, the error variance has to be reduced and number of replicate need to be at least 7. A few suggestions to reduce the variance between the sections were mentioned above.

The SPS-1 experiment design involves multiple factors; therefore to simulate this experiment, power curves were generated for a design having six factors with two levels each. These curves for three levels of error variance (σ^2 = 25, 100 and 625) are shown in Figure 5-26. The standard deviation (σ) for fatigue cracking within SPS-1 experiment is about 10 sq-m. To detect a means difference of at least 5 sq-m of fatigue cracking (minimum level of practical significance) between the levels of factors at 80% power assuming σ^2 = 100, a replication of at least 2 is required within each cell of the experiment [see Figure 5-26 (b)]. However, a single replication of each treatment will yield a power of less than 20% to detect the same mean difference. Furthermore, if σ is

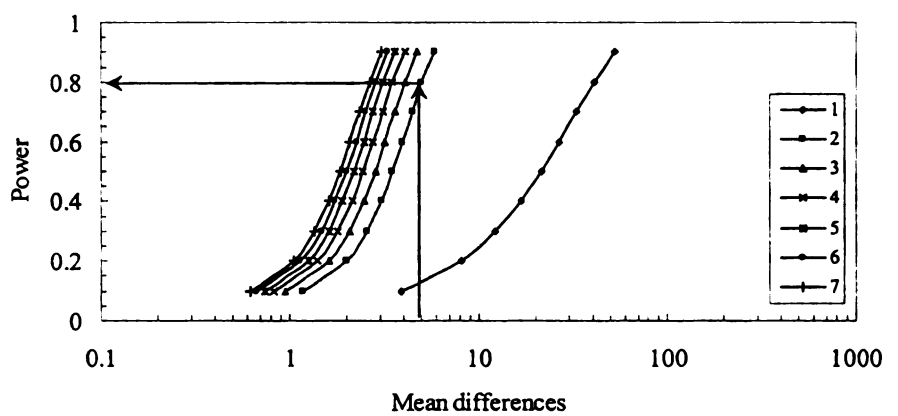
greater than 10 sq-m of fatigue cracking, a single replicate will have a very limited power to detect any mean difference less than 70 sq-m and even 2 replicates of each treatment will not be able to detect a mean difference of 10 sq-m of cracking at 80% power.

It should be noted that the above discussion of sample size (replication) and power is only valid for a balanced design, i.e. no missing value for any of the cells in the experiment. However, the actual scenario with real data may reflect a more complicated situation; for example Table 5-3 shows the real replication of the test sections for fatigue cracking at this point in time. It can be observed that the experiment design is quite unbalanced because of the many missing values for treatments. The implication of this unbalance on multi-factor ANOVA models may seriously hamper the power of the statistical analysis. Therefore, simple statistical analyses such as comparison of means between two populations or one-way ANOVA (where more than two means need to be compared) are warranted at this point in time. This suggestion strongly applies to cracking performances since a considerable number of sections have not yet shown any distress (see Chapter 4 for details on data availability).

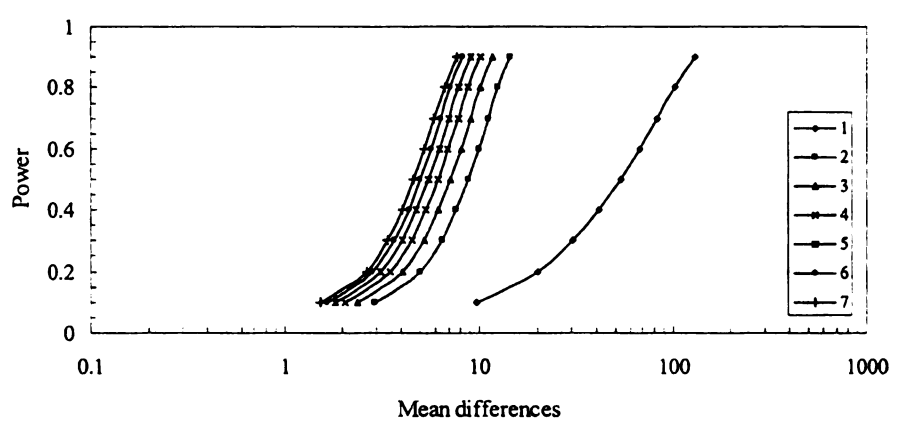
The simple analysis will have higher statistical power as the sample size will increase when some factors are ignored. However, these methods may not adjust the means for factors other than the ones considered in the analysis. Figure 5-27 shows the power curves for comparison of means between two populations. Each curve in the figure represents a detectable mean difference. To detect a mean difference of 5 sq-m for σ=10 sq-m at 80% power, a sample size of at least 60 each for both populations will be required [see Figure 5-27 (b)]. However, a sample size of about 20 is required to detect the same mean difference at the same power for σ=5 sq-m [see Figure 5-27 (a)].



(a) Error Variance = 25



(b) Error Variance = 100

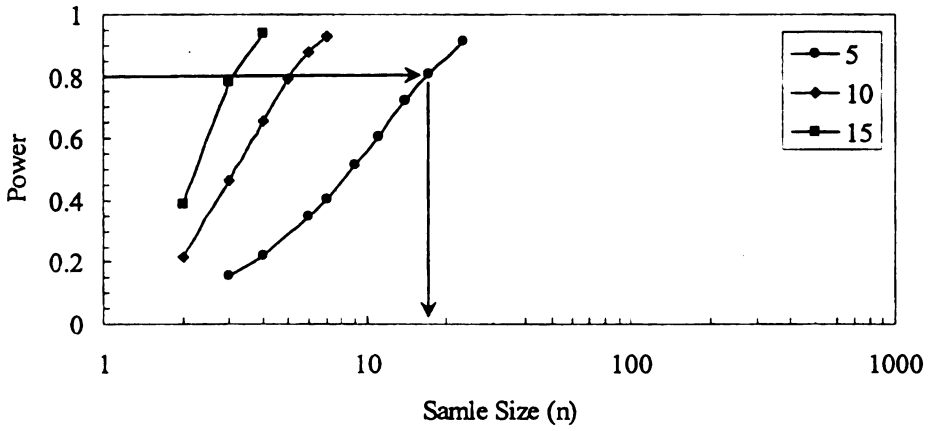


(c) Error Variance = 625

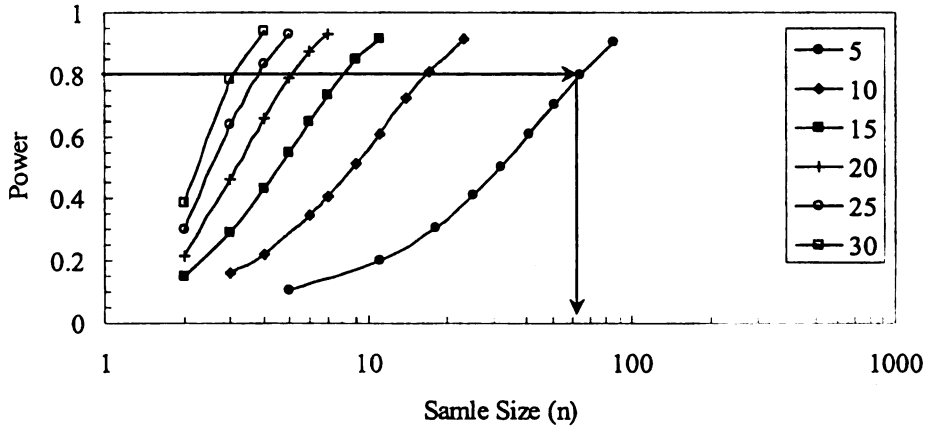
Figure 5-26 Effect of replication and variance on statistical power—Multi-Factorial ANOVA

Table 5-3 Actual replication of sections within SPS-1 experiment— Fatigue cracking

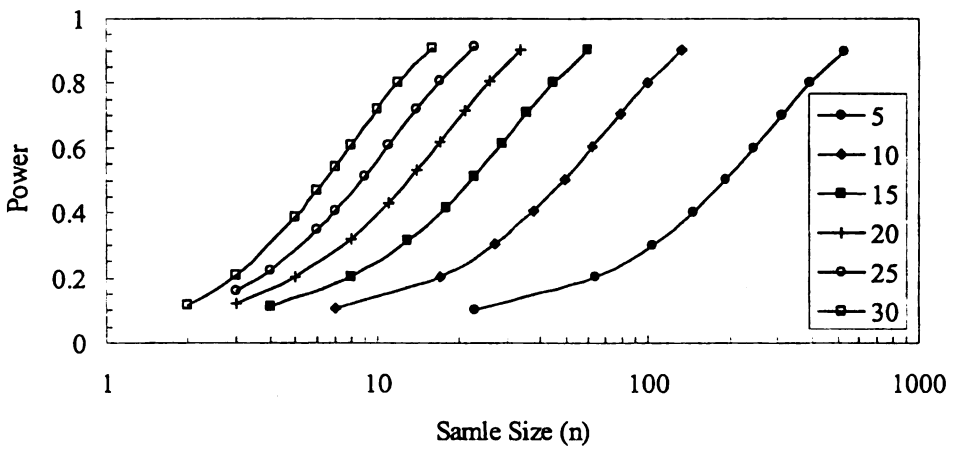
Drainage	Base Type	Base Thickness	HMA Thickness	WF		WNF		DF		DNF		Row Total
				F	C	F	C	F	C	F	C	
No	DGAB	8"	4"	1	2	1	2		1		1	8
			7"	1	1	1			1			4
		12"	4"	1	1	1	1		1	1		6
			7"	1	2	1	1		1		1	7
	ATB	8"	4"	3	1	1	1		1	1		8
			7"	1	1				1		1	4
		12"	4"	1	2		1		1		1	6
			7"	3	1	1			1	1		7
	ATB/DGAB	8"	4"	2	1	1	1		1			6
			7"	1	1		1		1		1	5
		12"	4"		1		2		1		1	5
			7"	3	1	1			1			6
Yes	PATB/DGAB	8"	4"	1	1	1	1		1	1		6
			7"		1		1		1		1	4
		12"	4"		1	1	1		1		1	5
			7"	3	1	1	1		1			7
		16"	4"		1	1	1		1		1	5
			7"	1	1	1			1			4
	ATB/PATB	8"	4"		1		2		1		1	5
			7"	3		1	1		1			6
		12"	4"	3	1	1			1			6
			7"		1		1		1		1	4
		16"	4"	2	1	1						4
			7"	1	1		1		1		1	5
Column Total				32	26	16	20	0	23	4	12	133



(a) Pooled variance = 25



(b) Pooled variance = 100



(c) Pooled variance = 100

Figure 5-27 Effect of mean difference and variance on statistical power—Test means for two populations

Based on the statistical power considerations and data availability for the SPS-1 experiment design, a simple univariate may be more suitable for detecting a lower mean difference (for various performance measures) between the levels of experimental factors.

Discussion on Analysis of Variance (ANOVA)

The analysis of variance (ANOVA) is a powerful method; however under certain conditions (limited available data) the application of this method has some restrictions. These issues are briefly summarized below:

- The un-balanced data makes it difficult to meet the equal variance assumption. Therefore, an appropriate transformation of the response variable may be adopted to address this issue.
- In case of fractional factorial design, the higher order interactions (between more than two factors) cannot be studied [11].
- Replication within each cell of the experiment design plays an essential role in determining power of hypothesis testing. The power of a hypothesis test is the probability of correctly rejecting the null hypotheses when the null hypothesis is not true. Lower number of replications within experiment design will reduce the power of detecting a mean difference between levels of a factor for a given variance.
- Time series ANOVA with repeated measures seems to be an appropriate choice of analysis for this type of experiment where each section is monitored over time. However, this type of analysis requires a more balanced data i.e., all pavement sections should be monitored at the same interval and up to an age long enough (about 15 years) for capturing long term pavement performance.

Comparison of Means (One-way ANOVA)

As mentioned before, the SPS-1 experiment was designed as fractional factorials. Further, since the number of sites constructed in each zone-subgrade combination is not the same and not all the sections are exhibiting cracking distress, the experiment is unbalanced. Thus only two-way interactions may be reliable for the experiment. In addition, transformation of the response variables becomes an essential choice to fulfill the requirements (assumptions) of ANOVA. When natural logarithmic transformation is applied to response variables, due to the nature of the transforming function (natural logarithm of zero or a negative value is not defined), only data pertaining to those test sections that have distressed (i.e. non-zero positive data) were considered in ANOVA. Hence, ANOVA results are based only on distressed sections.

In the SPS-1 experiment, each SHRP ID represents a unique design and thus there are 24 designs in the experiment. The performance of the designs with respect to each other was evaluated using the deviation from mean performance, which is the standard deviate. The designs were evaluated based on selected distresses, considering one distress at a time. The standard deviate was calculated for each of the twelve designs within each site. Table 5-4 shows a sample calculation for alligator cracking in the AL (1) site, which is calculated by using the following equation.

$$\text{Standard Deviate} = \frac{(\text{PI of a given design - Average PI for the given site})}{(\text{Standard Deviation of PIs for the given site})}$$

As this measure was calculated for each section, considering one site at a time, it indicates the relative standing of the section compared to other sections.

Table 5-4 Calculation of standard deviate for alligator cracking - Alabama (1)

Section ID	Performance Index	Average	Standard deviation	Standard deviate
0101	23.08	45.93	32.82	-0.70
0102	90.69			1.36
0103	25.43			-0.62
0104	3.83			-1.28
0105	95.90			1.52
0106	16.12			-0.91
0107	21.97			-0.73
0108	70.45			0.75
0109	75.47			0.90
0110	52.74			0.21
0111	65.33			0.59
0112	10.22			-1.09

This transformation thus helps nullify the variation in performance (due to site conditions) among sites, as the sections are weighed with respect to companion sections in each site. The standard deviate will show the relative comparison of various designs for a specific performance measure. This value can be interpreted in the following three possible ways:

- Lower value indicates better performance than the mean
- Zero value indicates the mean performance
- Higher value indicates worse performance than the mean.

The standard deviate for a particular performance can also be used to compare the effects of design factors, and for this one-way ANOVA was performed on the standard deviates of the sections. The analyses were performed on data from all sections and also on subsets of data stratified by different subgrade types, climates and combinations of these. This helps identify the effects of design factors under different site conditions. The standard deviate values of each design were averaged from the various sites to study the overall as well as the interaction effects of climatic zones and subgrade type.

To consider all available sections, the test sections were categorized as "cracked" and "non-cracked" for frequency based methods (LDA and BLR). These analyses methods (discussed below) will help in identifying the significant factors that discriminate between the two categories.

5.3.2 Extent of distress

The effect of the key experimental factors on performance, through the relationship between the magnitude and relative occurrence of the observed distresses, can be observed from the data. Simple bivariate plots between the percentage of test sections that have exceeded various levels of distress for the key performance measures, categorized by experimental design and site factors were plotted to display and explore the data. Note that the effect of climatic zone will be only shown for the wet regions because of the limited number of sites (4 sites) in the dry regions.

5.3.3 Linear Discriminant Analysis

Linear Discriminant Analysis (LDA) allows for distinguishing between two or more groups of data. This is done by identifying variables that are significant in classifying the data into various groups. The procedure for predicting membership is to initially analyze pertinent variables where group membership is already known. The details of theoretical background of LDA is available in literature [7, 12, 13]. For example, groups of observations can include one group of pavements with cracks and the second group with no cracks. The method allows for determining which variables discriminate between cracked and non-cracked pavements.

5.3.4 Binary Logistic Regression

Binary Logistic Regression (BLR) is used often in the case where the outcome variable is discrete (dichotomous). The difference between logistic and linear regression is reflected both in the choice of a parametric model and in the assumptions. This method is based on the maximum likelihood method for determining the parameters of

interest. The details of theoretical background of BLR are available in literature [14]. The interpretation of effects for various levels of the categorical variables (independent) is very convenient in terms of the odds ratio when this type of model is used. Logistic regression models are also very useful for discrimination analysis (of various groups) when categorical variables are used as independent variables.

5.4 SITE-LEVEL ANALYSIS METHODS

The site level analysis evaluates the performance of each section based on comparisons between the similar designs with in a site (state). It is assumed that within each site, climatic conditions, subgrade soil type and traffic volume are identical for all test sections. Thus the main advantage of this analysis is that comparisons are made among those sections that were subjected to similar loading and environmental conditions. Furthermore, construction methods, material sources and surveys are also assumed to be identical within each site. All site-level analyses were conducted using the Performance Indices (PIs) of the sections for various performance measures. The difference in performance is assessed based on average values. The details of analysis are discussed below.

Comparisons by Design Factors

The site-level analysis consisted of series of comparisons, each focusing on the effect of a particular design/construction factor, for SPS-1 experiment. Such comparisons are not possible for SPS-8 sections because of the limited number of sections in the experiment. For the site level analysis, each performance measure was analyzed in terms of its performance index (PI).

Comparisons were done at two levels—A and B. In level-A analyses, all designs (0101 through 0112, or 0113 through 0124) at a given site are compared such that only one factor is held common within the sections of each group. For example, in level-A analysis, the effects of HMA thickness (4" vs. 7") were studied, within a site, by ignoring base type & thickness, and drainage.

In level-B analyses, most of the factors are 'controlled' for comparisons. In other words, individual sections within a given site are paired such that all but one design parameter are the same. This parameter is the factor being studied. Comparing a given pair of sections will allow for determining the effect of the particular design factor with the highest possible level of constraint (level-B). In this case, there are four factors being studied, so the highest possible number of constraints is three. For example, comparing sections 111 and 112 (SPS-1) allows for determining the effect of base thickness (8"ATB versus 12"ATB), while comparing sections 216 and 220 (SPS2) allows for determining the effect of base type (DGAB versus LCB). Table 5-5 and show possible comparisons within a given site in the SPS-1 experiment.

The relative effects of levels within each design factor were studied based on the ratio of mean performance of the sections corresponding to a level over the mean performance of all levels of the factor. A sample calculation of relative performance is presented in Table 4-6. In the table, the comparison of relative performance indicates that pavement sections with 7" HMA surface thickness are performing better than those with 4" HMA surface thickness, since the relative performance is lower for sections with 7" HMA surface thickness (0.8 versus 1.2).

For factors with two levels (such as HMA surface thickness, PCC thickness, and drainage), the relative performance of each level ranges from 0 to 2, a value of 1 indicating no effect of the factor (i.e., the amount of distress (performance) corresponding to the two levels of the factor is the same). A value less than 1 indicates better performance compared to mean performance of sections corresponding to both the levels of a factor. Consequently, a value higher than 1 indicates the worse performance. The best possible performance translates to 0, and the worst possible performance translates to 2. For cases where there is no distress, each level of a given factor will have relative performance of 1 indicating no difference in performance.

For factors with more than two levels, similar logic can be extended. For the effect of base type, the relative performance of each base type ranges from 0 to 5, since there are five base types in SPS-1 experiment. In the case of the SPS-2 experiment, for the effect of base type, the relative performance of each base type ranges from 0 to 3, since there are three under comparison. A value of 1 for all base types indicates that the amount of distress is the same for all base types. Values close to 1 for all the base types being compared indicate that there is no significant effect of the base type. A higher value indicates more distress (worse performance) for a particular base type. For SPS-1, the worst possible performance translates to 5 (all other base types would show 0, indicating no distress), and the best possible performance translates to 0 (no distress).

The relative performance for various levels of the main factors was calculated for all the sites in SPS-1experiment, and for each performance measure. The concept of relative performance can be utilized across the sites without considering traffic variability because it is calculated at site-level.

Table 5-5 Site Level Comparisons for the SPS-1 Experiment

Effects	Level	Site with Sections 101 to 112		Site with Sections 113 to 124	
		Comparisons	Comments	Comparisons	Comments
I. Effect of Base Type DGAB vs. ATB vs. ATB/DGAB vs. PATB/DGAB vs. ATB/PATB	A	(101,102) vs.(103,104) vs. (105,106) vs. (107,108,109) vs. (110,111,112)	By ignoring base thickness, AC thickness and drainage Comparison of all base types	(113,114) vs.(115,116) vs. (117,118) vs. (119,120,121) vs. (122,123,124)	By ignoring base thickness, AC thickness and drainage Comparison of all base types
	B	(103 vs. 105) (104 vs. 106)	Base thickness=8", AC=4",ND Base thickness=12", AC=7",ND	(116 vs. 118) (115 vs. 117)	Base thickness=12", AC=4",ND Base thickness=8", AC=7",ND
II. Effect of Base Thickness 8" vs. 12" vs. 16"	A	(101,103,105,107,110) vs. (102,104,106,108,111) vs. (109,112)	By ignoring base type, AC thickness and drainage 8" vs. 12" vs. 16"	(113,115,117,119,122) vs. (114,116,118,110,123) vs. (121,124)	By ignoring base type, AC thickness and drainage 8" vs. 12" vs. 16"
	B	(111 vs. 112) (108 vs. 109)	ATB, D, AC=4" DGAB, D, AC=7"	(120 vs. 121) (123 vs. 124)	DGAB, D, AC=4" ATB, D, AC=7"
III. Effect of Drainage	A	(102,103,105,101,104,106) vs. (107,111,112,108,109,110)	By ignoring base type, thickness and AC Thickness	(113,116,118,114,115,117) vs. (120,121,122,119,123,124)	By ignoring base type, thickness and AC Thickness
	B	(103 vs. 111) (101 vs. 108)	ATB=8", AC=4" DGAB=8", AC=7"	(113 vs. 120) (115 vs. 123)	DGAB=8", AC=4" ATB=8", AC=7"
IV. Effect of AC Thickness 4" vs. 7"	A	(102,103,105,107,111,112) vs. (101,104,106,108,109,110)	By ignoring base type, thicknesses and drainage	(113,116,118,120,121,122) vs. (114,115,117,119,123,124)	By ignoring base type, thicknesses and drainage
	B	(101 vs. 102) (103 vs. 104) (105 vs. 106)	By ignoring the base thickness ND, DGAB ND, ATB ND, ATB/DGAB	(113 vs. 114) (116 vs. 115) (118 vs. 117)	By ignoring the base thickness ND, DGAB ND, ATB ND, ATB/DGAB

Table 5-6 Example calculation of relative performance (State 1-Alligator Cracking)

4" AC Thickness		7" AC Thickness	
Section ID	Performance Index	Section ID	Performance Index
102	90.69	101	23.08
103	25.43	104	3.83
105	95.90	106	16.12
107	21.97	108	70.45
111	65.33	109	75.47
112	10.22	110	52.74
Average	51.59	Average	40.28
Mean Performance	(51.59+40.28)/2 = 45.93		
Relative performance	51.59/45.93=1.12	Relative performance	40.28/45.93=0.88

5.5 METHODS FOR INVESTIGATING APPARENT RELATIONSHIP BETWEEN PAVEMENT RESPONSE AND PERFORMANCE

This analysis is aimed at investigating the relationship between the pavement responses (deflections) and performance measures (cracking, rutting, roughness, etc.). The usefulness of such relationships can be further divided in two ways:

- To provide an explanatory information for a given performance trend. For example a relationship between AC pavement rutting and the farthest FWD sensor would indicate that rutting is related to the subgrade.
- To provide a predictive capability of the future level of a given performance measure. For example the initial high average deflection of a section may explain its future cracking and rutting performance.

Relationships were sought by directly relating commonly collected pavement response data to the development of specific distresses using statistical analysis. This analysis was conducted at the site level as well as for the overall experiment. The following describes statistical techniques used to investigate the apparent relationship between response and performance measures.

5.5.1 Univariate Analysis

The simpler statistical methods include *Univariate and Bivariate analysis* of data. These methods include determination of data statistics such as mean, standard deviation and data frequencies. Simple histograms and box plots can also be generated to determine data distribution. These methods also allow for determining the degree of dependence between variables. The results of such an analysis can be graphically illustrated. Such an analysis can also provide summary statistics such as the coefficient of correlation. This analysis was used at the site level. Bivariate analysis can also assist in identifying outliers.

5.5.2 Multiple Regression Analysis

Regression analyses attempt to explain some dependent variable, y, in terms of many independent (explanatory) variables, x's. The model (equation) can be either linear or non-linear, and with actual, transformed, or interaction clusters of variables. The model coefficients can be estimated using best (least squares) fitting techniques. The objective of this method is to develop models explaining the apparent relationships between pavement performance measures and responses. This analysis was used for investigating explanatory relationship by using the data from the overall experiment.

CHAPTER 6 - ANALYSIS RESULTS FOR THE SPS-1 EXPERIMENT

6.1 INTRODUCTION

The purpose of this chapter is to provide a summary of findings from the previous studies and the results of the various analyses conducted for the SPS-1 experiment on flexible pavements in this study. The performance and structural response indicators used in the analysis include fatigue (alligator) cracking, rutting, longitudinal cracking in the wheel path and outside the wheel path, transverse cracking, IRI, and deflections measured by sensors 1 and 7. In this chapter, results from site-level analyses will be followed by results from overall analysis, and results from apparent relationship between response and performance, leading to the summary of findings. Before presenting the results from analyses, a discussion on the effect of construction on the performance of SPS-1 pavements and a brief discussion of the performance of test sections at each site are presented.

The analyses conducted include:

- Site-level analyses (on performance measures): Evaluation of the consistency of the effects of design factors across sites.
- A comprehensive overall analysis of the performance and response data, which includes the following methods of analysis:
  a) Extent of distress by experimental factors (Frequencies)
  b) Linear Discriminant Analysis (LDA)
  c) Binary Logistic Regression (BLR)
  d) Analysis of Variance (ANOVA)
- An investigation of apparent relationships between response and performance at the site level and for the overall population.

6.2 PREVIOUS STUDIES

This section summarizes the findings from the literature review of research reports that deal with pavement performance in the field using SPS-1 experiment. The review included FHWA/LTPP reports, NCHRP reports as well as additional literature, and was focused on research that has identified factors affecting pavement response and performance including roughness. The most relevant reports were found to be those from studies addressing the Long Term Pavement Performance (LTPP) experiments.

6.2.1 Summary of Findings

The information obtained from the literature review has been used to identify various factors that have been shown in past research as having an effect on response and performance progression.

6.2.1.1 Factors Affecting Flexible Pavement Performance

A study [1] entitled "Structural Factors for Flexible Pavements" was conducted using SPS-1 data. The summary of findings from this preliminary study is given below:

Layer Thickness

- The SPS-1 test sections with thick (178-mm (7-inch)) AC surface layers appear to be smoother and develop less fatigue cracking than those sections with thin (102-mm (4-inch)) surface layers. This confirms a similar finding from earlier studies.
- In the SPS-1 experiment, AC surface thickness and the age of the project appear to influence the amount of fatigue cracking that occurs. The test sections that are younger and have thicker AC surface layers have the least fatigue cracking.

Base Layer

- Hot-mix asphalt (HMA) pavements with unbound aggregate base layers show greater rut depths than those sections with asphalt-treated base layers. This suggests that a portion of the rutting measured at the surface is a result of permanent deformations in the unbound aggregate base layer, which is consistent with a previous finding from analysis of the GPS test sections.

- The HMA pavements with unbound aggregate layers have slightly more fatigue cracking and higher IRI values than those sections with asphalt-treated base layers.

- The test sections with coarse-grained soils, asphalt-treated base layers, permeable base layers, thicker bases, and thicker HMA layers were found to be smoother.

- The test sections with permeable asphalt-treated base layers exhibit more fatigue cracking than those without permeable base layers.

Subgrade

- HMA pavements built over coarse-grained subgrade soils are smoother than pavements built over fine-grained subgrade soils. This is consistent with the finding in the SPS-2 JPCP: A stiffer foundation contributes to smoother pavements.

- HMA pavements built over coarse-grained subgrade soils and in a no-freeze climate are smoother and stay smoother over a longer period of time than do those built over fine-grained subgrade soils in a freeze climate. HMA pavements built

over fine-grained sub-grades and in a wet-freeze climate are substantially rougher than those built in other climates.

- HMA pavements built over fine-grained subgrade soils have more fatigue cracking than those projects built over coarse-grained subgrade soils.
- Subgrade soil type and, to a lesser degree, age are important to the amount of transverse cracking measured at each site. More transverse cracking has occurred on the HMA pavements built on fine-grained soils than on pavements built on coarse-grained soils.

Another study [2]was conducted using SPS-1 data to identify the factors affecting pavement smoothness. A summary of main findings from this research is given below:

- A significant difference between early age IRI of pavements placed on DGAB and ATB was observed. No significant difference in early age IRI was obtained on pavements placed on PATB when compared to other two base types.
- The SPS-1 projects that showed the highest increase in IRI were located in Kansas, Iowa and Ohio, reasons being fatigue and transverse cracking for KS (20), transverse and longitudinal cracking (WP) for IA (19), and rutting for OH (39) site. Some of the test sections in TX (48) are showing higher increase in IRI of over 10% within an approximate 6-month period, which is attributed to rutting.
- Although the pavements in IA (19), KS (20), and OH (39) achieved a smooth pavement initially, many sections, including very thick sections had high increases in roughness during the initial life of the pavement. Achieving a smooth

pavement initially does not guarantee that it will remain smooth even during the initial life.

- Mix design problems in the AC, inadequate preparation of the subgrade prior to placing the pavement, or other construction problems can cause smoothly built pavements to have higher increase in roughness within a short time period.

Two studies[3, 4] were conducted to investigate the effects of sub-drainage on the performance of asphalt and concrete pavements. The following is a summary of their findings for asphalt pavements:

- Based on 7 years (on average) of SPS-1 data, those HMA sections built on permeable bases without edge drains were found to perform better than those with edge drains.
- The ranking of performance in terms of IRI and cracking for various base types with all other design features matched is from poor to good performance: un-drained dense-graded aggregate bases, drained permeable asphalt-treated bases, and un-drained dense-graded asphalt-treated bases.
- The results in terms of rutting for the above three sub-drainage designs were inconclusive.

It was observed from the performance data (see Chapter 4) that a number of sections located in various sites have shown high distresses (cracking and rutting). Keeping in view, the early age of the sections, it was felt necessary to investigate effects of the construction on the performance of all test sections in the SPS-1 experiment. These effects are discussed in the next section.

6.3 EFFECT OF CONSTRUCTION ON PAVEMENT PERFORMANCE

For the SPS-1 experiment, detailed construction guidelines were developed by LTPP (summarized in Chapter 2) for the participating agencies to control variability in construction across sites. The lesser variability across sites, the lower the "noise", and easier it is to determine the "pure" effects of the design factors on pavement performance. However, some deviations have occurred during construction of various sites and some of those issues were highlighted in the construction reports prepared by the participating agencies (see Chapter 3). In general, sections with serious construction deviations will perform poorer than those with normal construction conditions and inclusion of such sections in the analysis may distort (bias) the effects of design factors. Sections with deviations may be identified at least in three ways, based on:

- Construction issues highlighted in construction reports for each site,
- Unusual performance trend of individual sections, and
- Unusual material properties of pavement layers.

Depending on the nature of construction issues (construction quality) the performance of a pavement is affected. Minor issues (such as minor thickness deviation in base course) may not seriously impact initial performance where as major issues (such as HMA mix issues, compaction or drainage problems) have greater chances of affecting performance early in the life of a pavement. Hence, a construction deviation (poor quality) may be used to identify sections that may potentially show an abnormal performance. Also, some sections with serious early performance concerns were "de-assigned" from LTPP database (for example, 39-101 and 20-101).

In this study, any abnormality in early performance was used as an indicator to identify substandard sections. The performance of all the sections, over time, was

observed for this purpose and those sections that had premature "failure" (within first 2 to 3 years of service life) were identified. It was observed that most of the identified sections are from a few sites (for example KS for fatigue cracking and TX for rutting in SPS-1) in the experiments, indicating consistent construction problems in those sites. In such cases, all sections from the identified sites were excluded from related analyses. Material properties (example: HMA mix properties) of pavement layers may also indicate non-compliance in construction. Limited material data are available in DataPave (Release 17). This data were also considered to explain the probable causes of unexpected performance wherever possible.

In order to further investigate the construction-related performance issues, each performance measure for all pavement sections in SPS-1 experiment was examined over time. This analysis helped minimize the bias, if any, in the results. The analysis is discussed next with illustrations. This section of the report is followed by site-wise performance summaries of the test sections.

6.3.1 Construction-related Issues

A brief discussion of construction-related performance issues for each performance measure in SPS-1 is presented in this section of the report. Based on the time-series plots for all distress measures it was found that premature "failure" was predominantly observed in rutting for a relatively large number of pavement sections. Hence, this is presented first among all the performance measures.

Rutting

Figure 6-1 shows rutting in all the SPS-1 test sections over time. It can be observed that a considerable number of sections have noticeably high initial rutting. The

premature rutting in these pavements can be further classified into two types based on the causes of rutting:

- Mix-related rutting, and
- Base layer rutting (this could be because of wet base and poor drainage or poor compaction in un-bound pavement layers).

Therefore, the premature or early rutting in pavements was separated from the structural rutting in order to study the effects of structural factors on the long-term pavement performance. Table 5-1 is a summary of details regarding the pavement sections that exhibited early (pre-mature) rutting in the SPS-1 experiment. The causes of rutting were identified by using the transverse profile data available in the LTPP database based on the criteria developed in NCHRP Report 468 [5]. Figure 6-3 through Figure 6-6 show the average transverse profiles of some sections, from four of the SPS-1 sites, which showed premature rutting. HMA material-related data from the field cores were extracted from DataPave (Release 17.0). Unfortunately a limited amount of material data is available at this point in time; therefore, only a few of the mix-related properties could be calculated. The summary of the mix-related data for the sites that showed early rutting is given in Table 6-2. It can be seen from these asphalt mix properties that there is high variation in the field air void content between these identified sites. High air voids in the pavement sections at the Kansas site KS (20) and very low air voids (high VFA) at the Texas site TX (48), are noticeable. The pavements built at these two sites have shown extensive cracking and rutting, respectively.

To investigate the effect of structural factors on the rutting performance, the pavement sections were separated, as explained above, into two categories: (a) pavements with premature rutting, and (b) pavements which exhibited structural rutting. Figure 6-2

shows the rutting performance of sections with probable "structural" rutting. The effect of outliers (sections with premature rutting) on the rutting performance for SPS-1 pavements can be observed by comparing Figure 6-7 and Figure 6-8.

An analysis of variance (ANOVA) was performed separately on two subsets of the data as well as on data from all the sections (superset of the two subsets). The first subset represents the pavement sections which have shown higher rutting at an early age (mix-related or material-related). The second subset includes the pavements which have shown normal rutting growth, and these pavements were assumed to be exhibiting "structural rutting". Table 6-3 is the summary of results from ANOVA. Initially, only the main effects of structural factors were considered in the analysis by blocking the site. The results indicated that none of the structural factors has a significant effect on premature rutting. These results are reasonable, as it is expected that pavements will undergo accelerated rutting early in their service life (irrespective of the pavement structure) if the asphalt layer has mix-related issues or when the base has drainage-related issues. Next, ANOVA was performed by taking all the experimental factors and the results are shown in Table 6-5. The mean rut depths from both analyses by each experimental factor are shown in Table 6-4 and Table 6-6, respectively, to illustrate the effects. A brief discussion of the results from analysis of structural rutting is given below:

HMA Thickness: Pavement sections with "thin" (4-inch) HMA surface layer have undergone higher rutting compared to those with "thick" (7-inch) HMA surface layer. However, this difference was not found to be of practical significance at this point in time.

Base Type: No statistically significant effect of base type was observed on the structural rutting, at this point. On average, sections with DGAB have shown slightly higher rutting than those built with treated bases. A slight (0.05<p-value <0.1) interaction effect was observed between base type and subgrade type. Among pavements built with DGAB, those built on fine-grained soils have shown higher rutting than those built on coarse-grained soils.

Base Thickness: Higher rutting occurred in sections with 8-inch base than those with 12-inch or 16-inch base. However, the effect is statistically marginally significant, is not of practical significance.

Drainage: On average, sections with no drainage have shown slightly higher rutting than those with drainage. However, this effect of drainage on the structural rutting was not found to be statistically significant at this point in time.

Subgrade Type: Rutting in sections built on fine-grained soils is comparable with rutting in sections built on coarse-grained soils. The effect of subgrade types was not found to be statistically significant for structural rutting.

Climatic Zone: Effect of climate was found to be statistically significant. Pavements located in WNF zone have shown higher rutting than those located in WF zone. However, this effect is not of practical significance.

Given the above findings, all subsequent statistical analyses on rutting performance, (presented later in this chapter) were performed on data from the sections with structural rutting. This assumes that only pavement sections which have exhibited structural rutting will capture the effects of design factors on the long-term rutting performance.

Table 6-1 Identified sites and sections with rutting problems

Site	Sections deleted due to extensive rutting	Probable cause of rutting	Comments
Arizona, AZ (4)	All 12 sections	HMA and Base	The rutting is either occurring due to HMA mix problems or base or both (from transverse profile)
Kansas, KS (20)	All 12 sections	HMA and Base	Some sections have shown HMA mix related rutting and others showed base problems (from transverse profile)
Michigan, MI (26)	113, 114, 119, 122 120	HMA	The first four sections were deleted from the LTPP database just after construction
Nebraska, NE (31)	All 12 sections	HMA	From transverse profile
Ohio, OH (39)	101, 102	Base	From transverse profile
Texas, TX (48)	All 12 sections	HMA	See reference[6]
Virginia, VA (51)	113	Base	Only one section has shown accelerated deterioration at this site due to base problems (from transverse profile)
Total	56 sections		

Table 6-2 Average asphalt mixture properties in the field

State	Asphalt Content (%)	Air Void Content (%)	VMA[a] (%)	VFA[a] (%)	Superpave Specifications	
					VMA[1] (%)	VFA[2] (%)
4	4.4	10.6	19.1	44.3	>14	65-75
19	4.2	10.5	-	-		
20	4.1	15.3	21.4	29.1		
26	5.0	6.5	-	-		
31	4.2	6.2	-	-		
39	6.6	11.2	-	-		
48	4.4	1.8	12.2	85.0		
51	4.9	9.7	20.4	52.6		

Note: [a] G_{sb} values are missing therefore, these properties can not be calculated, [1] the minimum VMA requirement for nominal maximum size of 12.5 mm, [2] VFA requirements for traffic > 100 million ESALs (Source: Superpave Mix Design, SP-2)

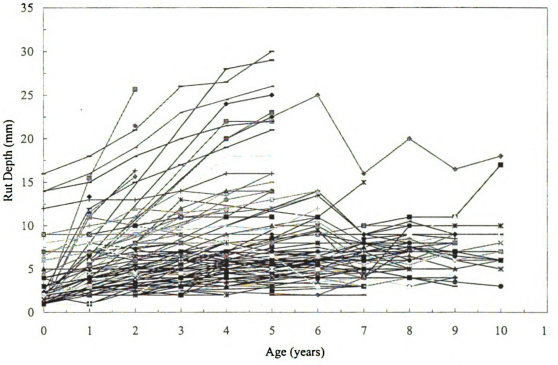


Figure 6-1 Rutting with time for SPS-1 pavements - All sections

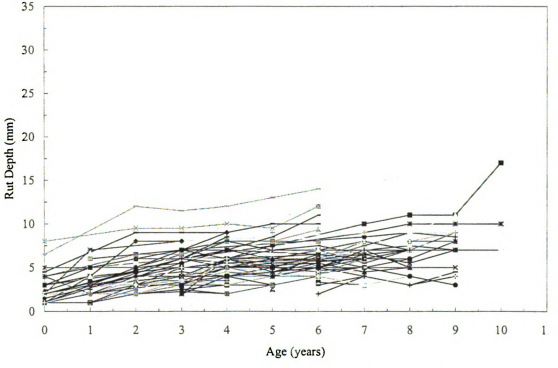


Figure 6-2 Rutting with time for SPS-1 pavements – Selected sections

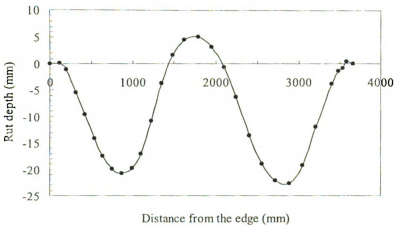


Figure 6-3 Transverse profile for base rutting—Section 20-0102

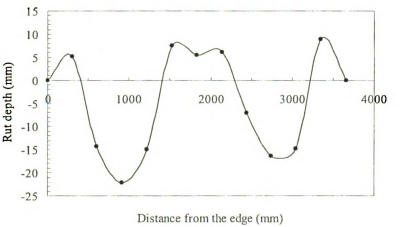


Figure 6-4 Transverse profile for asphalt rutting— Section 31-0113

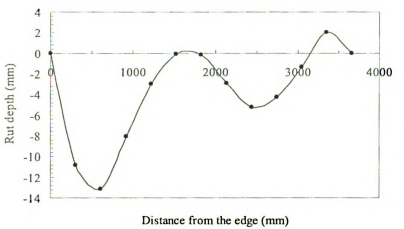


Figure 6-5 Transverse profile for (HMA + base) rutting— Section 39-0101

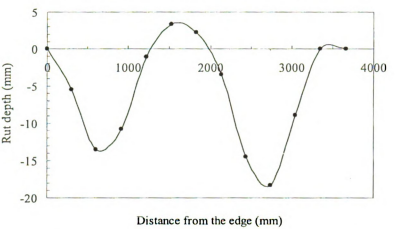


Figure 6-6 Transverse profile for (Base) rutting— Section 51-0113

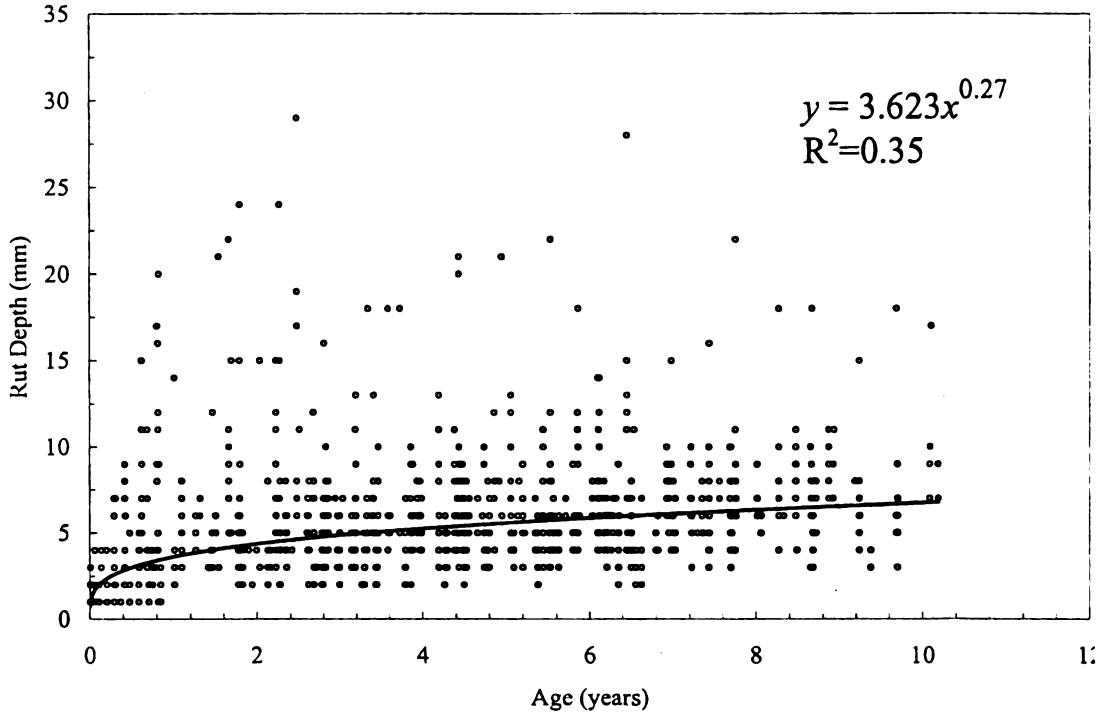


Figure 6-7 Rutting growth with time for SPS-1 pavements – All sections

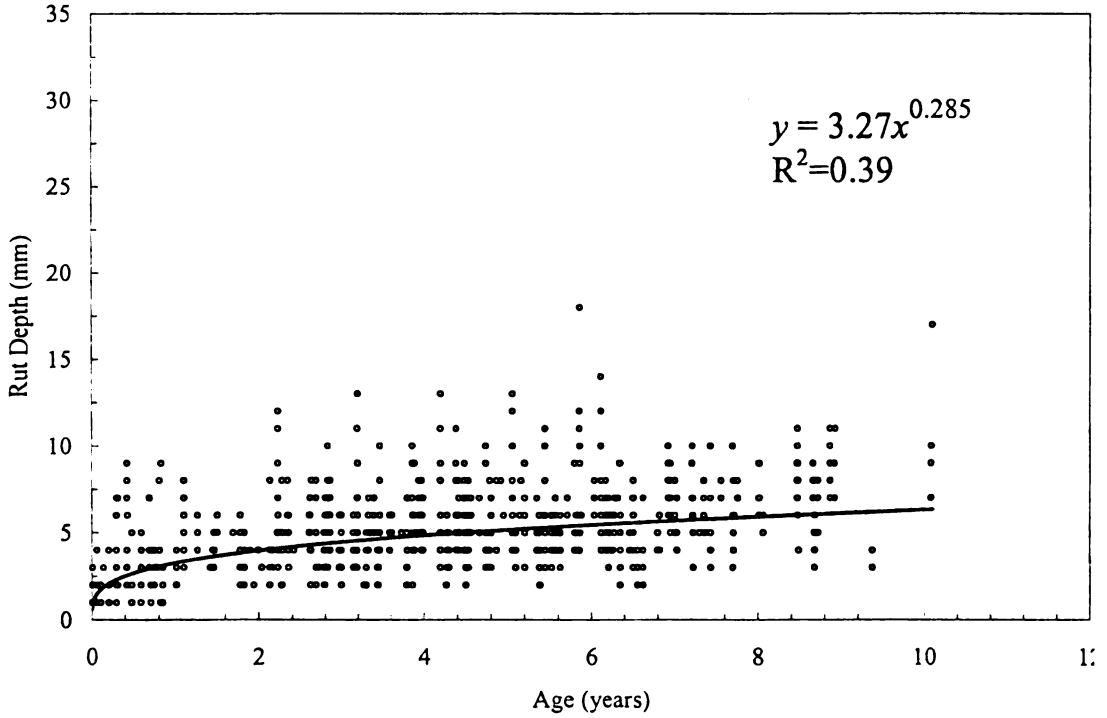


Figure 6-8 Rutting growth with time for SPS-1 pavements – Selected sections

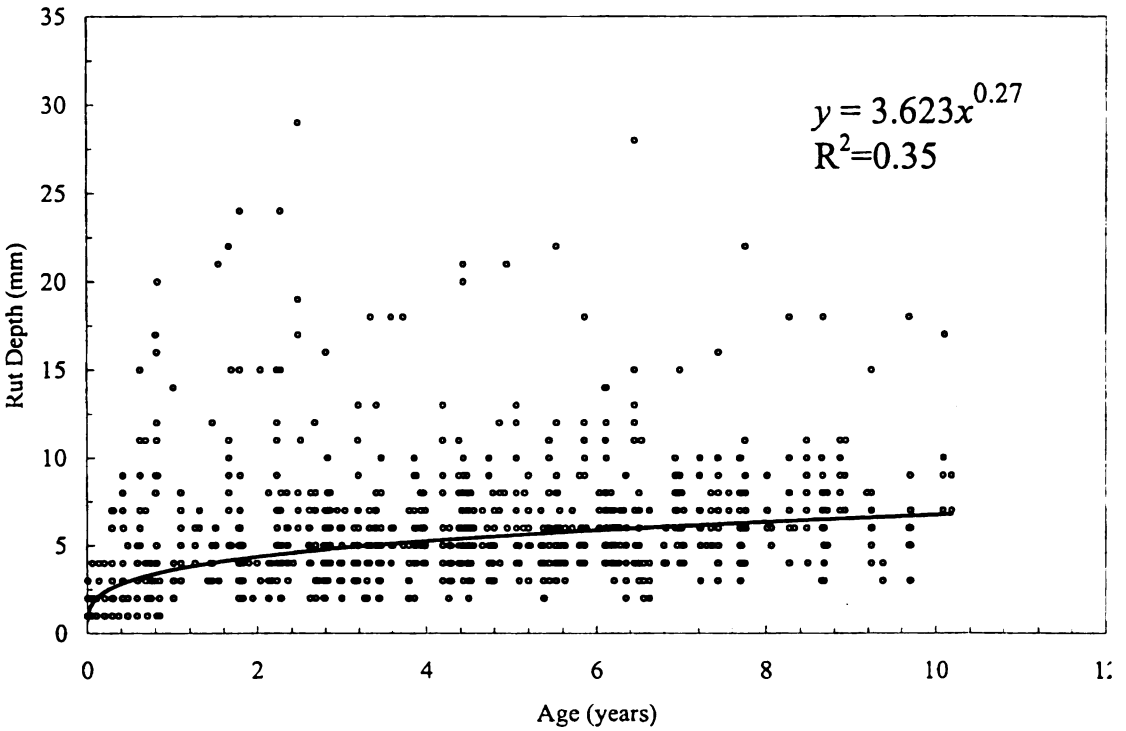


Figure 6-7 Rutting growth with time for SPS-1 pavements – All sections

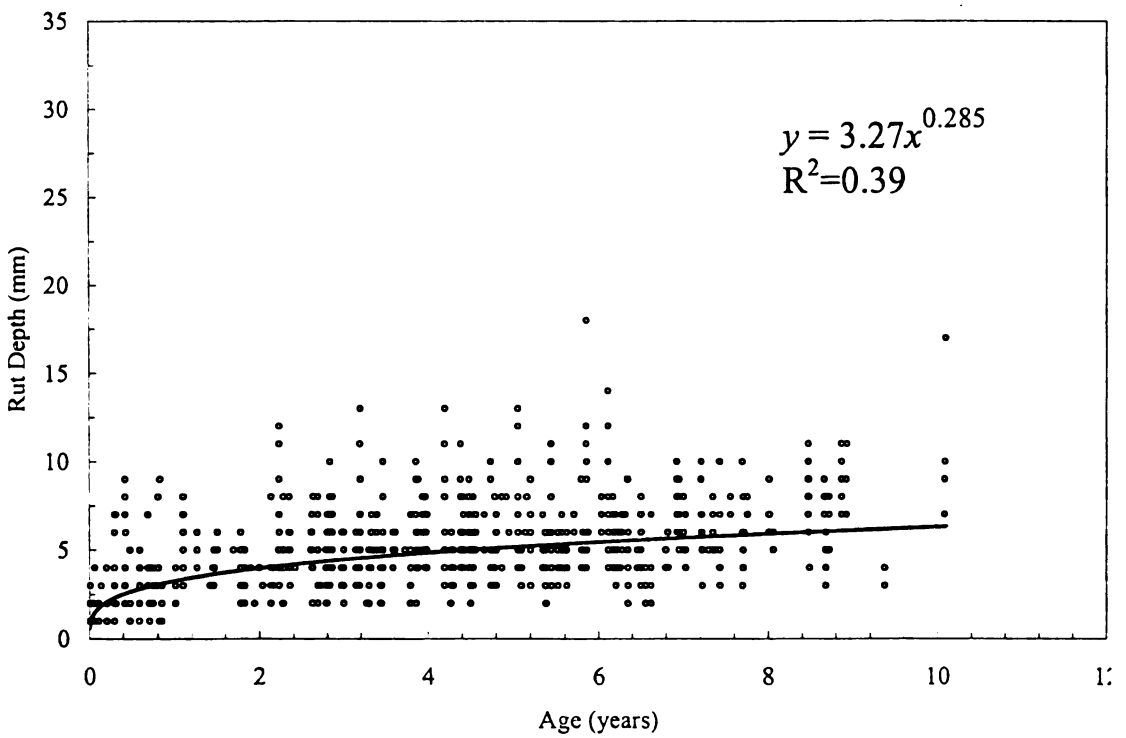


Figure 6-8 Rutting growth with time for SPS-1 pavements – Selected sections

Table 6-3 Summary of p-values from ANOVA for determining the effect of main design factors on pavement rutting

Design Factor	Rutting Type		
	Non-structural rutting[a]	Structural rutting	Overall
HMA thickness	0.71	0.074	0.20
Base type	0.20	0.51	0.017
Base thickness	0.99	0.08	0.195
Drainage	0.12	0.25	0.030
Site (blocked)	0.15	0.00	0.00
	R^2=0.343 N=53	R^2=0.55 N=159	R^2=0.57 N=212

Note: [a] Mix-related or premature rutting in un-bound layers.

Table 6-4 Summary of marginal means from ANOVA for determining the effect of main design factors on pavement rutting

Design Factor		Rutting Type		
		Non-structural (mm)	Structural rutting (mm)	Overall (mm)
HMA thickness	4"	9.0	5.3	6.1
	7"	10.0	4.9	5.8
Base type	DGAB	11.0	5.2	6.5
	ATB	9.05	4.9	5.7
	ATB/DGAB	8.2	5.1	5.6
Base thickness	8"	10.0	5.6	6.3
	12"	10.0	5.1	5.8
	16"	9.05	5.0	5.8
Drainage	N	11.0	5.3	6.3
	Y	9.0	4.9	5.6
MSE[a]		0.206	0.062	0.10

[a] MSE is in natural log.

Table 6-5 Summary of p-values from ANOVA for determining the effect of experimental factors on pavement rutting

Experimental Factor	Rutting Type	
	Non-structural rutting[a]	Structural rutting
HMA thickness	0.16	0.043
Base type	0.94	0.54
Base thickness	0.76	0.09
Drainage	0.50	0.28
Subgrade	0.46	0.43
Zone	0.27	0.00
Traffic	0.000	0.013
	R^2=0.552 N=53	R^2=0.55 N=159

Note: [a] Mix-related or premature rutting in un-bound layers.

Table 6-6 Summary of marginal means from ANOVA for determining the effect of experimental factors on pavement rutting

Design Factor		Rutting Type	
		Non-structural rutting	Structural rutting
HMA thickness	4"	9.7	5.7
	7"	11.8	5.0
Base type	DGAB	10.7	5.4
	ATB	10.7	5.2
	ATB/DGAB	10.7	5.3
Base thickness	8"	9.7	5.7
	12"	10.7	5.0
	16"	10.7	5.2
Drainage	N	11.8	5.5
	Y	10.7	5.1
Subgrade	F	13	5.2
	C	9.7	5.4
Zone	WF	6.5	4.9
	WNF	13	5.6
	DF	14.4	4.0
	DNF	-	7.1
MSE[a]		0.137	0.091

[a] MSE is in natural log.

Fatigue Cracking

It was observed that sections from the Kansas, KS (20), site exhibited the highest area of cracking at an early age compared to sections from other sites. The KS (20) site had a wet subbase during construction (based on the construction report). Also, from the materials data (DataPave) it was found that the test sections at KS (20), on average, have "high" air void content in the HMA (see Table 6-2). These reasons could have caused the abnormally high cracking in the sections at this site. Figure 6-9 and Figure 6-10 are time-series plots of fatigue cracking for all the pavements sections, before and after exclusion of sections from the Kansas site, KS (20). All statistical analyses pertaining to fatigue cracking, presented in this chapter, were conducted without including data from sections at Kansas site, KS (20).

Roughness and other Performance Measures

Figure 6-11, through Figure 6-14 show the time-series plots for IRI, transverse cracking and longitudinal (WP and NWP) cracking, respectively. It can be observed that only a few sections have exhibited an abnormal performance. Exclusion of data from these sections was not considered necessary as their inclusion will not impact the results considerably. Therefore, all the pavement sections were included in the analyses of roughness, longitudinal cracking (WP and NWP) and transverse cracking.

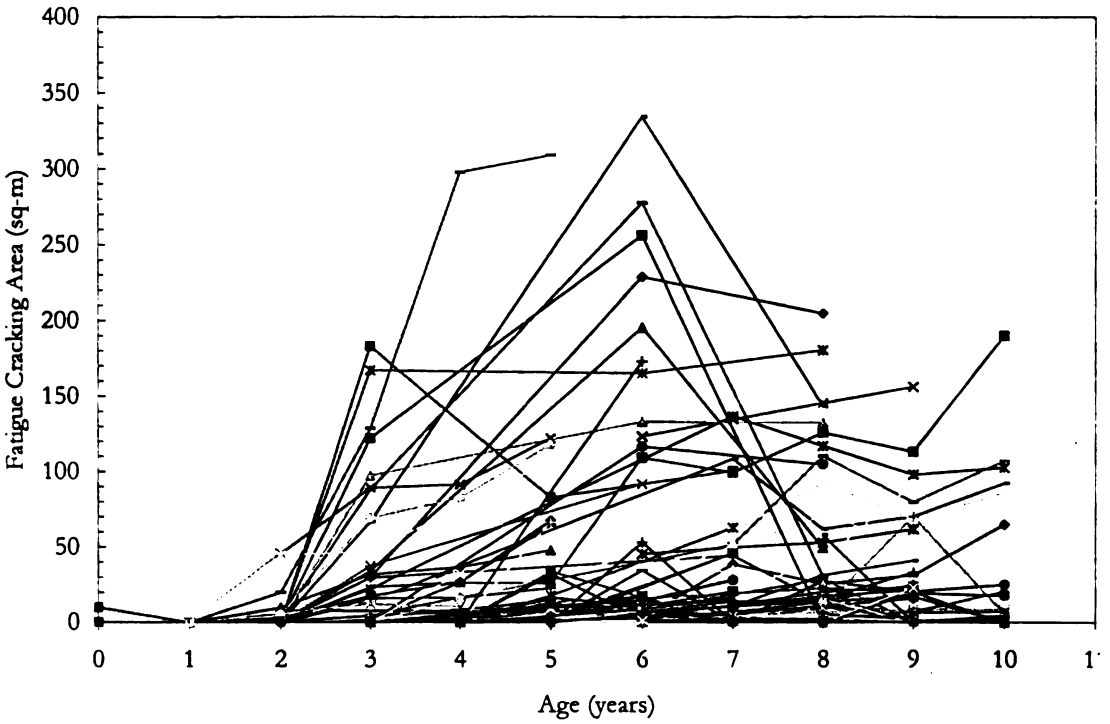


Figure 6-9 Fatigue cracking with time for SPS-1 pavements – All sections

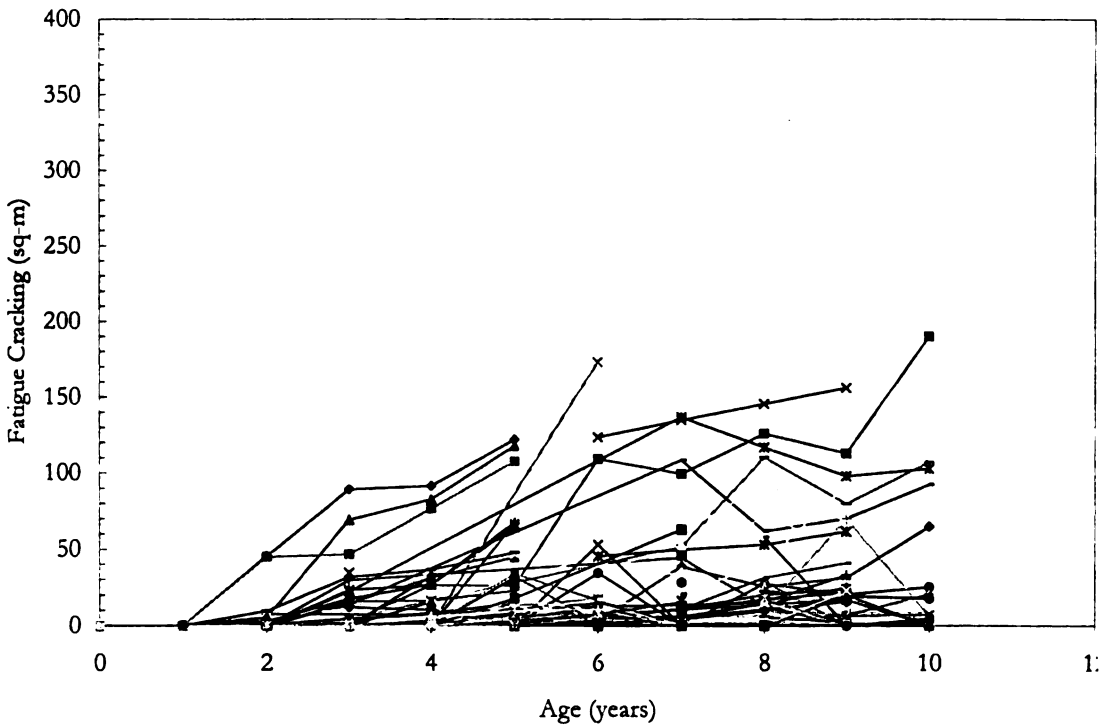


Figure 6-10 Fatigue cracking with time for SPS-1 pavements – Selected sections

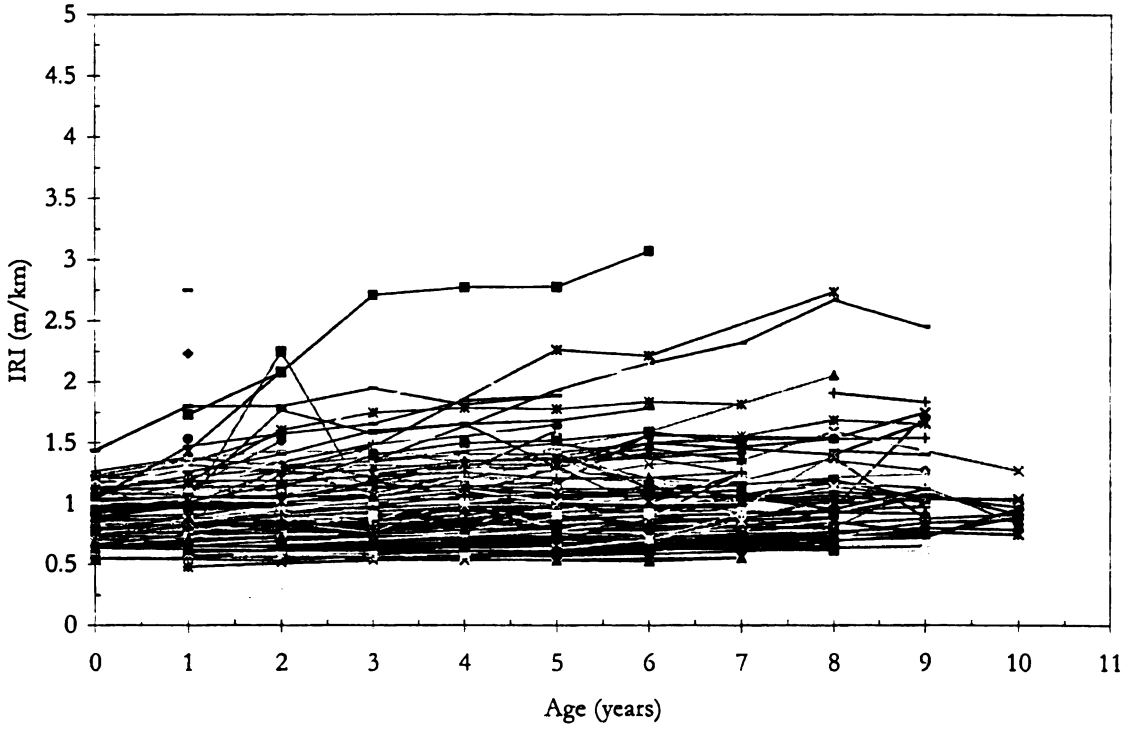


Figure 6-11 IRI with time for SPS-1 pavements – All sections

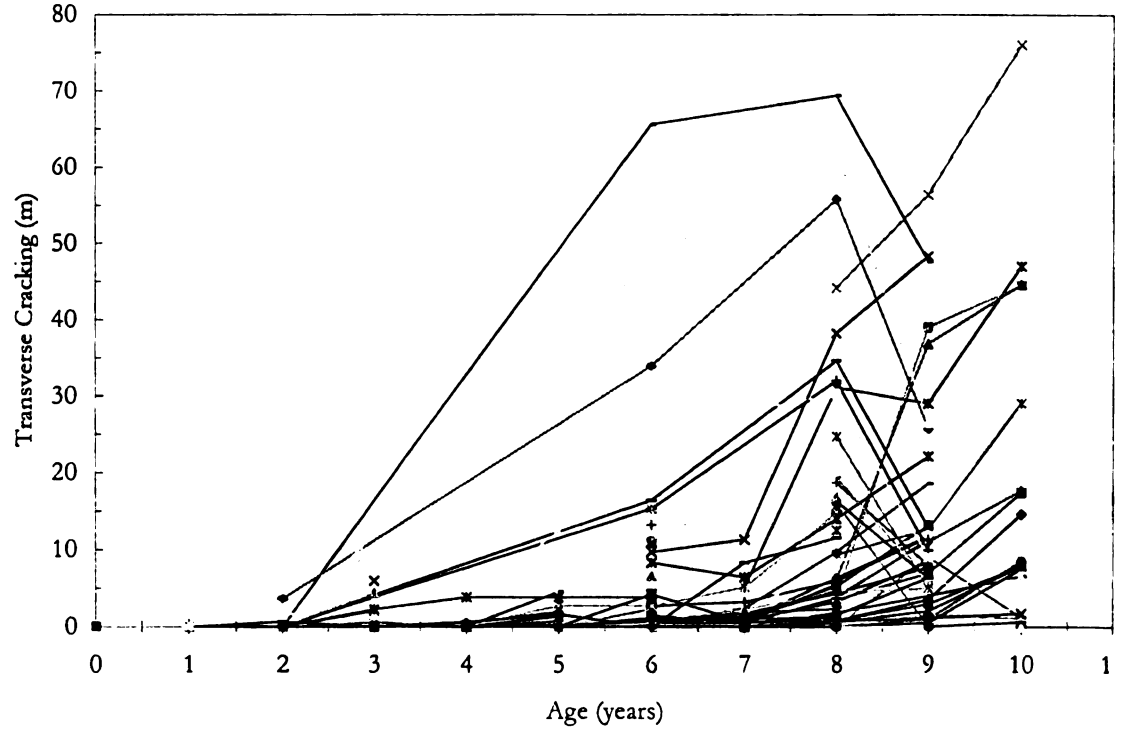


Figure 6-12 Transverse cracking with time for SPS-1 pavements – All sections

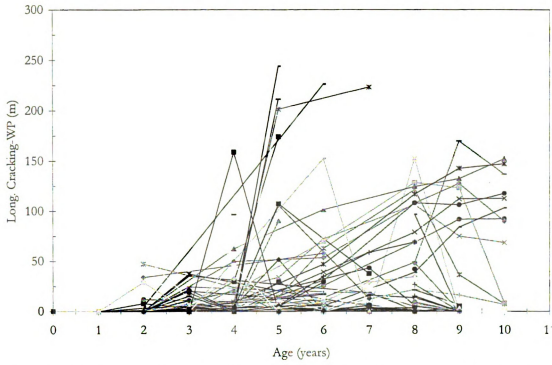


Figure 6-13 Longitudinal cracking-WP with time for SPS-1 pavements – All sections

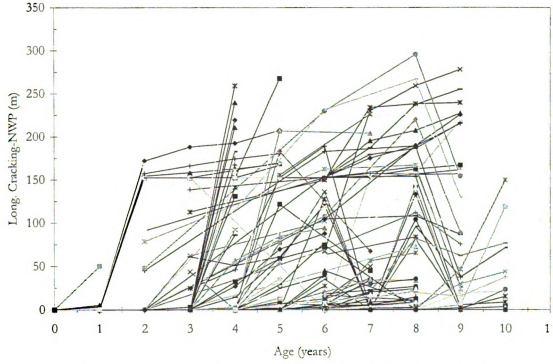


Figure 6-14 Longitudinal cracking-NWP with time for SPS-1 pavements – All sections

6.3.2 Drainage-related issues

In the above section, construction-related issues have been linked to the poor performance of some pavement sections. Some construction and/or maintenance related issues with respect to the in-pavement drainage were also identified in previous research [3, 4]. The in-pavement drainage for some of the SPS-1 flexible pavement sections was found to have some deviations from design. All the drained sections of the SPS-1 experiment were video taped to assess the condition of the drainage in the project 1-34C [4]. A subjective assessment of the quality of the drainage functioning in each test section as "good" or "poor" was reported. The ratings assigned to each section are summarized in Table 5-7. The "poor" rating was indicative of; (i) buried lateral outlet, (ii) outlet fully blocked with silt, gravel or other debris (iii) longitudinal drains being fully blocked, or (iv) a considerable amount of stagnant water in the longitudinal drain. A "good" rating was given to the drainage if a reasonably sufficient flow of water was evident even if some amount of material was present in the drains. Hall et al [4] conducted preliminary analysis of the performance of SPS-1 test sections in light of their assessment of drainage, and a brief summary of their findings are presented below:

- Undrained pavement sections built on DGAB may develop cracking, rutting and roughness more rapidly than drained sections built on ATB.
- Undrained pavement sections built on ATB may develop roughness and cracking more slowly than those built with drained DGAB, while the un-drained sections may develop rutting more rapidly.
- Undrained pavement sections built on ATB/DGAB may develop roughness and rutting more quickly than those on drained DGAB, while the undrained sections may develop cracking more slowly.
- Also, among the drained sections, those with "good" rating for drainage performed better than undrained sections, while those with "poor" rating did not.

However, the above trends were based only on the average performance and in no case, were the differences detected statistically significant. These preliminary findings (from Hall et al) should be considered during the interpretation/validation of the results (from this study) regarding the effect of drainage.

The results from the site level analysis for SPS-1 experiment are summarized in the next section.

Table 6-7 Subjective ratings of drainage functioning at SPS-1 test sections based on video inspection results (source: [4])

State	Test Section											
	0101 0113	0102 0114	0103 0115	0104 0116	0105 0117	0106 0118	0107 0119	0108 0120	0109 0121	0110 0122	0111 0123	0112 0124
	Base Type											
	DGAB		ATB		ATB/DGAB		PATB/DGAB			ATB/PATB		
	Un-drained						Drained					
AL (1)							G[1]	G	G	G	G	G
AZ (4)							G	G	G	G	G	?[2]
AR (5)							P[3]	P	P	P	P	P
DE (10)							G	G	G	G	G	G
FL (12)							P	P	P	P	P	P
IA (19)		P	G	P	P	G	?	G	P	?	?	P
KS (20)							?	P	?	?	?[4]	?
LA (22)							?	P	P	?	?	?
MI (26)						P	?	?	P	?	P	?
MT (30)							G	G	G	G	G	G
NE (31)							G	G	G	G	G	G
NV (32)							P	P	P	P	P	P
NM (35)							P	P	?	P	P	P
OH (39)		G	G	G		P	?	G	?	G	G	G
OK (40)							?	?	?	?	?	?
TX (48)							P	P	P	P	P	P
VA (51)			G	G	G		?	G	G	G	G	G
WI (55)							?	?	?	?	?	?

[1] G= Drainage function rated as good

[2] ? = Drainage outlet not found

[3] P = Drainage function rated as poor

[4] ?* = Camera could not be inserted

6.4 SPS-1 PROJECT PERFORMANCE SUMMARIES

This section summarizes the performance trends for each site within the SPS-1 experiment based on the latest year data. The performance summary for each site is based on the data available in the Release 17.0 of the DataPave. The severity levels for all types of cracking were combined to calculate its total magnitude. This descriptive summary is intended to help the reader gain an understanding of performance of test sections at each site. The performance of pavement sections regarding selected distresses is presented here, for each site. The identified distresses include fatigue cracking (sq-m), longitudinal cracking-WP (m), longitudinal cracking-NWP (m), transverse cracking (m), rutting (mm) and roughness (m/km).

Additional details about each of the sites can be found in site-level summaries presented in Appendix A1 and performance data tables in Appendix A2 of reference [7].

Alabama, AL (1)

Performance data is available for 10 years (1994-2003) at this site. Fatigue cracking is the dominant distress type at this site. Section 103, 104, 106, 107 and 112 have less than 10% (area) cracking while all other sections have fatigue cracking of range 10% to 15%. A wide range of longitudinal cracking-WP (between 5 m and 30 m) occurred on all the sections. Longitudinal cracking-NWP, between 80 m and 200 m, occurred on all the sections, by year 8. Transverse cracking, of range 15 m to 50 m, was observed in sections 101, 102, and 105 respectively. Sections 102 and 105 have shown 10 mm and 17 mm of rutting, respectively, while other sections have rutting between 6 mm

to 9 mm. Sections 102 and 107 have IRI of 1.4 m/km and 1.7 m/km, respectively, while other sections have IRI less than 1.0 m/km.

Arizona, AZ (4)

The performance data is available for 10 years (1994-2003) at this site. Less than 3% (of area) of fatigue cracking occurred in sections 113, 119 and 124, while less than 1% cracking occurred in other sections. Longitudinal cracking-WP is the dominant type of cracking with all the sections showing a cracking between 10 to 150 m. Longitudinal cracking-NWP of 150 m and 120 m occurred on sections 114 and 120, respectively; while in other sections longitudinal cracking-NWP is less than 50 m. Transverse cracking of 76 m and 45 m occurred on sections 113 and 121, while other sections have less than 30 m of transverse cracking. Rutting of 14 mm and 25 mm occurred on sections 114 and 119, respectively, after 6 years. In other sections rutting ranged from 3 mm to 9 mm. All sections except 113, 120 and 122 have IRI greater than 1 m/km while other sections have IRI less than 1.0 m/km.

Arkansas, AR (5)

The performance data is available for 9 years (1995-2003) for this site. Fatigue cracking area ranged from 10% to 25% in sections 119, 120 and 121; whereas, all other sections have exhibited less than 10% of fatigue cracking area. All the sections exhibited longitudinal cracking-WP less than 10 m. All the sections have exhibited longitudinal cracking-NWP, which range from 140m to 280 m. Transverse cracking of 48 m was observed only in section 119, whereas all other sections have less than 20 m of cracking. Rutting between 5 mm to 9 mm was observed at the site. Sections 119 and 120 have IRI of about 1.7 m/km while other sections have IRI of about 1 m/km.

Delaware, DE (10)

The performance data is available for 7 years (1996-2003) for this site. Fatigue cracking is the dominant distress type at this site. Sections 101 and 102 have cracking of about 10% and 20%, while in other sections cracking was less than 10%. Longitudinal and transverse cracking did not occur on any of the sections. All sections except 102 have shown rutting of range 2 to 4 mm while, sections 102 has 7 mm of rut depth. IRI for all the test section is less than 1.0 m/km.

Florida, FL (12)

The performance data is available for 7 years (1996-2003) for this site. Fatigue cracking less than 1% occurred in the sections. Longitudinal cracking-WP, less than 10 m, occurred in sections 107, 108, 110, and 112. Longitudinal cracking-NWP, less than 50 m was observed in sections 101,105,108 and 110. Transverse cracking was not observed on any of the sections. Rutting of about 4 mm occurred in all sections, except sections 103, 105, 110 and 111, which exhibited rutting of about 6 mm. All the test sections have shown less than 1 m/km of IRI.

Iowa, IA (19)

The performance data is available for 9 years (1995-2003) at this site. Fatigue cracking of 10% (area) was observed on section 102, and all other sections have fatigue cracking less than 2%. Longitudinal cracking-WP of range 50 m to 100m occurred in 102, 104, 105, and 107, while in other sections this cracking is less than 30 m. Longitudinal cracking-NWP of range 110 m to 295 m occurred at this site, in all the sections. Transverse cracking of 30 m to 70 m was also observed in sections 101 through 106, while in other sections this cracking is less than 20 m. Sections 107, 108, and 109

have rut depth of about 7 mm while other sections have rutting between 3 mm and 6 mm. Sections 101 and 102 have IRI values of 1.8 m/km and 2.5 m/km, while IRI in other sections ranged from 1.0 m/km to 1.6 m/km.

Kansas, KS (20)

The performance data is available for 8 years (1993-2001) at this site. Fatigue cracking is the main distress type in all the sections. Fatigue cracking ranged from 5% to 20%. Longitudinal cracking-WP has not occurred at this site. Sections 103, 104, 105 and 110 have longitudinal cracking-NWP of 72 m to 189 m, while in other sections the this cracking is less than 20 m. Sections 103, 105 and 110 have transverse cracking less than 16 m. Sections 101, 102, 107 (data available for first 2 years only) have shown rut depth between 16 mm to 25 mm, whereas section 105 has exhibited 13 mm of rutting, after 3 years. Other sections have rutting less than 5 mm. Section 105 has an IRI of 2.7 m/km while sections 104, 109 through 112 have IRI less than 1.2 m/km. Other remaining sections have IRI between 1.5 and 2.0 m/km.

Louisiana, LA (22)

The cracking data is available for only 2 years (1997-1999) at this site. The rutting data is available for 6 years (1998-2003) while roughness data is available only for one year (1997). No cracking occurred on any of the sections. Sections 119, 122 and 123 have rutting less than 4.0 mm while; other sections have rutting of about 5 mm. All sections have initial roughness less than 0.8 m/km.

Michigan, MI (26)

The performance data is available for 7 years (1996-2003) for this site. The performance data is only available for eight sections for this site, as four sections (112, 114, 119 and 122) have been 'de-assigned' from LTPP due to construction issues. Fatigue cracking of 2 to 10% was observed on sections 115, 116, 117 and 124, while no cracking occurred in other sections. Longitudinal cracking-WP and transverse cracking has not occurred on any of the test sections. Longitudinal cracking-NWP was observed on all test sections. Section 116 has longitudinal cracking-NWP of 35 m while this cracking in other sections ranged from 120 m to 188 m. Sections 115 and 117 have rutting of 9 mm and 12 mm while other sections have rutting of about 6 mm. Sections 118 and 117 have IRI of 1.2 m/km and 1.4 m/km whereas other sections have IRI less than 1.0 m/km.

Montana, MT (30)

The performance data is available for 5 years (1998-2003) at this site. Fatigue cracking was observed in sections 115, 117, 120, 121, and 122, with a range of 5% to 10%. Other sections have fatigue cracking of less than 5%. Longitudinal cracking-WP occurred only in sections 113, 114, 115, 118, and 124, with a range of 5 to 10 m. Longitudinal cracking-NWP occurred in all sections, with a range of 150 to 208 m. Transverse cracking were observed only in sections 113, 115 and 121, and this cracking was between 5 m to 10 m. Rutting of 8 mm occurred in sections 120 and 121, while it ranged from 3 to 5 mm in other sections. Sections 120 and 121 have IRI of about 1.5 m/km whereas other sections have IRI of range 0.8 m/km to 1.1 m/km.

Nebraska, NE (31)

The performance data is available for 7 years (1995-2002) at this site. Fatigue cracking has just initiated only in sections 113 and 114. Longitudinal cracking-WP of 97 m was observed in section 113. Longitudinal cracking-NWP and transverse cracking did not occur in any of the sections. Rutting of 29 mm was observed on section 113 by year 5. Among other sections, 114, 115, 118, 123, and 124 have rutting between 11 mm and 15 mm, while others have rutting between 5 mm and 8 mm. Sections 113 has an IRI of 1.9 m/km and all other sections have IRI of about 1.0 m/km. All test sections have an initial IRI between 0.9 and 1.4 m/km.

Nevada, NV (32)

The performance data was collected for 8 years (1996-2003) for this site. Fatigue cracking area is less than 1% in all the test sections. No noticeable longitudinal cracking-WP was observed in any of the sections. More than 50 m of longitudinal cracking-NWP have been observed only in section 103 and sections 101, 102, 104, 105 and 109 have less than 35 m of longitudinal cracking-NWP. Less than 15 m of transverse cracking occurred in sections 102, 107 and 109. Sections 101, 104, 107, and 108 have rut depth less than 4 mm while other sections have rut depth of about 5 mm. All sections except 102 have IRI less than 1 m/km. Section 102 has an IRI value of 1.4 m/km.

New Mexico, NM (35)

The performance data was collected for 8 years (1997-2003) at this site. Fatigue cracking, less than 1% of area, occurred in sections 102 to 104. Sections 101, 103,105 and 107 have exhibited less than 45 m of longitudinal cracking-WP, while section 102 has 70 m of this cracking. Furthermore, less than 25 m of longitudinal cracking-NWP

was observed only in sections 101, 102, and 107. No transverse cracking was observed at this site. All sections at this site have exhibited rut depth of about 7 mm. All sections except 101, 102, 103, 107 and 112 have IRI between 1.0 to 1.3 m/km while other sections have less than 1.0 m/km of IRI.

Ohio, OH (39)

The performance data was collected for 7 years (1996-2002) for this site. At this site, cracking data for sections 101, 102,105 and 107 are only available for the initial year. These sections were 'de-assigned' from the LTPP because of premature rutting. Fatigue cracking is the dominant cracking distress within all sections at this site. All sections, except 109, have fatigue cracking between 3 to 15% of area. All sections, except 104, 111, and 112, have between 175 to 245 m of longitudinal cracking-WP. Also, all sections except 109, 110, 111 and 112, have longitudinal cracking-NWP between 200 to 260 m. No transverse cracking was observed in any section at this site. Sections 101 and 102 had more than 10 mm of rut depth after only 1 year. Also sections 103 108, and 109 have exhibited rutting of about 10 mm. IRI more than 2 m/km was observed on sections 101 and 102 after only 1 year of construction. All sections have IRI greater than 1.4 m/km, while sections 103 and 108 have IRI of 3 and 2 m/km, respectively.

Oklahoma, OK (40)

The performance data was collected for 6 years (1997-2003) for this site. Less than 3% (area) of fatigue cracking was observed in all the sections. No longitudinal cracking-WP occurred on any section at this site. Longitudinal cracking-NWP between 20 m and 120 m was observed in all sections except, section 113, which has no longitudinal cracking-NWP. Transverse cracking less than 4 m was observed in sections

113, 115 and 121. Sections 113, 117 and 122 have more than 10 mm of rut depth, while the other sections have rutting between 4 to 8 mm. Sections 113 through 118 have IRI of about 1.1 m/km whereas, all other sections have less than 1.0 m/km of IRI.

Texas, TX (48)

At this site, performance data was collected for 5 years (1997-2002). Fatigue cracking, less than 1% (area) and longitudinal cracking-WP (between 20 m and 50 m), was observed only in sections 113, 117, 118 and 122. Only sections 112 and 119 have exhibited longitudinal cracking-NWP of 54 m and 77 m, respectively. No transverse cracking was observed in any of the sections. Severe early rutting, between 10 mm to 18 mm, was observed in sections 114, 115, 116, 119, 123 and 124, after 1 year. Rutting in these sections progressed to about 14 mm to 26 mm, by year 4. All other sections have rutting less than 11 mm, by year 4. Sections 115, 116 and 119 have IRI between 1.2 and 1.8 m/km, after only 2 years and all other sections have IRI less than 1.0 m/km.

Virginia, VA (51)

The performance data was collected for 6 years (1995-2002) for this site. More than 25% of fatigue cracking was observed only on sections 103 and 120. Longitudinal cracking-WP and transverse cracking has not occurred on any of the sections. Sections 114 and 120 have exhibited longitudinal cracking-NWP of 40 m and 4 m, respectively. Section 103 has rut depth of 12 mm just after 1 year and this progressed to 21 mm by year 5. All other sections have exhibited rutting ranging between 4 mm and 6 mm. All sections, except 113, have IRI less than 1.3 m/km. Section 113 has an IRI of 1.9 m/km.

Wisconsin, WI (55)

The performance data was collected for only 4 years (1998-2002) at this site. Fatigue cracking less than 5% was observed only on sections 113, 114, and 116. Longitudinal cracking-WP and transverse cracking has not occurred on any of the sections. A wide range (between 90 m and 300 m) of longitudinal cracking-NWP was observed on all the sections. Sections 120 and 121 have rutting of 4.0 mm while all other sections have rutting between 6 to 9 mm. All sections have IRI less than 1.3 m/km.

6.5 SITE-LEVEL ANALYSIS

The site-level analysis deals with each SPS-1 project separately. The main advantage of this analysis is that it eliminates the variability between SPS-1 sites. For each site, the climatic conditions, subgrade type and traffic are the same. Construction conditions, material sources and surveys can also be considered the same for a given SPS-1 project. Two approaches were used: (1) paired comparisons by design factor; and (2) evaluation of the various designs within the project.

As described in Chapter 5, the site-level analyses consists of two types of comparisons: (i) Level-A— In this analysis all designs (0101 through 0112, or 0113 through 0124) at a given site are compared such that only one factor is held common within the sections of each group. For example in level-A analysis, the effects of HMA thickness (4" vs. 7") were considered for all twelve sections within a site by ignoring base type & thickness, and drainage. (ii) Level-B—analysis- In this analyses, most of the factors are 'controlled' for comparisons. In level B analysis, the effect of HMA thickness (4" vs. 7") was compared for only those sections for which all other factors are the same.

The concepts of Performance Index (PI) and the Relative Performance Index (RPI) were used in the site level analysis. PI and RPI were introduced in Chapter 5 of this report. The site-level analysis process is summarized in Figure 6-15. Each analysis was conducted separately for each performance measure. The pavement performance measures considered include:

- Fatigue cracking
- Rutting
- Roughness (IRI)
- Transverse cracking, and
- Longitudinal cracking (WP and NWP)

The PIs and relative performance ratio for the main design factors were calculated at all sites. Because the relative performance is a ratio, it can be used across the sites. The relative performance ratios for a given design factor from all eighteen states were used to test the significance of its effect. A two-level non-parametric (Wilcoxon Signed Ranks) test was done to evaluate the effect of HMA thickness and drainage, and a multiple level non-parametric comparison test was done to evaluate the effect of base type. The p-values from these tests are reported in the discussion of the results. To evaluate the interactive effect of the design factors with climatic zone and subgrade type, the average relative performance ratios within the same climatic zones and for both subgrade types were compared.

The computed PIs and relative performance ratios for all the distresses are summarized and presented in Appendix A4 of reference [7]. The following is a summary of the main findings from each method of analysis, categorized by design factor and performance measure.

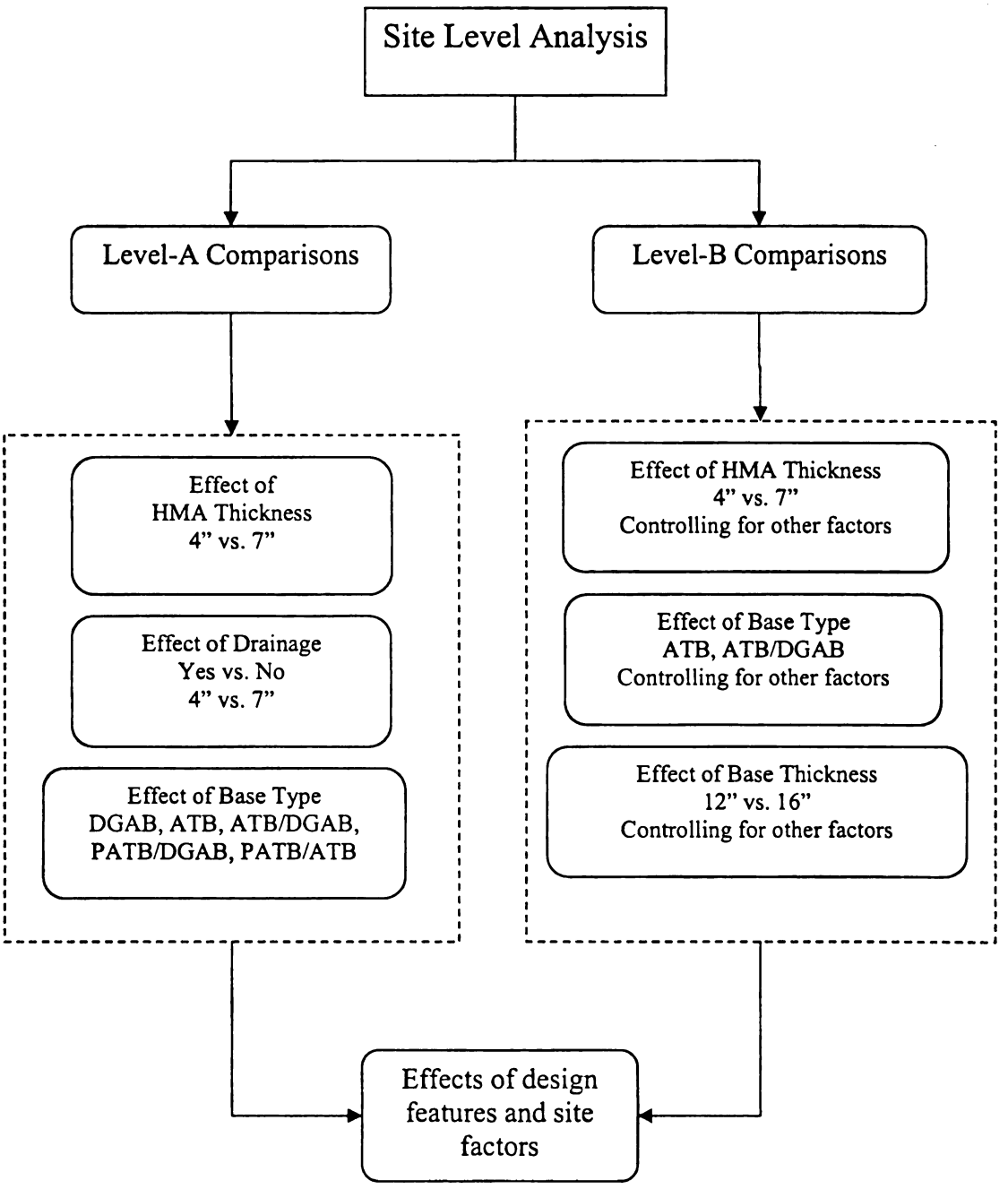


Figure 6-15 Methodology for site level analysis (SPS-1)

6.5.1 Effects of design features on performance – Paired Comparisons at Level-A

A summary of p-values obtained from the non-parametric tests on RPIs (for all 18 sites) from level-A analyses is in Table 6-8. It is important to note that the significance of a factor indicates consistency in its effect across all the sites but not necessarily a significance of its effect on the magnitude of distress.

Table 6-8 Summary of p-values (non-parametric test) for Site Analysis - Level-A

Design Factor	Performance Measures					
	Fatigue cracking	Rutting	Roughness	Transverse cracking	Longitudinal cracking	
					WP	NWP
HMA thickness	0.013	0.408	0.009	0.050	0.311	0.368
Base type	0.033	0.529	0.000	0.079	0.599	0.883
Base thickness	-	-	-	-	-	-
Drainage	0.047	0.056	0.020	0.040	0.035	0.028

The following is a summary of the main findings from paired comparisons at level-A categorized by design factor and performance measures:

HMA Thickness

The effect of HMA thickness is consistent on roughness (IRI), fatigue cracking, and transverse cracking, with 7-inch HMA pavements showing better performance. It is not consistent for longitudinal cracking (WP and NWP)) and rutting (see Table 6-8).

- **Fatigue Cracking**: HMA thickness appears to have a consistent effect on fatigue cracking. Sections with 7-inch HMA thickness have consistently (across sites) performed better than those with 4-inch HMA thickness. Twelve sites show a positive effect, compared to five sites showing a negative effect and one site showing no effect.

The effect of HMA thickness is less seen among the sites located in WF zone. Also, on average, the superior performance of 7- over 4-inch HMA sections can be seen more for sections on coarse-grained subgrade than for those on fine-grained subgrade.

- **Rutting**: The effect of HMA thickness on rutting is not consistent. Nine sites show a positive (lesser rutting in sections with 7-inch HMA surface thickness) effect and the other nine show a negative effect, which shows no definitive trend of the effect across sites.
- **Roughness (IRI)**: The effect of HMA thickness on IRI is consistent across sites. On average, sections with 7-inch HMA surface thickness have consistently performed better than those with 4-inch HMA thickness. This trend was observed in thirteen out of eighteen sites and two sites showed no effect.
- **Transverse Cracking**: The effect of HMA thickness on transverse cracking is consistent across sites. Sections with 7-inch HMA thickness have consistently performed better than those with 4-inch HMA thickness. Eight out of eighteen sites showed less transverse cracking for "7 inch" sections; seven sites showed no transverse cracking, while three sites showed more transverse cracking for "7 inch" HMA sections.

In terms of climatic zones, positive HMA thickness effect can be seen in all zones except for WF. This could be attributed to the severe environmental conditions in WF zone where even thicker HMA may not be able to inhibit cracking. Averaging over all climatic zones, the HMA thickness effect is essentially the same for both subgrade types.

- **Longitudinal Cracking-WP**: The effect of HMA thickness on longitudinal cracking-WP is not consistent. Nine sites show a positive effect and the other nine sites show a negative effect, which shows no definitive trend of the effect across sites.
- **Longitudinal Cracking-NWP**: HMA thickness seems to have no consistent effect on longitudinal cracking-NWP. Nine sites show a positive effect and the eight sites show a negative effect, indicating no definitive trend of the effect across sites.

Effect of Base Type

The effect of base type is consistent across sites on fatigue cracking, roughness, and transverse cracking, with sections on DGAB showing the worst performance and sections on ATB+PATB showing the best performance.

- **Fatigue cracking**: Base type has a consistent effect, across sites, on fatigue cracking. Sections with DGAB have shown the most amount of cracking while sections with ATB+PATB have shown least amount of cracking. The order of performance from best to worst is as follows: (1) PATB+ATB, (2) ATB, (3) ATB+DGAB (4) PATB+DGAB and (5) DGAB.
- **Rutting**: The effect of base type is not consistent. However, on average, sections with PATB+ATB have slightly lesser rutting compared to sections with DGAB.
- **Roughness (IRI)**: The effect of base type on IRI is consistent across sites. Sections with DGAB have consistently shown the highest IRI-values while sections with ATB+PATB have shown the lowest values. The order of performance from best to worst is as follows: (1) PATB+ATB, (2) ATB+DGAB, (3) ATB, (4) PATB+DGAB, and (5) DGAB.

- **Transverse cracking**: The effect of base type appears to be marginally consistent across sites (p=0.079). Sections with PATB+ATB have somewhat lesser cracking than sections with DGAB.
- **Longitudinal cracking-WP**: The effect of base type is not consistent across sites. Nonetheless, on average, the permeable bases appear to be performing better. The worst performance was shown by the sections with DGAB.
- **Longitudinal cracking-NWP**: The effect of base type is not consistent across sites. However, on average, the best performing bases are the permeable bases- PATB+ATB and PATB+DGAB, and the worst performing base is DGAB.

Effect of Drainage

The effect of drainage on all performance measures is consistent across sites, with drained pavements showing better performance than un-drained pavements (see Table 6-8).

- **Fatigue cracking**: Drainage has a consistent effect across sites on fatigue cracking. Sections with drainage have performed better than those without drainage. Twelve sites show a positive effect (better performance of drained sections), compared to five sites that show a negative effect. Averaging over all climatic zones, the effect of drainage can be seen better for sections with fine-grained subgrade as opposed to coarse-grained subgrade.
- **Rutting**: The effect of drainage on rutting is consistent. Sections with drainage have consistently performed better than those without drainage. Twelve sites show a positive effect, compared to four sites showing a negative effect and two sites showing no effect.

- **Roughness**: Drainage has a consistent effect on roughness. Sections with drainage have consistently (across sites) performed better than those without drainage. In fifteen out of eighteen sites drained sections have shown a better performance, while reverse trend was found in only three sites. This effect is less seen among sections in DF zone and could be attributed to the fact that drainage is not as important in this zone.
- **Transverse cracking**: Drainage has somewhat consistent effect on transverse cracking. Sections with drainage have performed better than those without drainage in most of the sites. Seven sites show a positive effect, while no transverse cracking occurred in seven of the sites. Averaging over all climatic zones, the effect of drainage is better seen for sections built on fine-grained subgrade than for sections built on coarse-grained subgrade.
- **Longitudinal cracking-WP**: The effect of drainage on longitudinal cracking-WP is consistent across sites. On average, sections with drainage have consistently performed better than those without drainage. Eleven sites show a positive effect, compared to three sites showing a negative effect and four sites showing no effect. Moreover, the effect is more prominent for sections on fine-grained subgrades as opposed to those on coarse-grained subgrades.
- **Longitudinal cracking-NWP**: Drainage has a consistent effect on longitudinal cracking-NWP. On average, sections with drainage have consistently performed better than those without drainage. Thirteen sites show a positive effect, compared to three sites showing a negative effect and two sites showing no effect.

6.5.2 *Effects of design features – Paired Comparisons at Level-B*

As explained in Chapter 4, level-B comparisons are more "controlled" compared to level-A comparisons. To study the consistency of the effect of HMA thickness across sites, nonparametric testing was performed on relative performance corresponding to 4-inch and 7-inch HMA thicknesses, within each base type. Similarly, to investigate the effect of base thickness, nonparametric testing was performed on relative performance corresponding to 12-inch and 16-inch base thicknesses, within PATB/DGAB and ATB/PATB. Also, nonparametric testing was performed on relative performance corresponding to ATB and ATB/DGAB, within 8-inch and 12-inch base thicknesses. For each of these effects, the corresponding p-values are presented in Table 6-9. A brief summary of results from the level-B comparisons follows.

Table 6-9 Summary of p-values (non-parametric test) for Site Analysis - Level-B

Design Factor	Performance Measures					
	Fatigue cracking	Rutting	Roughness	Transverse cracking	Longitudinal cracking	
					WP	NWP
HMA thickness	0.041	0.080	0.005	0.010	0.203	0.110
Base type	0.969	0.214	0.150	0.552	0.929	0.551
Base thickness	0.307	0.022	0.046	0.933	0.499	0.387

HMA Thickness

The effect of HMA thickness can be examined for sections with different base types. Among sections with DGAB, it was observed that HMA thickness has a positive effect of on fatigue cracking, transverse cracking, and roughness (IRI). Also, on average, a positive effect of HMA thickness was observed on sections with ATB for fatigue cracking, but it is not consistent. The same effect was observed for rutting. This suggests

that increasing HMA thickness is more effective in pavement sections with unbound bases as compared to those with treated bases.

Base Type

The effect of all five base types cannot be evaluated effectively for Level-B comparisons since the only base types that can be compared are ATB and ATB+DGAB. Nonetheless, the results show that the difference in performance between these two base types is not statistically significant.

Base Thickness

The effect of base thickness can be better seen when using Level-B comparisons, mainly because it is a more secondary effect relative to HMA layer thickness and base type (treated versus untreated). Therefore it is more useful to look at this effect for two types of (permeable) bases: (1) DGAB and (2) ATB.

The effect of base thickness was found to be consistent for rutting and roughness in pavement sections with DGAB. The effect of base thickness is not consistent for fatigue cracking and longitudinal cracking-NWP. However, on average, thicker (16-inch) bases have shown better performance than thinner (12-inch) bases. Finally, the effect of base thickness is not consistent for transverse cracking and longitudinal cracking-WP.

6.6 OVERALL ANALYSIS

The results obtained from statistical analyses performed on the SPS-1 data are presented in this section. Both the performance and response variables were analyzed to study the effects of various design and site-factors on the pavement sections. Analyses were performed combining all data and is referred to as 'Overall' analyses. Analyses were also conducted in each climatic zone combining data from all sections within a zone as per the recommendation of the project panel. Linear Discriminant Analysis (LDA), Binary Logistic Regression (BLR), and Analysis of Variance (ANOVA) are the statistical methods that were employed for analyses. Before presenting the results from statistical analyses, the extent of distresses that occurred on the test sections is discussed.

6.6.1 Extent of Distress by Experimental Factor

This section discusses the effect of the key experimental factors on performance through the relationship between the magnitude and relative occurrence of the observed distresses. Figures 6-16 through 6-21 show the percentage of test sections that have exceeded various levels of distress for the key performance measures, categorized by experimental (design and site) factors. Note that the effect of climatic zone is only shown for the wet regions because of the limited number of sites in the dry regions (only four). The following is a brief interpretation of these figures:

Fatigue cracking: Figure 6-16 indicates that about 70% of all test sections have shown some fatigue cracking, with about 10% of all test sections showing 20% or higher cracking by area. The effects of specific design and site factors are discussed below.

a) HMA Thickness: About 75% of sections with thin HMA surface layer have shown some fatigue cracking as compared to about 65% of sections with thick HMA surface layer; the effect of HMA thickness tends to be larger for higher levels of fatigue cracking.

b) Base Type: The difference in the percentage of test sections that have shown fatigue cracking between those with unbound (DGAB) and those with treated (ATB) bases is highest among all experimental factors (about 15%), with sections built on DGAB bases showing the highest percentages.

c) Base Thickness: The effect of base thickness on fatigue cracking was found to be insignificant.

d) Drainage: The effect of drainage in terms of higher percentage of test sections showing fatigue cracking is more pronounced at the lower levels of fatigue; the effect becomes insignificant at the later stages of fatigue. This could mean that drainage is more effective in the early life of the pavement, and becomes less effective later in the pavement life. Also, the effect of drainage is slightly more visible for fine-grained than for coarse-grained subgrade soils [Figure 6-16 (d)].

e) Climatic Zone: There are consistently more sections in wet-freeze (WF) than wet-no-freeze (WNF) climate that have shown fatigue cracking exceeding various levels, with about 10% more sections in WF than in WNF climate.

f) Subgrade Type: There are consistently about 15% more sections built on fine-grained than coarse-grained subgrade soils that have shown fatigue cracking exceeding various levels, and the effect of subgrade soils tends to be larger for higher levels of fatigue cracking.

Rutting: Figure 6-17 indicates that about 60% of all test sections have shown rut depths higher than 0.25 inch (6.25 mm), and about 20% of all test sections showing rut depths higher than 0.5 inch (12.5 mm). The effects of specific design and site factors are discussed below.

a) HMA Thickness: The effect of HMA thickness on rutting was found to be negligible.

b) Base Type: There are about 10% to 15% more sections with unbound (DGAB) bases that have rut depths greater than 7.5 mm than those with treated (ATB) bases. This difference is relatively constant at higher rut depths.

c) Base Thickness: There is a slight effect of base thickness for sections that have rut depths that are less than 7.5 mm, with about 5% more sections with thinner (8 inch) bases than those with thicker (16 inch) bases. The effect becomes less apparent for rut depths greater than 7.5 mm.

d) Drainage: There are consistently about 5% more sections without drainage than with drainage that have exceeded various rut depth levels. Also, the effect of drainage is slightly more visible at the higher rut depths and for fine-grained subgrade soils [Figure 6-17 (d)].

e) Climatic zone: The effect of climatic zones (within wet regions) on rutting appears to be more significant at rut depth higher than 7.5 mm, with about 10% more sections in wet-freeze than in wet-no-freeze climate exceeding various rut depths.

f) Subgrade Type: There are consistently about 10% more sections built on fine-grained than coarse-grained subgrade soils that exceed various rut depths.

Roughness: Figure 6-18 indicates that about 60% of all test sections have shown IRI values higher than 1 m/km, with about 20% of all test sections showing IRI values higher than 1.4 m/km. The effects of specific design and site factors are discussed below.

a) HMA Thickness: The percentage of test sections with thin (4 inch) HMA surface layer that have exceeded an IRI of 1.2 m/km is about 40% as compared to about 20% for test sections with thick (7 inch) HMA surface layer. The percentage of test sections with thin (4 inch) HMA surface layer exceeding higher IRI levels is 5% to 10% more than that of test sections with thick (7 inch) HMA surface layer.

b) Base Type: The percentage of test sections with unbound aggregate base (DGAB) that have exceeded an IRI of 1.2 m/km is about 40% as compared to about 20% for test sections with asphalt treated base (ATB). The percentage of test sections with a 8-inch base exceeding higher IRI levels is about 10% to 15% more than that of test sections with an 16-inch base.

c) Base Thickness: The effect of base thickness on roughness is more pronounced than for other performance measures, showing a percentage of test sections with a DGAB base exceeding 1.2 m/km and higher IRI levels that is about 10% to 15% more than that of test sections with an ATB base.

d) Drainage: There are consistently about 5% more sections without drainage than with drainage that have exceeded various IRI levels.

e) Climatic zone: The effect of climatic zones (within wet regions) on roughness appears to be the most significant, with about 20% to 30% more sections in wet-freeze than in wet-no-freeze climate exceeding 1.2m/km and higher IRI levels.

f) Subgrade Type: The effect of subgrade type on roughness is more pronounced than for other performance measures, with about 15% to 30% more sections on fine-grained than on coarse-grained subgrade exceeding 1.2m/km and higher IRI levels.

Transverse cracking: Figure 6-19 indicates that about 40% of all test sections have shown some transverse cracking, with about 10% of all test sections showing 20m or higher length of transverse cracking. The effects of specific design and site factors are discussed below.

a) HMA Thickness: Only a slight effect of HMA thickness was found on transverse cracking.

b) Base Type: Base type appears to be a significant factor affecting transverse cracking. About 10% to 15% more test sections with unbound aggregate base (DGAB) than those built with an asphalt treated base (ATB) at various levels of transverse cracking.

c) Base Thickness: Only a slight effect of base thickness was observed.

d) Drainage: Only a slight effect of drainage on transverse cracking was observed.

e) Climatic Zone: Climate seems to be a significant factor affecting transverse cracking. There are about 15% to 20% more test sections in WF zone than those built in WNF zone at various levels of transverse cracking.

f) Subgrade Type: Subgrade soil type seems to have some effect on transverse cracking, in that, slightly higher proportion of sections built on fine-grained soils have shown cracking compared to those built on coarse-grained soil.

Longitudinal cracking: Figures 6-20 and 6-21 indicate that about 50% of all test sections have shown some longitudinal cracking-WP and about 75% of all test sections have shown some longitudinal cracking-NWP. The effects of experimental factors are discussed below.

a) HMA thickness: HMA thickness appears have a negligible effect on longitudinal cracking.

b) Base Type: There seems to be a slight effect of base type on longitudinal cracking, in that, sections with ATB has shown lesser cracking than those with DGAB.

c) Base Thickness: Only a slight effect of base thickness was observed. Sections with 16-inch base thickness have slightly less occurrence of cracking than those with 8-inch base.

d) Drainage: There appears to be some positive effect of drainage on lower levels of longitudinal cracking-WP. However this effect was observed to be negligible for higher levels of cracking.

e) Climatic Zone: The effect of climatic zone (within wet regions) on longitudinal cracking is more pronounced than other effects, especially for longitudinal cracking-NWP. About 10% to 20% more test sections in wet-freeze than in wet-no-freeze climate exceed 100 m or more of longitudinal cracking-WP, and about 20% to 35% more test sections in wet-freeze than in wet-no-freeze climate exceed 100 m or more of longitudinal cracking-NWP.

f) Subgrade Type: Only a slight positive effect of subgrade type was observed for longitudinal cracking-WP.

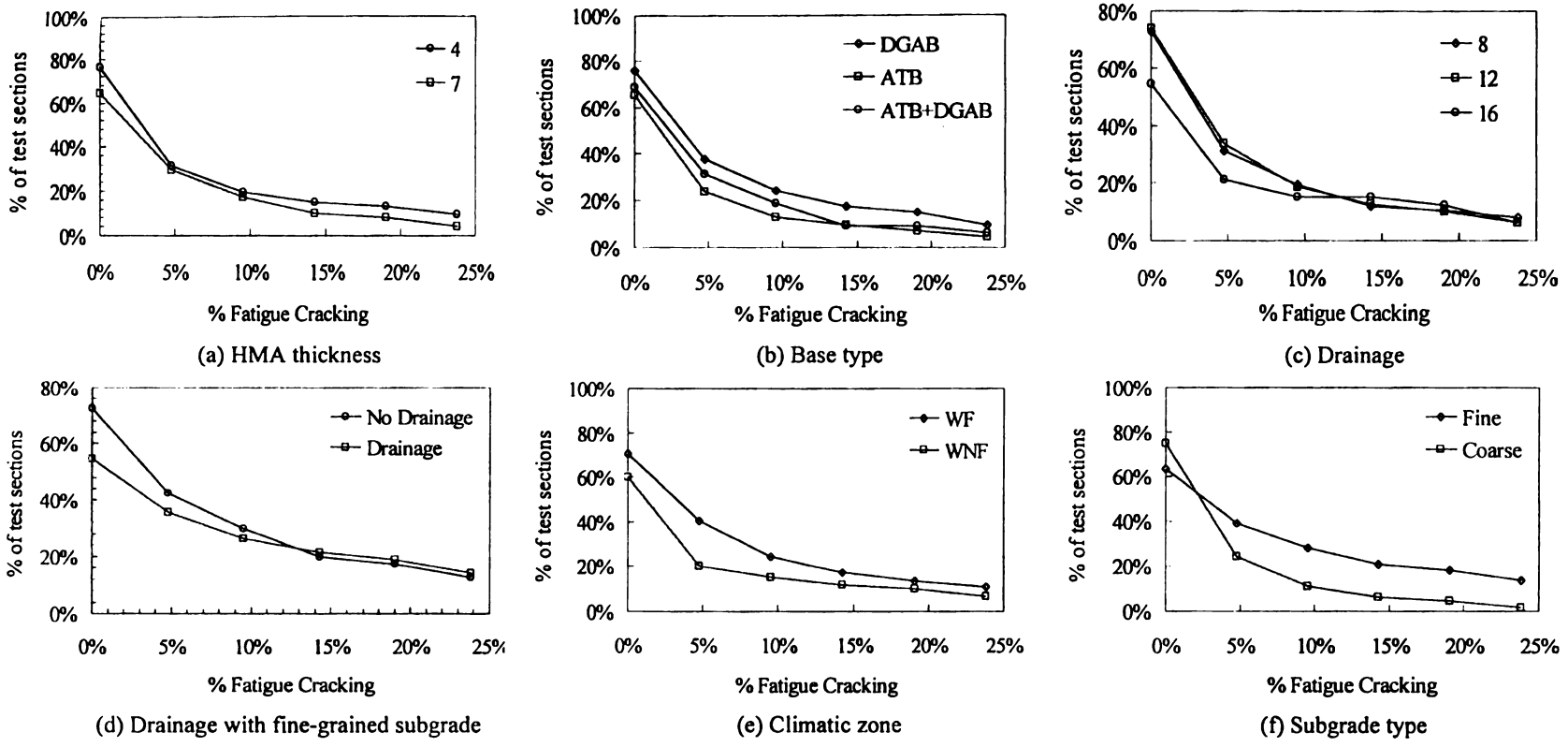
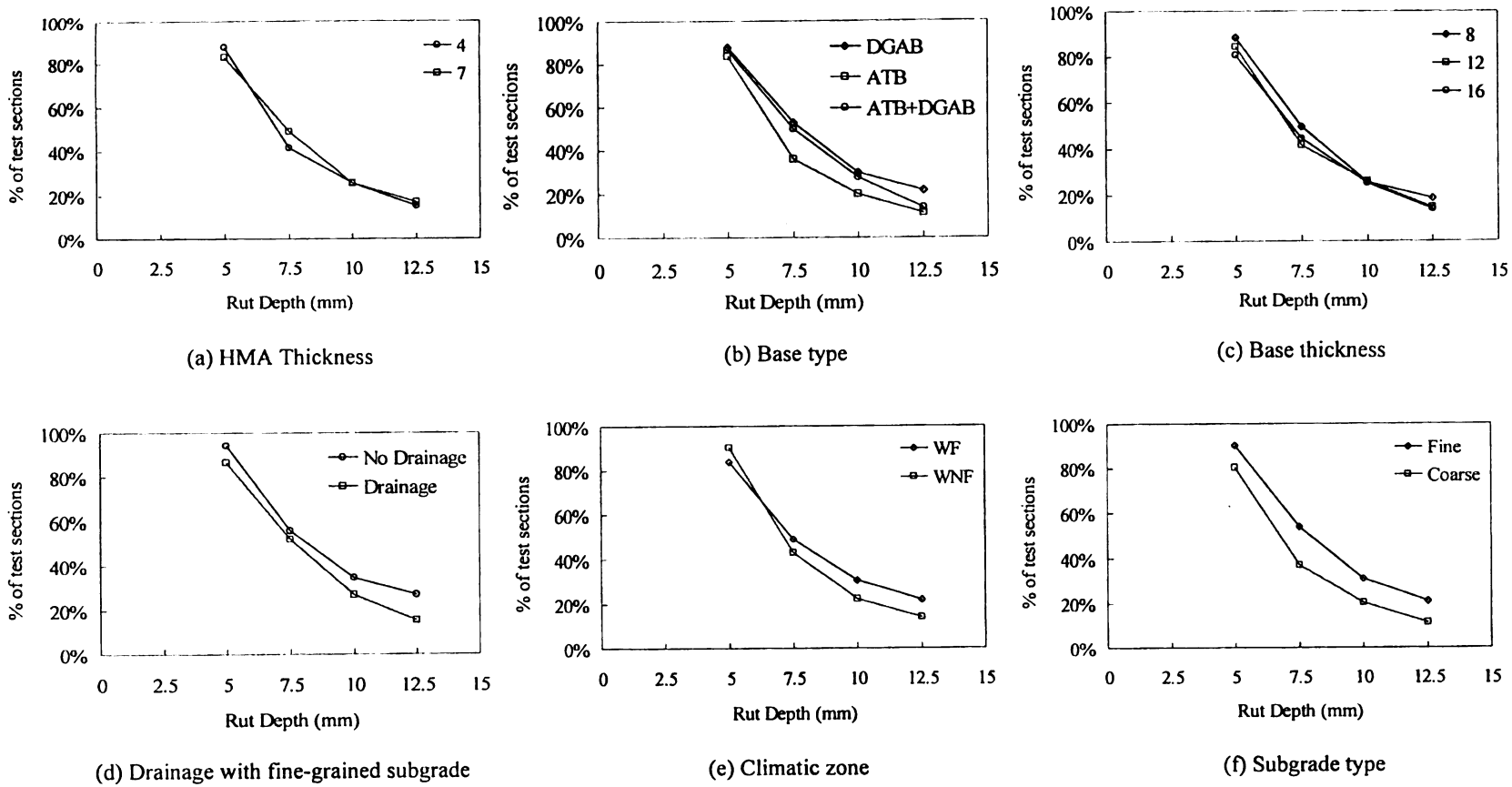


Figure 6-16 Effect of experimental factors on fatigue cracking



(a) HMA Thickness

(b) Base type

(c) Base thickness

(d) Drainage with fine-grained subgrade

(e) Climatic zone

(f) Subgrade type

Figure 6-17 Effect of experimental factors on rutting

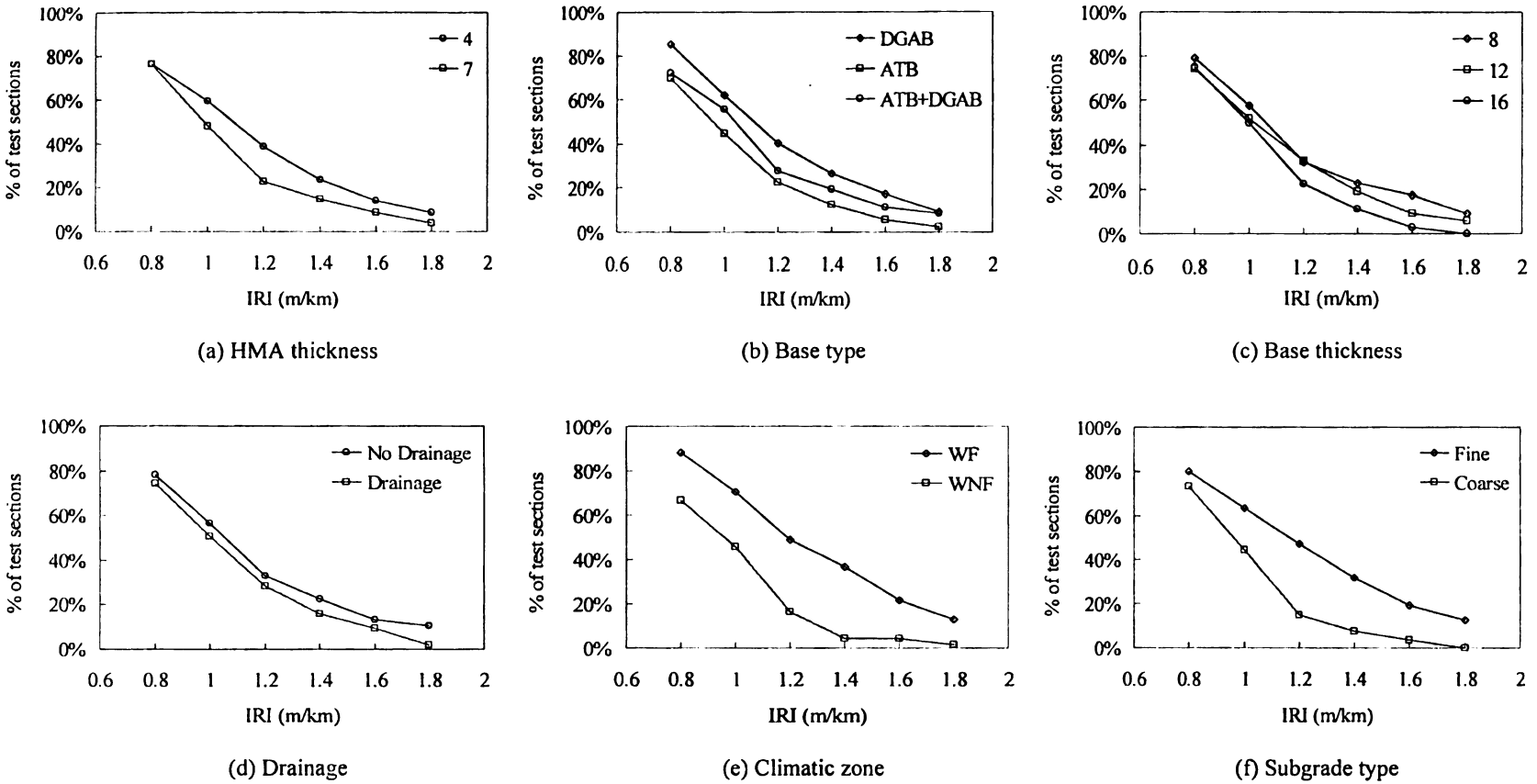


Figure 6-18 Effect of experimental factors on roughness

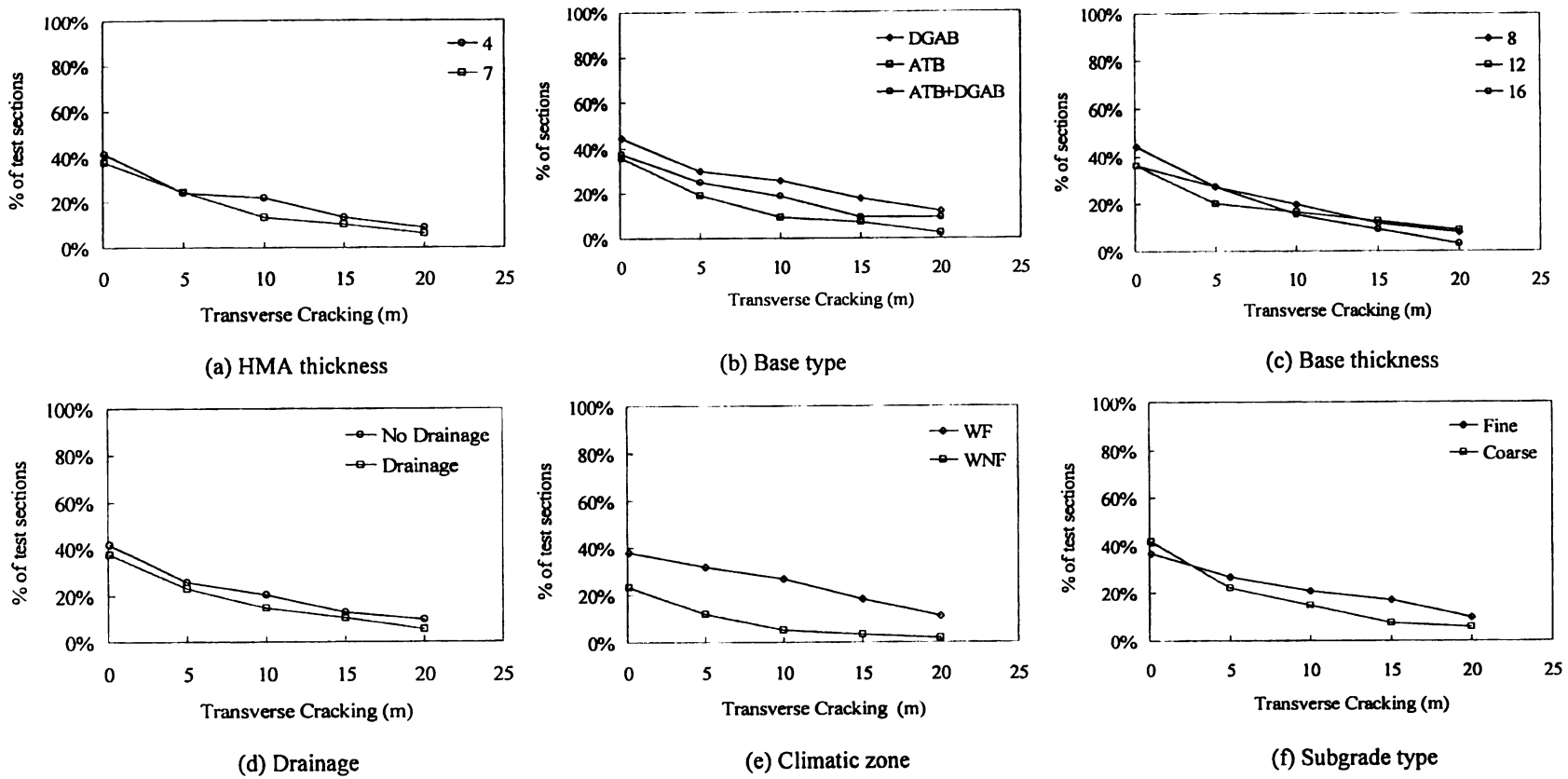
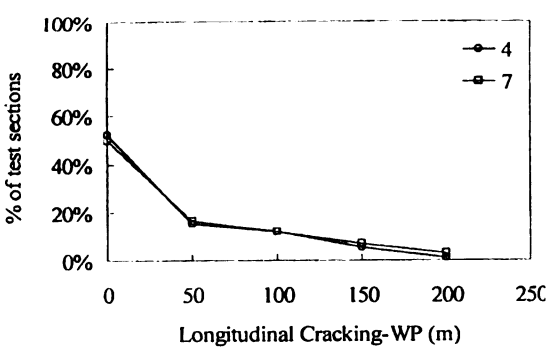
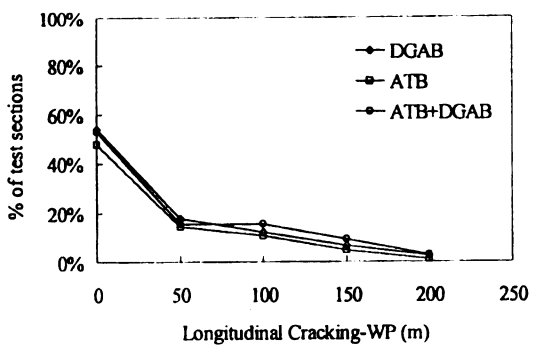


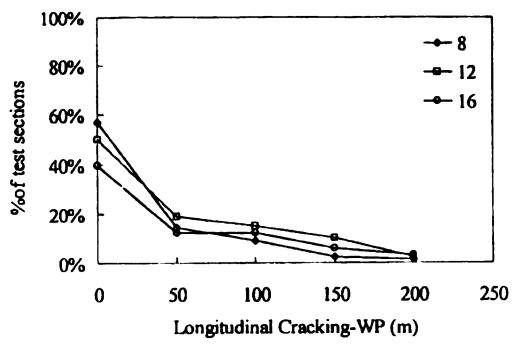
Figure 6-19 Effect of experimental factors on transverse cracking



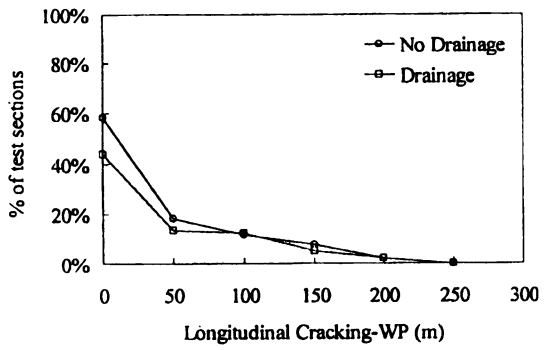
(a) HMA thickness



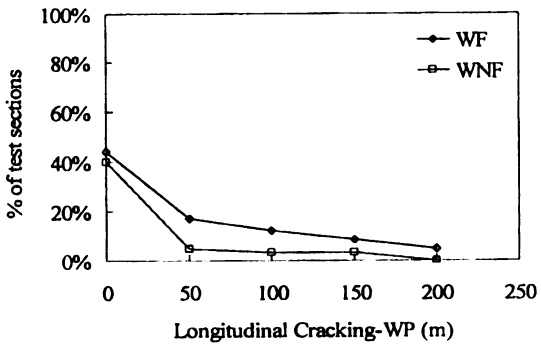
(b) Base type



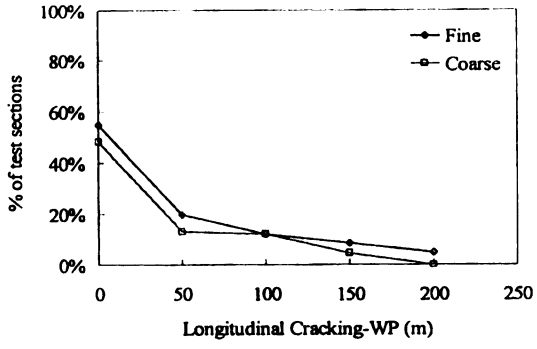
(c) Base thickness



(d) Drainage

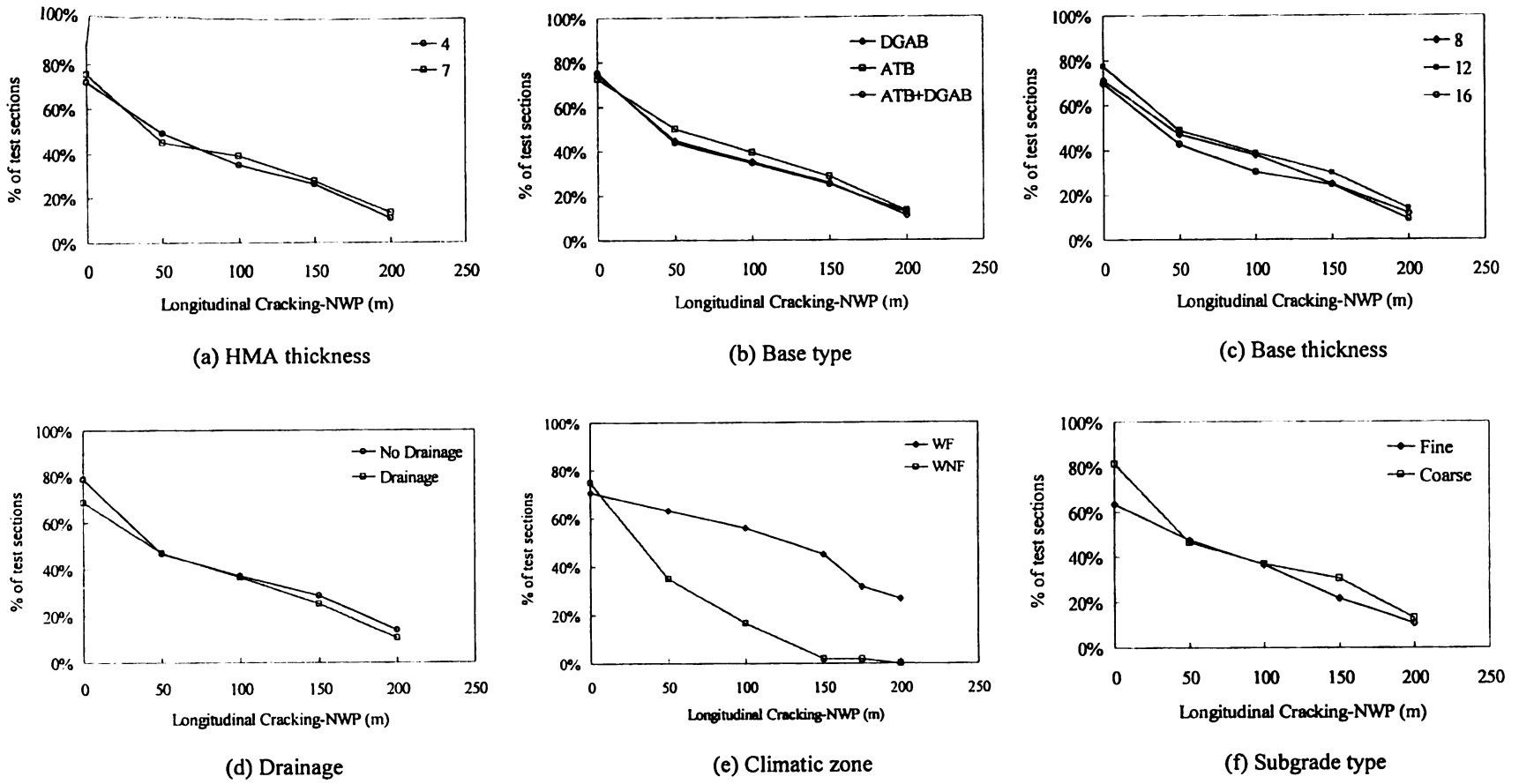


(e) Climatic zone



(f) Subgrade type

Figure 6-20 Effect of site factors on longitudinal cracking-WP



(a) HMA thickness

(b) Base type

(c) Base thickness

(d) Drainage

(e) Climatic zone

(f) Subgrade type

Figure 6-21 Effect of site factors on longitudinal cracking-NWP

6.6.2 Frequency-based methods

Two frequency-based methods were used- Linear Discriminant Analysis and Binary Logistic Regression (details in Chapter 4). The results from these analyses are as follows:

Discriminant Analysis

In this analysis, two mutually exclusive groups were defined as follows:

- Alligator, transverse and longitudinal cracking: Cracked versus non-cracked.
- Rutting: Rut depth < 7 mm versus rut depth > 7mm
- Roughness: IRI < 1.4 m/km versus IRI>1.4 m/km

This analysis was intended to identify the experimental factors which help in discriminating the cracked versus non-cracked pavement sections. As most of the pavements in the SPS-1 experiment have not shown a high level of distress, this analysis will help in finding the significant design and site factors contributing to occurrence of distress. In order to include the effect of traffic and pavement age, these were considered as covariate in this analysis. Table 6-10 summarizes the results of network level analysis.

The performance measures were defined as dichotomous variables and all design and site factors were used as independent variables. The following summarizes the results from this analysis:

- *Fatigue cracking*: The effects of drainage condition and base type were found to discriminate between cracked and non-cracked sections. Test sections without drainage built on unbound (DGAB) bases are more likely to crack.
- *Rutting*: The effects of drainage condition, subgrade soil, base thickness were found to discriminate between sections having rut depths greater or less than 7mm. Test

sections without drainage with thinner bases and built on fine-grained subgrade soils in wet zones are more likely to exhibit severe rutting.

- *Roughness*: The effects of climatic zone, subgrade soil and base thickness were found to discriminate between sections having IRI greater or less than 1.4 m/km. Test sections with thinner bases built on fine-grained subgrade soil and in wet freeze zone are more likely to exhibit higher roughness. Sections with higher initial roughness are more likely to become rougher with age.
- *Transverse cracking*: The effect of base type to a lesser degree was found to discriminate between cracked and non-cracked sections. Test sections in wet freeze zone with unbound (DGAB) bases are more likely to crack. Also, older sections are more likely to crack.
- *Longitudinal cracking*: The effect of climatic zone was found to discriminate between cracked and non-cracked sections (inside the wheel path). Test sections built in wet-freeze zone are more likely to crack outside the wheel path. Also, older sections are more likely to crack (in and outside the wheel path).

Table 6-10 Summary of p-values from LDA for determining the effect of experimental factors on pavement performance measures

Design Factor	Performance Measures					
	Fatigue cracking	Rutting	Roughness	Transverse cracking	Longitudinal cracking	
					WP	NWP
HMA thickness	0.39	1.000	0.370	0.320	0.88	0.310
Base type	0.098	0.517	0.250	0.139	0.77	0.690
Base thickness	0.92	0.077	0.076	0.736	0.19	0.421
Drainage	0.09	0.056	0.370	1.000	0.47	0.310
Subgrade type	0.045	0.011	0.000	0.177	0.184	0.001
Climatic Zone	0.392	0.578	0.000	0.417	0.002	1.000

Logistic Regression

The binary logistic regression model was used to model the probability of occurrence for the various performance measures. This method requires fewer assumptions than discriminant analysis and even when the assumptions required for discriminant analysis are not satisfied, it performs well. The overall models for each of the performance measures were found to be significant. The results using the maximum likelihood method are summarized in Tables 6-11 and 6-12.

- Fatigue cracking:

  HMA Thickness— Thin (4-inch) pavement sections have a slightly higher probability of cracking than thick (7-inch) sections when all other variables are held constant.

  Base Type— Pavement sections with unbound (DGAB) base have a significantly higher probability of cracking than those with bound (ATB/DGAB) bases.

  Drainage— Pavement sections with no drainage have a higher probability of cracking than those sections with drainage.

  Climatic Zone— Pavement sections in freeze zones have a significantly higher probability of cracking than those in the no-freeze environments.

- Rutting:

  Drainage— Pavement sections with no drainage have a slightly higher probability of rutting (rut depth > 7 mm) than those sections with drainage.

Subgrade Type— Pavement sections built on fine-grained subgrade soils have a significantly higher probability of rutting than those sections built on coarse-grained subgrade soils.

Climatic Zone— Pavement sections in WNF zones have a significantly higher probability of rutting than those in the WF environments.

- Roughness:

  HMA Thickness— Thin (4 inch) pavement sections have a higher probability of showing higher roughness (IRI > 1.4 m/km) than thick (7 inch) sections.

  Base Type— Pavement sections with unbound (DGAB) base have a significantly higher probability of showing higher roughness than those with bound (ATB/DGAB) bases.

  Base Thickness— Pavement sections with thin bases have a significantly higher probability of showing higher roughness than those with thick bases.

  Subgrade Type— Pavement sections built on fine-grained subgrade soils have a significantly higher probability of showing higher roughness than those sections built on coarse-grained subgrade soils.

  Climatic Zone— Pavement sections built in wet-freeze zone have a significantly higher probability of showing higher roughness than those sections built in wet-no-freeze zone.

Table 6-11 Summary of p-values from BLR for determining the effect of experimental factors on pavement performance measures (Wet zones)

Design Factor	Performance Measures					
	Fatigue cracking	Rutting	Roughness	Transverse cracking	Longitudinal cracking	
					WP	NWP
HMA thickness	0.160 (1.8)	0.833 (1.1)	0.068 (3.7)	0.493 (0.6)	0.360 (1.5)	0.31 (0.7)
Base type	0.024 (2.4)	0.972 (1.0)	0.006 (33)	0.711 (1.2)	0.437 (1.7)	0.396 (1.7)
Base thickness	0.420 (1.7)	0.212 (2.5)	0.038 (14)	0.632 (1.3)	0.410 (1.8)	0.733 (0.8)
Drainage	0.045 (2.8)	0.124 (2.2)	0.278 (2.4)	0.316 (2.7)	0.40 (0.6)	0.837 (0.9)
Subgrade type	0.960 (1.0)	0.015 (3.4)	0.000 (571)	0.345 (0.005)	0.000 (22)	0.009 (0.34)
Climatic Zone	0.088 (2.2)	0.098 (.42)	0.000 (420)	0.316 (10)	0.976 (1.0)	0.73 (0.862)

Note: The values in parenthesis are odds ratios

Table 6-12 Summary of p-values from BLR for determining the effect of experimental factors on pavement performance measures (All zones)

Design Factor	Performance Measures					
	Fatigue cracking	Rutting	Roughness	Transverse cracking	Longitudinal cracking	
					WP	NWP
HMA thickness	0.19 (1.5)	0.98 (1.0)	0.068 (3.7)	0.224 (0.5)	0.527 (1.25)	0.34 (0.7)
Base type	0.013 (2.2)	0.98 (1.0)	0.006 (33)	0.043 (3.2)	0.55 (1.4)	0.22 (2.0)
Base thickness	0.81(0.8)	0.4 (2.3)	0.038 (14)	0.974 (1.0)	0.376 (1.6)	0.77 (1.2)
Drainage	0.009 (3.1)	0.22 (1.7)	0.278 (2.4)	0.473 (1.6)	0.700 (1.2)	0.297 (1.5)
Subgrade type	0.073 (0.5)	0.87 (1.1)	0.000 (571)	0.006 (0.001)	0.076 (2.2)	0.019 (0.42)
Climatic Zone WF-DNF WNF-DNF DF-DNF WF-WNF	0.000 0.000 (12) 0.002 (9) 0.000 (23) (1.4)	0.747 0.72 (0.83) 0.611(1.4) (0.6)	0.001 - - - (420)	0.019 - - - (17)	0.254 - - - (0.76)	0.002 0.003 (5.1) 0.003 (6.4) 0.000 (27) (0.8)

Note: Very high value of odds ratio is caused by the un-balanced data or too few sections in one of the categories.

- Transverse cracking:

  Climatic Zone— Pavement sections built in wet-freeze zone have a higher probability of cracking than those sections built in wet-no-freeze zone.

  Also, older pavement sections have a significantly higher probability of cracking.

- Longitudinal cracking:

  Subgrade Type— Pavement sections built on fine-grained subgrade soils have a higher probability of longitudinal cracking in the wheel path than those sections built on coarse-grained subgrade soils.

  Climatic Zone— Pavement sections built in freeze zone have a significantly higher probability of cracking (outside the wheel path) than those sections built in no-freeze zone.

  Also, older pavement sections have a significantly higher probability of cracking.

6.6.3 Analysis of Variance

Several analyses of variance (ANOVA) were conducted for each of the performance measures and response indicators. The first ANOVA was targeted at determining the significance of only the main structural design factors considered in the experiment. This was achieved by blocking the site factor (to neutralize the effects of subgrade type, climatic conditions, traffic, age, and construction variability) as well as accounting for the variability in target layer thicknesses. The main structural design factors included in the ANOVA are listed below:

- HMA thickness (4-inch versus 7-inch)
- Base type (DGAB, ATB or DGAB+ATB)
- Base thickness (8 inch, 12 inch or 16 inch)
- Drainage condition (with versus without Permeable ATB)

To meet the assumptions of ANOVA, the dependent variables (performance measures) had to be transformed using the natural logarithm. This was particularly relevant for all cracking distresses because of the large number of zeroes in those populations. A negative consequence from this is that the number of sections used in the analysis is reduced.

6.6.3.1 Effect of Design Factors on Pavement Performance

The results from this analysis are summarized in Table 5-13 and indicate that the most significant design factor is the base type, which has a significant effect, statistically as well as operationally, on all performance measures. The ΔIRI (which is the change in IRI between initial and latest value) is also significantly affected by base thickness. The initial roughness is significantly affected by all the design factors except for drainage.

Also, the effect of drainage condition on rutting, and the effect of base thickness on longitudinal cracking-NWP and change in roughness are statistically significant.

For investigating the mean difference between the levels of design factors, the marginal means (predicted cell means from the model) were transformed back to the original scale of the distress. These conversions were necessary in order to find out the practical/operational mean difference. The marginal means were back transformed using the properties of lognormal distribution. A random variable X is considered to have a lognormal distribution if Y=ln (X) has a normal probability distribution, where ln (X) is the natural logarithm to the base e. Equations (5-1) and (5-2) are used to calculate the mean and variance of a random variable X.

$$\mu_x = \exp\left(\mu_y + \frac{1}{2}\sigma_y{}^2\right) \tag{5-1}$$

$$\sigma_x{}^2 = \mu_x{}^2\left[\exp\left(\sigma_y{}^2\right) - 1\right] \tag{5-2}$$

Where: μ_y and σ_y^2 are the mean and the variance of lognormal distribution.

The marginal means of performance measures (for which natural logarithmic transformation was necessary to meet the ANOVA assumptions) were estimated by using equation 5-1. The mean squared error (MSE) was considered as the "best" estimate of the variance for lognormal distribution in all analyses. Table 6-14 shows the back transformed marginal means for all levels of design factors in the SPS-1 Experiment. The following discussion summarizes the effect of key design factors on performance:

- **Effect of base type**: The effect of base type was found to be significant for all performance measures except for rutting. Pavement sections with dense-graded aggregate bases (DGAB) have shown the worst performance for all distresses while those with asphalt treated bases (ATB) have shown the best performance. Sections built with DGAB have shown significantly (operationally and statistically) higher fatigue cracking compared to those built with ATB. On average higher rutting was observed on pavement sections built with DGAB than those constructed with ATB. In the case of other distresses (change in roughness, transverse cracking, and longitudinal cracking) the difference in the performance of sections built on DGAB and sections built on ATB were only found to be statistically significant (i.e., they are not of operational significance at this point in time).

- **Effect of HMA thickness**: In general, thin [4-inch (102 mm)] pavement sections were built rougher than thick [7-inch (178 mm)] pavements. On an average, thin pavements [4-inch (102 mm)] have shown slightly higher fatigue cracking and rutting than thick [7-inch (178 mm)] pavements. However, this effect was found to be of marginal statistical significant.

- **Effect of base thickness**: Sections with thicker bases [12-inch (305 mm) and 16-inch (406 mm)] were built smoother compared to those with thinner base [8-inch (203 mm)]. Also, sections with thinner base [8-inch (203 mm)] have shown more change in roughness than those with thicker base [12-inch (305 mm) and 16-inch (406 mm)]. However, this change in roughness is not of practical significance. On an

average, pavement sections with 8-inch (203 mm) base have shown more rut depths than those with 12-inch (305 mm) and 16-inch (406 mm) thick bases. However, this effect is not of practical significance. More longitudinal cracking-NWP occurred in sections built with 8-inch (203 mm) base compared to those with 16-inch (406 mm) base.

- **Effect of drainage condition**: On average, pavement sections with drainage have shown slightly lower rutting than those without drainage; however this difference in performance is not statistically significant.

Table 6-13 Summary of p-values from ANOVA for determining the effect of main design factors on pavement performance measures—Overall

Design Factor	Performance Measures						
	Fatigue cracking	Rut[1] depth	IRI		Transverse cracking	Longitudinal cracking	
			ΔIRI	IRI_0		WP	NWP
HMA Thickness	0.163	0.074	0.870	0.006	0.758	0.737	0.787
Base Type	0.000*	0.510	0.004	0.000	0.016	0.079	0.031
Base Thickness	0.951	0.080	0.027	0.028	0.697	0.488	0.008*
Drainage	0.347	0.250	0.293	0.160	0.544	0.645	0.874
Site (blocked)	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	R^2=0.712 N=133	R^2=0.55 N=159	R^2=0.624 N=163	R^2=0.603 N=212	R^2=0.630 N=75	R^2=0.72 N=97	R^2=0.770 N=140

Note: The model considered for this analysis only has main effects for all design factors.
* Also shows operational/practical significance, [1] Structural rutting only

Table 6-14 Summary of marginal means from ANOVA for determining the effect of main design factors on pavement performance measures—Overall

Design Factor		Average Performance						
		Fatigue crackin g (sq-m)	Rut depth (mm)	IRI		Transverse cracking (m)	Longitudinal cracking	
				ΔIRI (m/km)	IRI_0 (m/km)		WP (m)	NWP (m)
HMA Thickness	4"	10.5 (1.6)	5.3 (1.76)	0.45 (-0.86)	0.837	1.5 (-0.204)	8.2 (1.43)	21 (2.6)
	7"	7.6 (1.28)	4.9 (1.7)	0.45 (-0.85)	0.786	1.7 (-.12)	7.6 (1.346)	22.6 (2.68)
Base Type	DGAB	17.4 (2.1)	5.2 (1.82)	0.5 (-0.77)	0.87	2.8 (0.40)	9.8 (1.6)	26.8 (2.85)
	ATB	6.3 (1.04)	4.9 (1.69)	0.4 (-0.975)	0.79	1.2 (-0.48)	5.4 (1.0)	18 (2.45)
	ATB/ DGAB	7.0 (1.12)	5.1 (1.67)	0.47 (-0.825)	0.776	1.3 (-0.38)	9.4 (1.56)	21.5 (2.63)
Base Thickness	8"	9.2 (1.46)	5.6 (1.78)	0.51 (-0.74)	0.843	1.8 (-0.054)	9 (1.51)	34.4 (3.1)
	12"	8.5 (1.39)	5.1 (1.7)	0.45 (-0.87)	0.79	1.4 (-0.313)	6.4 (1.18)	25.5 (2.8)
	16"	9.3 (1.47)	5.0 (1.71)	0.42 (-0.94)	0.80	1.7 (-0.12)	8.7 (1.48)	12.6 (2.2)
Drainage	N	10 (1.55)	5.3 (1.79)	0.47 (-0.82)	0.838	1.8 (-0.06)	8.5 (1.46)	21.5 (2.63)
	Y	8 (1.32)	4.6 (1.67)	0.44 (-0.89)	0.785	1.4 (-0.264)	7.4 (1.32)	22 (2.65)
MSE		(1.51)	(0.06)	(0.13)	-	(1.3)	(1.4)	(0.87)

Note: Values in parenthesis are the lognormal marginal mean values.

The ANOVA was conducted for the design factors within each climatic zone as per the project panel recommendations. However, this analysis suffers from the lack of data within zones, especially within "Dry" zones, where only 2 sites each are available for DF and DNF zones. The results of ANOVA for "Wet" zones are more reliable as 8 and 6 sites are available within WF and WNF zones, respectively. The following discussion summarizes the effect of key design factors on performance in WF climatic zone (see Table 6-15 and Table 6-16):

- **Effect of HMA thickness**: In general, thin (4-inch) pavement sections were built rougher than thick (7-inch) pavements. On average, thin pavements (4-inch) have shown slightly more fatigue cracking and rutting than thick (7-inch) pavements. However, this effect was not found to be statistically significant.

- **Effect of base type**: The effect of base type was found to be significant for all performance measures except for transverse and longitudinal cracking. In the case of distresses that are significantly affected by base type, pavement sections with dense-graded aggregate bases (DGAB) have shown the worst performance while those with asphalt treated bases (ATB) have shown the best performance. Sections built with DGAB have shown significantly (operationally and statistically) more fatigue cracking, rutting, and change in roughness compared to those built with ATB.

- **Effect of base thickness**: Sections with 12-inch (305 mm) bases were built smoother compared to those with 8-inch (203 mm) base. Also, sections with 8-inch (203 mm) base have shown significantly (practically and statistically) higher change in

roughness than those with 16-inch base. However, the change in roughness was not found to be practically significant between sections with 8-inch (406 mm) base and those with 12-inch (305 mm) base. On average, pavement sections with 8-inch (203 mm) base have shown more rut depth than those with 12-inch (305 mm) and 16-inch (406 mm) thick bases. However, this effect was not found to be statistically significant. More longitudinal cracking-NWP occurred in sections built with 8-inch base compared to those with 16-inch base.

- **Effect of drainage condition**: Pavement sections with drainage have shown less rutting than those without drainage. This difference in performance was found to be both statistically and practically significant.

Table 6-15 Summary of p-values from ANOVA for determining the effect of design factors on flexible pavement performance—WF Zone

Design Factor	Performance Measures						
	Fatigue cracking	Rut Depth	IRI		Transverse cracking	Longitudinal cracking	
			ΔIRI	IRI_0		WP	NWP
HMA thickness	0.745	0.688	0.277	0.133	0.560	0.893	0.762
Base type	0.004*	0.001*	0.076*	0.012	0.128	0.232	0.400
Base thickness	0.832	0.504	0.040*	0.084	0.278	0.873	0.069*
Drainage	0.674	0.012*	0.874	0.003*	0.359	0.813	0.885
Site (blocked)	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	R^2=0.638 N=58	R^2=0.631 N=92	R^2=0.620 N=76	R^2=0.628 N=92	R^2=0.71 N=31	R^2=0.695 N=36	R^2=0.813 N=60

* Also shows operational/practical significance

Table 6-16 Summary of marginal means from ANOVA for determining the effect of main design factors on pavement performance measures— WF Zone

Design Factor		Average Performance						
		Fatigue cracking (sq-m)	Rut depth (mm)	IRI		Transverse cracking (m)	Longitudinal cracking	
				ΔIRI (m/km)	IRI_0 (m/km)		WP (m)	NWP (m)
HMA Thickness	4"	17.6 (2.3)	6.1 (1.76)	0.50 (-0.773)	0.891	8.7 (1.92)	24.5 (2.26)	29.3 (3.22)
	7"	15.9 (2.2)	5.9 (1.73)	0.55 (-0.673	0.844	7.4 (1.76)	23.1 (2.2)	31.75 (3.3)
Base Type	DGAB	34.4 (3.0)	7.5 (1.96)	0.55 (-0.665)	0.932	11.0 (2.15)	23.1 (2.2)	26.5 (3.12)
	ATB	10.3 (1.8)	5.7 (1.68)	0.45 (-0.865)	0.833	5.7 (1.5)	11.5 (1.5)	33.4 (3.35)
	ATB/DGAB	14.0 (2.1)	5.2 (1.6)	0.56 (-0.64)	0.84	8.3 (1.87)	47.5 (2.92)	30.2 (3.25)
Base Thickness	8"	17.1 (2.3)	6.4 (1.8)	0.61 (-0.561)	0.91	11.5 (2.2)	28.5 (2.41)	38.8 (3.5)
	12"	18.8 (2.4)	5.8 (1.7)	0.55 (-0.673)	0.833	8.3 (1.87)	23.6 (2.22)	33.0 (3.34)
	16"	15.4 (2.2)	6.0 (1.74)	0.42 (-0.94)	0.86	5.7 (1.49)	19.7 (2.04)	21.3 (2.9)
Drainage	N	18.1 (2.4)	6.8 (1.86)	0.53 (-0.714)	0.924	9.4 (2.0)	22 (2.15)	30.5 (3.26)
	Y	15.4 (2.2)	5.4 (1.63)	0.52 (-0.732)	0.811	6.9 (1.69)	25.5 (2.3)	29.6 (3.23)
MSE		(1.073)	(0.113)	(0.139)		(0.493)	(1.881)	(0.316)

Note: Values in parenthesis are the lognormal marginal mean values.

The following discussion summarizes the effect of key design factors on performance in WNF climatic zone (see Table 6-17 and Table 6-18):

- **Effect of base type**: The effect of base type was found to be significant only for the change in roughness. Pavement sections with dense-graded aggregate bases (DGAB) have shown higher change in roughness compared to those with asphalt treated bases (ATB). However, difference was not practically significant.
- **Effect of HMA thickness**: On average, thin pavements (4-inch) have shown slightly more fatigue cracking compared to thick (7-inch) pavements. This effect was found to be both statistically and practically significant.
- **Effect of base thickness**: The effect of base thickness on various performance measures was found to be statistically insignificant. However, on average, higher change in roughness was observed in sections with 8-inch base compared to those with 12-inch or 16-inch base.
- **Effect of drainage condition**: Pavement sections with drainage have shown less rutting than those without drainage. This difference in performance was found to be both statistically and practically significant.

The fractional factorial design for the SPS-1 experiment calls for a tradeoff between selecting the number of "runs" and testing all possible interactions. Therefore, all possible two-way interactions were considered in the analysis. An ANOVA was run with two-way interaction effects between the main structural design factors. No significant interaction effect was detected.

Table 6-17 Summary of p-values from ANOVA for determining the effect of design factors on flexible pavement performance—WNF Zone

Design Factor	Performance Measures						
	Fatigue cracking	Rut Depth	IRI		Transverse cracking	Longitudinal cracking	
			ΔIRI	IRI_o		WP	NWP
HMA thickness	0.077*	0.576	0.948	0.141	0.383	0.759	0.532
Base type	0.545	0.547	0.065*	0.117	0.470	0.803	0.110
Base thickness	0.703	0.476	0.144	0.559	0.806	0.937	0.265
Drainage	0.298	0.031*	0.725	0.032	0.306	0.760	0.142
Site (blocked)	0.000	0.000	0.008	0.000	0.276	0.037	0.000
	R^2=0.834 N=36	R^2=0.561 N=72	R^2=0.503 N=49	R^2=0.680 N=72	R^2=0.965 N=14	R^2=0.662 N=24	R^2=0.579 N=45

* Also shows operational/practical significance

Table 6-18 Summary of marginal means from ANOVA for determining the effect of main design factors on pavement performance measures— WNF Zone

Design Factor		Average Performance						
		Fatigue cracking (sq-m)	Rut depth (mm)	IRI		Transverse cracking (m)	Longitudinal cracking	
				ΔIRI (m/km)	IRI_o (m/km)		WP (m)	NWP (m)
HMA Thickness	4"	6.4 (1.2)	6.2 (1.78)	0.34 (-1.157)	0.82	0.3 (-1.412)	3.5 (0.562)	13.7 (1.96)
	7"	2.9 (0.43)	5.9 (1.74)	0.33 (-1.165)	0.785	1.0 (-0.363)	4.1 (0.73)	17.5 (2.2)
Base Type	DGAB	5.5 (1.05)	5.9 (1.744)	0.40 (-0.989)	0.84	0.6 (-0.872)	3.7 (0.614)	19.7 (2.32)
	ATB	3.2 (0.524)	6.4 (1.815)	0.29 (-1.3)	0.81	1.6 (0.143)	2.8 (0.356)	8.5 (1.48)
	ATB/ DGAB	4.4 (0.84)	5.8 (1.712)	0.32 (-1.2)	0.764	0.2 (-1.93)	5.2 (0.97)	21.8 (2.42)
Base Thickness	8"	5.8 (1.104)	6.4 (1.82)	0.39 (-1)	0.82	0.4 (-1.171)	4.5 (0.812)	26.6 (2.62)
	12"	4.7 (0.89)	5.8 (1.72)	0.31 (-1.23)	0.793	0.5 (-0.982)	3.6 (0.603)	14.7 (2.03)
	16"	2.9 (0.42)	5.9 (1.73)	0.30 (-1.26)	0.79	0.8 (-0.51)	3.3 (0.523)	9.6 (1.6)
Drainage	N	3.3 (0.54)	6.6 (1.85)	0.57 (-1.136)	0.833	0.9 (-0.385)	3.4 (0.54)	11.1 (1.75)
	Y	5.8 (1.1)	5.2 (1.6)	0.53 (-1.2)	0.772	0.3 (-1.391)	4.2 (0.752)	21.3 (2.4)
MSE		(1.3)	(0.09)	(0.146)		(0.647)	(1.37)	(1.321)

Note: Values in parenthesis are the lognormal marginal mean values.

6.6.3.2 *Effect of Site Factors on Pavement Performance*

The third ANOVA was targeted at determining the significance of subgrade type and climatic zone. Traffic, age and variability in target layer thicknesses were considered as covariates. The subgrade type and climatic zone were included as main factors in addition to the structural design factors. For fatigue cracking, the analysis was run with and without the Kansas (20) data, since the test sections in Kansas (20) have a large amount of fatigue cracking and the project is known to have had construction problems with a wet subbase and variable densities. The analysis for rutting was also done with and without the Texas (48) data since rutting for these sections is believed to be due to the asphalt mix [6].

The results from this analysis are summarized in Tables 6-19 and 6-20, and generally show lower R^2 values than those in Table 6-13. This may be partially due to the effects of variations in environmental conditions within a given climatic zone and variations in material properties within different pavement layers among sites. In addition, construction and material variability is not accounted for in this analysis, since the site factor is not blocked. The results seem to indicate that the effect of the climatic zone is significant for all performance measures and that the effect of subgrade type can be significant for most of them. However, caution must be exercised in interpreting these results given the unbalanced nature of the design with respect to climatic zone: there are only two projects in each of the dry zones, Dry-Freeze (DF) and Dry-No-Freeze (DNF), as opposed to eight projects in the Wet-Freeze (WNF) and six projects in the Wet-No-Freeze (WF) zones.

Finally, ANOVA was conducted by considering the main and interaction (two-way) effects for all six experimental factors. The results of this analysis are summarized in Tables 6-21 and 6-22. The conclusions are based on the main effects when interaction between site factors is not significant.

In case of significant interaction between site factors, the interpretation of results are based on the comparison of cell means, i.e., the mean performance of sections corresponding to each subgrade type should be compared within each climatic zone. The following discussion summarizes the effect of climatic zone and subgrade type on the key performance measures:

- Fatigue cracking: More fatigue cracking was observed on sections located in "wet" climates. The interaction effect between subgrade soil type and climatic zone is statistically significant (see Table 6-21); therefore the conclusions are based on the interaction effect. More cracking was observed in pavement sections built on fine-grained subgrade especially in WNF zone. Among the sections located in WNF zone, the difference between the mean cracking of sections built on fine-grained and sections built on coarse-grained soil is also practically significant.
- Structural Rutting: Rutting was higher among sections located in "wet" climate and was generally higher for pavement sections on fine-grained subgrade. Both of these effects statistically and practically significant. There were high rut depths observed in the Dry-No-Freeze (DNF) zone; however, it is believed that this is more related to the asphalt mix as opposed to structural rutting.

Table 6-19 Summary of p-values from ANOVA for determining the effect of site factors on pavement performance measures (Main effects only)

Site Factor	Performance Measures						
	Fatigue cracking	Rut Depth	IRI		Transverse cracking	Longitudinal cracking	
			ΔIRI	IRI_o		WP	NWP
Subgrade type	0.680	0.432	0.000*	0.067	0.020	0.015	0.013
Climatic Zone	0.000*	0.000*	0.000*	0.001	0.000*	0.000	0.000
Traffic Level	0.000	0.024	0.083	-	0.000	0.437	0.000
Age	0.068	0.013	0.091	-	0.000	0.050	0.009
	R^2=0.288 N=124	R^2=0.305 N=159	R^2=0.401 N=163	R^2=0.215 N=212	R^2=0.674 N=67	R^2=0.434 N=95	R^2=0.534 N=134

Note: The model considered for this analysis has main effects for all six experiment factors. KS (20) was not considered for analysis of all cracking measures, whereas, rut depth analysis was conducted without TX (48). * Also shows operational/practical significance.

Table 6-20 Summary of marginal means from ANOVA for determining the effect of site factors on pavement performance measures (Main effects only)

Site Factor		Performance Measures						
		Fatigue cracking	Rut Depth	IRI		Transverse cracking	Longitudinal cracking	
				ΔIRI	IRI_o		WP	NWP
Subgrade type	F	15.4 (1.16)	5.2 (1.6)	0.59 (-0.62)	0.81	1.2 (-0.38)	44 (2.5)	20.8 (2.2)
	C	18.5 (1.34)	5.4 (1.65)	0.44 (-0.92)	0.76	4.9 (1)	13.2 (1.3)	42.0 (2.9)
Climatic Zone	WF	53.4 (2.4)	4.9 (1.56)	0.59 (-0.628)	0.86	13.2 (2)	26.5 (2.0)	139 (4.1)
	WNF	24.0 (1.6)	5.6 (1.68)	0.336 (-1.186)	0.797	1.5 (-0.15)	16.1 (1.5)	25.4 (2.4)
	DF	26.5 (1.7)	4.0 (1.34)	0.58 (-0.641)	0.784	3.2 (0.6)	5.92 (0.5)	56.5 (3.2)
	DNF	4.5 (-0.08)	7.1 (1.92)	0.596 (-0.613)	0.695	0.4 (-1.5)	177 (3.9)	3.12 (0.3)
MSE		(3.156)	(0.091)	(0.19)		(1.161)	(2.556)	(1.668)

Table 6-21 Summary of p-values from ANOVA for determining the effect of site factors on pavement performance measures (With interaction effects)

Site Factor	Performance Measures						
	Fatigue cracking	Rut Depth	IRI		Transverse cracking	Longitudinal cracking	
			ΔIRI	IRI_0		WP	NWP
Subgrade type	0.626	0.886	0.018	0.653	0.247	0.480	0.191
Climatic Zone	0.049	0.007*	0.000*	0.003	0.004*	0.002	0.000*
Subgrade*Zone	0.000*	0.257	0.562	0.000*	0.092	0.000*	0.496
Traffic Level	0.031	0.028	0.150	-	0.197	0.655	0.000
Age	0.068	0.025	0.565	-	0.077	0.014	0.202
	R^2=0.575 N=124	R^2=0.47 N=159	R^2=0.525 N=163	R^2=0.435 N=212	R^2=0.931 N=67	R^2=0.755 N=95	R^2=0.648 N=134

Note: The model considered for this analysis has main effects for all six experiment factors and all possible two-way interactions between them. * Also shows operational/practical significance.

Table 6-22 Summary of marginal means from ANOVA for determining the effect of site factors on pavement performance measures (Interaction effects only)

Site Factor			Performance Measures						
			Fatigue cracking	Rut Depth	IRI		Transverse cracking	Longitudinal cracking	
					ΔIRI	IRI_0		WP	NWP
Subgrade type			-	-	-	-	-	-	-
Climatic Zone			-	-	-	-	-	-	-
Subgrade*Zone	WF	F	22.4 (1.7)	-	-	0.95	-	60.8 (3.1)	-
		C	67 (2.8)	-	-	0.76	-	3.0 (0.1)	-
	WNF	F	135 (3.5)	-	-	0.78	-	27.3 (2.3)	-
		C	2.5 (-0.5)	-	-	0.83	-	5.5 (0.7)	-
MSE			(2.816)					(2.015)	

Note: The cell means are only given when interaction is significant. For main effects see Table 5-20 for marginal means.

- Roughness: Both subgrade type and climatic zone are very significant factors affecting roughness growth (see Table 6-19). The pavements constructed on fine-grained soils have shown higher changes in roughness than those constructed on coarse-grained soils. Also, pavements located in the WF zone have shown higher change in roughness than those located in WNF zone. These effects were found to be statistically and practically significant (see Table 6-20). The effect of subgrade soil appears to be mainly caused by the initial roughness being significantly higher in sites with fine-grained subgrade. The initial IRI (IRI_o) was found to be associated with future roughness, especially among sections built on fine-grained soils and among sections located in "wet" climate.
- Transverse cracking: The effects of subgrade type and climate on transverse cracking are significant. More cracking was observed in sections built on coarse-grained soil compared to those built on fine-grained soil. However, the magnitude of cracking at this point in time is too low to conclude on the effect. More cracking occurred in sections located in WF zone compared to those located in other zones, and this effect was found to be statistically and practically significant.
- Longitudinal cracking: As the interaction effect between subgrade type and climatic zone is significant for longitudinal cracking-WP, the conclusions are based on comparing cell means for sections built on each subgrade type within each climatic zone. It was found that among pavements located in WF zone, those constructed on fine-grained soils have shown significantly more cracking than those constructed on coarse-grained soils. The effects of subgrade type and climate were significant in the case of longitudinal cracking-NWP. Significantly more cracking

was observed in the sections built on coarse-grained soil compared to those built on fine-grained soil. Also pavements located in "freeze" climate have shown significantly more cracking compared to those in "no-freeze" climate.

Given the unbalanced design of the experiment with respect to climatic zone, a one-way ANOVA was performed to investigate the effects of subgrade type (fine-grained versus coarse-grained soils) and climatic zones (wet versus dry, freeze versus no-freeze), one at a time. The p-values and mean performances by site factors, from this analysis, are summarized in Tables 6-23 and 6-24, respectively. To indicate the direction of effects for site factors, the "+" and "-"signs are also reported along with the p-values. The "+" indicates that, within a factor, the first level is exhibiting more distress than the second level, while "-"indicates otherwise. For example, in the case of the effect of subgrade on fatigue cracking, the "+" indicates more cracking in pavements constructed on fine-grained soils (first level for subgrade) compared to those constructed on coarse-grained soils (second level for subgrade).

The p-values indicate that subgrade type appears to be significantly affecting fatigue cracking, rut depth, roughness, transverse and longitudinal cracking-WP. The pavements built on fine-grained subgrade have shown higher distress than those constructed on coarse-grained subgrade. The effect was found to be practically significant in the case fatigue cracking, rutting, change in roughness and transverse cracking. The effects of site factors by performance measure are listed below:

- Fatigue Cracking: Climate appears to be significantly affecting fatigue performance. Pavements located in "wet" or "freeze" climate have exhibited significantly higher amount of fatigue cracking than those located in "dry" or "no-freeze" climate, respectively. This effect was found to be practically significant.
- Rut Depth: On average, rutting appears to be higher in wet climate. Also pavements located in DNF zone were found to have significantly more rutting compared to those located in DF zone. However, it is believed that this is more related to the asphalt mix as opposed to structural rutting, as mentioned before at the beginning of this chapter.
- Roughness: Significantly higher growth in roughness was observed for pavements located in WF zone compared to those located in WNF zone. This effect was found to be practically significant.
- Transverse Cracking: It was found that pavements located in WF zone have exhibited significantly higher transverse cracking than those located in WNF zone. This effect was found to be practically significant.
- Longitudinal Cracking-WP: Significantly more longitudinal cracking-WP was observed in pavements located in WF zone compared to those located in WNF zone. Also, significantly more longitudinal cracking-WP was exhibited by the pavements located in DNF zone compared to those located in DF. In DNF zone, longitudinal cracking-WP and rutting is mainly contributed by sections in the Arizona, AZ (4), site, where HMA-related issues are believed to be causing the distresses.

- Longitudinal Cracking-NWP: Significantly more longitudinal cracking-NWP was exhibited by the pavements located in WF zone compared to those located in other zones. Also, more cracking was observed in pavements located in DF zone compared to those built in DNF zone. These effects indicate that this distress could be related to "freeze" environment.

It should be noted that the data from the four projects in the dry climatic zones show negative trends in several performance measures. This may be in part due to the lower number of projects in these zones.

6.6.3.3 Effect of Design Factors on Pavement Performance (univariate) based on standard deviate

As explained before, the experiment design and the performance of the test sections have rendered the SPS-1 experiment "unbalanced". Fourteen out of eighteen sites, in the experiment are located in "wet" climate, of which eight are in the WF zone. In addition, all 24 unique designs were not built in every soil-climate combination. Furthermore, non-occurrence of distresses in a considerable number of sections contributed to the unbalance. This could be a reason for insignificance of interaction effects between the design and site factors from multivariate analyses presented above. In light of the above concerns, a simplified analysis considering one design factor at a time (univariate) was performed using one-way ANOVA (as in the case of analysis of the effects for site factors).

Table 6-23 Summary of p-values from one-way ANOVA for determining the effect of site factors on pavement performance measures

Site Factor	Performance Measures						
	Fatigue cracking	Rut depth	IRI		Transverse cracking	Longitudinal cracking	
			ΔIRI	IRI_0		WP	NWP
Subgrade Type							
Fine vs. Coarse	0.03 (+)*	0.002(+)*	0.00 (+)*	0.011 (+)*	0.016 (+)*	0.001 (+)	0.26 (-)
Climatic Zone							
Wet vs. Dry	0.021 (+)*	0.087 (+)	0.596 (-)	0.000 (+)*	0.005 (+)*	0.919 (+)	0.040 (+)
F vs. NF	0.011 (+)*	0.001 (-)	0.000 (+)*	0.010 (+)*	0.060 (+)*	0.038 (-)	0.000 (+)
WF vs. WNF	0.063 (+)*	0.893 (-)	0.000 (+)*	0.030 (+)*	0.000 (+)*	0.096 (+)	0.000 (+)
DF vs. DNF	0.054 (+)*	0.000 (-)	0.281 (-)	0.231 (+)	0.055 (-)	0.000 (-)	0.001 (+)

* Also shows operational/practical significance

Table 6-24 Summary of marginal means from one-way ANOVA for determining the effect of site factors on pavement performance measures

Site Factor		Performance Measures						
		Fatigue cracking	Rut depth	IRI		Transverse cracking	Longitudinal cracking	
				ΔIRI	IRI_0		WP	NWP
Subgrade Type	Fine	54.2	5.8	0.613	0.86	18.97	64.5	163.0
	Coarse	24.2	4.8	0.451	0.79	6.70	17.4	113.0
Climatic Zone	Wet	41.7	5.4	0.524	0.85	15.10	39.7	150.5
	Dry	17.4	4.7	0.552	0.73	04.83	38.1	74.0
	Freeze	45.6	4.8	0.616	0.85	13.57	24.5	189.7
	No Freeze	18.4	5.8	0.425	0.78	06.20	57.7	27.1
	WF	53.7	5.2	0.635	0.88	24.3	31.3	244.7
	WNF	24.5	5.6	0.360	0.81	2.80	14.8	29.4
	DF	26.2	4.0	0.480	0.76	2.25	4.10	83.1
	DNF	07.8	6.8	0.572	0.70	6.00	108.8	14.7

The performance of test sections was not found to be consistent across sites indicating the influence of site conditions (see Chapter 3). The site conditions that could have contributed to this variation in performance are traffic, age, construction quality, measurement variability, and material properties, apart from the experimental site factors (i.e. subgrade and environment). In order to separate the "true" effects of the experimental factors, this "noise" had to be nullified.

The standard deviate for each performance measure was calculated, within each site, for all the sections using equation 5-3. This measure indicates the relative performance of a design compared to the other designs. As this measure was calculated for each section, considering one site at a time, it indicates the relative standing of the section compared to other sections. It thus helps nullify the variation in performance (due to site conditions) among sites, as the sections are weighed with respect to companion sections in each site.

$$Std.Deviate = \left(\frac{x - \mu}{\sigma} \right) \qquad (5\text{-}3)$$

The above approach of using the standard deviate is similar to blocking of the site factors performed in the multivariate analysis. One-way analysis of variance (univariate) was performed on the standard deviates of the sections to study the effects of each design factor by taking one design factor at a time. The analyses were performed on data from all sections and also on subsets of data stratified by different subgrade types, climates and combinations of these. This helps identify the effects of design factors under different site conditions.

In the SPS-1 experiment, HMA thickness and drainage have two levels (i.e. 4" vs. 7" and drainage vs. no drainage). But for base thickness and base type, three levels (8"

vs. 12" vs. 16" and DGAB vs. ATB vs. ATB/DGAB) are present. Moreover, 16" base thickness was provided only for drained sections and ATB/DGAB was built only for un-drained sections making the design unbalanced. Therefore, for studying the effects of base thickness and base type, analyses were done separately among drained sections and among un-drained sections.

To see the "pure" effect of each design factor, comparisons of standard deviates were also made between the levels of each design factor while controlling the other factors, as in the case of level-B analyses (site-level). The results from this analysis are in Appendix A5 of reference [7].

The effects of design factors, based on the above-mentioned analyses, on each performance measure are discussed next.

Fatigue Cracking

The effects of the design and site factors, in terms of standard deviate, are shown in Figure 6-22. In addition, the summary of p-values corresponding to the analyses performed to study the effects of design factors on fatigue cracking is shown in Table 6-25. The mean area (m^2) of fatigue cracking (PI) corresponding to each comparison presented in Table 6-25 is shown in Table 6-26. Though the univariate analyses were performed on standard deviates, the mean cracking was used to identify operationally significant effects. The effects of design factors on fatigue cracking, based on this analysis, are presented below:

HMA thickness: The effect of HMA surface thickness is statistically and operationally significant, especially among sections located in WNF zone. Sections built with "thin"

[4-inch (102 mm)] HMA surface have exhibited higher fatigue cracking than those built with "thick" [7-inch (178 mm)] HMA surface. On average, among sections built on fine-grained soils, higher fatigue cracking was observed on "thin" [4-inch (102 mm)] sections compared to "thick" [7-inch (178 mm)] sections. However, this effect is not significant. Similar trend was found among sections built on coarse-grained soils and the effect is statistically and operationally significant.

Base thickness: The effect of base thickness is marginal among sections built on fine-grained subgrade soil, in that sections with thick 16-inch (406 mm) permeable base have exhibited lesser cracking than those with 12-inch (305 mm) or 8-inch (203 mm) base thickness. Also among sections located in WF zone the effect of base thickness on alligator cracking is statistically and operationally significant. Sections built with 16-inch (406 mm) permeable base have exhibited lesser cracking than those with 12-inch (305 mm) or 8-inch (203 mm) base.

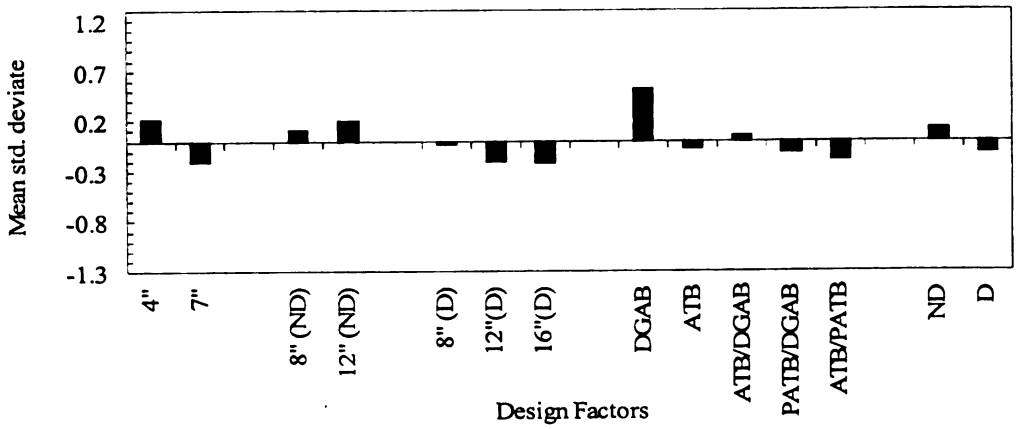
Base Type: The effect of base type (unbound versus treated base) is statistically and operationally significant, with ATB giving the "best" performance and DGAB showing the "worst" performance. This effect is more prominent among sections built on fine-grained soils compared to sections built on coarse-grained soils. Also, this effect is more noticeable among sections located in WF zone.

Drainage: The effect of drainage is significant (statistically and operationally) among those in WF zone and built on fine-grained subgrade soils. Sections with drainage have lesser cracking than those without drainage. This effect is more prominent for the sections constructed with DGAB than those with ATB. This shows that drainage is more

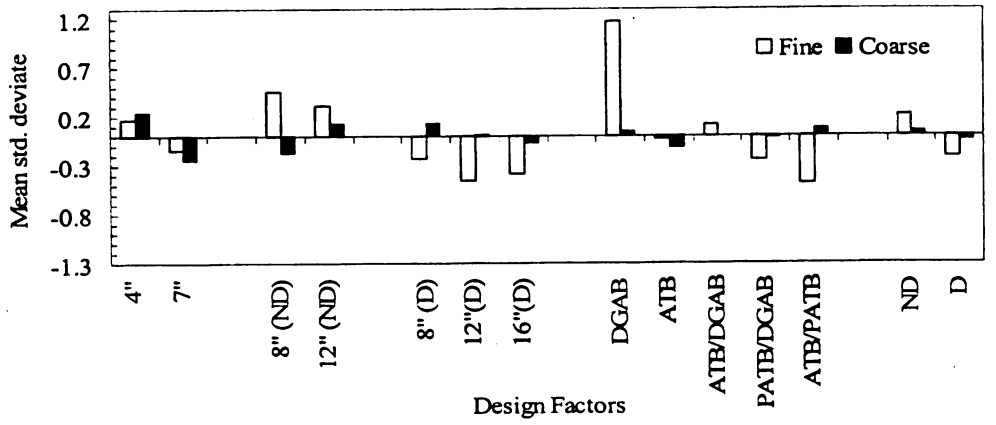
effective if provided with DGAB than when with ATB. The interaction effects among the experimental factors, on fatigue cracking, are reported below:

In general, "thin" sections with DGAB on fine-subgrade soils have exhibited the most alligator cracking while "thick" sections with ATB on coarse-grained subgrade soils have exhibited the least alligator cracking. Among un-drained pavements, on average, an increase in HMA surface thickness from 4-inch (102 mm) to 7-inch (178 mm) has a slightly higher effect on fatigue cracking for pavements with DGAB than for pavements with ATB.

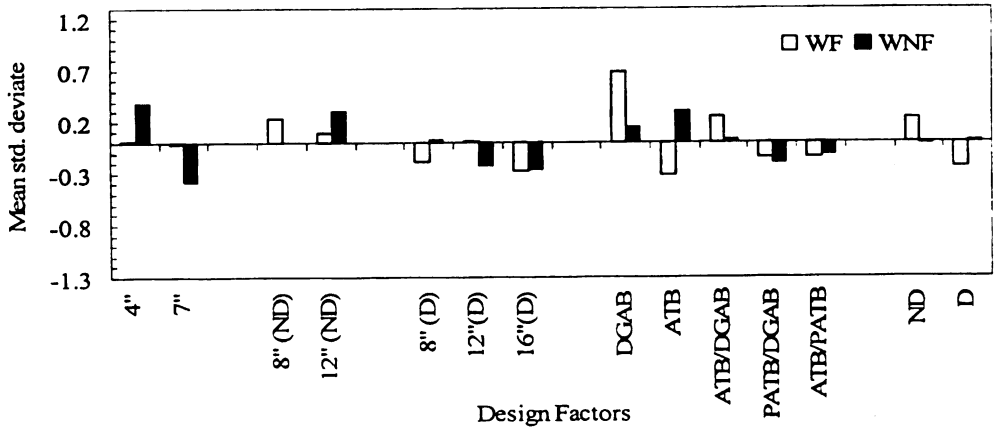
Among sections located in the WF zone, those with DGAB have shown the highest amount of cracking while those with ATB have the least. In addition, among pavements located in WF zone, those with 16-inch (406 mm) drained base have less fatigue cracking than others. These effects were found to be statistically and practically significant. Among pavements with DGAB and built on fine-grained soils, those with drained base have lesser fatigue cracking than others. Also, among sections with drainage and built on fine-grained soils, those with 16-inch base have lesser cracking. These effects were found to be statistically and practically significant.



(a) Overall



(b) By subgrade type



(c) By zone type

Figure 6-22 Effect of design factors on fatigue cracking

Table 6-25 Summary of p-values for comparisons of standard deviates— Fatigue cracking

Design Factor		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
				Fine	Coarse	WF	WNF	DF	DNF	WF		WNF		DF		DNF	
										F	C	F	C	F	C	F	C
HMA thickness		4" vs. 7"	0.003	0.167	0.008	0.890	0.002	0.102	0.213	0.900	0.900	0.160	0.005				
Base thickness	Overall	8" vs. 12" vs. 16"	0.211	0.086	0.890	0.043	0.410	0.63	0.265	0.070	0.040	0.850	0.420				
	ND	8" vs. 12"	0.737	0.802	0.480	0.271	0.168	0.181	0.028	0.817	0.062	0.336	0.374				
	D	8" vs. 12" vs. 16"	0.381	0.020	0.512	0.137	0.609	0.977	0.159	0.060	0.523	0.207	0.587				
Base type	Overall	DGAB vs. ATB vs. ATB/DGAB	0.000	0.004	0.011	0.003	0.057	0.390	0.500	0.060	0.050	0.150	0.350				
	ND	DGAB vs. ATB vs. ATB/DGAB	0.002	0.003	0.207	0.003	0.140	0.881	0.449	0.024	0.095	0.254	0.557				
	D	DGAB vs. ATB	0.001	0.058	0.008	0.027	0.157	0.101	0.313	0.186	0.060	0.270	0.680				
	All Bases	DGAB vs. ATB vs. ATB/DGAB vs. DGAB/PATB vs. ATB/PATB	0.000	0.000	0.036	0.000	0.165	0.522	0.290	0.003	0.060	0.270	0.680				
Drainage	Overall	Drainage vs. No-Drainage	0.111	0.058	0.610	0.038	0.884	0.740	0.180	0.050	0.330	0.870	0.750				
	DGAB	Drainage vs. No-Drainage	0.070	0.010	0.770	0.030	0.720	0.370	0.220	0.040	0.300	0.430	0.840				
	ATB	Drainage vs. No-Drainage	0.160	0.580	0.200	0.040	0.190	0.500	0.250	0.170	0.150	0.240	0.500				
N			188	80	108	80	60	24	24	44	36	24	36				

Note: Shaded cells show statistically significant at 90% or higher level of confidence.

Table 6-26 Summary of means of PI for fatigue cracking

Design Factors		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
										WF		WNF		DF		DNF	
				Fine	Coarse	WF	WNF	DF	DNF	F	C	F	C	F	C	F	C
HMA thickness		4"	14.6	19.2	11.8	10.7	19.8	24.3	3.4	5.5	15.2	47.0	1.6		24.3	0.2	6.5
		7"	9.4	10.3	8.7	11.2	8.4	13.0	2.3	7.4	15.5	20.3	0.6		13.0	0.0	4.5
Base thickness	Overall	8"	13.4	17.2	10.9	11.8	16.5	21.9	1.4	6.9	16.4	40.1	0.8		21.9	0.0	2.9
		12"	11.9	14.8	9.9	12.4	13.5	14.3	3.6	8.4	16.5	31.8	1.2		14.3	0.2	7.0
		16"	8.8	7.8	9.5	5.6	9.7	21.5	4.2	2.0	9.9	22.1	1.4		21.5	0.0	8.5
	ND	8"	13.4	22.3	7.5	8.7	22.1	18.0	1.1	8.3	9.1	54.4	0.6		18.0	0.0	2.2
		12"	9.8	10.5	9.4	14.0	8.1	5.9	5.9	8.3	19.6	18.7	1.0		5.9	0.4	11.5
	D	8"	13.4	8.8	16.2	17.0	8.2	27.9	1.9	4.5	27.4	18.7	1.2		27.9	0.1	3.8
		12"	14.9	21.4	10.5	10.1	21.6	26.9	0.2	8.5	11.8	51.5	1.6		26.9	0.0	0.4
		16"	8.8	7.8	9.5	5.6	9.7	21.5	4.2	2.0	9.9	22.1	1.4		21.5	0.0	8.5
Base type	Overall	DGAB	18.8	23.4	16.1	18.5	22.5	26.0	3.2	7.7	27.2	53.8	1.7		26.0	0.2	6.3
		ATB	6.8	7.7	6.2	5.8	6.8	14.7	2.7	5.3	6.4	15.8	0.8		14.7	0.0	5.4
		ATB/DGAB	8.8	13.4	5.7	8.2	11.5	10.3	2.0	8.2	8.3	28.0	0.5		10.3	0.0	4.1
	ND	DGAB	20.6	34.3	13.0	20.0	30.6	13.9	3.8	11.3	25.9	74.3	1.4		13.9	0.5	7.2
		ATB	6.4	6.0	6.7	7.7	3.2	11.5	4.7	6.9	8.9	7.3	0.4		11.5	0.1	9.3
		ATB/DGAB	8.8	13.4	5.7	8.2	11.5	10.3	2.0	8.2	8.3	28.0	0.5		10.3	0.0	4.1
	D	DGAB	17.7	16.9	18.1	17.6	17.1	34.0	2.8	5.9	28.0	40.1	1.8		34.0	0.0	5.7
		ATB	7.2	8.9	5.9	4.3	9.1	16.8	1.4	4.0	4.7	21.4	1.0		16.8	0.0	2.8
Drainage	Overall	ND	11.6	16.4	8.5	11.3	15.1	11.9	3.5	8.3	14.4	36.5	0.8		11.9	0.2	6.8
		D	12.3	12.7	12.0	10.6	13.1	25.4	2.1	4.8	16.3	30.7	1.4		25.4	0.0	4.2
	DGAB	ND	20.6	34.3	13.0	20.0	30.6	13.9	3.8	11.3	25.9	74.3	1.4		13.9	0.5	7.2
		D	17.7	16.9	18.1	17.6	17.1	34.0	2.8	5.9	28.0	40.1	1.8		34.0	0.0	5.7
	ATB	ND	6.4	6.0	6.7	7.7	3.2	11.5	4.7	6.9	8.9	7.3	0.4		11.5	0.1	9.3
		D	7.2	8.9	5.9	4.3	9.1	16.8	1.4	4.0	4.7	21.4	1.0		16.8	0.0	2.8

Structural Rutting

The effects of the design and site factors, in terms of standard deviate, are shown in Figure 6-23. The summary of p-values corresponding to the analyses performed to study the effects of design factors on structural rutting is presented in Table 6-27. The mean rut depth (PI), in mm, corresponding to each comparison presented in Table 6-27 are shown in Table 6-28. The effects of design factors on rutting, based on this analysis, are presented below:

HMA thickness: Among sections built on coarse-grained soils, the sections built with "thin" [4-inch (102 mm)] HMA surface have exhibited higher rut depths than those built with "thick" [7-inch (178 mm)] HMA surface. This effect is statistically significant but not operationally significant, at this point in time. Thus increasing HMA thickness from 4" to 7" may be more effective in retarding rutting in the case of sections with coarse-grained soils than in the case of sections with fine-grained soils. On average, sections built on fine-grained soils have slightly higher rutting than those with coarse-grained soils.

Base thickness: The effect of base thickness is significant (statistical and operational) among sections located in WNF zone where higher rutting was observed for the sections built with 8-inch (203 mm) thick base than for those built on 16-inch (406 mm) thick base. In addition, this effect seems to be more apparent for the sections built on coarse-grained subgrade soils.

Base Type: In general, the effect of base type (unbound versus treated base) is not statistically significant. However on average, sections built on ATB have shown the

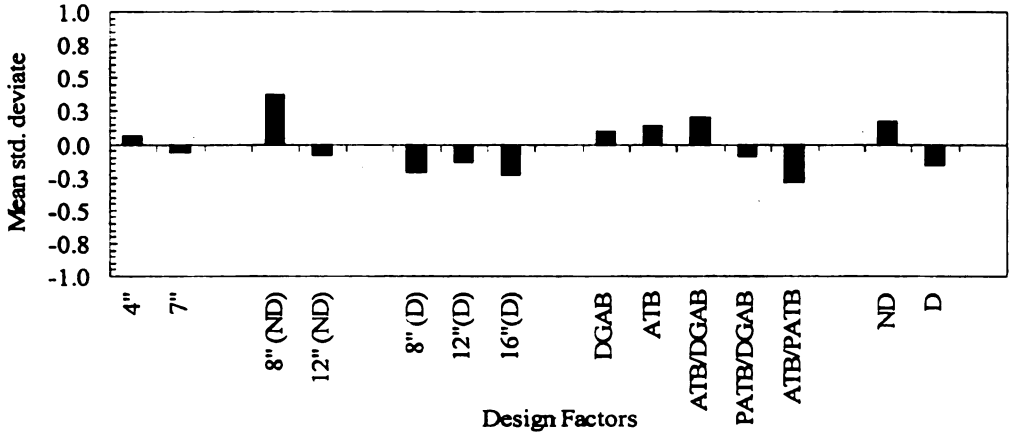
better performance than those sections built on DGAB. This effect (DGAB vs. ATB) is more prominent among sections located in WF zone and built on fine-grained soils.

Drainage: In general, the effect of drainage is statistically significant with un-drained sections showing higher rutting than those with drainage. However, this effect is not operationally significant. This effect is significant (statistical and operational) among sections located in WNF zone and built on fine-grained soils. Also the effect is significant (statistical and operational) among sections in WF zone and built on coarse-grained soils. The results suggest that drainage may be more effective in inhibiting rutting for pavements on fine-grained soils, when located in WNF zone.

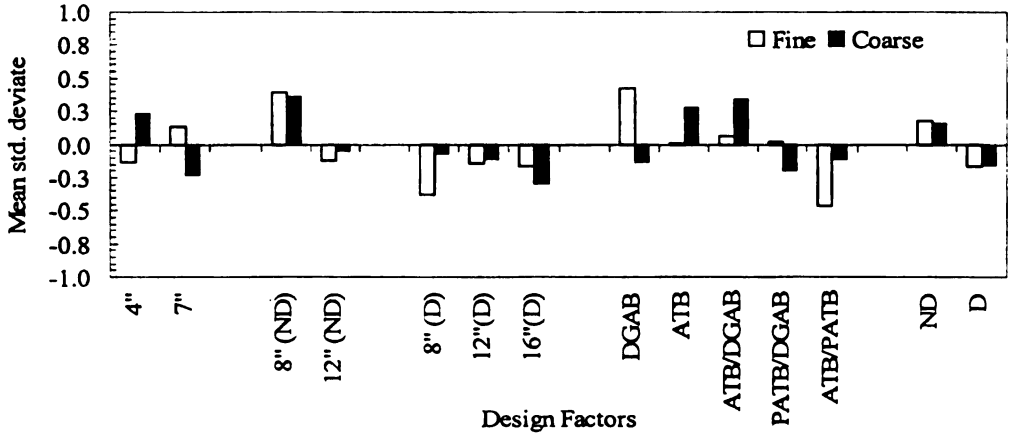
The interaction effects among the experimental factors, on structural rutting, are reported below:

A marginal effect of drainage was observed on pavements built with ATB and on fine-grained soils. Also, among drained pavements located in WF zone, those with DGAB have shown higher rutting than those with ATB. Furthermore, among sections located in WF zone and built with ATB, those with drainage have shown significantly less amount of rutting than those without drainage. Both of the above effects were found to be statistically significant and are of operational significance.

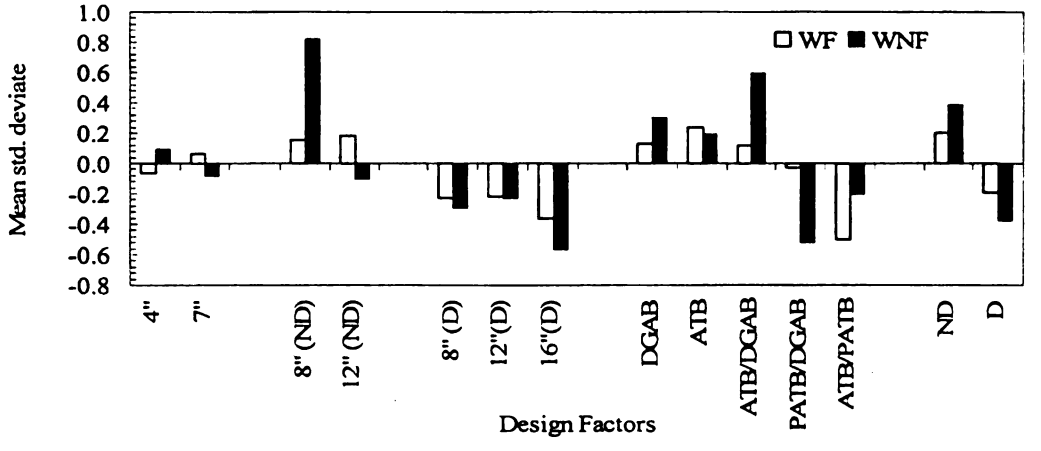
Among un-drained sections located in WNF zone, those with 12-inch (305 mm) base thickness have less amount of rutting than those with 8-inch (203 mm) base thickness. This effect was found to be statistically significance and is practically meaningful. For sections built on DGAB and located in WNF zone, those with drainage have shown slightly lesser rutting than those without drainage. The effect was not found to be statistically significant.



(a) Overall



(b) By subgrade type



(c) By zone type

Figure 6-23 Effect of design factors on structural rutting

Table 6-27 Summary of p-values for comparisons of standard deviates— Structural rutting

Design Factor		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
				Fine	Coarse	WF	WNF	DF	DNF	WF		WNF		DF		DNF	
										F	C	F	C	F	C	F	C
HMA thickness		4" vs. 7"	0.458	0.210	0.0270	0.580	0.501	0.001	0.078	0.420	0.900	0.900	0.300				
Base thickness	Overall	8" vs. 12" vs. 16"	0.270	0.850	0.240	0.560	0.046	0.390	0.620	0.600	0.200	0.300	0.002				
	ND	8" vs. 12"	0.080	0.193	0.245	0.947	0.049	0.241	0.745	0.036	0.159	0.990	0.001				
	D	8" vs. 12" vs. 16"	0.675	0.438	0.695	0.907	0.283	0.669	0.712	0.784	0.496	0.358	0.126				
Base type	Overall	DGAB vs. ATB vs. ATB/DGAB	0.180	0.225	0.230	0.520	0.003	0.770	0.370	0.004	0.170	0.002	0.440				
	ND	DGAB vs. ATB vs. ATB/DGAB	0.579	0.879	0.497	0.931	0.240	0.632	0.161	0.660	0.527	0.095	0.823				
	D	DGAB vs. ATB	0.332	0.110	0.692	0.044	0.070	0.471	0.487	0.000	0.400	0.156	0.308				
	All Bases	DGAB vs. ATB vs. ATB/DGAB vs. DGAB/PATB vs. ATB/PATB	0.140	0.274	0.368	0.214	0.004	0.323	0.409	0.017	0.053	0.030	0.578				
Drainage	Overall	Drainage vs. No-Drainage	0.028	0.110	0.140	0.095	0.002	0.069	0.980	0.700	0.007	0.004	0.170				
	DGAB	Drainage vs. No-Drainage	0.810	0.760	0.860	0.940	0.100	0.180	0.710	0.590	0.370	0.110	0.550				
	ATB	Drainage vs. No-Drainage	0.030	0.100	0.170	0.010	0.250	0.120	0.270	0.290	0.006	0.580	0.310				
N			161	77	84	66	59	24	12	30	36	35	24				

Note: Shaded cells show statistically significant at 90% or higher level of confidence.

Table 6-28 Summary of means of PI for structural rutting

Design Factors		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
										WF		WNF		DF		DNF	
				Fine	Coarse	WF	WNF	DF	DNF	F	C	F	C	F	C	F	C
HMA thickness		4"	5.4	5.7	5.1	5.2	5.8	4.7	6.3	5.3	5.1	5.9	5.7		4.7	6.3	
		7"	5.2	5.8	4.6	5.4	5.5	3.3	6.7	5.7	5.1	5.7	5.1		3.3	6.7	
Base thickness	Overall	8"	5.5	5.9	5.2	5.4	6.1	4.0	6.5	5.8	5.1	5.7	6.7		4.0	6.5	
		12"	5.2	5.7	4.7	5.2	5.4	3.8	6.4	5.1	5.4	6.0	4.5		3.8	6.4	
		16"	5.1	5.7	4.5	5.0	4.9	4.7	6.7	5.6	4.4	5.3	4.3		4.7	6.7	
	ND	8"	5.9	6.3	5.5	5.7	6.8	3.9	6.5	6.3	5.3	6.2	7.5		3.9	6.5	
		12"	5.1	5.5	4.6	5.2	5.4	3.2	6.4	4.2	5.9	6.2	4.2		3.2	6.4	
	D	8"	5.0	5.3	4.8	4.9	5.3	4.0	6.4	5.0	4.9	5.1	5.6		4.0	6.4	
		12"	5.3	6.0	4.7	5.3	5.4	4.7	6.3	6.2	4.5	5.7	5.0		4.7	6.3	
		16"	5.1	5.7	4.5	5.0	4.9	4.7	6.7	5.6	4.4	5.3	4.3		4.7	6.7	
Base type	Overall	DGAB	5.4	6.1	4.7	5.5	5.3	4.3	6.5	6.8	4.7	5.4	5.2		4.3	6.5	
		ATB	5.1	5.4	4.9	5.0	5.6	3.7	6.5	4.7	5.2	5.7	5.4		3.7	6.5	
		ATB/DGAB	5.6	6.0	5.2	5.4	6.5	3.8	6.1	5.1	5.8	6.9	5.8		3.8	6.1	
	ND	DGAB	5.3	6.0	4.9	5.2	5.9	3.5	6.5	5.6	5.1	6.0	5.8		3.5	6.5	
		ATB	5.4	5.7	5.1	5.6	5.8	3.3	6.8	5.3	5.9	5.7	5.8		3.3	6.8	
		ATB/DGAB	5.6	6.0	5.2	5.4	6.5	3.8	6.1	5.1	5.8	6.9	5.8		3.8	6.1	
	D	DGAB	5.4	6.2	4.6	5.7	5.0	4.9	6.6	7.1	4.4	5.1	4.7		4.9	6.6	
		ATB	4.9	5.2	4.7	4.5	5.4	4.1	6.3	4.2	4.8	5.6	5.2		4.1	6.3	
Drainage	Overall	ND	5.5	5.9	5.1	5.4	6.1	3.5	6.5	5.2	5.6	6.2	5.8		3.5	6.5	
		D	5.1	5.7	4.7	5.1	5.2	4.5	6.5	5.7	4.6	5.4	5.0		4.5	6.5	
	DGAB	ND	5.3	6.0	4.9	5.2	5.9	3.5	6.5	5.6	5.1	6.0	5.8		3.5	6.5	
		D	5.4	6.2	4.6	5.7	5.0	4.9	6.6	7.1	4.4	5.1	4.7		4.9	6.6	
	ATB	ND	5.4	5.7	5.1	5.6	5.8	3.3	6.8	5.3	5.9	5.7	5.8		3.3	6.8	
		D	4.9	5.2	4.7	4.5	5.4	4.1	6.3	4.2	4.8	5.6	5.2		4.1	6.3	

Roughness

The effects of the design and site factors, in terms of standard deviate, are shown in Figure 6-24. The summary of p-values corresponding to the analyses performed to study the effects of design factors on roughness is shown in Table 6-28. The mean PI corresponding to each comparison presented in Table 6-28 is shown in Table 6-29. The effects of design factors on roughness, based on this analysis, are presented below:

HMA thickness: In general, the effect of HMA surface thickness is statistically significant. Sections built with "thin" [4-inch (102 mm)] HMA surface have exhibited higher change in roughness than those built with "thick" [7-inch (178 mm)] HMA surface. This effect is more prominent among sections built on fine-grained soils.

Base thickness: On the whole, the effect of base thickness is statistically and operationally significant. Sections with "thick" [16-inch (406 mm)] permeable base have exhibited the least change in roughness whereas sections with "thin" [8-inch (203 mm)] base thickness have shown the highest change in roughness. This effect is more significant among sections located in "wet" climate than among sections located in "dry" climate.

Base Type: In general, the effect of base type is statistically and operationally significant. Sections built with DGAB have exhibited higher change in roughness than those built with ATB. This effect is more prominent among sections built on fine-grained soils.

Drainage: By and large, the effect of drainage is only statistically significant i.e. it is not of practical significance at this point in time. Sections without drainage have exhibited higher change in roughness than those built with drainage. This effect is significant (statistical and operational) among sections built on fine-grained soils and located in WF

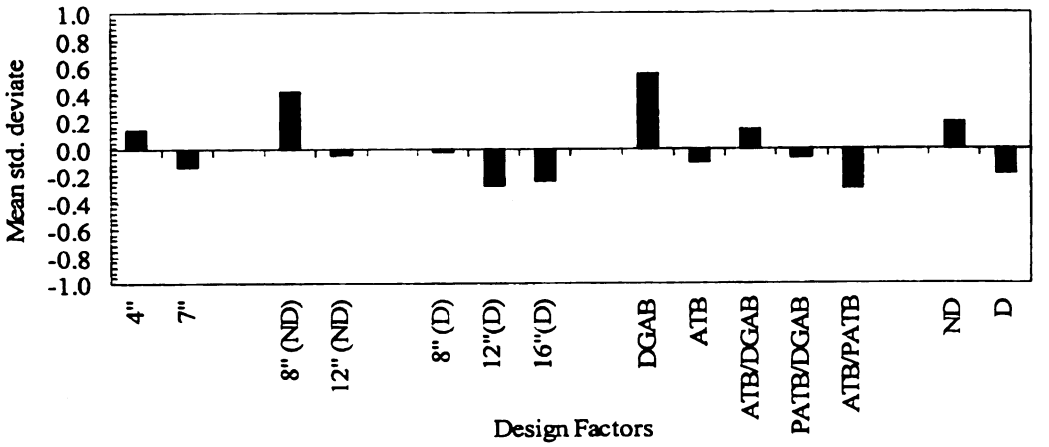
zone. This effect is more prominent for sections with DGAB. This suggests that drainage is more effective for pavements with DGAB on fine-grained soils, especially when in WF zone. The interaction effects among the experimental factors are reported below:

Also for un-drained pavements built on fine-grained soils, the effect of base type is significant, in that pavements with ATB have significantly lower ΔIRI. Furthermore, the effect of drainage for sections with DGAB and built on fine-grained soils, is significant. The above effects were found to be statistically significant and are of practical significance. For un-drained pavements built on coarse-grained soils, an increase in base thickness from 8-inch (203 mm) to 12-inch (305 mm) has a marginally significant effect, in that sections with thicker base have lower ΔIRI. However, this effect is not of practical significance at this point in time. It should be noted that, in general, pavements built on fine-grained soils have shown higher ΔIRI than those built on coarse-grained soils, especially among sections in WF zone. Also, ΔIRI among sections located in WF zone is higher than those in WNF zone.

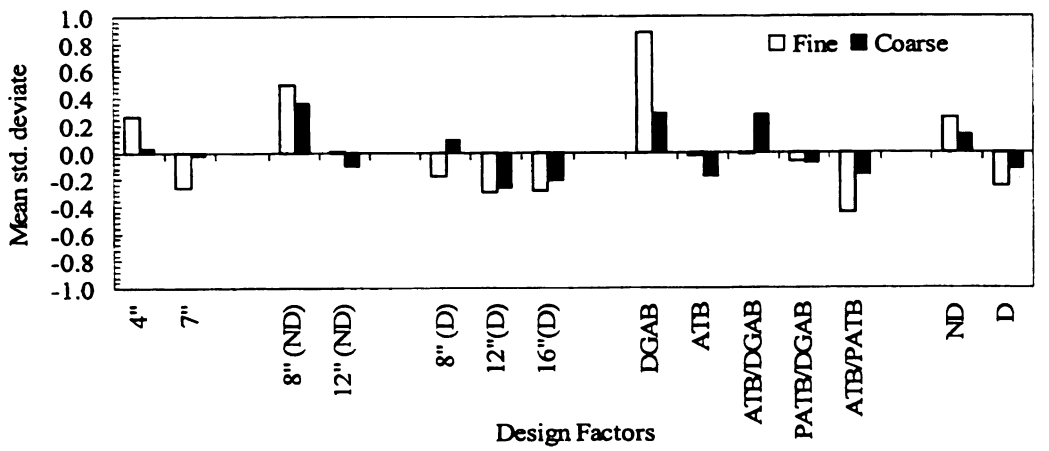
Transverse Cracking

The effects of the design and site factors, in terms of standard deviate, are shown in Figure 6-25. The summary of p-values corresponding to the analyses performed to study the effects of design factors on transverse cracking is presented in Table 6-31. The mean PI corresponding to each comparison presented in Table 6-31 is shown in Table 6-32. The effects of design factors are presented below:

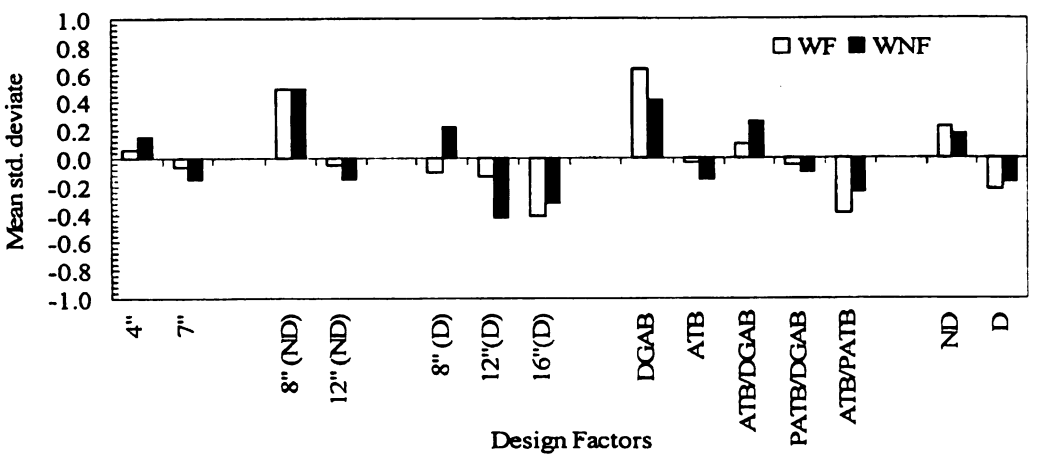
HMA thickness: The effect of HMA surface thickness is not significant. However, on an average, sections with "thin" HMA surface have slightly higher cracking than sections with "thick" HMA layer.



(a) Overall



(b) By subgrade type



(c) By zone type

Figure 6-24 Effect of design factors on change in IRI

Table 6-29 Summary of p-values for comparisons of standard deviates— Change in IRI

Design Factor		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
				Fine	Coarse	WF	WNF	DF	DNF	WF		WNF		DF		DNF	
										F	C	F	C	F	C	F	C
HMA thickness		4" vs. 7"	0.047	0.008	0.800	0.540	0.220	0.038	0.660	0.140	0.400	0.160	0.670				
Base thickness	Overall	8" vs. 12" vs. 16"	0.008	0.120	0.070	0.045	0.028	0.900	0.340	0.320	0.069	0.200	0.145				
	ND	8" vs. 12"	0.019	0.128	0.076	0.083	0.068	0.861	0.540	0.372	0.056	0.242	0.185				
	D	8" vs. 12" vs. 16"	0.445	0.898	0.498	0.501	0.265	0.884	0.133	0.287	0.381	0.448	0.629				
Base type	Overall	DGAB vs. ATB vs. ATB/DGAB	0.023	0.036	0.220	0.100	0.340	0.180	0.810	0.110	0.740	0.190	0.180				
	ND	DGAB vs. ATB vs. ATB/DGAB	0.028	0.030	0.252	0.193	0.418	0.205	0.992	0.086	0.997	0.446	0.416				
	D	DGAB vs. ATB	0.212	0.087	0.754	0.150	0.691	0.484	0.750	0.136	0.443	0.236	0.601				
	All Bases	DGAB vs. ATB vs. ATB/DGAB vs. DGAB/PATB vs. ATB/PATB	0.002	0.001	0.331	0.032	0.423	0.331	0.744	0.003	0.934	0.528	0.223				
Drainage	Overall	Drainage vs. No-Drainage	0.006	0.011	0.170	0.028	0.163	0.920	0.160	0.007	0.834	0.900	0.055				
	DGAB	Drainage vs. No-Drainage	0.014	0.009	0.300	0.060	0.340	0.520	0.180	0.001	0.830	0.900	0.23				
	ATB	Drainage vs. No-Drainage	0.420	0.120	0.970	0.260	0.720	0.280	0.610	0.240	0.580	0.630	0.800				
N			200	92	108	92	60	24	24	56	36	24	36				

Note: Shaded cells show statistically significant at 90% or higher level of confidence.

Table 6-30 Summary of means of PI for change in IRI

Design Factors		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
										WF		WNF		DF		DNF	
				Fine	Coarse	WF	WNF	DF	DNF	F	C	F	C	F	C	F	C
HMA thickness		4"	0.27	0.44	0.14	0.38	0.13	0.18	0.31	0.52	0.17	0.20	0.08		0.18	0.50	0.11
		7"	0.22	0.32	0.14	0.37	0.07	0.04	0.24	0.45	0.24	0.05	0.08		0.04	0.29	0.19
Base thickness	Overall	8"	0.30	0.48	0.16	0.46	0.16	0.10	0.29	0.59	0.26	0.24	0.11		0.10	0.45	0.14
		12"	0.22	0.34	0.12	0.35	0.05	0.14	0.23	0.47	0.17	0.03	0.06		0.14	0.34	0.12
		16"	0.18	0.25	0.11	0.23	0.06	0.08	0.33	0.28	0.15	0.08	0.05		0.08	0.41	0.24
	ND	8"	0.35	0.59	0.16	0.56	0.15	0.09	0.34	0.77	0.24	0.21	0.11		0.09	0.51	0.17
		12"	0.24	0.38	0.12	0.37	0.05	0.15	0.29	0.52	0.13	0.03	0.07		0.15	0.43	0.15
	D	8"	0.22	0.29	0.17	0.29	0.17	0.11	0.22	0.29	0.30	0.28	0.09		0.11	0.34	0.10
		12"	0.21	0.29	0.13	0.34	0.06	0.12	0.14	0.41	0.22	0.04	0.06		0.12	0.20	0.07
		16"	0.18	0.25	0.11	0.23	0.06	0.08	0.33	0.28	0.15	0.08	0.05		0.08	0.41	0.24
Base type	Overall	DGAB	0.31	0.49	0.16	0.47	0.14	0.19	0.27	0.63	0.24	0.24	0.07		0.19	0.41	0.12
		ATB	0.18	0.27	0.11	0.28	0.06	0.05	0.27	0.34	0.18	0.05	0.06		0.05	0.38	0.15
		ATB/DGAB	0.26	0.38	0.15	0.39	0.10	0.07	0.30	0.51	0.19	0.04	0.14		0.07	0.39	0.21
	ND	DGAB	0.41	0.73	0.16	0.66	0.15	0.27	0.34	1.02	0.18	0.26	0.07		0.27	0.53	0.14
		ATB	0.23	0.37	0.10	0.36	0.06	0.02	0.32	0.47	0.18	0.06	0.06		0.02	0.50	0.14
		ATB/DGAB	0.26	0.38	0.15	0.39	0.10	0.07	0.30	0.51	0.19	0.04	0.14		0.07	0.39	0.21
	D	DGAB	0.25	0.35	0.16	0.35	0.14	0.13	0.22	0.40	0.28	0.23	0.08		0.13	0.33	0.11
		ATB	0.16	0.21	0.11	0.22	0.05	0.08	0.23	0.25	0.17	0.04	0.06		0.08	0.31	0.16
Drainage	Overall	ND	0.30	0.48	0.14	0.46	0.10	0.12	0.32	0.64	0.19	0.12	0.09		0.12	0.47	0.16
		D	0.20	0.28	0.14	0.29	0.10	0.10	0.23	0.33	0.22	0.14	0.07		0.10	0.32	0.14
	DGAB	ND	0.41	0.73	0.16	0.66	0.15	0.27	0.34	1.02	0.18	0.26	0.07		0.27	0.53	0.14
		D	0.25	0.35	0.16	0.35	0.14	0.13	0.22	0.40	0.28	0.23	0.08		0.13	0.33	0.11
	ATB	ND	0.23	0.37	0.10	0.36	0.06	0.02	0.32	0.47	0.18	0.06	0.06		0.02	0.50	0.14
		D	0.16	0.21	0.11	0.22	0.05	0.08	0.23	0.25	0.17	0.04	0.06		0.08	0.31	0.16

Base thickness: The effect of base thickness is marginally significant among sections located in WF zone, especially among sections built on fine-grained soils. Sections built with 16" base have shown the least cracking while sections with 8" or 12" base have shown the highest cracking.

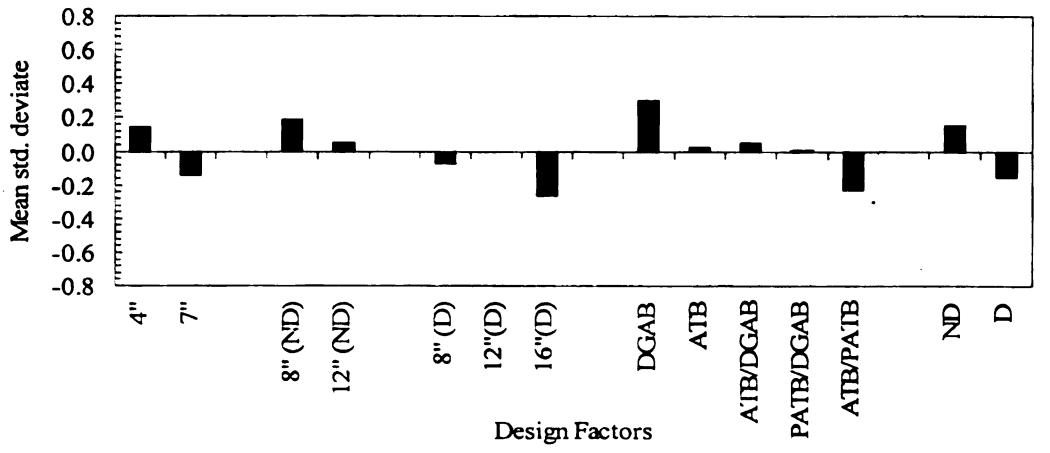
Base Type: On the whole, the effect of base type is statistically significant. Sections with ATB have exhibited the least cracking while sections with DGAB have shown the highest cracking. However, this effect is not operationally significant at this point in time.

Drainage: In general, sections with un-drained sections showing higher cracking than those with drainage. In addition, this effect is significant (statistically and operationally) among sections built on fine-grained subgrade and located in WF zone.

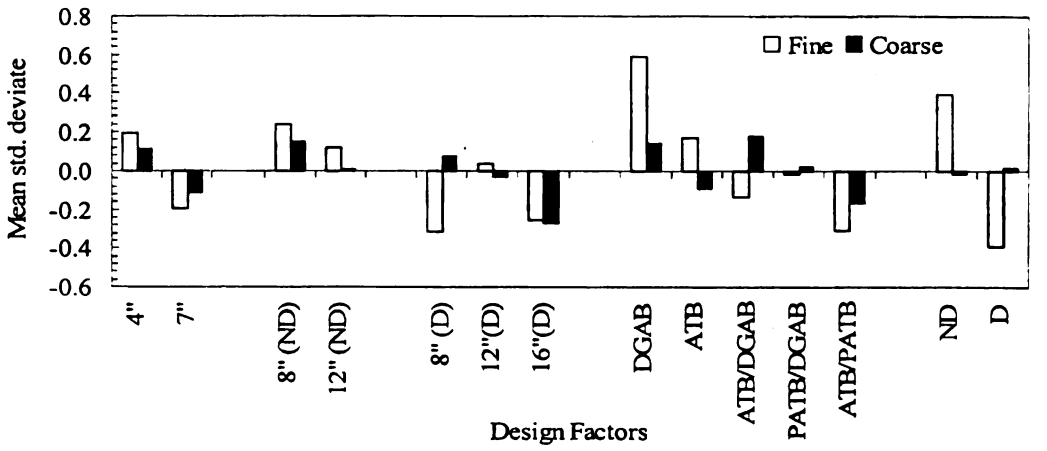
On the whole, at this point in time, sections in WF zone have shown higher cracking than those located in WNF zone indicating that transverse cracking is associated with low temperatures. Also, among drained pavements built on coarse-grained soils, those with ATB performed better than those with DGAB. However, among pavements with DGAB and built on fine-grained soils, those with drainage have shown significantly less transverse cracking than those without drainage. These effects were statistically significant and are of practical importance.

Longitudinal Cracking- WP

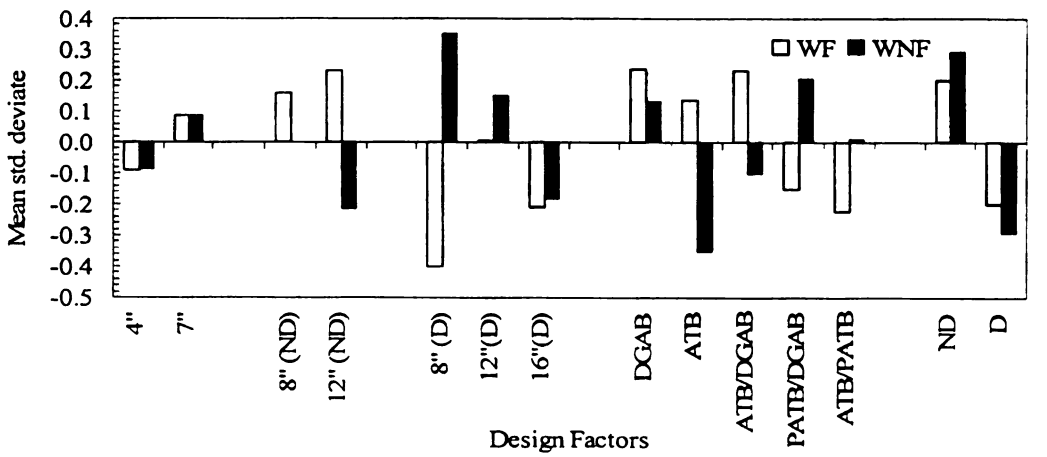
The effects of the design and site factors, in terms of standard deviate, are shown in Figure 6-26. The summary of p-values corresponding to the analyses performed to study the effects of design factors on longitudinal cracking-WP is presented in Table 6-33. The mean PI corresponding to each comparison presented in Table 6-33 is shown in Table 6-34.



(a) Overall



(b) By subgrade type



(c) By zone type

Figure 6-25 Effect of design factors on transverse cracking

Table 6-31 Summary of p-values for comparisons of standard deviates— Transverse cracking

Design Factor		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
				Fine	Coarse	WF	WNF	DF	DNF	WF		WNF		DF		DNF	
										F	C	F	C	F	C	F	C
HMA thickness		4" vs. 7"	0.100	0.170	0.330	0.610	0.310	0.310	0.097	0.850	0.630	0.180	0.940				
Base thickness	Overall	8" vs. 12" vs. 16"	0.480	0.410	0.255	0.342	0.038	0.405	0.880	0.100	0.700	0.200	0.240				
	ND	8" vs. 12"	0.391	0.865	0.168	0.083	0.022	0.830	0.840	0.013	0.713	0.123	0.135				
	D	8" vs. 12" vs. 16"	0.814	0.745	0.698	0.681	0.992	0.508	0.688	0.545	0.591	0.828	0.368				
Base type	Overall	DGAB vs. ATB vs. ATB/DGAB	0.008	0.240	0.012	0.353	0.240	0.025	0.013	0.660	0.120	0.345	0.370				
	ND	DGAB vs. ATB vs. ATB/DGAB	0.172	0.215	0.618	0.770	0.824	0.370	0.040	0.914	0.235	0.548	0.644				
	D	DGAB vs. ATB	0.003	0.630	0.001	0.240	0.474	0.040	0.049	0.374	0.120	0.794	0.318				
	All Bases	DGAB vs. ATB vs. ATB/DGAB vs. DGAB/PATB vs. ATB/PATB	0.011	0.006	0.037	0.542	0.285	0.107	0.010	0.167	0.185	0.329	0.434				
Drainage	Overall	Drainage vs. No-Drainage	0.089	0.004	0.910	0.330	0.037	0.440	0.530	0.008	0.330	0.140	0.160				
	DGAB	Drainage vs. No-Drainage	0.210	0.008	0.520	0.760	0.170	0.610	0.160	0.160	0.370	0.160	0.850				
	ATB	Drainage vs. No-Drainage	0.070	0.110	0.190	0.050	0.250	0.710	0.850	0.070	0.260	0.810	0.240				
N			120	48	72	24	48	24	24	12	12	24	24				

Note: Shaded cells show statistically significant at 90% or higher level of confidence.

Table 6-32 Summary of means of PI for transverse cracking

Design Factors		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
										WF		WNF		DF		DNF	
				F	C	WF	WNF	DF	DNF	F	C	F	C	F	C	F	C
HMA thickness		4"	4.5	6.1	3.4	12.5	1.4	1.5	5.7	18.9	6.2	2.8	0.0		1.5	0.0	11.4
		7"	3.7	5.7	2.4	14.5	0.6	0.8	2.1	20.7	8.2	1.0	0.3		0.8	0.0	4.1
Base thickness	Overall	8"	4.0	5.0	3.4	11.2	1.9	1.1	4.2	13.2	9.1	3.4	0.3		1.1	0.0	8.4
		12"	4.7	8.1	2.4	18.6	0.4	0.9	3.1	30.9	6.3	0.8	0.0		0.9	0.0	6.1
		16"	2.9	2.5	3.1	6.6	0.3	2.0	5.1	8.8	4.3	0.6	0.1		2.0	0.0	10.3
	ND	8"	4.7	7.1	3.1	11.5	2.8	0.7	5.7	18.1	4.8	5.1	0.5		0.7	0.0	11.4
		12"	5.6	11.2	1.8	24.4	0.4	1.1	1.7	43.4	5.5	0.7	0.0		1.1	0.0	3.4
	D	8"	3.1	2.0	3.8	10.8	0.5	1.7	1.9	5.9	15.6	1.0	0.0		1.7	0.0	3.7
		12"	3.3	3.5	3.2	9.9	0.5	0.5	5.1	12.1	7.7	0.9	0.0		0.5	0.0	10.2
		16"	2.9	2.5	3.1	6.6	0.3	2.0	5.1	8.8	4.3	0.6	0.1		2.0	0.0	10.3
Base type	Overall	DGAB	5.4	5.2	5.5	15.1	0.7	2.2	8.3	18.3	11.9	1.3	0.0		2.2	0.0	16.6
		ATB	2.5	4.7	1.1	10.5	0.5	0.4	0.8	17.4	3.6	0.7	0.3		0.4	0.0	1.6
		ATB/DGAB	4.8	10.6	1.0	17.0	3.1	0.4	0.5	29.8	4.2	6.3	0.0		0.4	0.0	0.9
	ND	DGAB	7.0	9.8	5.1	21.1	1.0	1.8	10.0	35.3	6.9	1.9	0.0		1.8	0.1	20.0
		ATB	3.6	7.1	1.4	15.8	0.7	0.5	0.7	27.2	4.4	0.6	0.8		0.5	0.0	1.3
		ATB/DGAB	4.8	10.6	1.0	17.0	3.1	0.4	0.5	29.8	4.2	6.3	0.0		0.4	0.0	0.9
	D	DGAB	4.3	2.2	5.8	11.1	0.5	2.5	7.2	7.0	15.3	0.9	0.0		2.5	0.0	14.3
		ATB	1.8	3.1	0.9	7.0	0.4	0.4	0.9	10.9	3.1	0.7	0.0		0.4	0.0	1.8
Drainage	Overall	ND	5.1	9.1	2.5	17.9	1.6	0.9	3.7	30.7	5.2	2.9	0.3		0.9	0.0	7.4
		D	3.1	2.6	3.4	9.1	0.4	1.4	4.0	8.9	9.2	0.8	0.0		1.4	0.0	8.1
	DGAB	ND	7.0	9.8	5.1	21.1	1.0	1.8	10.0	35.3	6.9	1.9	0.0		1.8	0.1	20.0
		D	4.3	2.2	5.8	11.1	0.5	2.5	7.2	7.0	15.3	0.9	0.0		2.5	0.0	14.3
	ATB	ND	3.6	7.1	1.4	15.8	0.7	0.5	0.7	27.2	4.4	0.6	0.8		0.5	0.0	1.3
		D	1.8	3.1	0.9	7.0	0.4	0.4	0.9	10.9	3.1	0.7	0.0		0.4	0.0	1.8

The effects of design factors on longitudinal cracking-WP, based on this analysis, are presented below:

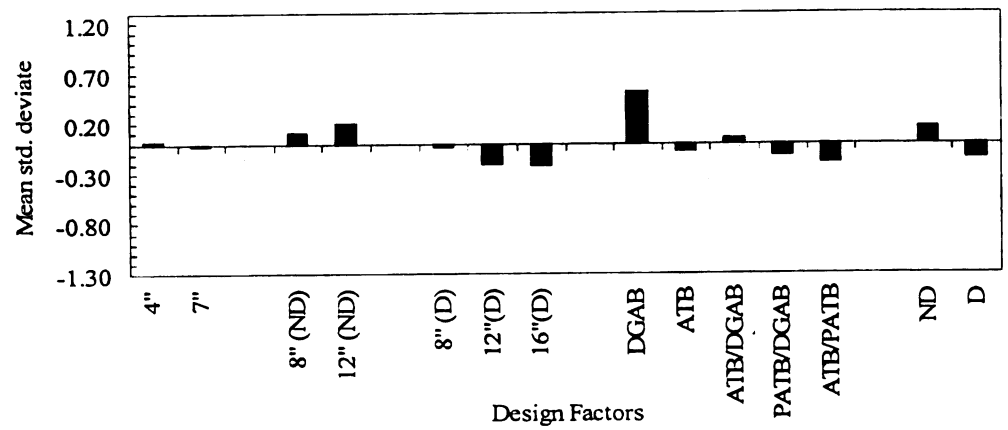
HMA thickness: The effect of HMA thickness on longitudinal cracking-WP is inconclusive. Sections with 4" HMA surface layer and sections with 7" HMA surface layer have shown comparable levels of longitudinal cracking-WP.

Base thickness: The effect of base thickness on longitudinal cracking-WP is inconclusive. In general, all sections have shown comparable performance.

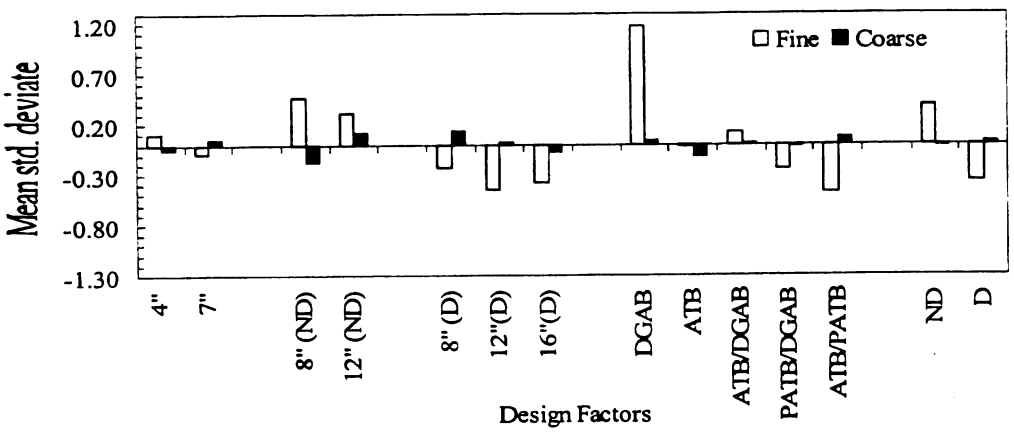
Base Type: The effect of base type on longitudinal cracking-WP is inconclusive. However, on average, sections built on ATB have exhibited least cracking compared to other sections, especially among sections built on fine-grained soils.

Drainage: In general, the effect of drainage is statistically significant with un-drained sections showing higher cracking than those with drainage. However, this effect is not operationally significant. This effect is statistically and operationally significant among sections built on fine-grained soils, especially among sections located in WF zone. In addition, drainage seems to be more effective for sections with DGAB.

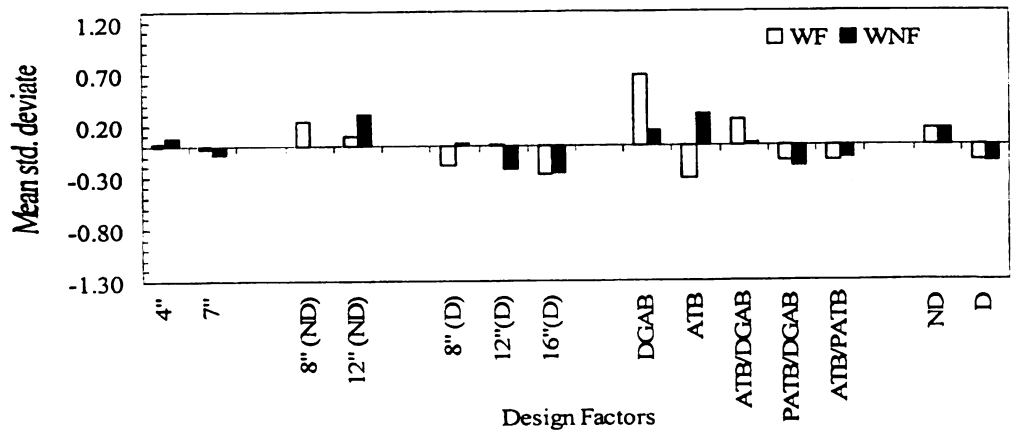
On the whole, at this point in time, sections in WF zone have exhibited much higher cracking than those in other climatic zones. Among pavements built on fine-grained soils, those built with DGAB have shown higher longitudinal cracking-WP and those built with ATB have shown the least longitudinal cracking-WP. This main effect of base type was statistically and operationally significance. Also among pavements built on fine-grained soils, drainage has a significant effect on longitudinal cracking and this effect is more pronounced (significant) among pavements built with DGAB. This effect is statistically significant and is of practical importance.



(a) Overall



(b) By subgrade type



(c) By zone type

Figure 6-26 Effect of design factors on longitudinal cracking-WP

Table 6-33 Summary of p-values for comparisons of standard deviates— Longitudinal cracking-WP

Design Factor		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
				Fine	Coarse	WF	WNF	DF	DNF	WF		WNF		DF		DNF	
										F	C	F	C	F	C	F	C
HMA thickness		4" vs. 7"	0.851	0.420	0.630	0.850	0.560	0.127	0.425	0.440	0.400	.730	0.290				
Base thickness	Overall	8" vs. 12" vs. 16"	0.451	0.237	0.813	0.690	0.620	0.685	0.780	0.512	0.570	0.770	0.790				
	ND	8" vs. 12"	0.703	0.720	0.379	0.787	0.499	0.550	0.653	0.754	0.969	0.934	0.325				
	D	8" vs. 12" vs. 16"	0594	0.362	0.839	0.803	0.558	0.248	0.929	0.480	0.497	0.364	0.439				
Base type	Overall	DGAB vs. ATB vs. ATB/DGAB	0.281	0.067	0.990	0.410	0.920	0.470	0.230	0.093	0.680	0.790	0.620				
	ND	DGAB vs. ATB vs. ATB/DGAB	0.172	0.045	0.923	0.240	0.890	0.321	0.256	0.193	0.097	0.501	0.718				
	D	DGAB vs. ATB	0.668	0.052	0.757	0.990	0.745	0.354	0.481	0.148	0.451	0.321	0.650				
	All Bases	DGAB vs. ATB vs. ATB/DGAB vs. DGAB/PATB vs. ATB/PATB	0.058	0.000	0.988	0.305	0.753	0.085	0.349	0.030	0.698	0.181	0.893				
Drainage	Overall	Drainage vs. No-Drainage	0.049	0.001	0.815	0.297	0.220	0.045	0.650	0.052	0.290	0.063	0.943				
	DGAB	Drainage vs. No-Drainage	0.020	0.001	0.850	0.130	0.340	0.080	0.710	0.080	0.450	0.110	0.510				
	ATB	Drainage vs. No-Drainage	0.650	0.090	0.630	0.700	0.360	0.190	0.180	0.510	0.480	0.170	0.950				
N			152	68	84	44	60	24	24	32	12	24	36				

Note: Shaded cells show statistically significant at 90% or higher level of confidence.

Table 6-34 Summary of means of PI for longitudinal cracking-WP

Design Factors		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
										WF		WNF		DF		DNF	
				Fine	Coarse	WF	WNF	DF	DNF	F	C	F	C	F	C	F	C
HMA thickness		4"	13.9	17.3	11.2	22.3	2.8	0.9	39.5	30.7	1.5	2.8	2.8		0.9	12.7	66.4
		7"	14.3	17.9	11.3	24.6	2.7	2.6	35.4	31.4	5.4	5.7	0.7		2.6	4.4	66.4
Base thickness	Overall	8"	13.8	18.0	10.5	23.2	2.5	1.7	37.9	32.5	0.8	3.5	1.9		1.7	12.3	63.5
		12"	14.3	18.5	10.8	24.7	3.7	2.2	33.0	31.3	6.3	5.7	2.3		2.2	8.2	57.8
		16"	14.5	14.7	14.3	21.3	1.3	0.6	47.6	27.5	2.7	2.7	0.3		0.6	0.0	95.2
	ND	8"	14.3	21.0	9.2	26.5	3.0	1.9	34.9	37.4	1.0	4.5	1.9		1.9	15.9	54.0
		12"	15.0	22.9	8.6	27.8	5.2	3.7	27.5	37.8	1.0	7.6	3.6		3.7	13.7	41.3
	D	8"	12.9	13.6	12.4	18.5	1.8	1.4	42.3	25.7	0.6	1.9	1.8		1.4	6.8	77.8
		12"	13.2	12.2	14.0	20.6	1.3	0.0	41.3	22.6	14.3	2.7	0.4		0.0	0.0	82.6
		16"	14.5	14.7	14.3	21.3	1.3	0.6	47.6	27.5	2.7	2.7	0.3		0.6	0.0	95.2
Base type	Overall	DGAB	15.3	19.8	11.7	25.2	2.1	2.5	44.1	35.0	1.6	3.8	0.9		2.5	15.3	72.9
		ATB	12.0	14.0	10.3	19.5	2.5	1.0	31.5	24.1	5.9	5.2	0.7		1.0	1.2	61.7
		ATB/DGAB	16.8	22.2	12.5	30.9	5.2	1.9	35.8	42.5	1.8	3.1	6.7		1.9	9.8	61.8
	ND	DGAB	16.9	27.7	9.2	32.2	3.2	5.5	39.5	47.9	0.9	5.5	1.6		5.5	31.5	47.5
		ATB	10.7	17.1	5.2	20.1	3.9	1.0	18.3	26.7	0.3	9.7	0.1		1.0	3.1	33.5
		ATB/DGAB	16.8	22.2	12.5	30.9	5.2	1.9	35.8	42.5	1.8	3.1	6.7		1.9	9.8	61.8
	D	DGAB	14.2	15.2	13.5	21.4	1.3	0.4	47.2	28.6	2.1	2.7	0.4		0.4	4.5	89.9
		ATB	12.8	11.9	13.7	19.1	1.6	0.9	40.3	22.3	9.7	2.1	1.2		0.9	0.0	80.5
Drainage	Overall	ND	14.7	22.0	8.9	27.2	4.1	2.8	31.2	37.6	1.0	6.1	2.8		2.8	14.8	47.6
		D	13.5	13.5	13.6	20.2	1.5	0.7	43.7	25.3	5.9	2.4	0.8		0.7	2.3	85.2
	DGAB	ND	16.9	27.7	9.2	32.2	3.2	5.5	39.5	47.9	0.9	5.5	1.6		5.5	31.5	47.5
		D	14.2	15.2	13.5	21.4	1.3	0.4	47.2	28.6	2.1	2.7	0.4		0.4	4.5	89.9
	ATB	ND	10.7	17.1	5.2	20.1	3.9	1.0	18.3	26.7	0.3	9.7	0.1		1.0	3.1	33.5
		D	12.8	11.9	13.7	19.1	1.6	0.9	40.3	22.3	9.7	2.1	1.2		0.9	0.0	80.5

Longitudinal Cracking- NWP

The effects of the design and site factors, in terms of standard deviate, are shown in Figure 6-27. The summary of p-values corresponding to the analyses performed to study the effects of design factors on longitudinal cracking-NWP is presented in Table 6-35. The mean PI corresponding to each comparison presented in Table 6-35 is shown in Table 6-36. The effects of design factors on longitudinal cracking-NWP, based on this analysis, are presented below:

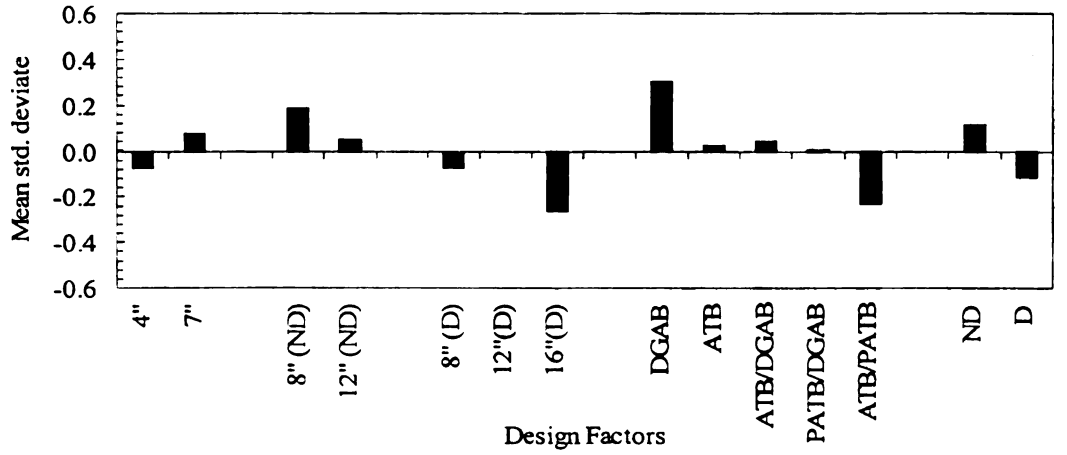
HMA thickness: The effect of HMA surface thickness is not significant. Comparable amount of cracking occurred in sections with "thin" and "thick" HMA surface.

Base thickness: The effect of base thickness is not significant. On average, sections with 16" base have shown slightly lesser cracking than other sections.

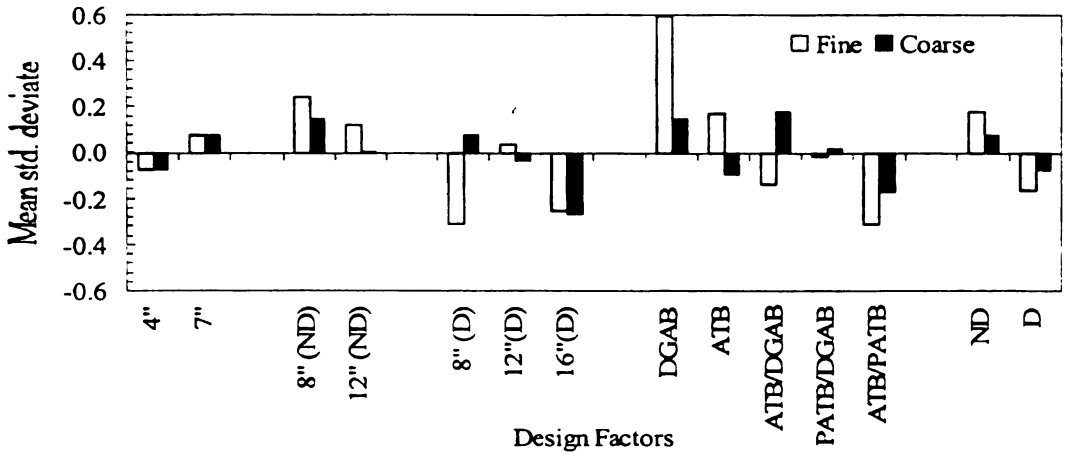
Base Type: The effect of base type is not significant. Comparable amount of cracking occurred in all sections, irrespective of base type.

Drainage: In general, sections with un-drained sections showing higher cracking than those with drainage. However, this effect is not significant. Also, this effect is more apparent among sections located in WF zone.

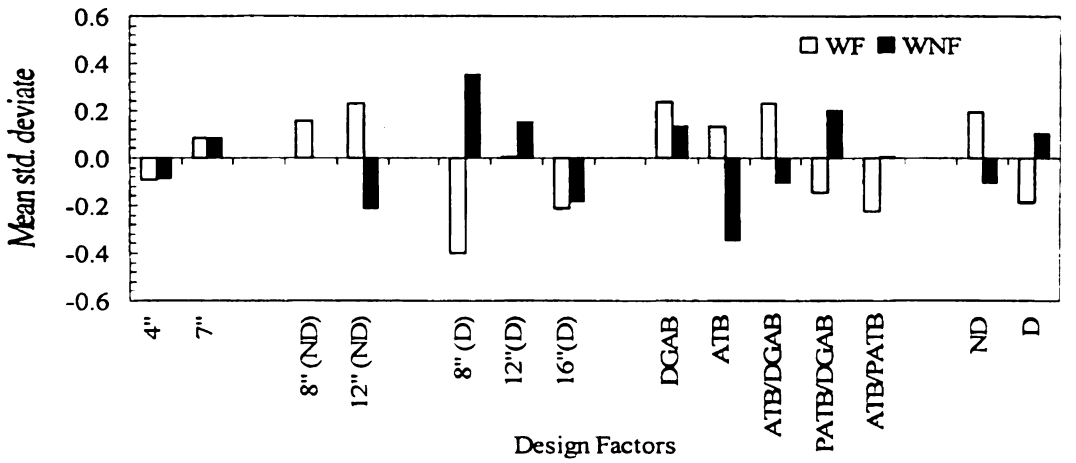
On the whole, at this point in time, it seems that longitudinal cracking-NWP is not a "structural" distress. It may be more affected by climate. It may be noted that the amount of longitudinal cracking-NWP is higher among sections located in "freeze" climate.



(a) Overall



(b) By subgrade type



(c) By zone type

Figure 6-27 Effect of design factors on longitudinal cracking-NWP

Table 6-35 Summary of p-values for comparisons of standard deviates— Longitudinal cracking-NWP

Design Factor		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
				Fine	Coarse	WF	WNF	DF	DNF	WF		WNF		DF		DNF	
										F	C	F	C	F	C	F	C
HMA thickness		4" vs. 7"	0.304	0.510	0.440	0.430	0.490	0.120	0.230	0.250	0.940	0.460	0.790				
Base thickness	Overall	8" vs. 12" vs. 16"	0.220	0.540	0.350	0.490	0.610	0.300	0.320	0.620	0.780	0.840	0.250				
	ND	8" vs. 12"	0.545	0.733	0.615	0.831	0.594	0.096	0.440	0.617	0.366	0.942	0.388				
	D	8" vs. 12" vs. 16"	0.467	0.513	0.513	0.523	0.377	0.919	0.584	0.146	0.988	0.415	0.090				
Base type	Overall	DGAB vs. ATB vs. ATB/DGAB	0.280	0.390	0.420	0.620	0.500	0.760	0.042	0.960	0.430	0.630	0.650				
	ND	DGAB vs. ATB vs. ATB/DGAB	0.524	0.278	0.702	0.963	0.621	0.857	0.014	0.740	0.650	0.679	0.327				
	D	DGAB vs. ATB	0.196	0.268	0.451	0.792	0.540	0.569	0.180	0.840	0.877	0.456	0.711				
	All Bases	DGAB vs. ATB vs. ATB/DGAB vs. DGAB/PATB vs. ATB/PATB	0.235	0.157	0.743	0.540	0.701	0.465	0.052	0.587	0.764	0.743	0.311				
Drainage	Overall	Drainage vs. No-Drainage	0.104	0.120	0.420	0.082	0.400	0.077	0.360	0.150	0.320	0.540	0.110				
	DGAB	Drainage vs. No-Drainage	0.220	0.190	0.660	0.330	0.880	0.380	0.330	0.280	0.790	0.580	0.420				
	ATB	Drainage vs. No-Drainage	0.210	0.090	0.780	0.240	0.370	0.130	0.007	0.250	0.670	0.430	0.084				
N			184	76	108	76	60	24	24	40	36	24	36				

Note: Shaded cells show statistically significant at 90% or higher level of confidence.

Table 6-36 Summary of means of PI for longitudinal cracking-NWP

Design Factors		Comparison	Overall	By subgrade		By climatic zone				By subgrade and zone							
										WF		WNF		DF		DNF	
				Fine	Coarse	WF	WNF	DF	DNF	F	C	F	C	F	C	F	C
HMA thickness		4"	47.7	35.9	55.8	78.0	14.9	77.2	6.7	59.9	97.2	13.7	15.7		77.2	4.1	9.3
		7"	49.0	40.8	54.9	77.7	15.6	84.1	4.3	66.6	90.5	15.7	15.5		84.1	0.8	7.8
Base thickness	Overall	8"	48.9	37.4	56.4	79.9	16.8	84.2	4.0	65.2	93.6	14.9	18.1		84.2	4.2	3.8
		12"	49.0	40.8	55.1	79.1	13.7	77.9	9.1	66.8	93.9	13.4	13.9		77.9	1.7	16.6
		16"	45.4	35.0	53.5	70.5	15.1	78.6	0.0	52.6	94.3	17.3	13.6		78.6	0.0	0.1
	ND	8"	52.4	47.7	55.5	88.0	13.9	89.3	4.6	81.2	94.8	19.9	9.9		89.3	3.0	6.3
		12"	46.9	35.9	54.6	77.9	9.8	78.1	10.1	61.1	96.6	10.4	9.4		78.1	2.8	17.4
	D	8"	43.6	20.4	57.7	66.7	21.2	76.6	3.1	36.6	91.8	7.4	30.3		76.6	6.1	0.0
		12"	52.2	47.4	55.8	80.8	19.6	77.7	7.7	74.0	89.8	18.0	20.7		77.7	0.0	15.5
		16"	45.4	35.0	53.5	70.5	15.1	78.6	0.0	52.6	94.3	17.3	13.6		78.6	0.0	0.1
Base type	Overall	DGAB	47.8	34.2	56.6	77.0	16.3	79.5	10.2	58.5	94.3	14.9	17.2		79.5	4.6	15.7
		ATB	48.9	41.6	54.4	77.9	15.4	80.2	2.7	65.7	93.4	16.2	14.8		80.2	0.7	4.7
		ATB/DGAB	48.3	39.6	54.6	79.5	12.3	85.0	0.7	67.2	93.7	10.3	13.5		85.0	1.3	0.0
	ND	DGAB	47.0	28.8	57.2	81.9	11.1	81.0	15.6	56.8	98.7	12.4	10.2		81.0	5.5	25.7
		ATB	53.1	52.7	53.3	86.6	12.2	85.2	5.9	80.5	94.7	22.7	5.2		85.2	1.8	9.9
		ATB/DGAB	48.3	39.6	54.6	79.5	12.3	85.0	0.7	67.2	93.7	10.3	13.5		85.0	1.3	0.0
	D	DGAB	48.3	37.0	56.2	74.4	19.8	78.5	6.6	59.1	91.4	16.6	21.9		78.5	4.1	9.1
		ATB	46.1	33.8	55.1	71.9	17.5	76.8	0.6	55.0	92.6	11.9	21.2		76.8	0.0	1.3
Drainage	Overall	ND	49.6	41.6	55.0	82.8	11.9	83.7	7.4	70.6	95.7	15.2	9.7		83.7	2.9	11.9
		D	47.2	35.4	55.7	73.1	18.6	77.6	3.6	56.9	92.0	14.2	21.6		77.6	2.0	5.2
	DGAB	ND	47.0	28.8	57.2	81.9	11.1	81.0	15.6	56.8	98.7	12.4	10.2		81.0	5.5	25.7
		D	48.3	37.0	56.2	74.4	19.8	78.5	6.6	59.1	91.4	16.6	21.9		78.5	4.1	9.1
	ATB	ND	53.1	52.7	53.3	86.6	12.2	85.2	5.9	80.5	94.7	22.7	5.2		85.2	1.8	9.9
		D	46.1	33.8	55.1	71.9	17.5	76.8	0.6	55.0	92.6	11.9	21.2		76.8	0.0	1.3

6.6.4 *Effect of Experimental Factors on Pavement Response*

This section of the report is a discussion of the results from analyses of FWD data (pavement response) of the SPS-1 sections. Three parameters were chosen for analyses – peak deflection under FWD load (d_0), far-sensor deflection (d_6), and AREA. The peak deflection under FWD load is indicative of the "overall capacity" of the pavement structure while the far-sensor deflection is illustrative of the subgrade "strength". The AREA is the area under the first three feet of the deflection basin. The computational details regarding the AREA can be found in the reference "LTPP Data Analysis: Feasibility of Using FWD Deflection Data to Characterize Pavement Construction Quality", NCHRP Project 20-50 [8], by Richard N. Stubstad, October 2002. The AREA is indicative of stiffness of the upper layers of the pavement relative to the stiffness of the underlying layers. Higher the AREA higher is the stiffness of upper layers in relation to underlying layers.

An ANOVA was conducted with the peak deflection under the FWD load plate (do), the far sensor deflection at 60 inches from the FWD load (d6) and the AREA as the dependent variables. All the response parameters have been calculated using the initial deflections of the test sections.

It should be noted that the pavement surface temperature at the time of testing was taken as a covariate along with the age at the time of testing and variability in the HMA and base layer thicknesses. The natural logarithmic transformation has been applied to the three response indicators to fulfill the ANOVA assumptions. The results from ANOVA are summarized in Tables 6-37 and 6-38. The brief discussion of the results is given below:

Peak Deflection under FWD Load (d_0)

When only design factors were considered by blocking the site effects, interactions between HMA thickness and base type (p=0.043), base thickness and base type (p=0.000), base type and drainage (p=0.000), have shown significant effects on the peak deflection.

Among the pavement sections built on DGAB, those with 4-inch HMA thickness have shown higher peak deflection than those with 7-inch HMA thickness. Also as expected, thicker bases for each base type have helped in reducing the peak deflection. However, the reduction in peak deflection was more significant in the case of ATB and ATB/DGAB base types. Furthermore, among pavement sections built on DGAB, those with drainage have shown lesser peak deflections than those without drainage. As mentioned before, DGAB with drainage refers to PATB/DGAB with drainage. It was assumed in the experiment, to study the effect of drainage among sections with DGAB that PATB has the same structural "strength" as DGAB. This assumption appears to be invalid from above result.

When all the site and design factors were considered simultaneously along with the two-way interactions between the main factors, the interaction between subgrade soil and climatic zone (p=0.005) was found to have a very significant effect on the peak deflection.

Among the pavement sections located in "wet" climates, those built on fine-grained subgrade soils have a significantly higher peak deflection (d_0) as compared to those built on coarse-grained subgrade soils. This effect is more prominent on pavements located in WNF zone than those located in WF zone.

Far Sensor Deflection (d_6)

When only design factors were considered by blocking the site effects, the main effects of base type (p=0.000), base thickness (p=0.005) and drainage (p=0.012) have significant effects on the far-sensor deflection (60-inches away from the center of the load).

Pavement sections built on DGAB bases have shown higher far-sensor deflections than those built on other base types. Pavement sections constructed on 8-inch bases have also shown significantly higher far-sensor deflections than those built on 12- or 16-inch bases. Furthermore, sections built with drainage have lesser far-sensor deflections than those without drainage. This effect can be attributed to PATB as the effect of drainage and the effect of PATB cannot be separated (confounded).

When all the site and design factors were considered simultaneously along with the two-way interactions between the main factors, an interaction between subgrade type and climatic zone (p=0.000) was found to have a significant effect on the far-sensor deflections. Among pavement sections located in "wet" climate, those built on fine-grained subgrade soils have higher deflections than those built on coarse-grained soils. This effect is significant among sections located in WNF zone.

The ANOVA results also show that HMA thickness and pavement mid depth temperature do not have a significant affect on the far-sensor deflection. The results seem reasonable, as this deflection (d_6) represent the subgrade strength, which is independent of the HMA thickness and pavement temperature.

AREA

When only design factors are considered by blocking the site effects, the interactions between HMA thickness and base type (p=0.002), base thickness and base type (p=0.03), and, drainage and base type (p=0.000) have shown significant effects on AREA.

Among pavement sections built on DGAB, those with "thin" HMA surface layer have significantly lower AREA values compared to those with "thick" HMA surface layer, implying that the upper layers of these pavement sections are "less stiff". Also, the increase in HMA thickness from 4 to 7 inches on ATB does not significantly increase AREA.

For sections built on DGAB, increasing base thickness from 8 to 12 inches has not shown a significant effect on AREA; however a two-fold increase in base thickness (from 8 to16 inch) has shown a significant increase in AREA. Also, base thickness does not seem to have a significant effect on AREA in pavement sections with ATB bases.

Among the pavement sections constructed on DGAB, those with drainage have a significantly different AREA compared to those without drainage; test sections with drainage have higher AREA, implying higher stiffness. This indicates that the structural capacity of the PATB layer is somewhat higher than that of the DGAB.

When all the site and design factors were considered simultaneously along with the two-way interactions between the main factors, the interaction between subgrade type and climatic zone (p=0.000) was found to have a very significant effect on AREA. Among the pavement sections located in WNF zone, those built on fine-grained subgrade soils have significantly higher AREA values than those built on coarse-grained soils.

However, in the case of sections located in WF zone, this effect is not significant indicating that AREA could be independent of the subgrade soil type.

Table 6-37 Summary of p-values from ANOVA for determining the effect of design factors on flexible pavement response — Overall

Design Factor	Performance Measures		
	Peak Deflection (d_o)	Far Deflection (d_6)	AREA
HMA thickness	0.000	0.560	0.000
Base type	0.000	0.000	0.000
Base thickness	0.000	0.005	0.214
Drainage	0.590	0.012	0.000
Mid depth temperature	0.000	0.738	0.000
Site (blocked)	0.000	0.000	0.000
	R^2=0.884 N=210	R^2=0.864 N=210	R^2=0.854 N=210

Table 6-38 Summary of p-values from ANOVA for determining the effect of site factors on flexible pavement response — Overall

Design Factor	Performance Measures		
	Peak Deflection (d_o)	Far Deflection (d_6)	AREA
Subgrade	0.000	0.000	0.353
Zone	0.000	0.000	0.000
Subgrade*Zone	0.005	0.005	0.005
Mid depth temperature	0.000	0.495	0.000
	R^2=0.865 N=210	R^2=0.658 N=210	R^2=0.682 N=210

6.7 APPARENT RELATIONSHIP BETWEEN RESPONSE AND PERFORMANCE

In this section of the report the observations regarding apparent relationships between flexible pavement response (FWD testing) and performance are presented. The usefulness of such relationships can be divided into two categories:

- Explanatory: To provide an explanatory information for a given performance trend. For example, a relationship between AC rutting and the farthest sensor deflection would indicate that rutting is related to the subgrade soil.
- Predictive: To provide a predictive capability of the future level of a given performance measure. For example, the initial high average deflection of a section may explain its future cracking and rutting (due to subgrade) performance.

Explanatory relationships were established using multiple regressions on data from all the test sections in the experiment. Predictive relationships were established based on bivariate correlation analyses at the site level, and using scatter plots on data from all sections. The DLR data were used for predictive relationships regarding the instrumented sections in Ohio.

6.7.1 Overall Analysis—Explanatory Relationships

In this section, the entire population of the SPS-1 experiment was used to seek apparent explanatory relationships between response and performance. This analysis was done irrespective of the experimental design matrix layout, since pavement response should reflect the effects of the various structural designs. In other words, the analysis spans over all the SPS-1 sections, as opposed to it being restricted to individual structural designs. The spatial variability of the deflections and deflection-based indices (within a section) was considered by taking the 95th percentile within each section. As deflection

on all sections was measured during different seasons and times, the impact of temperature and moisture conditions cannot be ignored. Additionally the deflection and deflection-based parameters (SCI, BDI etc.,) are influenced by variety of factors, such as:

- Asphalt temperature (at mid depth)
- Thickness of asphalt layer
- The layer moduli of various layers and overall pavement structure
- Subgrade strength
- Apparent stiff layer depth
- Pavement distresses etc.

To consider the effect of various variables on the response at the same time, the multiple linear regression technique was used. The pavement response parameters (surface deflection (d_0), SCI and BDI) were taken as dependent variables and all other variables (temperature, asphalt thickness, subgrade strength and pavement distresses) were considered as independent variables. As expected, the surface deflection is significantly correlated with the asphalt layer thickness, mid-depth asphalt temperature, and deflection at the outer most sensors that represents the subgrade strength. Furthermore, fatigue cracking, longitudinal cracking-WP and transverse cracking in all the sections have shown statistically significant relationships with the surface deflection. The results of multiple regression analyses within each zone have also shown that, on average, fatigue and transverse cracking has a significant positive effect on the surface deflection. An example of such multiple regression models is given by the following equation:

$$\ln(d_o) = 3.694 + 0.17T - 0.085H_{ac} - 0.031Age + 0.602\ln(d_6) + 0.001AC + 0.004TC + 0.001LCWP$$

(R^2=0.856, *SE=0.269*)
where: *ln* is the natural logarithm
T is the mid-depth asphalt concrete temperature (C)

H_{ac} is the HMA layer thickness
Age is the age of the pavement section at the time of FWD testing
AC is alligator cracking (*sqm*) at the time of testing
TC is transverse cracking length (m)
LCNWP is longitudinal cracking not in the wheel path (m)

The sensitivity analysis of the regression model for overall SPS-1 database was performed to observe the explanatory relationships between various independent variables with the surface deflection (d_0) under the FWD load plate. The following conclusions can be made from these results:

- The effect of asphalt thickness on the measured surface deflection (d_0) is very significant (*p=0.000*). The thicker the HMA layer, the lower the deflection will be.
- Mid-depth asphalt temperature at the time of testing has a significant positive effect on d_0 (*p=0.000*).
- The age of the pavement has indicated a negative effect on d_0 (*p=0.000*). Aging effect on HMA pavement may cause the stiffening of asphalt thus may reduce the deflections.
- The higher the "subgrade" deflection, d6 (deflection at the outer most sensor, or 60 inches in this case), the higher d_0 will be (*p=0.000*).
- Fatigue cracking (*p=0.000*), longitudinal cracking-WP (*p=0.006*) and longitudinal cracking-NWP (*p=0.012*) have a significant positive effect on d_0; i.e., higher cracking will cause an increase in d_0.
- Similarly, transverse cracking has a significantly positive effect on d_0 (*p=0.001*).

6.7.2 Site Level Analyses— Predictive Relationships

This section summarizes the findings regarding predictive relationships between initial response (FWD deflection or deflection-based indices) and future pavement

performance (cracking, rutting and roughness), at the site level. Various deflection-based indices [8, 9] were calculated based on the individual deflection basins for each section; these indices include:

- AREA (the area under first three feet of deflection basin),
- SCI— Surface Curvature Index, (d0 – d12),
- BDI— Base Damage Index, [9](d12 – d24),
- d36 — (d0-d36),
- do (peak deflection under the load),
- d6 (farthest deflection at 60 inches away from the load),
- ES (effective stiffness of upper (bound) layer), and
- Eg (subgrade modulus calculated from surface deflection at 36 inches from the load).

Bivariate correlation analyses between response parameters (deflections or deflection basin parameters) and performance (cracking, rutting and roughness) were conducted for all the states within SPS-1 experiment. The latest performance for each section within the SPS-1 experiment was used in these analyses. The effect of temperature on the measured deflection was taken into account by applying a temperature correction [10] . It is to be noted that for a site age, traffic, construction, material properties and environment are the same and thus this provides a good opportunity for seeking apparent relationships.

Figure 6-28 and Figure 6-29 are examples of bivariate relationships between SCI and AREA with fatigue cracking. The site in the state of Kansas (20) was chosen for this example because of high extent of cracking at the site. It can be seen that for the sections in this site, initial SCI and initial AREA have a slight association ($\rho = 0.4$) with the future fatigue cracking, in that higher the SCI or lower the AREA, higher is the cracking. Similarly, Figure 6-31 is the relationship between BDI and future rutting for the same

site. In this case, BDI has a strong correlation ($\rho = 0.77$) with future rutting i.e. sections that had higher initial BDI have higher rutting at a later stage. Also, Figure 6-30 shows the variation in future roughness (IRI) as a function of BDI. In this case, BDI has a correlation ($\rho = 0.42$) with the future roughness of all the pavement sections for this site (KS (20)).

Figure 6-32 and Figure 6-33 show relationships of roughness and rut depth with BDI and AREA for the sections in the site of OH (39). Strong correlations ($\rho = 0.85$ each) were observed between future roughness and rut depth. Sections that had higher initial BDI had higher future roughness, and sections with lower AREA had higher future rutting.

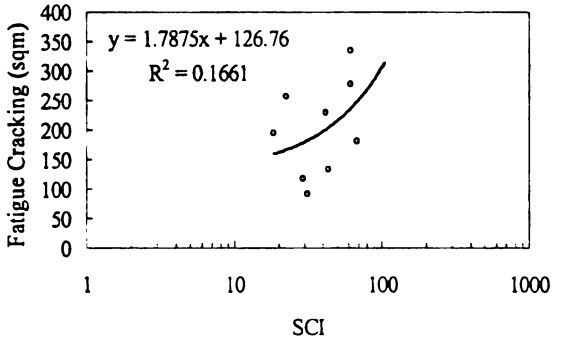


Figure 6-28 Fatigue cracking and SCI relationship— State (20) Kansas

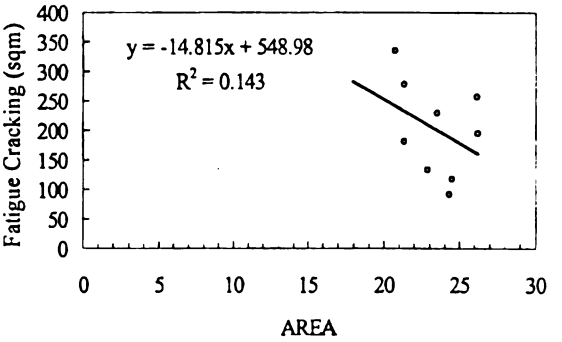


Figure 6-29 Fatigue cracking and AF relationship— State (20) Kansas

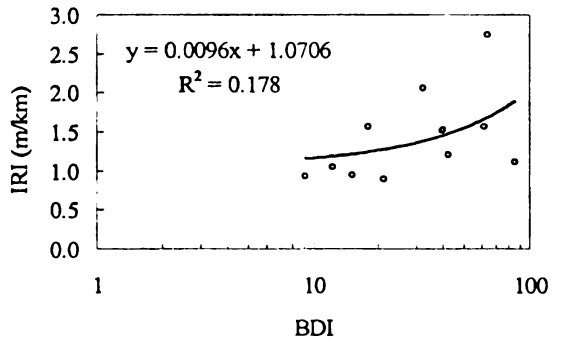


Figure 6-30 Roughness and BDI relationship— State (20) Kansas

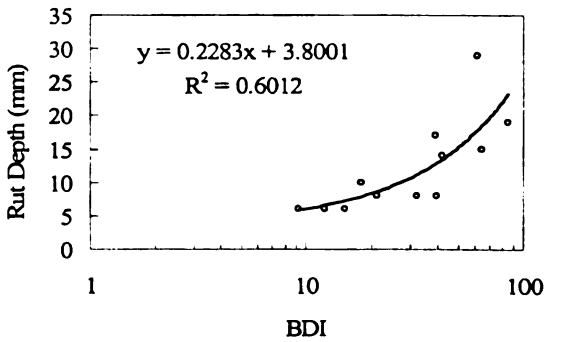


Figure 6-31 Rut depth and BDI relationship— State (20) Kansas

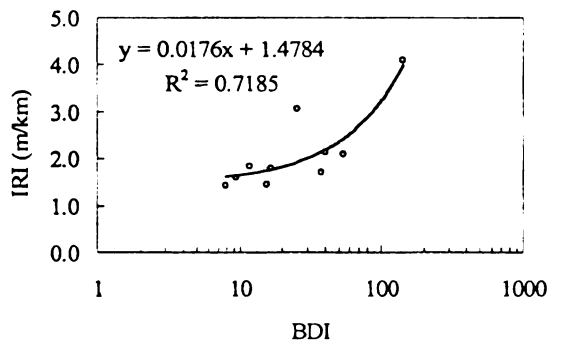


Figure 6-32 Roughness and BDI relationship— State (39) Ohio

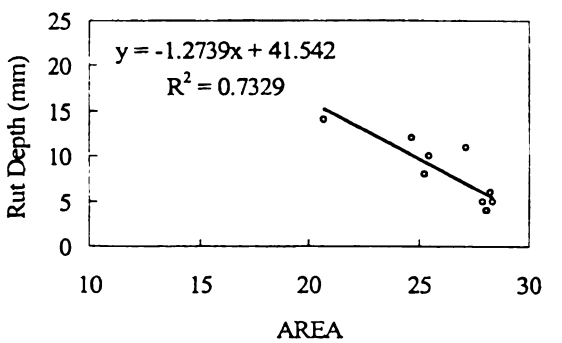


Figure 6-33 Rut depth and AF relationship— State (39) Ohio

Tables 6-39 through 6-41 are summaries of correlation coefficients from the bivariate analyses for three performances (fatigue cracking, rutting and roughness) and various deflection parameters between all the sites.

The results show that fifteen out of seventeen sites have a consistent trend of relationship between AREA, BDI, d_o and future cracking. Also, fourteen out of seventeen sites have a positive association between SCI and fatigue cracking. On an average, AREA, SCI and BDI have reasonable associations with fatigue cracking for all the sites in SPS-1 experiment. Sections that have higher fatigue cracking had higher initial SCI or BDI, and lower AREA.

The deflection basin parameters do not have a consistent association with rutting across the sites (see Table 6-40). This inconsistency may be explained in light of different rutting mechanisms for flexible pavements i.e., structural or asphalt mix rutting.

Consistent trends were observed only between BDI and future roughness across most of the sites (15 out of 17 sites) in the SPS-1 experiment (see Table 6-41). Sections that had higher BDI have higher roughness.

Apparent relationships between AREA and various performance measures (fatigue cracking, rutting and roughness) were found to be significant within sites that have shown considerable distress. Higher AREA means stiffer upper layers of a pavement. Sections that had higher AREA exhibited lesser cracking, rutting and roughness. Based on the magnitude of correlation coefficients, it was also found that sections that had stiffer bound layers are more likely to exhibit cracking than (structural) rutting.

Table 6-39 Summary of correlations for deflections and DBPs with fatigue cracking

State		*Area*	*ES/Esg*	*ES*	*Esg*	*d0*	*d6*	*SCI*	*BDI*	Zone	SG
31	Nebraska	-0.79	-0.66	-0.64	-0.37	0.90	0.43	0.94	0.91	WF	F
26	Michigan	0.48	0.59	0.42	0.21	-0.42	-0.03	-0.46	-0.45	WF	F
19	Iowa	-0.45	-0.11	-0.09	0.05	0.41	0.15	0.55	0.49	WF	F
20	Kansas	-0.38	-0.26	-0.25	-0.14	0.33	-0.01	0.41	0.30	WF	F
39	Ohio	-0.66	-0.44	-0.44	-0.43	0.58	0.20	0.63	0.63	WF	F
55	Wisconsin	-0.46	-0.38	-0.37	-0.02	0.38	0.04	0.51	0.22	WF	C
10	Delaware	-0.93	-0.78	-0.65	-0.04	0.72	-0.03	0.93	0.73	WF	C
5	Arkansas	-0.07	-0.02	-0.19	-0.54	0.34	0.59	0.17	0.34	WF	C
51	Virginia	-0.72	-0.57	-0.58	-0.32	0.75	0.04	0.79	0.75	WNF	F
1	Alabama	-0.79	-0.68	-0.64	-0.08	0.72	-0.27	0.75	0.73	WNF	F
48	Texas	-0.48	-0.33	-0.57	-0.49	0.74	0.65	0.78	0.58	WNF	C
40	Oklahoma	-0.47	-0.43	-0.59	-0.16	0.25	-0.05	0.28	0.28	WNF	C
12	Florida	-0.40	-0.37	-0.40	0.14	0.50	-0.12	0.46	0.55	WNF	C
30	Montana	-0.36	-0.31	-0.58	-0.74	0.53	0.83	0.34	0.42	DF	C
32	Nevada	-0.49	-0.355	-0.31	0.10	0.38	-0.17	0.41	0.29	DF	C
35	New Mexico	0.19	0.22	0.33	-0.14	0.31	-0.01	-0.14	0.60	DNF	F
4	Arizona	-0.13	-0.22	0.06	0.53	-0.10	-0.55	-0.03	-0.03	DNF	C
(-) ρ		15	15	14	12	2	9	3	2		
(+) ρ		2	2	3	5	15	8	14	15		
Mean		-0.41	-0.30	-0.32	-0.14	0.43	0.10	0.43	0.43		
Std		0.36	0.34	0.34	0.32	0.33	0.35	0.38	0.33		
CoV		0.89	1.12	1.03	2.21	0.76	3.55	0.89	0.76		

Note: The SPS-1 sections in State 22 (Louisiana) are young and have not shown any significant distress therefore, are not included in this analysis.

Table 6-40 Summary of correlations for deflections and DBPs with rut depth

State		*Area*	*ES/Esg*	*ES*	*Esg*	*d0*	*d6*	*SCI*	*BDI*	Zone	SG
31	Nebraska	-0.45	-0.48	-0.24	0.29	0.28	0.06	0.41	0.29	WF	F
26	Michigan	0.32	0.53	0.41	0.10	-0.14	-0.04	-0.16	-0.14	WF	F
19	Iowa	-0.56	-0.43	-0.41	-0.22	0.40	0.13	0.43	0.46	WF	F
20	Kansas	-0.80	-0.55	-0.59	-0.23	0.76	0.11	0.82	0.78	WF	F
39	Ohio	-0.86	-0.88	-0.88	-0.72	0.79	0.51	0.75	0.76	WF	F
55	Wisconsin	0.26	0.31	0.37	0.37	-0.60	-0.45	-0.48	-0.65	WF	C
10	Delaware	-0.72	-0.55	-0.62	-0.25	0.66	0.37	0.73	0.61	WF	C
5	Arkansas	0.37	0.51	0.51	0.03	-0.11	-0.02	-0.20	-0.03	WF	C
51	Virginia	-0.58	-0.40	-0.39	-0.22	0.69	0.02	0.78	0.68	WNF	F
1	Alabama	-0.51	-0.34	-0.26	0.23	0.63	-0.30	0.73	0.69	WNF	F
48	Texas	0.65	0.62	0.80	0.37	-0.67	-0.28	-0.65	-0.62	WNF	C
40	Oklahoma	0.02	0.23	-0.15	-0.43	0.60	0.67	0.54	0.55	WNF	C
12	Florida	0.56	0.60	0.40	-0.62	-0.33	0.66	-0.44	-0.39	WNF	C
30	Montana	-0.62	-0.66	-0.70	-0.59	0.68	0.56	0.60	0.63	DF	C
32	Nevada	0.05	-0.002	-0.03	-0.31	0.03	0.13	-0.10	0.25	DF	C
35	New Mexico	0.35	0.27	0.31	-0.24	-0.46	0.25	-0.47	-0.09	DNF	F
4	Arizona	-0.19	-0.31	-0.33	-0.20	0.06	0.02	0.05	0.04	DNF	C
(-) ρ		9	10	11	11	6	5	7	6		
(+) ρ		8	7	6	6	11	12	10	11		
Mean		-0.16	-0.09	-0.11	-0.16	0.19	0.14	0.20	0.22		
Std		0.51	0.50	0.49	0.34	0.51	0.33	0.52	0.48		
CoV		3.22	5.46	4.65	2.20	2.61	2.33	2.66	2.13		

Note: The SPS-1 sections in State 22 (Louisiana) are young and have not shown any significant distress therefore, are not included in this analysis.

Table 6-41 Summary of correlations for deflections and DBPs with IRI

State		*Area*	*ES/Esg*	*ES*	*Esg*	*d0*	*d6*	*SCI*	*BDI*	Zone	SG
31	Nebraska	-0.61	-0.44	-0.54	-0.43	0.66	0.49	0.62	0.72	WF	F
26	Michigan	-0.75	-0.71	-0.78	-0.82	0.78	0.74	0.73	0.76	WF	F
19	Iowa	-0.53	-0.31	-0.28	0.04	0.18	-0.12	0.30	0.25	WF	F
20	Kansas	-0.34	-0.31	-0.38	-0.38	0.40	-0.16	0.27	0.42	WF	F
39	Ohio	-0.79	-0.65	-0.65	-0.58	0.84	0.37	0.85	0.85	WF	F
55	Wisconsin	0.37	0.45	0.50	0.23	-0.54	-0.04	-0.46	-0.54	WF	C
10	Delaware	-0.71	-0.60	-0.68	-0.36	0.73	0.28	0.68	0.76	WF	C
5	Arkansas	-0.02	-0.03	-0.17	-0.41	0.21	0.42	0.05	0.21	WF	C
51	Virginia	-0.70	-0.50	-0.55	-0.38	0.78	0.06	0.83	0.77	WNF	F
1	Alabama	-0.47	-0.32	-0.31	-0.31	0.69	0.23	0.66	0.69	WNF	F
48	Texas	0.38	0.44	0.36	-0.17	-0.11	0.15	-0.20	-0.09	WNF	C
40	Oklahoma	0.45	0.49	0.25	-0.35	0.17	0.56	0.10	0.10	WNF	C
12	Florida	-0.31	-0.31	-0.42	0.14	0.47	-0.15	0.38	0.51	WNF	C
30	Montana	-0.55	-0.61	-0.69	-0.62	0.63	0.62	0.50	0.55	DF	C
32	Nevada	-0.47	-0.229	-0.20	-0.16	0.71	0.44	0.72	0.68	DF	C
35	New Mexico	0.39	0.50	0.49	-0.31	-0.02	0.33	-0.35	0.29	DNF	F
4	Arizona	-0.56	-0.46	-0.47	-0.08	0.63	0.05	0.64	0.65	DNF	C
(-) ρ		13	13	13	14	3	4	3	2		
(+) ρ		4	4	4	3	14	13	14	15		
Mean		-0.31	-0.21	-0.27	-0.29	0.43	0.25	0.37	0.45		
Std		0.44	0.42	0.42	0.27	0.39	0.28	0.41	0.37		
CoV		1.44	2.00	1.57	0.93	0.91	1.12	1.11	0.83		

Note: The SPS-1 sections in State 22 (Louisiana) are young and have not shown any significant distress therefore, are not included in this analysis.

6.7.3 Overall Analyses— Predictive Relationships

This section summarizes the findings of apparent relationships between initial response (FWD deflection or deflection base indices) and future pavement performance (cracking, rutting and roughness), based on data from all the test sections in the SPS-1 experiment. The relationships were explored using bivariate scatter plots between selected response parameters and performance measures for all the pavements in the experiment.

Though the sites differ in age, traffic, climate, and materials this analysis is intended to use the wealth of data from all the sections in the experiment. Moreover, the variation in age of the sites may not be very critical at this point in time as no definitive trends were observed between pavement age and performance (see Figure 6-34). Also, it is assumed in this analysis that deflection basin parameters (pavement response) will "characterize" the structural features such as HMA surface thickness, base type and base thickness. In other words, pavement response was assumed to be strongly correlated with the structural capacity of the pavement. In order to account for the effects of subgrade type and climate, relationships were explored for different subgrade soil types (fine- and coarse-grained soils) and climates (WF, WNF, DF and DNF).

Figure 6-35 shows a scatter plot between SCI (from initial response) and fatigue cracking. Among pavements constructed on fine-grained soils, ones with higher initial SCI have more cracking, especially in WNF zone. Similarly, from Figure 6-36 it seems that stiffer pavements (higher AREA) on fine-grained soils, especially if located in WF climatic zone, have higher fatigue cracking.

Higher longitudinal cracking-WP was observed for the pavement sections with higher initial AREA, especially among pavements located in WNF climatic zone (see Figure 6-37). The observation may imply that pavements with stiffer structural layers are more likely to exhibit this distress. No apparent relation was observed between AREA and longitudinal cracking-NWP (see Figure 6-38). The distress was found to be independent of the structural capacity (AREA) of various pavements. Thus the probable cause of this distress type may be the environment and not the loading (traffic).

Figure 6-39 shows a scatter plot between AREA and transverse cracking. No apparent trend was observed in the plot. This could imply that this distress type is not load-related and probably caused by the environment.

The apparent relationship between initial BDI and rutting is shown in Figure 6-40. It seems that among pavements constructed on fine-grained soils and located in WF zone, those with higher BDI experienced higher rutting. It can also be observed that some pavements with less BDI (stronger structure) have experienced higher rutting as compared to pavements with high BDI. These pavements could have experienced mix-related rutting (not structural rutting).

Figure 6-41 is the scatter plot between BDI and latest IRI. It seems that among pavements constructed on fine-grained soils and located in WF zone, those with higher BDI developed higher roughness.

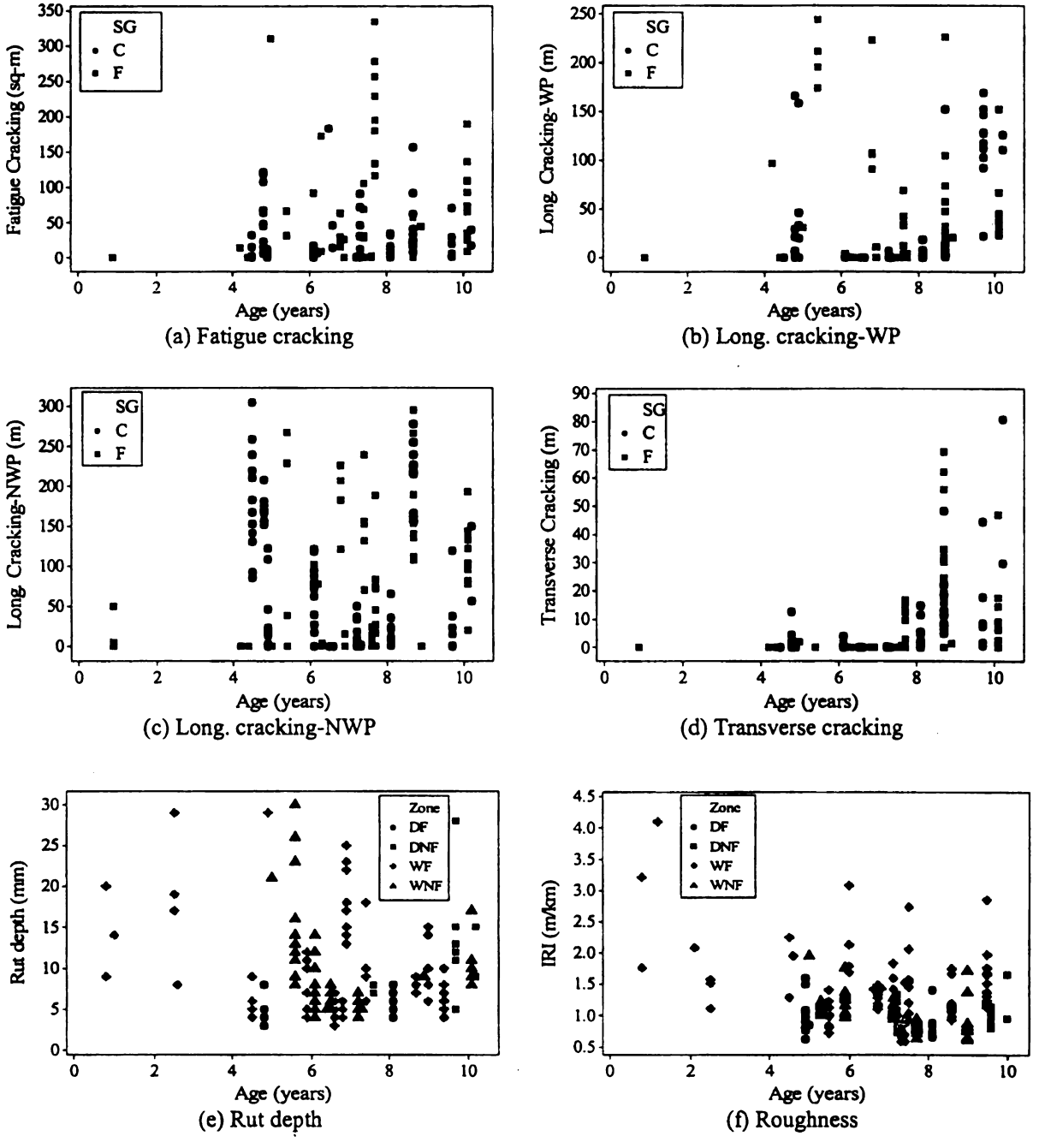
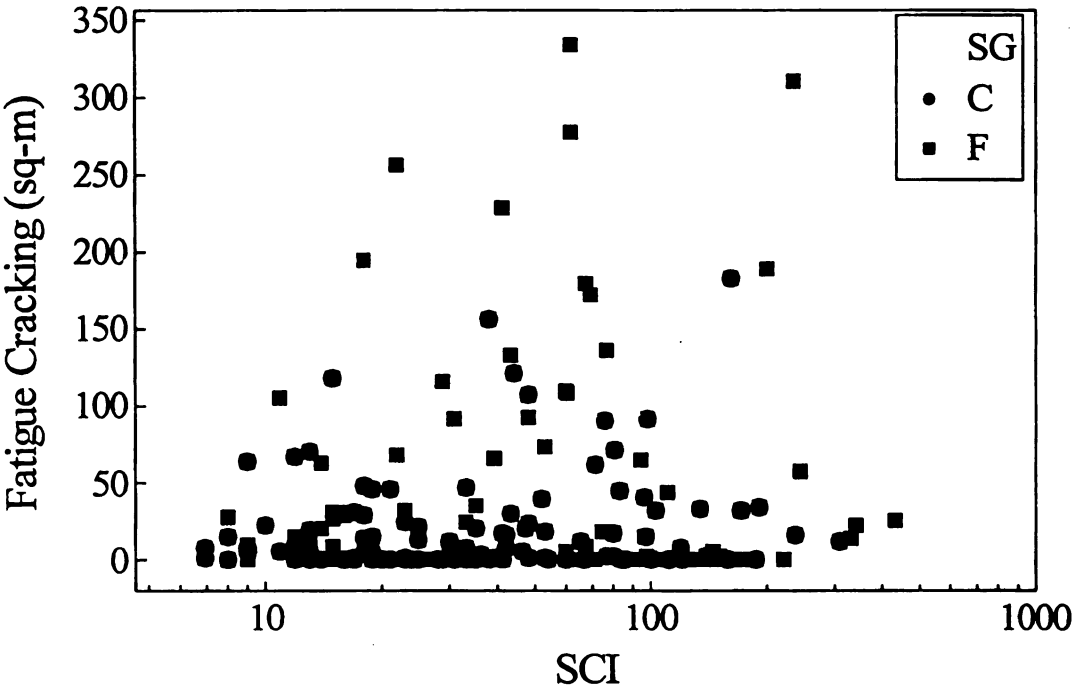
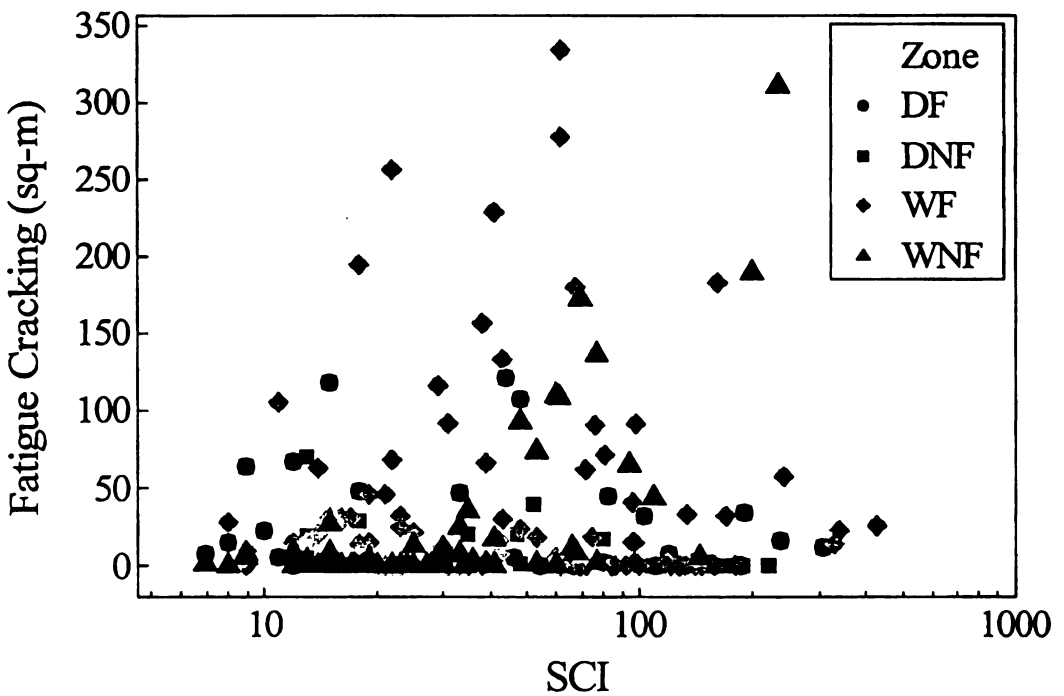


Figure 6-34 Relationships between age and different performance measures

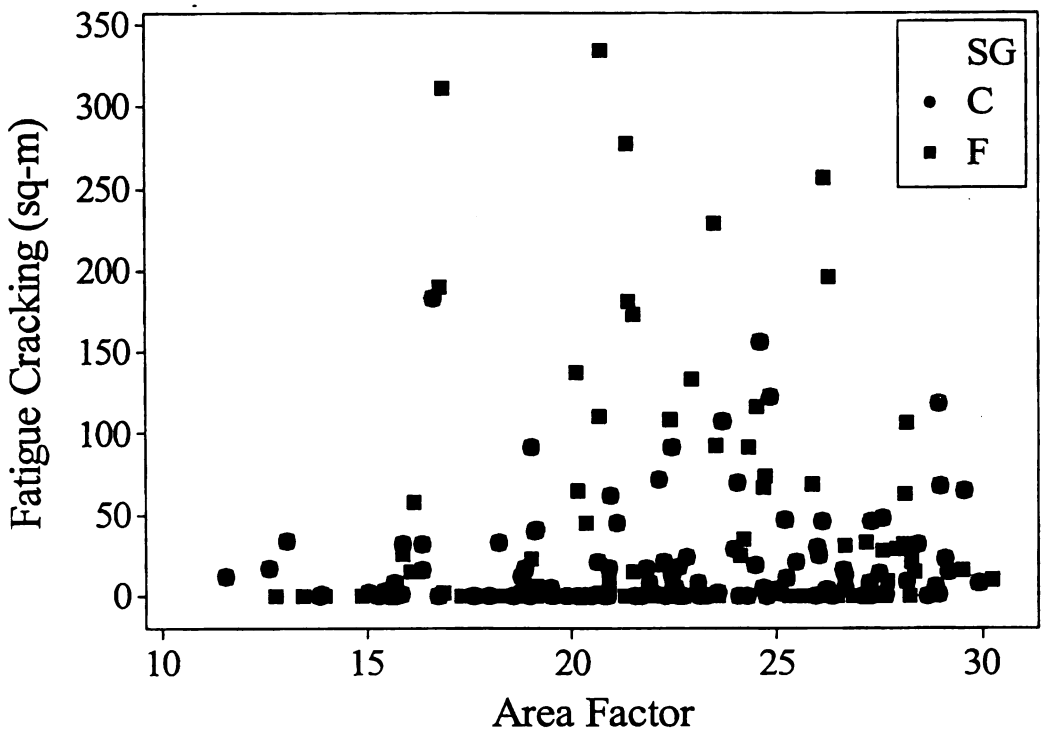


(a) Effect by subgrade soil type

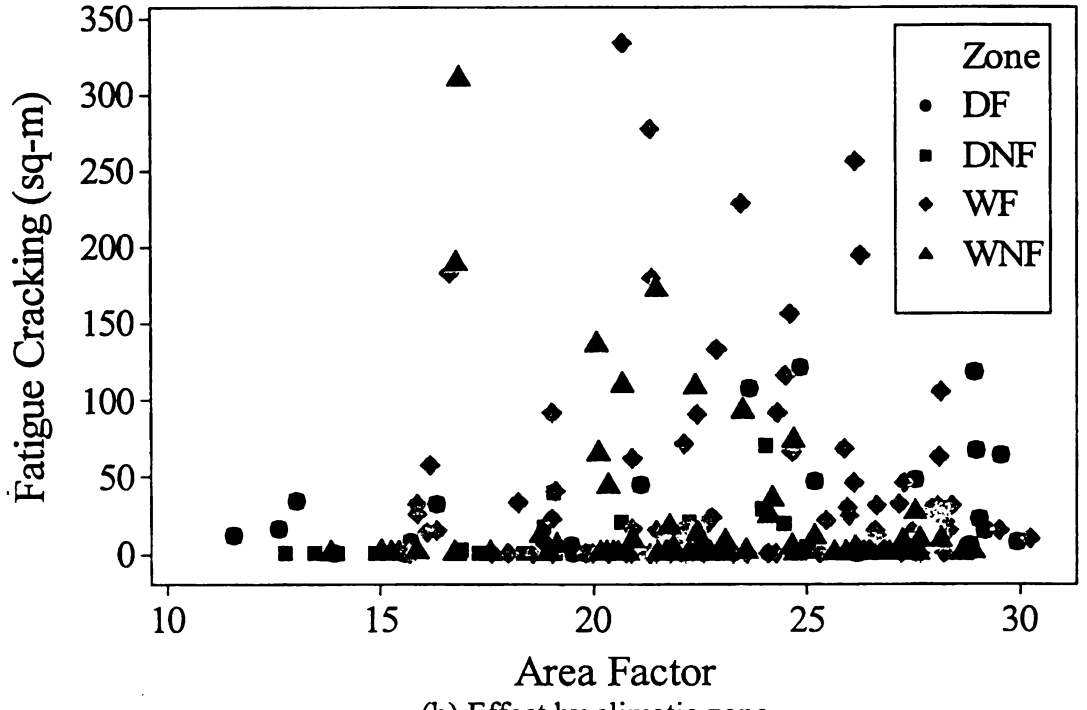


(b) Effect by climatic zone

Figure 6-35 Apparent relationships between SCI and fatigue cracking

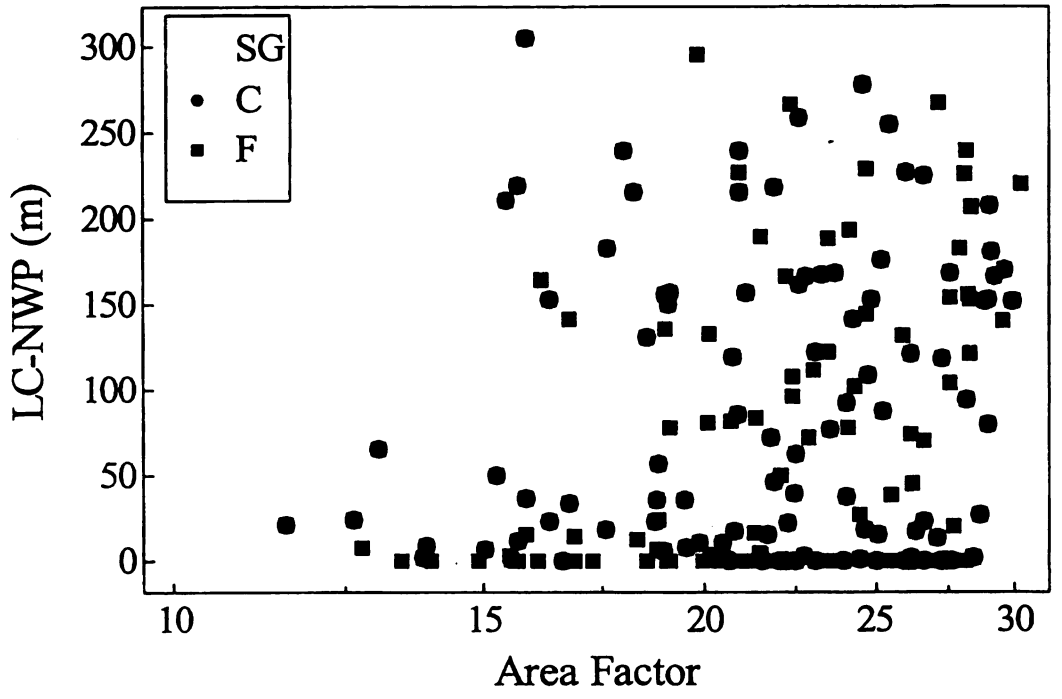


(a) Effect by subgrade soil type

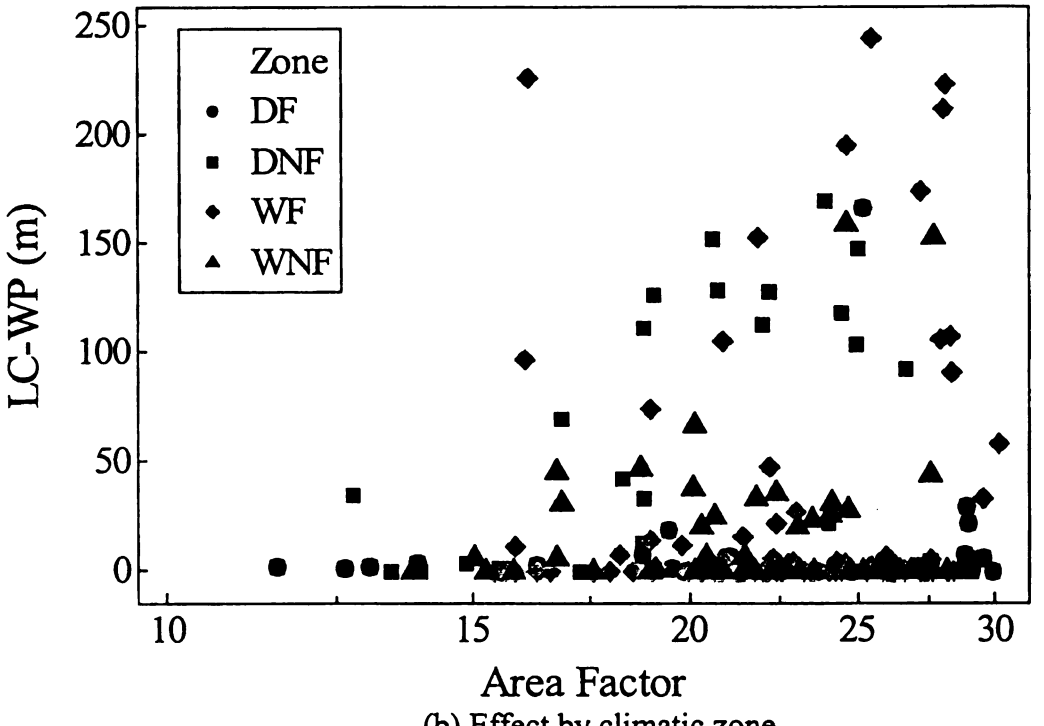


(b) Effect by climatic zone

Figure 6-36 Apparent relationships between AREA and fatigue cracking

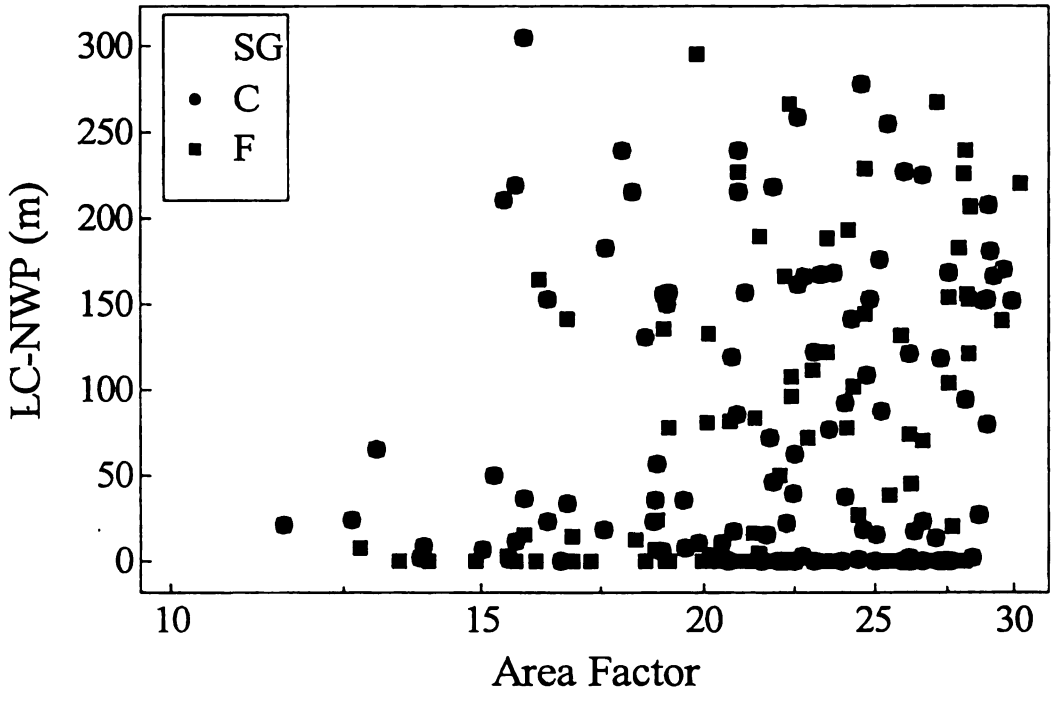


(a) Effect by subgrade soil type

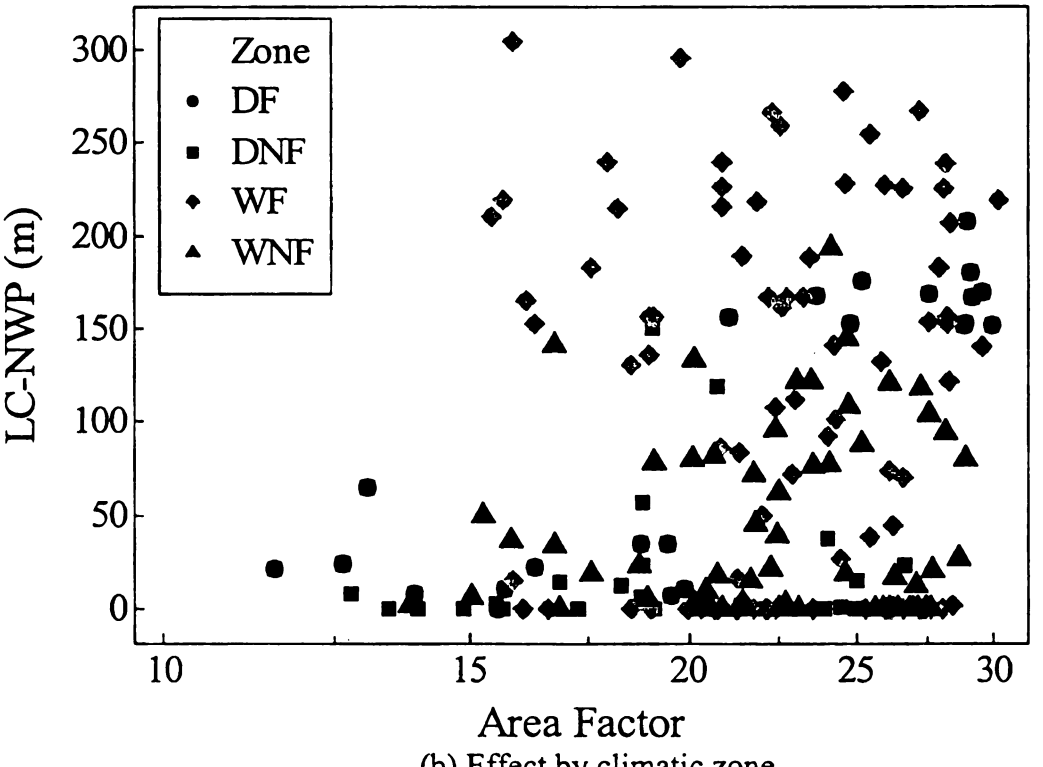


(b) Effect by climatic zone

Figure 6-37 Apparent relationships between AREA and longitudinal cracking-WP

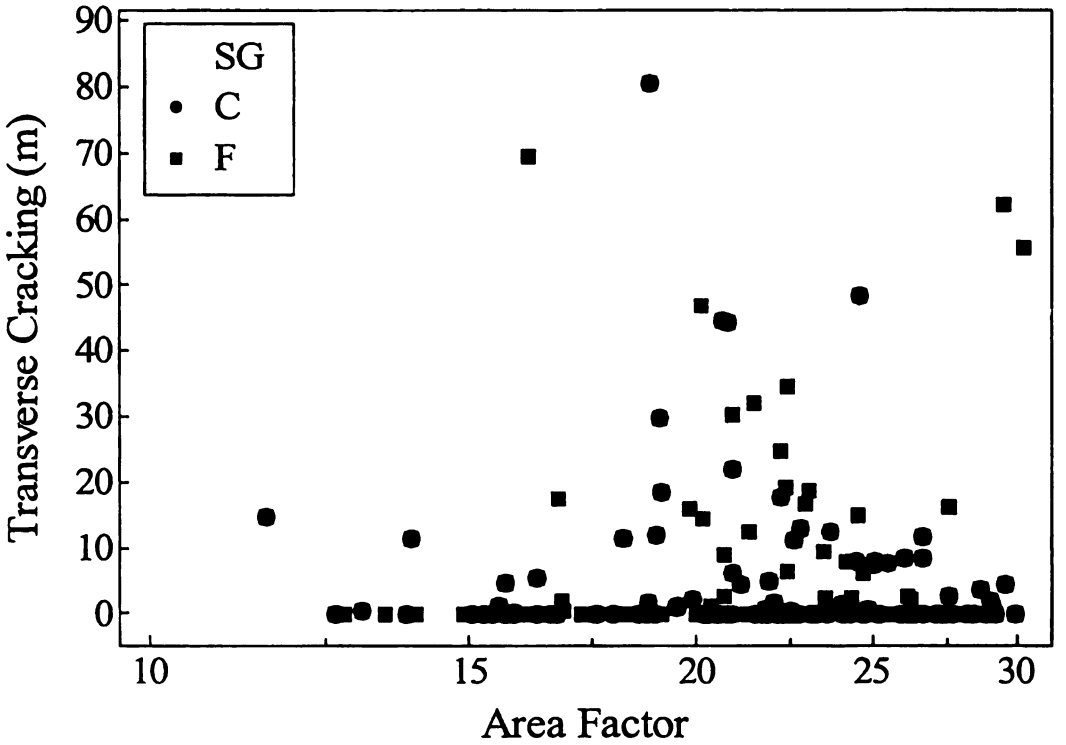


(a) Effect by subgrade soil type

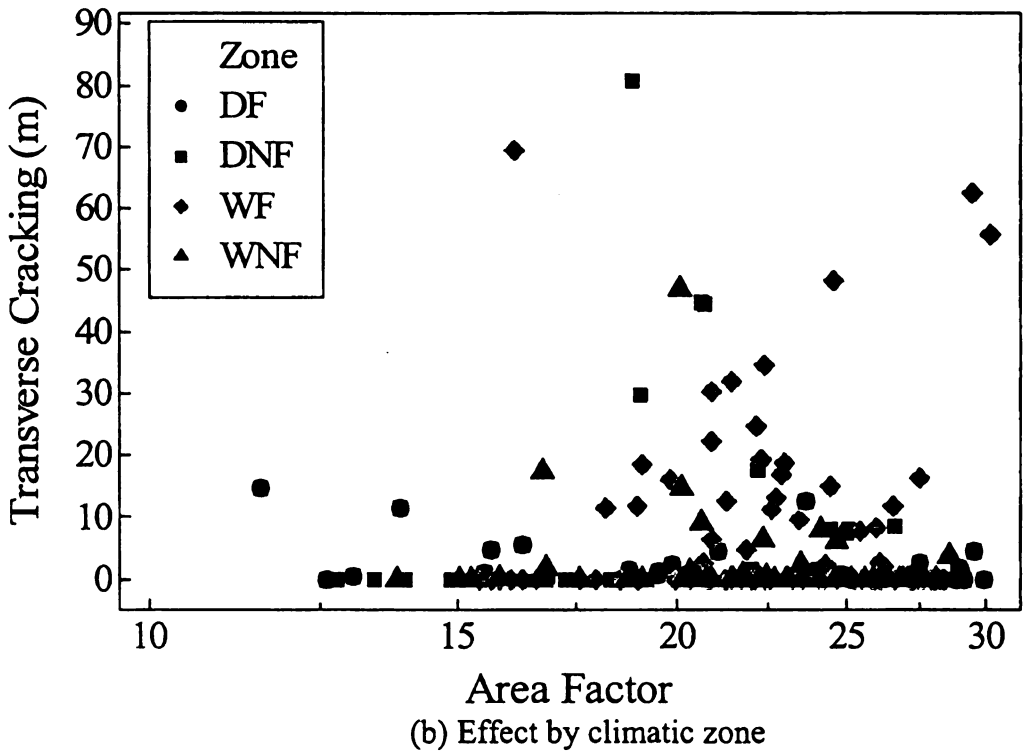


(b) Effect by climatic zone

Figure 6-38 Apparent relationships between AREA and longitudinal cracking-NWP

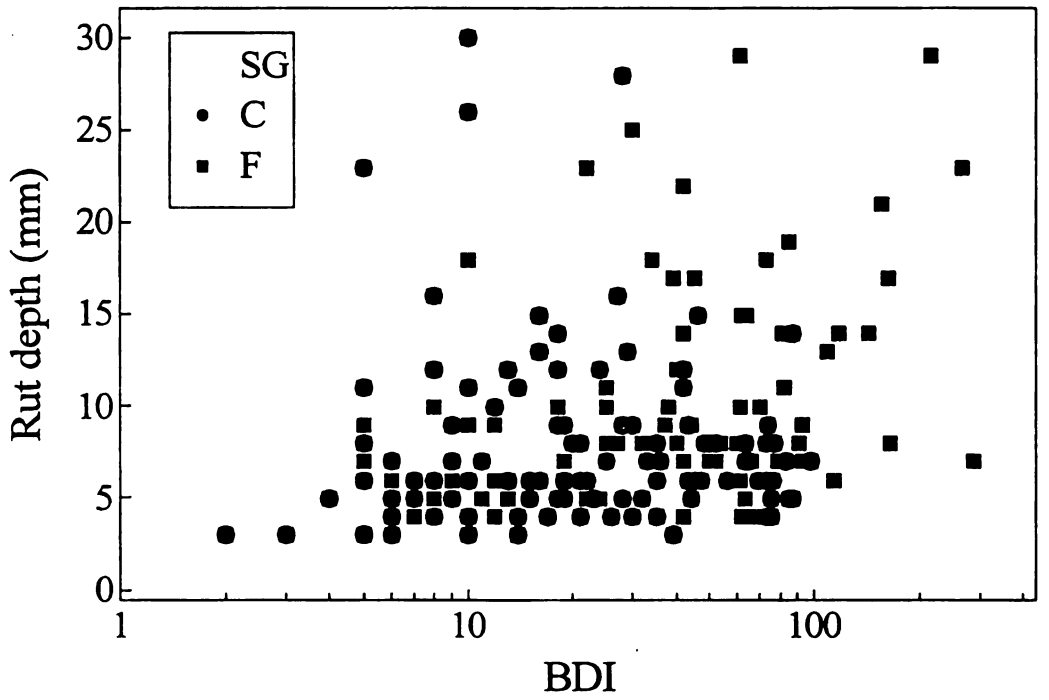


(a) Effect by subgrade soil type

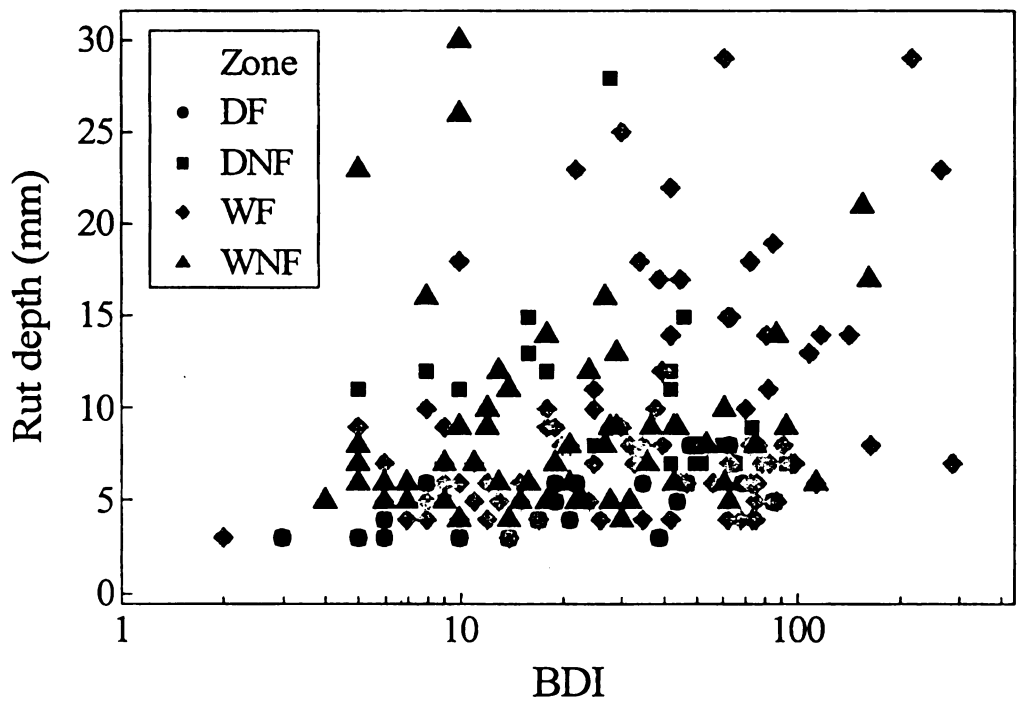


(b) Effect by climatic zone

Figure 6-39 Apparent relationships between AREA and transverse cracking

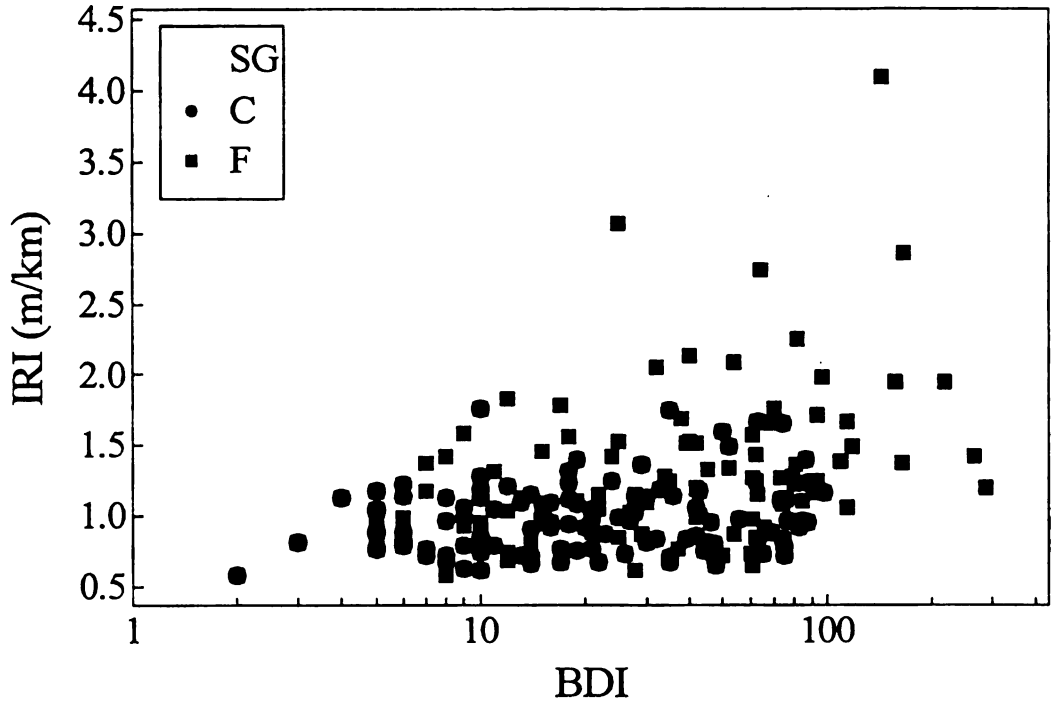


(a) Effect by subgrade soil type

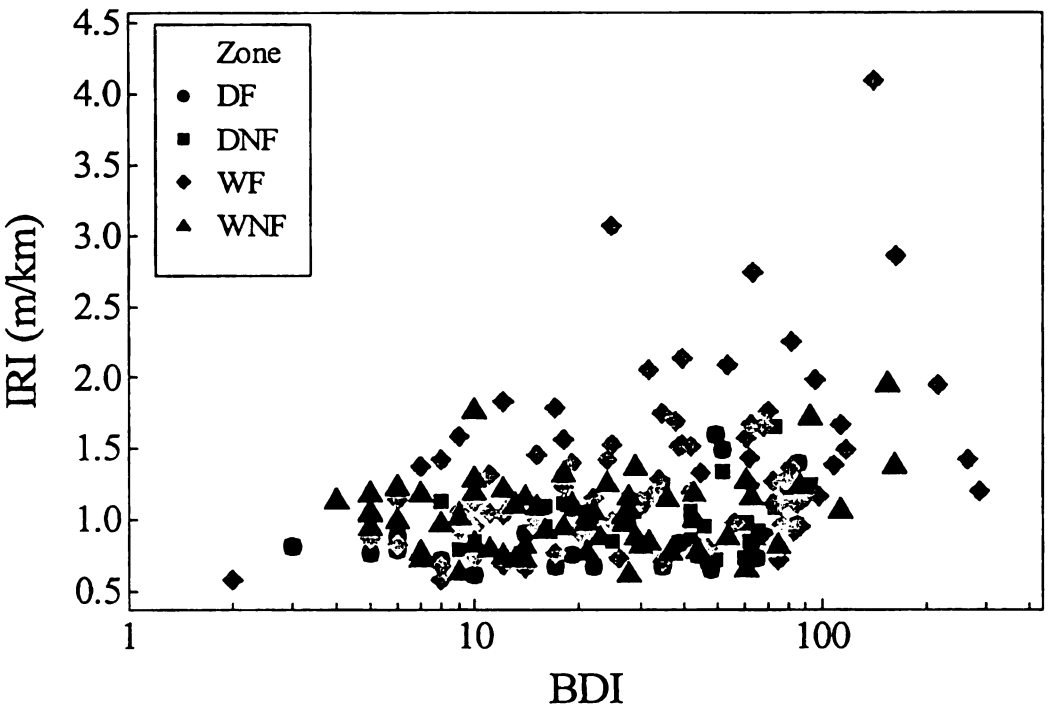


(b) Effect by climatic zone

Figure 6-40 Apparent relationships between BDI and rut depth



(a) Effect by subgrade soil type



(b) Effect by climatic zone

Figure 6-41 Apparent relationships between BDI and IRI

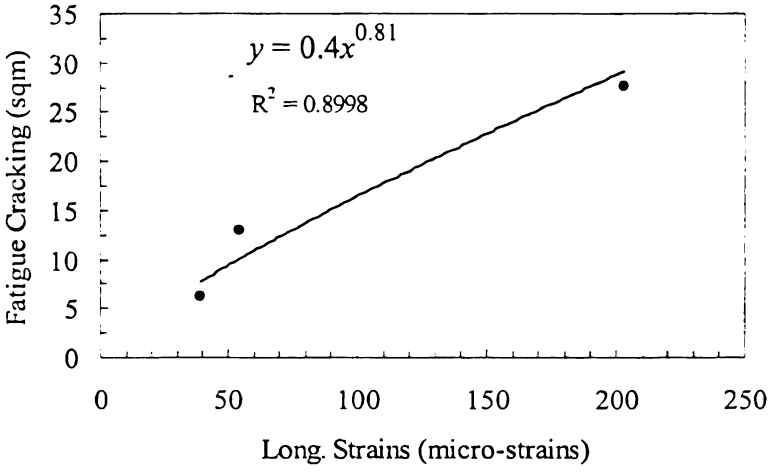
6.7.4 Dynamic Load Response for OH (39) test sections

This section presents the summary of findings from the analysis of Dynamic Load Response (DLR) data from the instrumented flexible pavement sections in the state of Ohio. These sections were instrumented with strain gauges, pressure cells and LVDTs to measure the pavement "response". The SHRP experiment in Ohio targeted four core sections (see Chapter 2) for the installation of sensors to monitor dynamic pavement response during controlled vehicle tests.

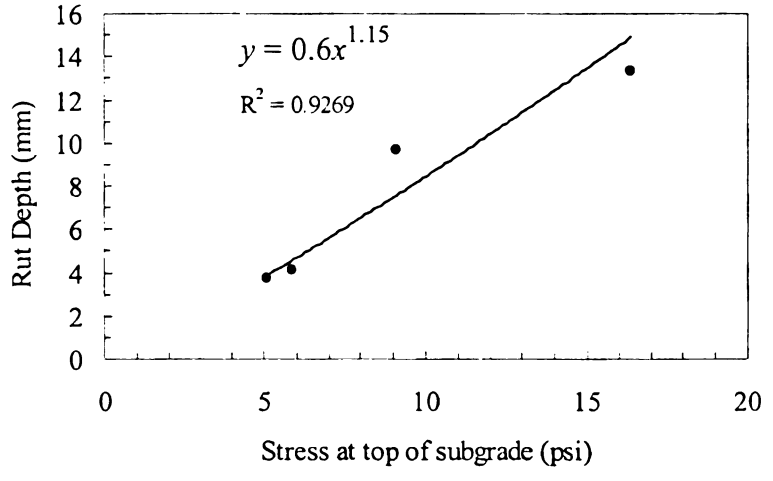
The main objective in this project was to study the response-performance relationship by using the measured dynamic load response and actual observed performance of the sections, in the SPS-1 experiment. Therefore, an attempt was made to relate the observed performance of these instrumented sections with measured responses (strains and surface deflections in HMA surface layer, and stress at top of subgrade) by means of bivariate scatter plots.

The bivariate relationships between measured responses (tensile strain at bottom of asphalt layer, compressive stress at top of subgrade and surface deflection) and observed performances (fatigue cracking and rutting) are shown in Figure 6-42. Also, the bivariate relationship between response (compressive stress at tope of subgrade and surface deflection) and observed performance (roughness in terms of IRI) is shown in Figure 6-43. However, these findings are limited to these four instrumented sections. The measured longitudinal strain (initial value) is "strongly" associated with future fatigue cracking, and the vertical stress at the top of the subgrade is "strongly" associated with future rutting. Other observations regarding the dynamic load response of the instrumented test sections are summarized below:

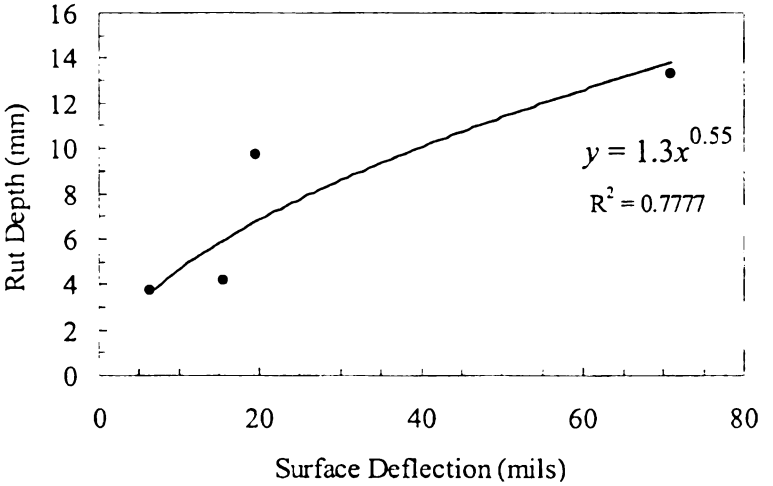
- In general, the strain in the longitudinal direction is higher than the strain in the transverse direction; this is consistent with the mechanistic analysis for flexible pavements.
- Sections with higher strain values have poor fatigue performance. These results are in agreement with the mechanistic-empirical design predictions as fatigue cracking in the flexible pavements is generally considered to be related to the initial tensile strain at the bottom of the HMA layer (bottom up cracking).
- The sections that exhibited high measured stress at the top of the subgrade and high surface deflection have shown poor rut performance.
- The sections that exhibited high measured stress and deflection have higher roughness.



(a) Relationship between strain and fatigue cracking

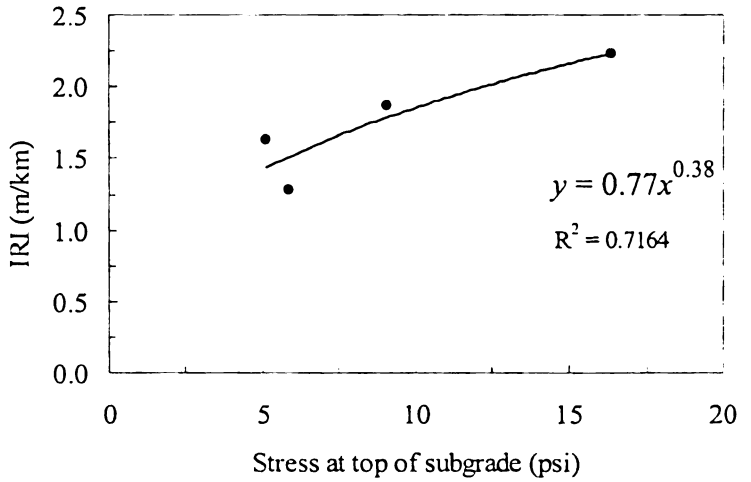


(b) Relationship between stress and rutting

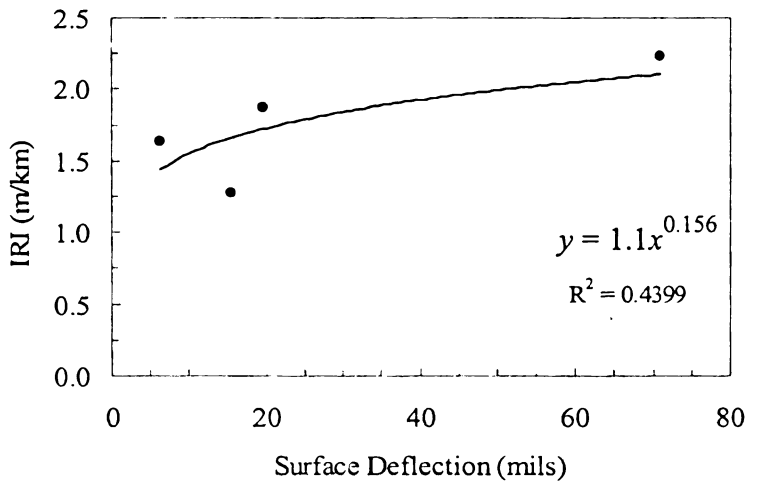


(c) Relationship between deflection and rutting

Figure 6-42 Relationship between measured responses and observed performances—Fatigue cracking and rutting



(d) Relationship between stress and roughness



(e) Relationship between deflection and roughness

Figure 6-43 Relationship between measured responses and observed performances—Roughness

6.8 SYNTHESIS OF RESULTS FROM ANALYSES

This section of the report summarizes the findings from various analyses performed on SPS-1 data. The methods employed in this study were explained in Chapter 4 and the results obtained from these analyses were presented above in this chapter.

Broadly two types of analyses were employed – magnitude-based and frequency-based. The magnitude-based analyses that were used are one-way (univariate) and multivariate ANOVA. These methods are used for comparison of means. The frequency-based analyses that were used are Binary Logistic Regression (BLR) and Linear Discriminant Analysis (LDA). These methods help identify the factors that significantly contribute to the occurrence of a distress based on the likelihood of occurrence or non-occurrence of distresses. In site-level analyses, the performance of pavements within each site was compared. The results from site-level analysis were used to ascertain the consistency of the effects of experimental factors across all sites.

The magnitude-based methods, though powerful, are more appropriate for analyses of distresses which have both high occurrence and magnitude (for example: fatigue cracking, roughness, and rutting). On the other hand, the frequency-based methods are more suitable when the occurrence of a distress is fairly high (for example: transverse cracking) but magnitude is low.

An attempt has been made to summarize the above said effects of design and site features on the performance and response measures. The results were interpreted in light of the type of analysis, and occurrence and extent of distress. ANOVA being the most "powerful" among the methods was given higher importance. However, the results from this analysis may not be reliable in case of limited (low occurrence of distress) or

unbalanced data. Therefore, in these cases, the effects of design features, on the occurrence of distresses were investigated using BLR and LDA.

All results need to be interpreted in light of the experiment design, occurrence and extent of distresses, and analyses methods used. A "weak" effect at this point in time may become a "medium" or "strong" effect in the long term. Hence, all the conclusions are based on "mid-term" performance of the ongoing SPS-1 experiment.

The synthesis of results is presented next for each performance measure separately.

6.8.1 Effects of structural factors for flexible pavements — SPS-1 experiment

This section is subdivided into three parts: (i) pavement performance, (ii) pavement response, and (iii) relationship between response and performance. The structural factors include HMA thickness, base thickness, base type, and drainage. The experiment also includes studying the secondary effects of site factors, namely subgrade type and climatic zones.

6.8.1.1 Effect of Design and Site Factors on Pavement Performance

The effects of the experimental factors on each performance measure are discussed below, one performance measure at a time.

Fatigue Cracking

All the experimental factors were found to be affecting fatigue cracking, though not at the same level. On the whole, pavements with "thin" 4-inch (102 mm) HMA surface layer have shown more fatigue cracking than those with "thick" 7-inch (178 mm) HMA surface layer. Also pavements constructed with only dense-graded aggregate base

(DGAB) have shown more fatigue cracking than those with dense-graded asphalt treated base over unbound aggregate base (ATB/DGAB) and those with ATB base only, with the latter base type showing the best performance. The effects of HMA surface thickness and base type were found to be statistically and practically significant. The main effect of base thickness was found to be statistically insignificant. However, on average, pavements with 16-inch (406 mm) base thickness have shown slightly better fatigue performance than those with 8-inch (203 mm) or 12-inch (305 mm) base thickness. It should be noted that only pavement sections with drainage have a 16-inch (406 mm) base thickness according to the SPS-1 experiment design; therefore, it is unclear whether this effect is caused by the increased base thickness or by drainage provided with the permeable asphalt treated base (PATB). In this regard, the frequency-based analyses did show that pavements with drainage have significantly lower chances of cracking than those without drainage.

In general, pavement sections built on fine-grained soils have more fatigue cracking than those built on coarse-grained soils. Also pavements located WF zone have shown more fatigue cracking than those located in WNF zone. These effects were found to be statistically significant and are of practical significance.

Among un-drained pavements, on average, an increase in HMA surface thickness from 4-inch (102 mm) to 7-inch (178 mm) has a slightly higher effect on fatigue cracking for pavements with DGAB than for pavements with ATB.

The above effect of HMA surface thickness is more significant for sections built on coarse-grained soils. On the other hand, among pavements built on fine-grained soils, the effect of drainage is seen only in those sections with DGAB; i.e., those with drainage

have less fatigue cracking than those without drainage. Also among drained pavements built on fine-grained soils, those with 8-inch (203 mm) base have more cracking than those with 12-inch (305 mm) and 16-inch (406 mm) base. These effects were found to be statistically and practically significant. Hence, for pavements built on fine-grained soils, drainage improves the fatigue performance, especially if built with thicker bases.

The main effect of HMA thickness, discussed above, is mainly seen among sections located in WNF zone. The effect is of practical and statistical significance. This may be an indication that an increase of HMA thickness from 4-inch (102 mm) to 7-inch (178 mm) is not sufficient in resisting fatigue cracking for pavements in WF zone as compared to WNF zone.

Among sections located in the WF zone, those with DGAB have shown the highest amount of cracking while those with ATB have the least cracking. In addition, those with 16-inch (406 mm) drained base have the least amount of fatigue cracking. These effects were found to be statistically and practically significant. This suggests that among pavements located in WF zone, "thick" 16-inch (406 mm) treated bases with drainage are less prone to cracking. The effects of HMA thickness and base thickness discussed above imply that, among sections located in WF zone, an increase in base thickness to 16-inch (with drainage) has a greater impact than an increase in HMA thickness from 4-inch (102 mm) to 7-inch (178 mm), suggesting that a thicker base and drainage helps in reducing frost effects.

Structural Rutting

The extent of structural rutting among the test sections in the SPS-1 experiment is 6.5 mm, on average, with a standard deviation of 2.4 mm. Their average age is about 7 years with a range between 4.5 and 10 years. The amount of rutting for the majority of these sections is within the normal range at this point in time. Therefore, the results at this point may only show initial trends and may not be of much practical significance.

Marginal main effects of drainage, HMA thickness, and base thickness on structural rutting were observed. Pavements with "thin" [4-inch (102 mm)] HMA surface layer have shown slightly more rutting than those with "thick" [7-inch (178 mm)] HMA surface layer. Also, on average, pavements with 16-inch (406 mm) drained base have shown somewhat better rut performance than those with 8-inch (203 mm) and 12-inch (305 mm) base. However, these effects of HMA surface thickness and base thickness were not found to be statistically significant. Pavements with drainage have less rutting than those without drainage. The effect of drainage on structural rutting was found to be statistically significant; however the effect is not of practical significance at this point in time.

In general, pavement sections built on fine-grained subgrade have shown more rutting than those built on coarse-grained subgrade. This effect is statistically significant and appears to be of practical significance. On the other hand, there is no apparent effect of climate (WF vs. WNF) on structural rutting.

Among the pavements built on coarse-grained soils, those with 7-inch (178 mm) HMA surface have shown slightly less rutting than those with 4-inch (102 mm) HMA surface. This effect was statistically significant; however it is not operationally

meaningful at this point. The above suggests that for sections built on fine-grained soils an increase in HMA thickness from 4-inch (102 mm) to 7-inch (178 mm) may not be sufficient in reducing the amount of rutting. On the other hand, among pavements built on fine-grained soils, a marginal positive effect of drainage is seen in sections with ATB.

Among drained pavements located in WF zone, those with DGAB have shown more rutting than those with ATB. Also, among sections located in WF zone and built with ATB, those with drainage have shown significantly less rutting than those without drainage. Both of these effects were found to be statistically significant and are of operational significance. This implies that, among pavements located in WF zone, those with ATB and drainage perform better than those with other combinations of base type and drainage.

Among un-drained sections located in WNF zone, those with 12-inch (305 mm) base have less rutting than those with 8-inch (203 mm) base. This effect was found to be statistically significant and of practically significance. For sections built on DGAB and located in WNF zone, those with drainage have shown slightly less rutting than those without drainage. The effect was found to be marginally significant. These early trends imply that the importance of drainage among pavements with DGAB is considerable in improving rut performance among sections located in WNF zone. On the other hand an increase in base thickness from 8-inch (203 mm) to 12-inch (305 mm) improves rut performance for un-drained sections, irrespective of base type.

Roughness

All the experimental factors were found to be affecting roughness, though not at the same level. Pavements with "thin" [4-inch (102 mm)] HMA surface layer have higher

change in IRI (ΔIRI) than those with "thick" [7-inch (178 mm)] HMA surface layer. This effect was found to be statistically significant but is not of practical significance at this point in time. Also, pavements constructed with DGAB have higher ΔIRI than those with ATB/DGAB and ATB, while pavements with ATB have the best performance for roughness. Pavements with thicker bases have lower ΔIRI. Also pavements with drainage have lower ΔIRI than un-drained pavements. The above main effects of base thickness, base type and drainage were found to be statistically significant and are of practical significance.

In general, pavements built on fine-grained soils have shown higher ΔIRI than those built on coarse-grained soils, especially among sections in WF zone. Also, the change in roughness among sections located in WF zone is significantly higher than those in WNF zone. These effects were found to be statistically significant and are of practical significance.

Among pavements built on fine-grained soils, an increase in HMA thickness from 4-inch (102 mm) to 7-inch (178 mm) has a significant positive effect on change in roughness. Also for un-drained pavements, those with ATB have significantly lower ΔIRI than those with DGAB. Finally the effect of drainage is significant only for sections with DGAB. The above effects were found to be statistically significant and are of practical significance. These effects suggest that, for pavements built on fine-grained soils, higher HMA thickness and/or treated base will help inhibit the increase in roughness. Also, drainage appears to be more effective in preventing an increase in roughness for sections with DGAB, especially among those located in WF zone.

For un-drained pavements built on coarse-grained soils, an increase in base thickness from 8-inch (203 mm) to 12-inch (305 mm) has a marginally significant effect, in that sections with thicker base have lower ΔIRI. However, this effect is not of practical significance at this point in time.

Transverse Cracking

The effect of base thickness on transverse cracking is insignificant, at this point. Pavements constructed with DGAB have more transverse cracking than those with ATB/DGAB and ATB, while pavements with ATB have shown the least amount of cracking. The effect was found to be statistically significant; however it is not of practical significance at this point in time. Slightly more cracking was observed on pavements with "thin" [4-inch (102 mm)] HMA surface layer. Also, pavements with drainage have shown slightly less cracking than un-drained pavements. However, these effects were not found to be statistically significant.

In general, pavements built on fine-grained soils have shown more transverse cracking than those built on coarse-grained soils. This effect was found to be statistically significant and is of practical significance.

Pavements located in WF zone have shown significantly more transverse cracking than those located in WNF zone. This main effect of climatic zone was found to be statistically significant and is of practical significance. This confirms that transverse cracking occurs mainly in freezing environment.

Among drained pavements built on coarse-grained soils, those with ATB performed better than those with DGAB. Also, among pavements with DGAB and built on fine-grained soils, those with drainage have shown significantly less transverse

cracking than those without drainage. These effects were statistically significant and appear to be of practical significance.

Longitudinal Cracking-WP

The effects of HMA and base thickness on longitudinal cracking-WP are insignificant at this point in time. Pavements with drainage have shown less cracking than un-drained pavements. The main effect of drainage was found to be statistically significant, but is not of practical significance at this point.

In general, pavements built on fine-grained soils have shown more longitudinal cracking-WP than those built on coarse-grained soils. This effect is of statistical and practical significance. Also, on average pavements in WF zone have shown higher levels of longitudinal cracking-WP than those in WNF, especially among pavements built on fine-grained subgrade. This effect was found to be marginally significant.

Among pavements built on fine-grained soils, those built with DGAB have shown more longitudinal cracking-WP, and those built with ATB have shown the least amount of cracking. This main effect of base type was statistically and operationally significant. Also among pavements built on fine-grained soils, drainage has a significant effect on longitudinal cracking, and this effect is more pronounced among pavements built with DGAB. This effect was statistically significant and is of practical significance. This trend implies that if a pavement on fine-grained subgrade is constructed with a DGAB base, better performance (in terms of longitudinal cracking-WP) can be achieved by providing drainage. These effects are seen in both WF and. WNF zones.

Longitudinal Cracking-NWP

The effects of HMA thickness, base thickness, and base type on longitudinal cracking-NWP are insignificant at this point in time. Pavements with drainage have shown slightly less cracking than un-drained pavements. However, the effect of drainage was found to be only marginally significant.

The effect of subgrade type was not found to be statistically significant. In general, more longitudinal cracking-NWP was observed among sections located in "freeze" climate compared to those in "no-freeze" climate. This main effect of climatic zone is statistically significant and is of practical significance. Also, the effect of drainage is more pronounced (with marginal statistical significance) among pavements located in "freeze" climate. However, this effect is not of practical significance.

These initial trends indicate that longitudinal cracking-NWP is caused by "freeze" climate (frost effects), and that pavements without drainage may be more prone to it.

6.8.1.2 Effect of Design and Site Factors on Pavement Response

Three pavement response parameters were chosen for ANOVA – peak deflection under FWD load (d_0), far-sensor deflection (d_6), and AREA. All the response parameters have been calculated using the initial deflections of the test sections. Also, the pavement surface temperature at the time of testing was taken as a covariate along with the age at the time of testing and variability in the HMA and base layer thicknesses. The natural logarithmic transformation has been applied to the three response indicators to fulfill the ANOVA assumptions. The following discussion summarizes the effects of design and site factors on each of the response parameters.

Peak Deflection under FWD Load (d_0)

The interactions between HMA thickness and base type, base thickness and base type, base type and drainage, have significant effects on the peak deflection (d_0).

Among the pavement sections built on DGAB, those with 4-inch (102 mm) HMA thickness have higher d_0 than those with 7-inch (178 mm) HMA thickness. Also as expected, thicker bases for each base type have lower d_0. However, this effect was more significant in the case of sections with treated bases (ATB or ATB/DGAB). Furthermore, pavement sections with PATB/DGAB have lower d_0 than those with DGAB.

The interaction between subgrade soil and climatic zone was found to have a very significant effect on d_0. Test sections built on fine-grained soils have shown significantly higher d_0 as compared to those built on coarse-grained soils. This effect is more prominent on pavements located in WNF zone.

Far Sensor Deflection (d6)

The effects of base type, base thickness and drainage have significant effects on the far-sensor deflection (d_6). HMA thickness and pavement mid depth temperature do not have a significant effect on d_6.

The interaction between subgrade soil type and climatic zone was found to have a significant effect on d_6. Test sections built on fine-grained soils have shown significantly higher d_6 as compared to those built on coarse-grained soils. This effect is more prominent on pavements located in WNF zone.

Pavement sections built with DGAB have shown higher far-sensor deflections than those built on other base types. Pavements constructed on 8-inch bases have also shown significantly higher far-sensor deflections than those built on 12-inch (203 mm) or

16-inch (406 mm) bases. Furthermore, pavement sections with PATB/DGAB have lower d_6 than those with DGAB. These effects of the design factors on d_6 are based on statistical analyses only, and may or may not be of practical importance.

AREA

The interactions between HMA thickness and base type, base thickness and base type, and, drainage and base type have significant effects on the AREA parameter.

Among pavement sections built on DGAB, those with "thin" HMA surface layer have lower AREA values compared to those with "thick" HMA surface layer, implying that the upper layers of these pavement sections are "less stiff". The increase in HMA thickness from 4-inch (102 mm) to 7-inch (178 mm) on ATB does not significantly increase the AREA value.

For sections built on DGAB, increasing base thickness from 8-inch (203 mm) to 12-inch (305 mm) has not shown a significant effect on AREA; however a two-fold increase in base thickness [from 8 to16 inch (203 to 406 mm)] has shown a significant increase in AREA. Also, base thickness does not seem to have a significant effect on AREA in pavement sections with ATB bases. Furthermore, pavement sections with PATB/DGAB have higher AREA values than those with DGAB. This indicates that the structural capacity of the PATB layer is somewhat higher than that of the DGAB.

Among the pavement sections located in WNF zone, those built on fine-grained subgrade soils have significantly higher AREA values than those built on coarse-grained soils. However, in the case of sections located in WF zone, this effect is not significant indicating that AREA could be independent of the subgrade soil type.

A simplified summary of results from all analyses is given in Table 6-42. The summary is only meant to give an overall assessment of the effects. The reader is strongly recommended to read the following write-up for a better understanding of all the effects. It is important to note that a "strong", "medium" or "weak" effect should only be interpreted in terms of the difference in effects at the various levels of a factor. As an example, a "strong" effect of HMA thickness and a "strong" effect of subgrade soil type should not be interpreted as HMA thickness and subgrade type having the same strength of effect.

A black circle indicates a "strong" effect (significant); a grey circle indicates a "medium" effect, and a white circle indicates a "weak" effect. Operational significance was determined only for "strong" or "medium" effects. It should be noted that an effect can be statistically significant (meaning that it is not a coincidence) but may not be operationally/ practically significant, at this point in time.

Table 6-42 'Simplified' summary of effects of design and site factors for flexible pavements

Design Factor	Performance Measures						Response Measures		
	Fatigue cracking	Rutting	Roughness	Transverse cracking	Longitudinal cracking		Peak deflection d_0	Peak deflection d_6	Area Factor
					WP	NWP			
HMA thickness	●	◐	●	◐	○	○	●	○	●
Base type	●	◐	●	●	◐	○	●	●	●
Base thickness	◐	◐	●	○	○	○	●	●	◐
Drainage	◐	◐	●	◐	●	◐	◐	●	●
Climatic Zone	●	●	●	●	●	●	●	●	●
Subgrade type	●	●	●	●	●	○	●	●	◐

Note: This table is solely for the purpose of summarizing some of the effects in a 'simple' format. The reader is urged to read relevant text in the report for a better understanding.

Symbol	Description
●	Strong Effect (Main effect exists)
◐	Medium Effect (Interaction effect)
○	Weak Effect

6.8.1.3 Apparent Relationship between Response and Performance

Two types of relations between flexible pavement response under (FWD testing) and performance were explored for the SPS-1 pavement sections—explanatory and predictive. Explanatory relationships were established using multiple regressions on data from all the test sections in the experiment. Predictive relationships were established based on bivariate correlation analyses at the site level, and using scatter plots on data from all sections. The dynamic load response (DLR) data from instrumented sections in Ohio were used for predictive relationships. The salient findings are briefly presented below:

Overall Analysis— Explanatory Relationship

A regression model was developed considering the peak deflection (d_0) as the dependent variable and variables such as temperature, asphalt thickness, subgrade strength and performance measures as independent variables. The observations based on the regression model are as follows:

- Pavements with "thick" [7-inch (178 mm)] HMA surface layer were observed (with statistical significance) to have significantly lower deflections than those with "thin" [4-inch (102 mm)] HMA surface layer.
- Mid-depth temperature of the HMA layer, at the time of testing, has a statistically significant effect on d_0. Irrespective of design features, pavement deflections (d_0) measured at higher temperatures is greater than those at lower temperatures.
- Older pavements have slightly lower deflections (d_0) compared to younger pavements, which could be due to stiffening (aging) of the asphalt.

- Pavements with "weaker" subgrade (higher d_6) have significantly higher d_0 (with statistical significance).
- Pavements with more cracking (fatigue cracking or longitudinal cracking) have a significantly higher d_0 (with statistical significance), compared to those with less cracking.

Site Level Analysis— Predictive Relationships

This section summarizes the findings regarding the predictive relationships between initial response (FWD deflection or deflection basin indices) and future pavement performance (fatigue cracking, rutting and roughness) at the site level. The data for sections from LA (22) were excluded from these analyses, as performance data for the sections are available for just one year.

- On average, AREA, SCI and BDI have shown reasonable correlations with fatigue cracking for sections in most of the sites in the SPS-1 experiment. In most of the sites, pavements with higher initial SCI or BDI, or lower initial AREA were found to have higher fatigue cracking.
- Consistent trends were observed between BDI and future IRI for the various sites in the SPS-1 experiment. In most of the sites, pavements with higher initial BDI were found to have higher IRI.
- The deflection basin parameters have not shown a consistent relationship with rut depth for the various sites in the SPS-1 experiment.

Overall Analysis— Predictive Relationships

Relationships were explored between initial response (FWD deflection basin indices) and pavement performance (cracking, rutting and roughness), using bivariate

scatter plots between selected response parameters and performance measures for all pavement sections in the experiment. The main observations based on these relationships are listed below:

- Among pavements constructed on fine-grained soils, ones with higher SCI have shown more fatigue cracking, especially in WNF zone. Also, stiffer pavements (higher AREA) on fine-grained soils have shown more fatigue cracking, especially if located in WF climatic zone.
- Higher longitudinal cracking-WP was observed for the pavement sections with higher AREA especially among pavements located in WNF climatic zone.
- No apparent relation was observed between AREA and longitudinal cracking-NWP, implying that this distress could be independent of the pavement structural capacity.
- No apparent trend was observed between AREA and transverse cracking. This could imply that this distress type is not load-related.
- Among pavements constructed on fine-grained soils and located in WF zone, those with higher BDI experienced slightly higher rutting. It was also observed that some pavements with lower BDI (stronger structure) have experienced higher rutting as compared to pavements with high BDI. These pavements could have experienced mix-related rutting (not structural rutting).
- Among pavements constructed on fine-grained soils and located in WF zone, those with higher BDI developed slightly higher roughness over time.

Dynamic Load Response for OH (39) test sections

This section of the report presents the summary of findings from the analysis of measured Dynamic Load Response (DLR) data from the instrumented flexible pavement sections in the state of Ohio. The observations from the analysis of these instrumented sections are summarized below:

- In general, the strains in the longitudinal direction are higher than the strains in the transverse direction; this is consistent with the results from mechanistic analysis of flexible pavements.
- The sections that were observed to have higher initial strain values have shown worse fatigue performance. These results are in agreement with the mechanistic-empirical design predictions that fatigue cracking in flexible pavements is related to the initial tensile strain at the bottom of the HMA layer (bottom up cracking).
- The sections that were observed to have high initial stress at the top of the subgrade layer and those that were observed to have high initial surface deflection under the load have shown poor rut performance.

CHAPTER 7 - ANALYSIS RESULTS FOR THE SPS-8 EXPERIMENT

7.1 INTRODUCTION

The purpose of this chapter is to provide a summary of the results of the analyses conducted for the SPS-8 experiment on flexible pavements. The performance measures used in the analysis include fatigue cracking, rutting, longitudinal cracking (in the wheel path and outside the wheel path), transverse cracking, and IRI. The results are summarized according to individual design and site factors.

As mentioned in Chapter 4 under section 4.4, all the flexible pavement sections in SPS-8 experiment are aged between 3 and 10 years, with an average age of about 6 years. Thus a majority of pavement sections are relatively "young" to exhibit any environment-related distresses. Only a few sections have shown some distresses as of Release 17.0. It is to be noted that the current status for SPS-8 experiment for flexible pavements shows that there a few sites located in the DF and DNF zones. The extent of various distresses exhibited by the pavements is presented in Chapter 4. Site-wise summaries of inventory data, construction issues and performance of flexible and rigid pavements can be found in Appendix C. Keeping in view the number of sections constructed for SPS-8 experiment (32 flexible pavements in 15 sites) and the extent of distresses at present, statistical analysis as in the case of SPS-1 experiment may not be applicable. Therefore, simple mean comparisons (only for sections that exhibited distresses) were performed to identify the effects of experimental factors on various performance measures. Some initial trends obtained from these comparisons are reported below.

7.2 EFFECTS OF ENVIRONMENTAL FACTORS IN SPS-8 EXPERIMENT FOR FLEXIBLE PAVEMENTS

The objective of the SPS-8 experiment is to develop conclusions concerning environmentally induced serviceability loss and the contribution of environment and subgrade to the distress of pavements. The experiment will also develop conclusions concerning the effects of base and surface thickness variations on retarding environmentally driven distress.

7.2.1 Site-Level Analysis

The analysis of the data from SPS-8 sections was done based on the concepts of performance index (PI) and relative performance as in the case of SPS-1 experiment. At the site-level, various performance measures (fatigue cracking, longitudinal cracking (WP and NWP), transverse cracking, raveling, rutting and roughness) were analyzed to investigate the effects of the main site factors (climatic zone and subgrade type) on performance. The summary of results from this analysis is given below:

- The results of the available data indicate that WF zones have shown relatively higher potential for fatigue cracking; however as expected, the magnitude of distress is not significant.
- Results for longitudinal cracking-NWP were inconclusive.
- Transverse cracking occurred mainly in freeze zones.
- There was higher amount of raveling observed in WNF zone.
- Rutting performance was similar in all environments and for different subgrade types; this is to be expected since rutting is essentially a load-related distress.

- The results of roughness in terms of IRI show that sections built in WF zones appear to have higher roughness, followed by those built in WNF zones.

7.2.2 Overall Analysis

The overall initial trends which show the effect of SPS-8 experimental factors on various performance measures will be discussed in this section. These comparisons were carried out only for the performance measures which have shown some extent in the SPS-8 flexible pavements.

Fatigue Cracking: Fatigue cracking was observed in only 12 out of 32 pavement sections. Among the cracked sections, the area of fatigue cracking varies from 0.2% to about 19% with an average of 3%. Excluding section 36-0801, where 19% of area has fatigue cracking, the average cracking area of cracked sections is about 1% with a range of 0.2% to 4.5%. The average fatigue cracking by experimental factors is shown in Figure 7-1. The average fatigue performance shows that pavements in WF zone have a higher potential for cracking. On average, pavements constructed on active subgrade (frost susceptible or expansive) soils and pavements with "thin" [4-inch (102 mm)] HMA surface layer have exhibited more cracking than those built on other subgrade types and with "thick" [7-inch (178 mm)] HMA surface layer. In order to show the effect of active subgrade within fine and coarse soil types, the average performance is presented in Figure 7-2. It was observed that flexible pavements constructed on the active coarse-grained subgrade soils have shown higher potential for fatigue cracking.

Longitudinal Cracking (WP and NWP): Longitudinal cracking-WP was observed in only 13 out of 32 pavement sections, while longitudinal cracking-NWP occurred in 20

pavement sections. Among the cracked sections, longitudinal cracking-WP length varies from 1 to 97 m with an average of 18 m, while longitudinal cracking-NWP length varies from 1 to 305 m with an average of 115 m. Excluding both sections from MT (30) site, where 78 m and 97 m of longitudinal cracking-WP occurred in sections 0805 and 0806, among cracked sections, the average crack length is 5 m with a range of 1 to 22 m. The average longitudinal cracking in the wheel path (WP) and non-wheel path (NWP) by experimental factors are shown in Figure 7-3 and Figure 7-5. More longitudinal cracking-WP is observed on sections located in DF zone. This cracking is mainly contributed by sections in the MT (30) site, which are constructed on coarse subgrade type. More longitudinal cracking-NWP is observed in all pavements constructed on active subgrade soils and located in WF zone. Also more longitudinal cracking-NWP is observed in sections located in DNF, which is contributed by sections in the NM (35) site only; these sections were constructed on fine-grained subgrade soils. The flexible pavements constructed on active fine-grained soils have shown slightly more longitudinal cracking-WP; the opposite trend is observed for pavements constructed on coarse-grained subgrade; however this is due to the contribution of only one section at the MT (30) site (see Figure 7-4). More longitudinal cracking-NWP is observed for flexible pavements constructed on active soils (see Figure 7-6).

Transverse Cracking: Transverse cracking was observed in only 10 out of 32 pavement sections. Among the cracked sections, transverse cracking length varies from 1 to 44 m with an average of 11 m. The average transverse cracking by experimental factors is shown in Figure 7-7. Pavements with "thick" [7-inch (178 mm)] HMA surface layer have shown less transverse cracking than those with "thin" [4-inch (102 mm)] HMA surface

layer. On average, more transverse cracking is exhibited by flexible pavements constructed on active soils and pavements located in "freeze" climates. The flexible pavements constructed on active subgrade have exhibited more transverse cracking than those constructed on non-active subgrade soils especially within fine subgrades (see Figure 7-7).

Roughness: The average roughness, in terms of IRI, by experimental factors is presented in Figure 7-9. The average initial IRI of the SPS-8 flexible pavement sections is 1.1 m/km, with a range of 0.8 to 3.2 m/km. The average change in IRI (ΔIRI) for pavements is 0.32 m/km with a range of 0.0 to 2.4 m/km. Excluding both sections from OH (39) site, where 2.2 m/km and 1.7 m/km of ΔIRI occurred in sections 0803 and 0804, the average ΔIRI is 0.2 m/km with a range of 0 to 1 m/km. On average, pavements located in "wet" climate have higher change in IRI than those in "dry" climate. Furthermore, pavements located in WF zone and those built on active soils have the higher changes in IRI. Also pavements constructed on active subgrade soils have exhibited more roughness than other pavements (see Figure 7-10).

Rut Depth: The average rut depth for flexible pavements by experimental factors is shown in Figure 7-11. The average latest rut depth of the sections is 5 mm with a range of 1 to 24 mm. Excluding section 39-0803, where 24 mm of rut depth was observed, the average rutting is about 4 mm with a range of 1 to 9 mm. On average, the magnitude of rutting observed is "low" for the sections. Slightly higher average rut depths were observed for the pavements located in WF zone.

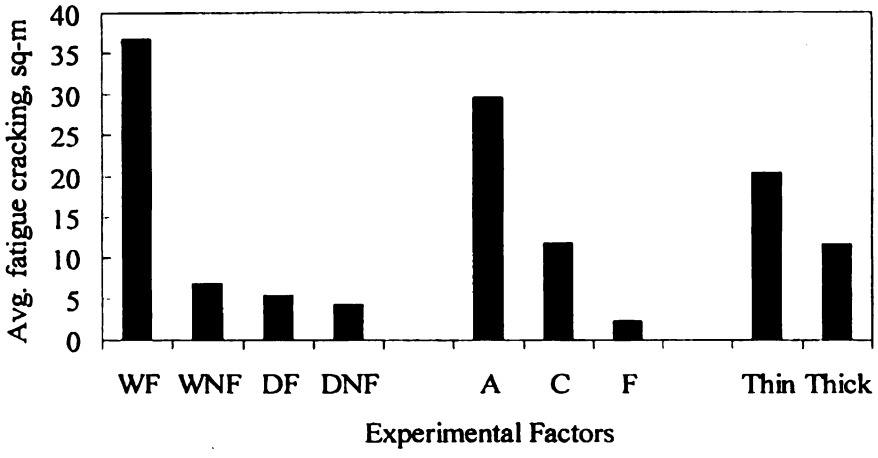


Figure 7-1 Average fatigue cracking by experimental factors— SPS-8 flexible pavements

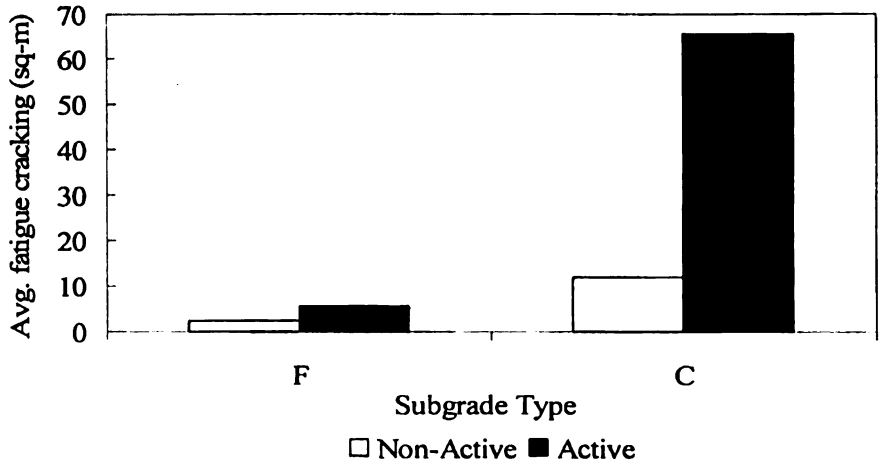


Figure 7-2 Average fatigue cracking by subgrade type— SPS-8 flexible pavements

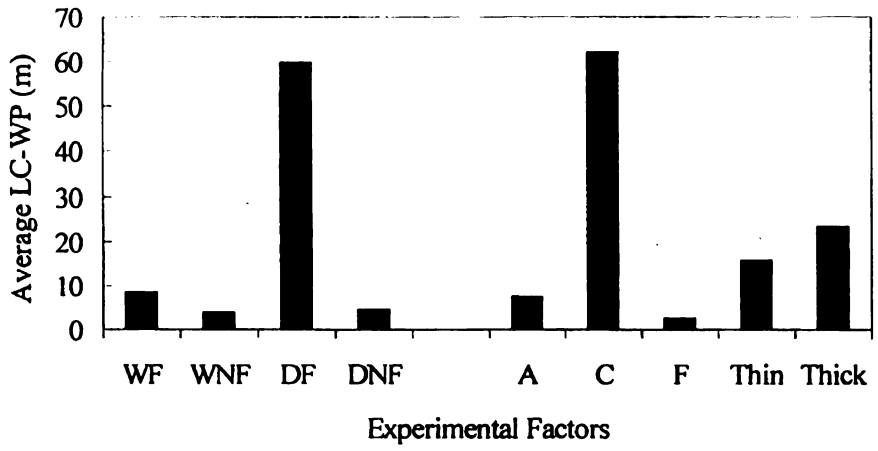


Figure 7-3 Average LC-WP by experimental factors— SPS-8 flexible pavements

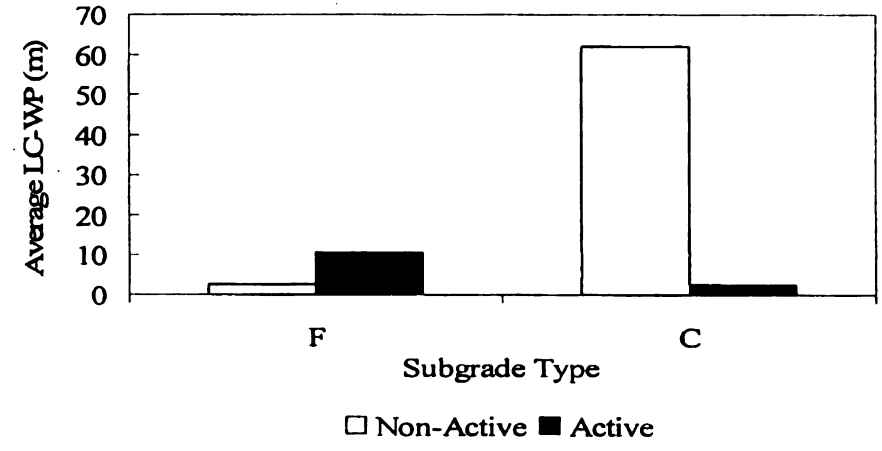


Figure 7-4 Average LC-WP by subgrade type— SPS-8 flexible pavements

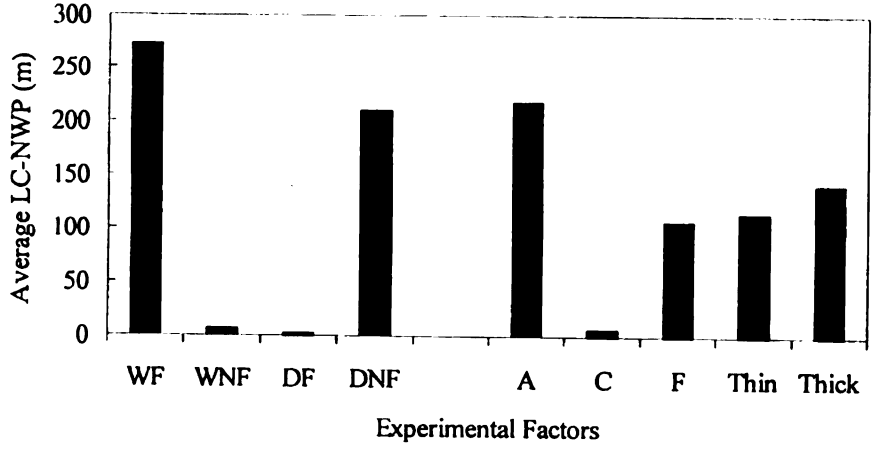


Figure 7-5 Average LC-NWP by experimental factors— SPS-8 flexible pavements

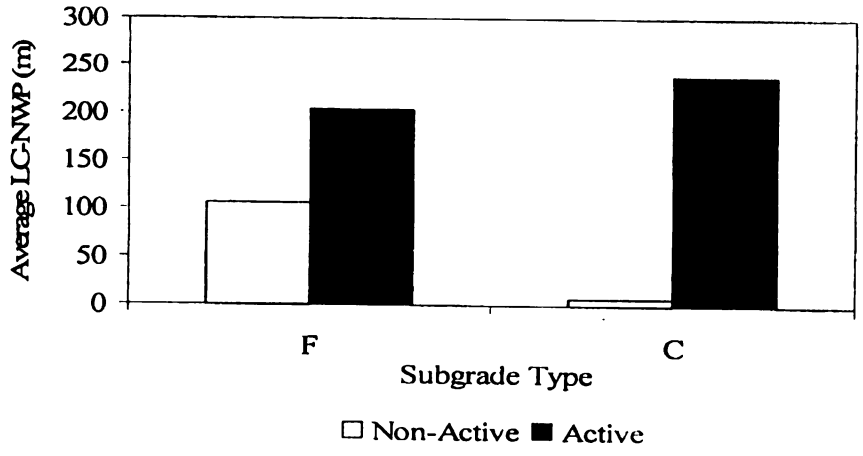


Figure 7-6 Average LC-NWP by subgrade type— SPS-8 flexible pavements

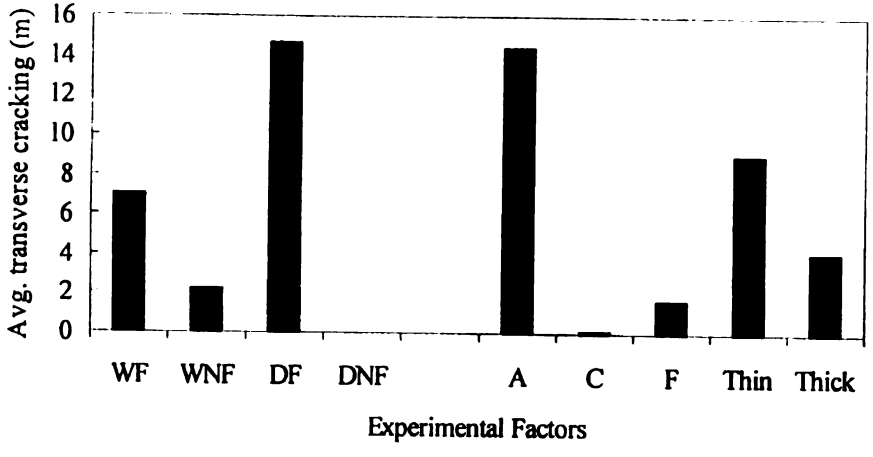


Figure 7-7 Average transverse cracking by experimental factors— SPS-8 flexible pavements

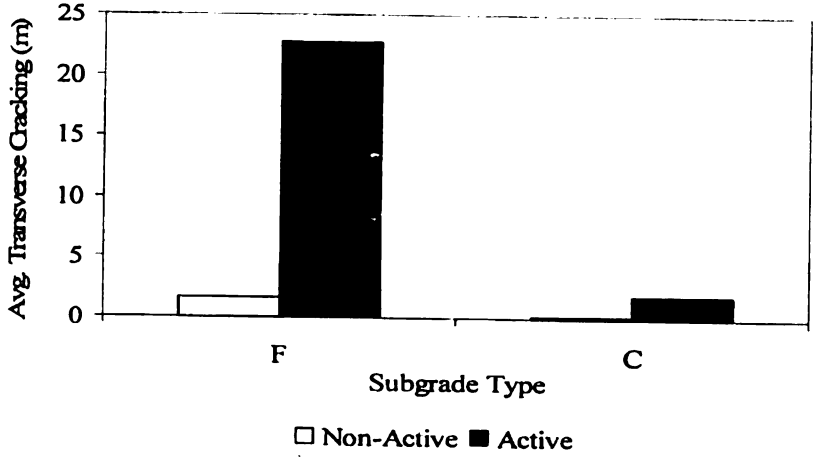


Figure 7-8 Average transverse cracking by subgrade type— SPS-8 flexible pavements

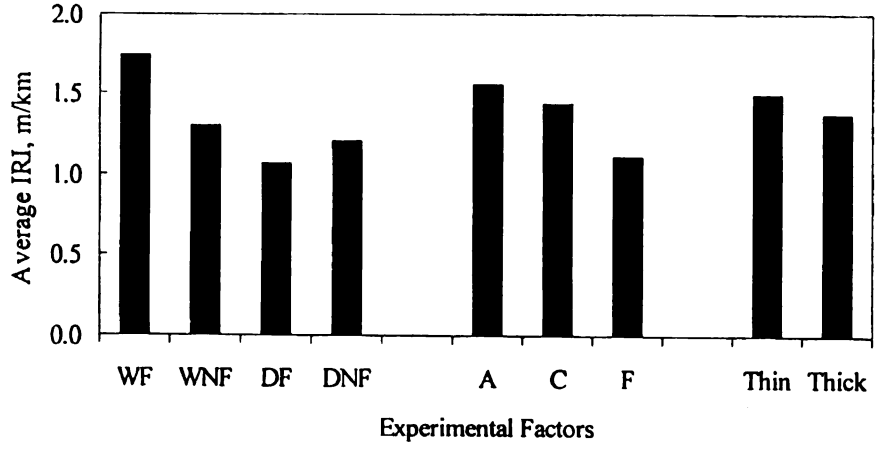


Figure 7-9 Average roughness by experimental factors— SPS-8 flexible pavements

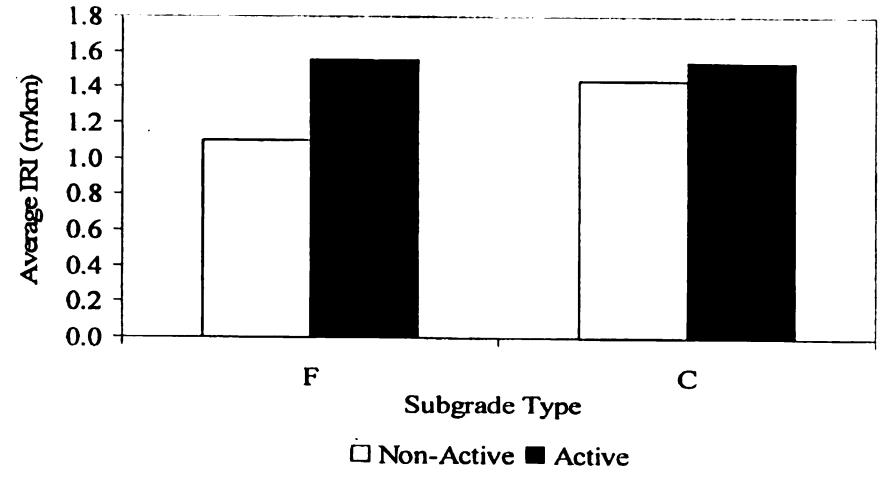


Figure 7-10 Average roughness by subgrade type— SPS-8 flexible pavements

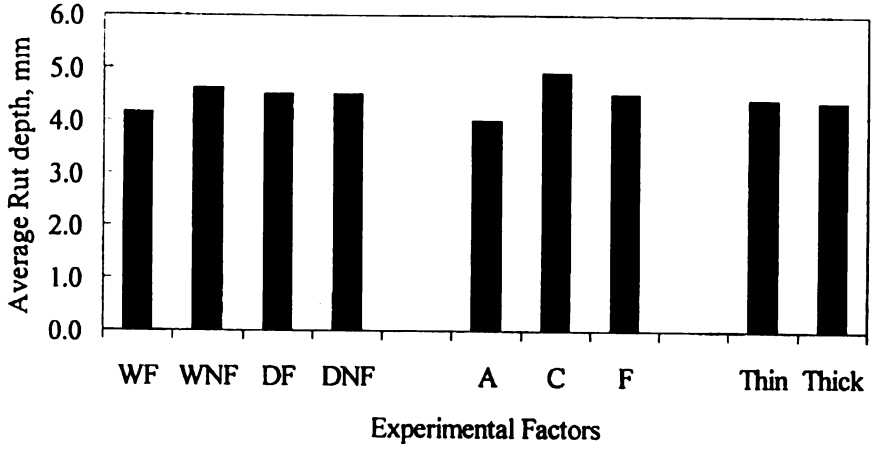


Figure 7-11 Average rut depth by experimental factors— SPS-8 flexible pavements

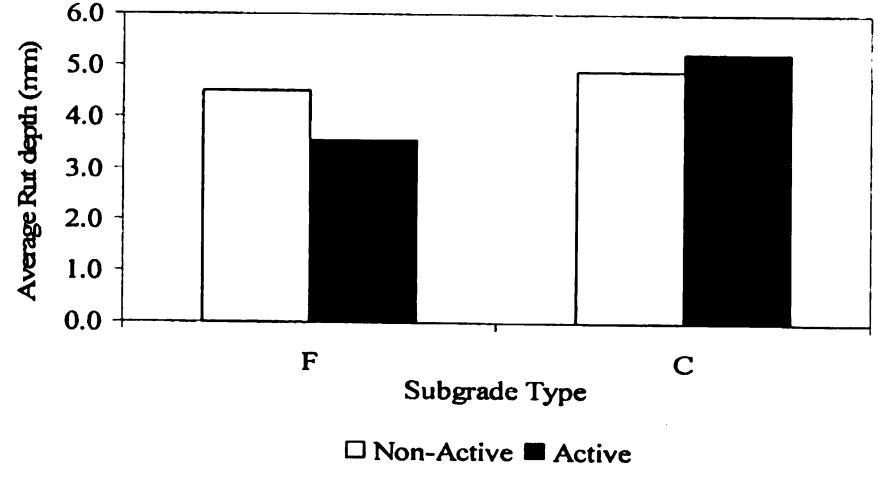


Figure 7-12 Average rut depth by subgrade type— SPS-8 flexible pavements

Also pavement sections with "thin" [4-inch (102 mm)] HMA surface layer have exhibited higher rutting than pavements with "thick" [7-inch (178 mm)] HMA surface layer. Furthermore, active subgrade seems to contribute towards more rutting (Figure 7-12). The magnitude of rutting was observed to be "low" for all sections in the experiment, which is expected since rutting is essentially a load-related distress.

7.2.3 Comparison between Similar Designs of SPS-8 and SPS-1 Experiments

To investigate the effect of traffic loading, similar designs in SPS-8 and SPS-1 experiments can be compared for various performance measures. This comparison may help in identifying load-related and environment-related distresses.

The median comparisons (non-parametric) were performed on similar sections in both experiments. From the experiment design matrix for SPS-1, sections 113 & 114 in all states were identified to be similar to the structural design of SPS-8 flexible pavement sections. To investigate the median difference between the performances of these sections, the sections were analyzed in two groups [4-inch (102 mm) and 7-inch (178 mm) HMA thickness].

It was found that fatigue cracking is essentially a load-related distress, while transverse cracking and longitudinal cracking-NWP may be attributed to environment. Table 7-1 shows the summary of results for these comparisons.

Table 7-1 Summary results for comparison between similar sections (SPS-8 vs. SPS-1)

Comparison	P-value	Remarks
Alligator cracking for 4"HMA thickness (SPS-8 vs. SPS-1)	0.0058	There is statistical significant difference between median performances of sections within both experiments. This implies that alligator cracking is more critical to traffic loads.
Longitudinal cracking-WP for 4"HMA thickness (SPS-8 vs. SPS-1)	0.0439	There is statistical significant difference between median performances of sections within both experiments. This shows that this distress is load related.
Longitudinal cracking-NWP for 4"HMA thickness (SPS-8 vs. SPS-1)	0.7068	There is no statistical significant difference between median performances of sections within both experiments. This means that this distress may not be load related.
Transverse cracking for 4"HMA thickness (SPS-8 vs. SPS-1)	0.7665	There is no statistical significant difference between median performances of sections within both experiments. This means that this distress may not be load related and can be attributed to environment.
Alligator cracking for 7" HMA thickness (SPS-8 vs. SPS-1)	0.0184	There is statistical significant difference between median performances of sections within both experiments. This implies that alligator cracking is more critical to traffic loads.
Longitudinal cracking-WP for 7" HMA thickness (SPS-8 vs. SPS-1)	0.7022	There is no statistical significant difference between median performances of sections within both experiments. This means that this distress may not be load related for thicker pavement sections.
Longitudinal cracking-NWP for 7" HMA thickness (SPS-8 vs. SPS-1)	0.1841	There is no statistical significant difference between median performances of sections within both experiments. This means that this distress may not be load related and can be attributed to environment.
Transverse cracking for 7" HMA thickness (SPS-8 vs. SPS-1)	N.A	There is no statistical significant difference between median performances of sections within both experiments.

Note: All the comparisons were made at 0.05 level of significance. N.A – Both median values are zero.

7.3 SUMMARY OF RESULTS

The SPS-8 experiment is entitled *Strategic Study of Environmental Effects in Absence of Heavy Loads for Flexible and Rigid Pavements*. The study examines the effect of climate and subgrade type (active, fine, and coarse) on pavement sections incorporating different flexible and rigid pavements, which are subjected to very limited traffic as measured by ESAL accumulation. The effects of the experimental factors on flexible and rigid pavements, based on the initial trends, are summarized below.

7.3.1 Effect of SPS-8 Experimental Factors on Performance of Flexible Pavements

Currently a total of 32 flexible pavement sections, in 15 sites, are present in the experiment. There are 14, 8, 6 and 4 pavement sections in WF, WNF, DF and DNF climatic zones, respectively. A total of 14 pavement sections were constructed on coarse-grained soils among which 4 sections are on "active" soils and 10 sections are on "non-active" soils. Also, 18 pavement sections were built on fine-grained soils, among which 12 sections are on "active" soils and 6 sections are on "non-active" soils. These test sections have an average age of about 6 years with a range of 3 to 10 years.

The SPS-8 pavements have "low" occurrence and extent of distresses, at this point. Most of the pavements in the experiment are performing at comparable levels. No formal statistical methods can be employed due to this. Therefore the observations presented here are just based on average performance of the distressed pavements. The observations, presented below, need to be considered as initial trends in light of these limitations. The effects of the design and site factors based on initial trends, as of Release 17.0, on key performance measures are summarized below:

- On average, pavements in WF zone have more fatigue cracking, longitudinal cracking-NWP, and roughness than pavements in other climates. Also, in general, pavements constructed on "active" subgrade (frost susceptible or expansive) soils have higher longitudinal cracking-NWP, transverse cracking, and fatigue cracking than pavements on "non-active" soils.
- Pavements located in "wet" climate, on average, have higher change in IRI than those in "dry" climate. Furthermore, pavements located in WF zone and those built on active soils have the higher changes in IRI.

CHAPTER 8 - CONCLUSIONS AND RECOMMENDATIONS

8.1 INTRODUCTION

This chapter presents a summary of findings from a comprehensive evaluation of SPS-1and SPS-8 experiments, based on mid-term performance trends [Release 17 (Level-E) of DataPave]. The current status of the experiment, construction quality, and data availability are also briefly discussed in this chapter for each experiment. Finally, the limitations of the findings from this research and recommendations for future data collection and research are presented.

A detailed description of the experiment designs and the current status of SPS-1, and SPS-8 experiments were presented in Chapter 3. The SPS-1 experiment is a fractional factorial experiment that was aimed at finding the relative influence of design and construction features on performance of new flexible pavements.

Each site within the SPS-1 experiment has twelve pavement test sections with each section representing a different structural design. There are eighteen sites in the SPS-1 experiment; these sites are distributed throughout the United States by climatic zones (wet-freeze, wet-no-freeze, dry-freeze and dry-no-freeze) and subgrade type (fine and coarse-grained). The SPS-1 experiment is designed to investigate the effects of HMA layer thickness, base type, base thickness, and drainage on flexible pavement performance. The current status (details in Chapter 3) of the experiments indicates some deviations from the intended experiment design for SPS-1 experiment. The most important deviation from the experiment design is the distribution of sites by climatic zone, which caused an unbalance in the number of sites per climatic zone.

The average age of test sections in the SPS-1 experiment is 7 years with a range of 3 to 11 years. It may thus be said that the pavements are "fairly young", and high occurrence and levels of distresses may not be expected at this point in time. Thus, all conclusions from the analyses presented in this report should only be interpreted as "mid-term" performance findings.

The SPS-8 experiment was designed to study the effects of the environment on pavement performance, in the absence of heavy traffic. The experimental factors include climate and subgrade soil type. A total of 32 flexible pavement sections in 15 were constructed for the experiment. The average age of the flexible pavement sections in SPS-8 is 6 years with a range of 3 to 10 years.

A summary of data availability was presented for SPS-1and SPS-8 experiments in Chapter 4. The extent and occurrence of distresses in the test sections within each of the experiments were also presented in Chapter 4. Experimental factors (design features and site factors) were compared to the as-constructed details obtained from the LTPP database and from construction reports. The deviations observed were reported in the chapter.

Based on the extent and occurrence of distresses, different methods of analysis were employed. A brief description of each of the methods used for this research is in Chapter 5. The majority of results from these analyses should be interpreted with caution in light of the "low" occurrence of distresses in the test sections within the SPS-1and SPS-8 experiments. This is especially true for the SPS-8 experiment. Also, it is suggested that the conclusions be considered while keeping in view the other limitations of the data, as explained later in this chapter.

A synopsis of the salient findings from all the analyses presented in previous chapters (Chapters 6 and 7), for each experiment, is presented in this chapter. This summary is intended to give the reader a brief overview of the effects of design and construction features on pavement performance and response.

The findings are presented by each performance or response measure, and not by design factor, as most of the experimental factors (design and site factors) are interacting among each other.

8.2 EFFECTS OF STRUCTURAL FACTORS FOR FLEXIBLE PAVEMENTS — SPS-1 EXPERIMENT

The SPS-1 experiment, entitled *Strategic Study of Structural Factors for Flexible Pavements*, is one of the nine special pavement studies in the LTPP program. The effects of the experimental factors on fatigue cracking, structural rutting, roughness, transverse cracking, and longitudinal cracking (WP and NWP) are discussed below.

It should be noted that the effects presented herein are statistically significant and of practical significance unless mentioned otherwise.

8.2.1 Effect of Design and Site Factors on Pavement Performance

The effects of the experimental factors on each performance measure are discussed below, one performance measure at a time.

Fatigue Cracking

- All the experimental factors were found to be affecting fatigue cracking, though not at the same level. On the whole, pavements with "thin" 4-inch (102 mm) HMA surface layer have shown more fatigue cracking than those with "thick" 7-inch (178 mm) HMA surface layer. Also pavements

constructed with only DGAB have shown more fatigue cracking than those with ATB-over-DGAB, and those with ATB base only, with the latter base type showing the best performance. The main effect of base thickness is not statistically significant. However, on average, pavements with 16-inch (406 mm) base thickness have shown slightly better fatigue performance than those with 8-inch (203 mm) or 12-inch (305 mm) base thickness. It should be noted that only pavement sections with drainage have a 16-inch (406 mm) base thickness according to the SPS-1 experiment design; therefore, it is unclear whether this effect is caused by the increased base thickness or by drainage provided with PATB. In this regard, the frequency-based analyses did show that pavements with drainage have significantly lower chances of cracking than those without drainage.

- In general, pavement sections built on fine-grained soils have more fatigue cracking than those built on coarse-grained soils. Also pavements located WF zone have shown more fatigue cracking than those located in WNF zone.
- Among un-drained pavements, on average, an increase in HMA surface thickness from 4-inch (102 mm) to 7-inch (178 mm) has a slightly higher effect on fatigue cracking for pavements with DGAB than for pavements with ATB. However, this effect is not statistically significant.
- The main effect of HMA surface thickness is more significant for sections built on coarse-grained soils.
- Among pavements built on fine-grained soils, the effect of drainage is seen only in those sections with DGAB; i.e., those with drainage have less fatigue

cracking than those without drainage. For drained pavements built on fine-grained soils, those with 8-inch (203 mm) base have more cracking than those with 12-inch (305 mm) and 16-inch (406 mm) base. Hence, for pavements built on fine-grained soils, drainage improves the fatigue performance, especially if built with thicker bases.

- The main effect of HMA thickness, discussed above, is mainly seen among sections located in WNF zone. This may be an indication that an increase of HMA thickness from 4-inch (102 mm) to 7-inch (178 mm) is not sufficient in resisting fatigue cracking for pavements in WF zone as compared to WNF zone.
- Among sections located in the WF zone, those with DGAB have shown the highest amount of cracking while those with ATB have the least cracking. In addition, those with 16-inch (406 mm) drained base have the least amount of fatigue cracking. This suggests that among pavements located in WF zone, "thick" 16-inch (406 mm) treated bases with drainage are less prone to cracking. The effects of HMA thickness and base thickness discussed above imply that, among sections located in WF zone, an increase in base thickness to 16-inch (with drainage) has a greater impact than an increase in HMA thickness from 4-inch (102 mm) to 7-inch (178 mm), suggesting that a thicker base and drainage helps in reducing frost effects.

Structural Rutting

The extent of structural rutting among the test sections in the SPS-1 experiment is 6.5 mm, on average, with a standard deviation of 2.4 mm. Their average age is about 7 years with a range between 4.5 and 10 years. The amount of rutting for the majority of these sections is within the normal range at this point in time. Therefore, the results at this point may only show initial trends and may not be of much practical significance.

- Marginal main effects of drainage, HMA thickness, and base thickness on structural rutting were observed. Pavements with "thin" [4-inch (102 mm)] HMA surface layer have shown slightly more rutting than those with "thick" [7-inch (178 mm)] HMA surface layer. Also, on average, pavements with 16-inch (406 mm) drained base have shown somewhat better rut performance than those with 8-inch (203 mm) and 12-inch (305 mm) base. However, these effects of HMA surface thickness and base thickness were not found to be statistically significant. Pavements with drainage have less rutting than those without drainage. The effect is not of practical significance, at this point in time.
- In general, pavement sections built on fine-grained subgrade have shown more rutting than those built on coarse-grained subgrade. On the other hand, there is no apparent effect of climate (WF vs. WNF) on structural rutting.
- Among the pavements built on coarse-grained soils, those with 7-inch (178 mm) HMA surface have shown slightly less rutting than those with 4-inch (102 mm) HMA surface. However, this effect is not operationally significant at this point.

- The above suggests that for sections built on fine-grained soils an increase in HMA thickness from 4-inch (102 mm) to 7-inch (178 mm) may not be sufficient in reducing the amount of rutting. Among pavements built on fine-grained soils, a marginal positive effect of drainage is seen in sections with ATB.
- Among drained pavements located in WF zone, those with DGAB have shown more rutting than those with ATB. Also, among sections located in WF zone and built with ATB, those with drainage have shown significantly less rutting than those without drainage. This implies that, among pavements located in WF zone, those with ATB and drainage perform better than those with other combinations of base type and drainage.
- Among un-drained sections located in WNF zone, those with 12-inch (305 mm) base have less rutting than those with 8-inch (203 mm) base. For sections built on DGAB and located in WNF zone, those with drainage have shown slightly less rutting than those without drainage. This effect was found to be marginally significant. These early trends imply that the importance of drainage among pavements with DGAB is considerable in improving rut performance among sections located in WNF zone. On the other hand an increase in base thickness from 8-inch (203 mm) to 12-inch (305 mm) improves rut performance for un-drained sections, irrespective of base type.

Roughness

- All the experimental factors were found to be affecting roughness, though not at the same level. Pavements with "thin" [4-inch (102 mm)] HMA surface

layer have higher change in IRI (ΔIRI) than those with "thick" [7-inch (178 mm)] HMA surface layer. This effect is not of practical significance at this point in time. Also, pavements constructed with DGAB have higher ΔIRI than those with ATB/DGAB and ATB, while pavements with ATB have the best performance for roughness. Pavements with thicker bases have lower ΔIRI. Also pavements with drainage have lower ΔIRI than un-drained pavements.

- In general, pavements built on fine-grained soils have shown higher ΔIRI than those built on coarse-grained soils, especially among sections in WF zone. Also, the change in roughness among sections located in WF zone is significantly higher than those in WNF zone.
- Among pavements built on fine-grained soils, an increase in HMA thickness from 4-inch (102 mm) to 7-inch (178 mm) has a significant positive effect on change in roughness. Also for un-drained pavements, those with ATB have significantly lower ΔIRI than those with DGAB. Finally the effect of drainage is significant only for sections with DGAB. These effects suggest that, for pavements built on fine-grained soils, higher HMA thickness and/or treated base will help inhibit the increase in roughness. Also, drainage appears to be more effective in preventing an increase in roughness for sections with DGAB, especially among those located in WF zone.
- For un-drained pavements built on coarse-grained soils, an increase in base thickness from 8-inch (203 mm) to 12-inch (305 mm) causes lower ΔIRI. However, this effect is marginally significant and is not of practical significance at this point in time.

Transverse Cracking

- The effect of base thickness on transverse cracking is insignificant, at this point. Pavements constructed with DGAB have more transverse cracking than those with ATB/DGAB and ATB, while pavements with ATB have shown the least amount of cracking. However the effect is not of practical significance at this point in time. Slightly more cracking was observed on pavements with "thin" [4-inch (102 mm)] HMA surface layer. Also, pavements with drainage have shown slightly less cracking than un-drained pavements. However, these effects were not found to be statistically significant.
- In general, pavements built on fine-grained soils have shown more transverse cracking than those built on coarse-grained soils.
- Pavements located in WF zone have shown significantly more transverse cracking than those located in WNF zone. This confirms that transverse cracking occurs mainly in freezing environment.
- Among drained pavements built on coarse-grained soils, those with ATB performed better than those with DGAB.
- Among pavements with DGAB and built on fine-grained soils, those with drainage have shown significantly less transverse cracking than those without drainage.

Longitudinal Cracking-WP

- The effects of HMA and base thickness on longitudinal cracking-WP are insignificant at this point in time. Pavements with drainage have shown less

cracking than un-drained pavements. The main effect of drainage is not of practical significance at this point.

- In general, pavements built on fine-grained soils have shown more longitudinal cracking-WP than those built on coarse-grained soils.
- On average pavements in WF zone have shown higher levels of longitudinal cracking-WP than those in WNF, especially among pavements built on fine-grained subgrade. This effect was found to be only marginally significant.
- Among pavements built on fine-grained soils, those built with DGAB have shown more longitudinal cracking-WP, and those built with ATB have shown the least amount of cracking. Also, drainage has a significant effect on longitudinal cracking, and this effect is more pronounced in pavements built with DGAB. This trend implies that if a pavement on fine-grained subgrade is constructed with a DGAB base, better performance (in terms of longitudinal cracking-WP) can be achieved by providing drainage. These effects are seen in both WF and. WNF zones.

Longitudinal Cracking-NWP

- The effects of HMA thickness, base thickness, and base type on longitudinal cracking-NWP are insignificant at this point in time. Pavements with drainage have shown slightly less cracking than un-drained pavements. However, the effect of drainage was found to be only marginally significant.
- The effect of subgrade type was not found to be statistically significant.
- In general, more longitudinal cracking-NWP was observed among sections located in "freeze" climate compared to those in "no-freeze" climate.

- The effect of drainage is more pronounced (with marginal statistical significance) among pavements located in "freeze" climate. However, this effect is not of practical significance.

These initial trends indicate that longitudinal cracking-NWP is caused by "freeze" climate (frost effects), and that pavements without drainage may be more prone to it.

In summary, based on the above discussion for SPS-1 experiment, base type seems to be the most critical design factor for fatigue cracking, roughness (IRI), and longitudinal cracking-WP. This is not to say that the effect of HMA surface thickness is not significant. In fact, the effect of base type should be interpreted in light of the fact that a dense graded asphalt treated base effectively means thicker HMA layer. Drainage and base type, when combined also play an important role in improving flexible pavement performance, especially in terms of fatigue and longitudinal cracking. Base thickness has secondary effects on performance, especially in the case of roughness and rutting.

Subgrade soil type seems to be playing an important role in flexible pavement performance. In general, pavements built on fine-grained soils have shown worst performance, especially in the case of roughness. Also, climate is a critical factor in determining flexible pavement performance. Longitudinal cracking-NWP, transverse cracking, and longitudinal cracking-WP appear to be affected by climate. Longitudinal cracking-WP and transverse cracking seems to be associated with Wet Freeze environment, while longitudinal cracking-NWP seems to be the dominant in "freeze" climate.

8.2.2 Effect of Design and Site Factors on Pavement Response

Three pavement response parameters were chosen for ANOVA – peak deflection under FWD load (d_0), far-sensor deflection (d_6), and AREA. A summary of the effects of design and site factors on each of the response parameters follows.

Peak Deflection under FWD Load (d_0): For pavement sections built on DGAB, those with 4-inch (102 mm) HMA surface thickness have higher d_0 than those with 7-inch (178 mm) HMA surface thickness. Also pavements with thicker bases, irrespective of base type, have lower d_0. This effect is more prominent in the case of sections with treated bases (ATB or ATB/DGAB). Also, pavement sections with PATB/DGAB have lower d_0 than those with DGAB.

In general, pavements built on fine-grained soils have shown significantly higher d_0 as compared to those built on coarse-grained soils. This effect is more prominent on pavements located in WNF zone.

Far Sensor Deflection (d6): Pavements built on fine-grained soils have higher d_6 values as compared to those built on coarse-grained soils. This effect is more prominent on pavements located in WNF zone.

Pavements built with DGAB have shown higher d_6 values than those built on other base types. Pavements constructed on 8-inch (203 mm) bases have also shown significantly higher d_6 values than those built on 12-inch (305 mm) or 16-inch (406 mm) bases. Furthermore, pavements with PATB/DGAB have smaller d_6 values than those with DGAB. These effects of the design factors on d_6 are based on statistical analyses only, and may or may not be of practical importance.

AREA: For pavements built on DGAB, those with "thin" HMA surface layer have lower AREA values compared to those with "thick" HMA surface layer, implying that the upper layers of these pavements are "less stiff".

For pavements built on DGAB, increasing base thickness from 8-inch (203 mm) to 12-inch (305 mm) has not shown a significant effect on AREA; however a two-fold increase in base thickness [from 8 to16 inch (203 to 406 mm)] has shown a significant increase in AREA. Furthermore, pavements with PATB/DGAB have higher AREA values than those with DGAB. This may be an indication that the structural capacity of the PATB layer is somewhat higher than that of the DGAB.

8.2.3 Apparent Relationship between Response and Performance

Two types of relations between flexible pavement response (FWD) and performance were explored for the SPS-1 pavements— explanatory and predictive. The salient findings are briefly presented below:

Overall Analysis— Explanatory Relationship

Following are findings regarding relationship between response and performance based on regression analysis, after adjusting for HMA surface thickness and pavement mid-depth temperature at the time of testing:

- Older pavements have slightly lower deflections (d_0) compared to younger pavements, which could be due to stiffening (aging) of the asphalt.
- Pavements with "weaker" subgrade (higher d_6) have higher d_0 values.
- Pavements with more cracking (fatigue cracking or longitudinal cracking) have a higher d_0 values, compared to those with less cracking.

Site Level Analysis— Predictive Relationships

This section summarizes the observations regarding the predictive relationships between initial response and future pavement performance, based on site-level analysis.

- In most of the sites, pavements with higher initial SCI or BDI, or lower initial AREA have higher fatigue cracking.
- In most of the sites, pavements with higher initial BDI have higher IRI.
- The deflection basin parameters have not shown a consistent relationship with rut depth for the various sites in the SPS-1 experiment.

Overall Analysis— Predictive Relationships

The main observations based on the analyses are as follows:

- For pavements constructed on fine-grained soils, ones with higher SCI have shown more fatigue cracking, especially in WNF zone.
- Stiffer pavements (higher AREA) built on fine-grained soils have shown more fatigue cracking, especially if located in WF climatic zone.
- Higher longitudinal cracking-WP was observed for the pavement sections with higher AREA, especially among pavements located in WNF climatic zone.
- No apparent relation was observed between AREA and longitudinal cracking-NWP or transverse cracking, implying that these distresses could be independent of the pavement structural capacity.

Dynamic Load Response for OH (39) test sections

The observations based on analysis of DLR data from instrumented sections are:

- In general, the strains in the longitudinal direction are higher than the strains in the transverse direction.

- The sections that were observed to have higher initial strain values have shown worse fatigue performance.
- The sections that were observed to have high initial stress at the top of the subgrade layer and those that were observed to have high initial surface deflection under the load have shown poor rut performance.

8.3 EFFECTS OF THE ENVIRONMENT IN THE ABSENCE OF HEAVY TRAFFIC FOR FLEXIBLE PAVEMENTS — SPS-8 EXPERIMENT

The SPS-8 experiment is entitled *Strategic Study of Environmental Effects in the Absence of Heavy Loads for Flexible and Rigid Pavements*. The study examines the effect of climate and subgrade type (active, fine, and coarse) on pavement sections incorporating different flexible and rigid pavements, which are subjected to very limited traffic as measured by ESAL accumulation.

The SPS-8 pavements have "low" occurrence and extent of distresses, at this point. Most of the pavements in the experiment are performing at comparable levels. No formal statistical methods can be employed due to this. Therefore the observations presented here are just based on average performance of the distressed pavements. The observations, presented below, need to be considered as initial trends in light of these limitations.

On average, pavements in WF zone have more fatigue cracking, longitudinal cracking-NWP, and roughness than pavements in other climates. Also, in general, pavements constructed on "active" subgrade (frost susceptible or expansive) soils have higher longitudinal cracking-NWP, transverse cracking, and fatigue cracking than pavements on "non-active" soils.

Pavements located in "wet" climate, on average, have higher change in IRI than those in "dry" climate. Furthermore, pavements located in WF zone and those built on active soils have the higher changes in IRI.

8.4 LIMITATIONS OF THE EXPERIMENTS AND ANALYSES

All the above findings/observations on the effects of design and construction features on pavement performance and response should be considered in light of the limitations discussed herein. These limitations can be broadly classified under two categories— experiment-related and data-related.

8.4.1 Experiment-related issues

- The SPS-1and SPS-8 experiments, which are fractional-factorial designs, were rendered unbalanced because unequal numbers of sites were constructed in each zone-subgrade combination. This unbalanced design limits the "power" of the experiments. In the SPS-1 experiment only 2 sites each are located in DF and DNF zones, compared to 8 and 6 sites in WF and WNF zones. Moreover, both the sites of SPS-1 in the DF zone are located on coarse-grained soils. Furthermore, in some of the sites not all sections were constructed on the same subgrade soil type [for example, KS (20)].
- Initially, the SPS-1 experiment was designed to have all 24 designs at a site. But later due to some implementation issues, 12 designs were constructed per site. Hence, the experiments do not have any "true" (statistical) replication of designs. In other words, though two sites (with 12 designs at each site) are located in a climate-subgrade soil combination, the traffic, age, and material-related properties vary between the sites.

- The variation in age of the sites is considerably high for the experiments. If the sites were reasonably similar in age, the findings would be more reliable. It should be noted that age of the test sections was included as a covariate in all statistical analyses to address the above issue to some extent.
- In the SPS-1 experiment, in-pavement drainage was provided only for sections built with PATB. Moreover, all sections with PATB were provided with drainage. As a result of this, the effect of PATB and the effect of drainage are inseparable (confounded).
- In the SPS-1 experiment, a 16-inch (406 mm) thick base was only provided for sections with drainage. In other words, none of the sections without drainage have a 16-inch (406 mm) base thickness. Hence, the effect of a 16-inch (406 mm) thick base and the effect of drainage are also inseparable.
- Among flexible pavement sections of SPS-1, all sections built with ATB-over-DGAB were not provided with drainage. Hence, any interaction effects of drainage and ATB-over-DGAB cannot be studied.
- Only the lower limit for traffic volume was specified for the SPS-1 experiment. This resulted in considerable variability in traffic across sites.
- In the SPS-8 experiment, the effects of HMA surface thickness and base thickness are confounded. Therefore, the "pure" effects of any of these factors cannot be studied.

8.4.2 Data-related issues

- Reasonably accurate monitored traffic data is not available for all the sites in the experiments. This has further complicated the issue of controlling for traffic.

- Large measurement variability was observed, over time, for some of the distresses (for example, longitudinal cracking in SPS-1). This variability has made the time-series trends unclear for some of the performance measures.
- The measurement variability discussed above is believed to be due to maintenance activity at some sites, which is a deviation from the experiment design. These activities tamper with the actual long-term field performance of the pavement sections.
- The frequency of distress surveys is not uniform across sites, especially for SPS-1 experiments. Wide gaps in distress surveys necessitate interpolation of performance, which may not always be accurate.
- Though thorough construction guidelines were developed by the LTPP to minimize construction variability across sites, some deviations occurred. These deviations along with the material variability have added to the variability in performance across sites. If material-related information were available for all the sections, the issues caused by performance variability could be better addressed.
- Backcalculated layer moduli are unavailable for most of the sections in the SPS-1 experiment. Some of the material-related issues could be dealt with if the data were present.

8.5 RECOMMENDATIONS FOR FUTURE DATA COLLECTION AND RESEARCH

Based on the above issues and the experience with the LTPP data, recommendations for future data collection and research are given below.

1) Reasonably accurate monitored traffic data should be made available for all the sites to allow for better adjustment of traffic loading variation across sites.
2) All the core sections of the SPS-1 experiment should be closely monitored until failure or to a stage when the long-term performance (at least 15-20 years) has been captured.
3) The core sections of the SPS-1experiment should be strictly supervised to prevent any maintenance activity, as per the experiment designs. This will ensure that the actual long-term performance of the pavements is observed.
4) Most of the test sections will soon enter a critical stage in their service life; in light of this, to reap maximum benefits from the experiments, the sections should be monitored at regular intervals and with greater accuracy.
5) Hall et al [1] have identified issues regarding in-pavement drainage for sections in SPS-1 and SPS-2 experiments. The findings from this study should be considered for inclusion in the DataPave IMS database. This may help study the effect of in-pavement drainage more accurately.
6) Some of the sections in the SPS-1 experiment have shown premature "failure". These sections should be considered for exclusion from DataPave, as they do not contribute to the study of long-term pavement performance.
7) Some of the sites in the SPS-1 experiment are very close to the thresholds (regarding average annual precipitation and freeze index) defined for delineation of climatic zones. The definitions of the climatic zones may need reconsideration in light of this.

8) Accurate material data should be made available for all the sections of the experiments to allow for addressing the variability in material quality across sites. Also, backcalculated layer moduli data should be made available for all sections in DataPave to help perform mechanistic analyses.

9) Most of the construction-related issues are available only from construction reports. These issues and/or deviations should be better highlighted well within the DataPave database.

10) The spatial location of some distresses (such as cracking) is sometimes important for research. It is practically cumbersome for the users to obtain distress maps and interpret the spatial location of the distresses. It is therefore recommended that each section be "discretized" (segmented) for data collection and the related data be made available in DataPave. This would greatly decrease the level of subjectivity in the data.

11) In general, the extent of distresses on the SPS-8 test sections is "low", at this point in time. The performance data should thus be collected for sufficiently long time (15 to 20 years for all the sections in the experiment) to capture the effects of environment. A meaningful statistical analysis may then be performed to study the effects of environment.

12) The DLR instrumentation location (spatial location), alignment and designation details should be made more accurate in DataPave.

13) It is recommended that the complete actual traces of data from the instrumented DLR sections be considered for inclusion in the DataPave database (in addition to the "peak" and "valley" data) after proper quality checks. Also more data (i.e.

more test series results) should be included in the DataPave. This data could be stored separately as in the case of profiles and FWD time histories.

When the long-term performance data is available for most of the sections of the SPS-1 and SPS-8 experiments, the methods employed in this research would be more "powerful" to study the effect of design and construction features. Methods that analyze the time-series data (such as survival analysis and ANOVA with repeated measures) can also be employed when the performance data for most of the sections is available for about 15 years.

REFERENCES

CHAPTER 1

1. National Research Council, S., Specific Pavement Studies Experimental Design and Research Plan for Experiment SPS-1, Strategic Study of Structural Factors for Flexible Pavements. Feb, 1990, SHRP: Washington, DC.

CHAPTER 2

1. Embacher, R.A., M.B. Snyder, and T.D. Odden, Using the Minnesota accelerated loading facility to test retrofit dowel load transfer systems. Transportation Research Record, 2001(1769): p. 134-141.

2. Haung, Y.H., Pavement Analysis and Design. 2nd Edition ed. 2004, Upper Saddle River, NJ 07458: Pearson Prentice Hall.

3. Yoder, E.J. and M.W. Witczak, Principles of Pavement Design. 1975, John Wiley & Sons, Inc.

4. Dempsey, B.J., Laboratory and Field Studies of Channeling and Pumping. Transportation Research Record. 849.

5. Quintus, H.V. and A.L. Simpson, Structural Factors for Flexible Pavements-Initial Evaluation of the SPS-1 Experiment, Final Report. 2001, FHWA: Washington, DC.

6. Hall, T.K., Effectiveness of Subsurface Drainage for HMA and PCC Pavements. 2002, National Corporative Highway Research Program: Washington, DC.

7. Hall, T.K. and E.C. Carlos, Effects of Subsurface Drainage on Performance of Asphalt and Concrete Pavements. 2003.

8. Mladenovic, G., Y.J. Jiang, and M.I. Darter, Study of Environmental Effects in the Absence of the Heavy Traffic Load-SPS-8. 2002, FHWA: Washington, DC.

9. Perera, R.W. and S.D. Kohn, LTPP Data Analysis: Factors Affecting Pavement Smoothness. 2001, BRE.

10. Rauhut, J.B., A. Eltahan, and A.L. Simpson, Common Characteristics of Good and Poorly Performing AC Pavements. 1999, FHWA.

11. Perera, R.W., C. Byrum, and S.D. Kohn, Investigation of Development of Pavement Roughness. 1998, FHWA, FHWA-RD-97-147: Washington, DC.

12. Ali, A.H. and D.T. Shiraz, Mechanistic Evaluation of Test Data from LTPP Flexible Pavement Test Sections. 1998.

13. Quintus, H.V. and K. Brian, Analysis Relating to Pavement Material Characterizations and their Effects on Pavement Performance. 1998, FHWA: Washington, DC.

14. Kim, R.Y. and S.R. Ranjithan, Assessing Pavement Layer Condition Using Deflection Data. 2000, NCHRP 10-48.

15. Stubstad, R.N., et al., LTPP Data Analysis: Feasibility of Using FWD Deflection Data to Characterize Pavement Construction Quality. 2002, NCHRP, NCHRP's Project 20-50(9), Final Report.

16. AASHTO, Guide for Design of Pavement Structures. 1993, American Association of State Highway and Transportation Officials: Washington, DC.

17. Ullidtz, P., Pavement Analysis. 1987, Amsterdam, The Netherlands: Elsevier Science Publisher B.V.

18. Ullidtz, P., Modeling Flexible Pavement Response and Performance. 1998, Gylling, Denmark: Narayana Press.

19. Simpson, A.L., et al., Sensitivity Analysis for Selected Pavement Distresses. 1994, Strategic Highway Research Program, National Research Council: Washington, DC.

CHAPTER 3

1. Hanna, A.N., S.D. Tayabji., and J.S. Miller, SHRP-LTPP Specific Pavement Studies: Five-Year Report. 1994, Strategic Highway Research Program, National Research Council.

2. National Research Council, S., Specific Pavement Studies Experimental Design and Research Plan for Experiment SPS-1, Strategic Study of Structural Factors for Flexible Pavements. Feb, 1990, SHRP: Washington, DC.

3. Kutner, M.H., et al., Applied Linear Models. 5th ed. 2005: McGraw-Hill, Irwin.

4. Mee, R.W., Efficient Two-Level Designs for Estimating All Main Effects and Two-Factor Interactions. Journal of Quality Technology, 2004. 36(No. 4): p. 400-412.

5. Chatti, K., et al., Final Report NCHRP 20-50 (10/16), LTPP Data Analysis: Influence of Design and Construction Features on the Response and Performance of New Flexible and Rigid Pavements. 2005, Michigan State University: East Lansing.

6. Quintus, H.V. and A.L. Simpson, Structural Factors for Flexible Pavements-Initial Evaluation of the SPS-1 Experiment, Final Report. 2001, FHWA: Washington, DC.

CHAPTER 4

1. Chatti, K., et al., Final Report NCHRP 20-50 (10/16), LTPP Data Analysis: Influence of Design and Construction Features on the Response and Performance of New Flexible and Rigid Pavements. 2005, Michigan State University: East Lansing.

2. Quintus, H.V. and A.L. Simpson, Structural Factors for Flexible Pavements-Initial Evaluation of the SPS-1 Experiment, Final Report. 2001, FHWA: Washington, DC.

3. Hall, T.K. and E.C. Carlos, Effects of Subsurface Drainage on Performance of Asphalt and Concrete Pavements. 2003.

4. Chen, D.H., et al., Forensic Evaluation of Premature Failures of Texas Specific Pavement Study-1 Sections. Journal of Performance and Constructed Facilities, 2003. Vol. 17(No. 2): p. 67-74.

5. Hanna, A.N., S.D. Tayabji., and J.S. Miller, SHRP-LTPP Specific Pavement Studies: Five-Year Report. 1994, Strategic Highway Research Program, National Research Council.

6. ORITE, Continued Monitoring of Instrumented Pavement in Ohio. 2001, Institute for Transportation and the Environment (ORITE), Ohio University: Ohio.

7. Holtz, R.D. and W.D. Kovacs, An Introduction to Geotechnical Engineering. 1981: Prentice-Hall.

CHPATER 5

1. Rauhut, J.B., A. Eltahan, and A.L. Simpson, Common Characteristics of Good and Poorly Performing AC Pavements. 1999, FHWA.

2. Khazanovich, L., et al., Common Characteristics of Good and Poorly Performing PCC Pavements. 1998, FHWA.

3. Quintus, H.V. and A.L. Simpson, Structural Factors for Flexible Pavements-Initial Evaluation of the SPS-1 Experiment, Final Report. 2001, FHWA: Washington, DC.

4. Hall, T.K. and E.C. Carlos, Effects of Subsurface Drainage on Performance of Asphalt and Concrete Pavements. 2003.

5. Larsen, J.R. and L.M. Morris, An Introduction to Mathematical Statistics and Its Applications. 3rd ed. 2001: Prentice Hall.

6. Kutner, M.H., et al., Applied Linear Models. 5th ed. 2005: McGraw-Hill, Irwin.

7. Dillon, W.R. and G. Matthew, Multivariate Analysis-Methods and Applications. 1984, New York: John Wiley & Sons, Inc.

8. Ott, R.L. and L. Michael, An Introduction to Statistical Methods and Data Analysis. 5th Edition ed. 2001, CA: Duxbury.

9. Khazanovich, L., et al., Common Characteristics of Good and Poorly Performing PCC Pavements. 1998, FHWA.

10. Stroup, W.W., Mixed Models Procedures to Assess Power, Precision and Sample Size in the Design of Experiments. 2004: University of Nebraska, Lincoln, NE.

11. Neter, J., et al., Applied Linear Statistical Models. 4th ed. 1996, Illinois: Richard D. Irwin, Inc.

12. Hastie, T., T. Tobert, and J. Friedman, The Elements of Statistical Learning: Data Mining, Inference, and Prediction. 2001, New York: Springer-Verlag.

13. Hair, J.F., Jr., et al., Multivariate Data Analysis. 5th ed. 1998, New Jersey: Prentice Hall.

14. Hosmer, D.W. and L. Stanley, Applied Logistic Regression. 2nd ed. 2000, NY.: John Wiley & Sons, Inc.

CHAPTER 6

1. Quintus, H.V. and A.L. Simpson, Structural Factors for Flexible Pavements-Initial Evaluation of the SPS-1 Experiment, Final Report. 2001, FHWA: Washington, DC.

2. Perera, R.W. and S.D. Kohn, LTPP Data Analysis: Factors Affecting Pavement Smoothness. 2001, BRE.

3. Hall, T.K., Effectiveness of Subsurface Drainage for HMA and PCC Pavements. 2002, National Corporative Highway Research Program: Washington, DC.

4. Hall, T.K. and E.C. Carlos, Effects of Subsurface Drainage on Performance of Asphalt and Concrete Pavements. 2003.

5. White, T.D., et al., Contribution of Pavement Structural Layers to Rutting of Hot Mix Asphalt Pavements. 2002, Purdue University: West Lafayette, IN.

6. Chen, D.H., et al., Forensic Evaluation of Premature Failures of Texas Specific Pavement Study-1 Sections. Journal of Performance and Constructed Facilities, 2003. Vol. 17(No. 2): p. 67-74.

7. Chatti, K., et al., Final Report NCHRP 20-50 (10/16), LTPP Data Analysis: Influence of Design and Construction Features on the Response and Performance of New Flexible and Rigid Pavements. 2005, Michigan State University: East Lansing.

8. Stubstad, R.N., et al., LTPP Data Analysis: Feasibility of Using FWD Deflection Data to Characterize Pavement Construction Quality. 2002, NCHRP, NCHRP's Project 20-50(9), Final Report.

9. Kim, R.Y. and S.R. Ranjithan, Assessing Pavement Layer Condition Using Deflection Data. 2000, NCHRP 10-48.

10. Lukanen, E.O., R. Stubstad, and R. Briggs, Temperature Predictions and Adjustment Factors for Asphalt Pavements. 2000, FHWA, FHWA-RD-98-085: Washington, DC.

Chapter 8

1. Hall, T.K. and E.C. Carlos, Effects of Subsurface Drainage on Performance of Asphalt and Concrete Pavements. 2003.


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