NCHRP Web Document 40 (Project 20-50[8/13]): Contractor's Final Report

LTPP Data Analysis: Factors Affecting Pavement Smoothness

Prepared for:

National Cooperative Highway Research Program
Transportation Research Board
National Research Council

Submitted by:

R. W. Perera
S. D. Kohn
Soil and Materials Engineers, Inc.
Plymouth, Michigan

August 2001

ACKNOWLEDGMENT

This work was sponsored by the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program (NCHRP), which is administered by the Transportation Research Board (TRB) of the National Research Council.

CONTENTS

ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 20-50(08/13) by Soil and Materials Engineers (SME). Dr. Starr D. Kohn, Manager of Pavement Services, and Dr. Rohan W. Perera, Project Engineer, served as co-principal investigators for this project. The analysis work for this project was performed by Dr. Rohan Perera. Ken Dani of SME created the analysis database, while Christopher Byrum of SME worked on development of models for GPS-1 sections and cumulative traffic analysis. Dr. Julian Faraway of University of Michigan provided input on the statistical analysis, and introduced the team to longitudinal data analysis.

SUMMARY

It is believed that the general public perceives a good road as one that provides a smooth ride. Studies at the road test sponsored by the American Association of State Highway Officials showed that the subjective evaluation of the pavement based on mean panel ratings was primarily influenced by roughness. Therefore, the development of roughness on pavements is a major issue for highway agencies.

Although pavement smoothness has been recognized as one of the important measures of pavement performance, the contribution of factors such as pavement structure, rehabilitation techniques, climatic conditions, traffic levels, layer materials and properties, and pavement distress to changes in pavement smoothness are not well documented. Without this information, the selection of appropriate pavement design structure, design features, and rehabilitation strategies that will ensure long-term smoothness is a difficult task. The data collected for the Long Term Pavement Performance (LTPP) study provides an opportunity to investigate the effect of these factors on the development of roughness.

In this research project, data available in the LTPP Information Management System (IMS) was used to determine the effect of factors such as design and rehabilitation parameters, climatic conditions, traffic levels, material properties, and extent and severity of distress that cause changes in pavement smoothness. For the purposes of this research, the International Roughness Index (IRI) was used as the measure of pavement smoothness. The IRI is a smoothness index that is widely used in the United States, and can be calculated for any profile that is measured by an inertial profiler. The LTPP program consists of two complementary programs, the General Pavement Studies(GPS) and Specific Pavement Studies (SPS).

The General Pavement Studies (GPS), is a study of the performance of in-service pavement test sections that were in either their original design phase or in their first overlay phase. The pavement types in the GPS experiment that were studied in this research project were: asphalt concrete (AC) on granular base, AC on stabilized base, jointed plain concrete, jointed reinforced concrete, continuously reinforced concrete, AC overlays of AC pavements,

and AC overlays on concrete pavements. Roughness trends over time for each of these pavement types were studied. Subgrade, climatic and pavement material properties that influence the roughness progression on each of these pavement types were identified.

The SPS projects that were analyzed in this project were the SPS-1, SPS-2, SPS-5 and SPS-6 experiments. The SPS projects are located throughout the United States. Each SPS project consists of several test sections, with the number of test sections being different for each SPS project. In the SPS-1 experiment, the structural factors affecting the performance of flexible pavements is studied. New flexible pavements were built for this study. The SPS-2 experiment is a study of structural factors affecting rigid pavement performance. New PCC pavements were built for this study. The SPS-5 experiment studies different treatment factors that can be used to rehabilitate AC pavements. All of these treatment factors involve overlays, with the factors being studied being overlay thickness, millimg, and type of AC used (virgin and recycled). The SPS-6 experiment studies different rehabilitation treatments that can be applied to rigid pavements. The treatments studied in this experiment include repairs to existing PCC, diamond grinding, AC overlays (with and without intensive restoration of existing surface prior to overlay), and crack/break seat with different AC thicknesses. The roughness characteristics of the different test sections in each of these projects were studied. Differences in performance between different rehabilitation strategies that were used for rehabilitation of flexible and rigid pavements were analyzed.

CHAPTER 1

INTRODUCTION AND PROJECT OBJECTIVES

INTRODUCTION

It is believed that the general public perceives a good road as one that provides a smooth ride. Studies at the road test sponsored by the American Association of State Highway Officials showed that the subjective evaluation of the pavement based on mean panel ratings was primarily influenced by roughness (1). Therefore, the development of roughness on pavements is a major issue for highway agencies.

Although pavement smoothness has been recognized as one of the important measures of pavement performance, the contribution of factors such as pavement structure, rehabilitation techniques, climatic conditions, traffic levels, layer materials and properties, and pavement distress to changes in pavement smoothness are not well documented. Without this information, the selection of appropriate pavement design structure, design features, and rehabilitation strategies that will ensure long-term smoothness is a difficult task. The data collected for the Long Term Pavement Performance (LTPP) study provides an opportunity to investigate the effect of these factors on the development of roughness.

PROJECT OBJECTIVES AND SCOPE

The objectives of this research project are to use the Level E data available in the LTPP Information Management System (IMS) to determine the effect of factors such as design and rehabilitation parameters, climatic conditions, traffic levels, material properties, and extent and severity of distress that cause changes in pavement smoothness, and to quantify the contribution of these factors to pavement smoothness. For the purposes of this research, the International Roughness Index (IRI) was used as the measure of pavement smoothness. The IRI is a smoothness index that is widely used in the United States, and can be calculated for any profile that is measured by an inertial profiler (2). The IRI values that are available in the IMS have been computed from profile measurements that have been obtained at test sections. The research was limited to using Level E data that is available in the IMS. The data at Level E have passed a series of quality control checks. The findings of this research will provide guidance for considering long-term smoothness in the design of new and rehabilitated pavements.

The LTPP program consists of two complementary programs, the General Pavement Studies and Specific Pavement Studies. The General Pavement Studies (GPS), is a study of the performance of in-service pavement test sections that were in either their original design phase or in their first overlay phase. Table 1 shows the GPS experiments that were studied in this research project.

Table 1. GPS experiments.

GPS Experiment Number	Description
GPS-1	AC on Granular Base
GPS-2	AC on Stabilized Base
GPS-3	Jointed Plain Concrete
GPS-4	Jointed Reinforced Concrete
GPS-5	Continuously Reinforced Concrete
GPS-6	AC Overlay of AC Pavements
GPS-7	AC Overlay of PCC Pavement

The Specific Pavement Studies (SPS), investigated the effect of specific design features on pavement performance. The SPS experiments that were studied in this research project are shown in table 2.

Table 2. SPS experiments.

SPS Experiment	Description
SPS-1	Strategic Study of Structural Factors for Flexible Pavements
SPS-2	Strategic Study of Structural Factors for Rigid Pavements
SPS-5	Rehabilitation of Asphalt Concrete Pavements
SPS-6	Rehabilitation of Jointed Concrete Pavements

The work performed for the research project was divided into five tasks. The following is a brief description of the work performed for each task.

Task 1: Perform a literature review of LTPP reports that deal with pavement smoothness to obtain information needed to accomplish project objectives. From the data elements available in the LTPP database, identify elements needed to conduct the research and determine the extent of availability of each.

Task 2: Based on the information obtained in Task 1, develop a data analysis plan to address the changes in smoothness encountered at the GPS and SPS experiments that were studied in this research project.

Task 3: Submit for NCHRP review and approval a progress report that documents the research performed under Tasks 1 and 2, and giving details of the data analysis plan.

Task 4: Revise the data analysis plan in accordance with the review comments, and execute the approved data analysis plan.

Task 5: Submit a final report that documents the entire research effort.

ORGANIZATION OF REPORT

Chapter 2 presents the review of literature related to factors affecting pavement smoothness and roughness development in pavements. Chapter 3 presents the data elements that were selected for analysis and data synthesis methods that were used with the data obtained from the IMS. Chapter 4 presents the data analysis methods that were utilized during the study. Chapter 5 describes roughness characteristics of new pavements, and describes the results obtained from the SPS-1 and SPS-2 experiments. Chapter 6 describes roughness characteristics of rehabilitated pavements, and describes results obtained from SPS –5 and SPS-6 experiments.

Chapter 7 presents the results obtained for GPS experiments in the first design phase, which are GPS experiments 1 through 5. Chapter 8 presents the results obtained for GPS experiments 6 and 7, which are overlaid pavements. Chapter 9 presents the conclusions and recommendations for future research.

CHAPTER 2

LITERATURE REVIEW

LONG TERM PAVEMENT PERFORMANCE PROGRAM

The LTPP program is a 20-year study that was started in 1987. The objectives of the LTPP program are to: (1) evaluate existing design methods; (2) develop improved design methods and strategies for the rehabilitation of existing pavements; (3) develop improved design equations for new and reconstructed pavements; (4) determine the effects of loading, environment, material properties and variability, construction quality, and maintenance levels on pavement distress and performance; (5) determine the effects of specific design features on pavement performance; and (6) establish a national long-term pavement database (3). The Strategic Highway Research Program (SHRP) administrated the first five years of the program, and thereafter, the administration of the program was transferred to Federal Highway Administration (FHWA).

The GPS experiments that were analyzed in this study were the GPS experiments 1 through 7. GPS experiments 1 through 5 study the performance of different types of pavements in the first design phase, while experiments 6 and 7 study the performance of AC overlays on AC and PCC pavements, respectively. Table 1 (in Chapter 1) gives the pavement type in each GPS experiment. Each GPS section is 152 m long. The GPS sections generally represent pavements that incorporate materials and structural designs used in standard engineering practice in the United States. The GPS test sections had been in service for some time when they were accepted into the LTPP program. Roughness data collection at these test sections have been performed at regular intervals after the test sections were accepted into the LTPP program. However, the initial IRI of these test sections are not known.

The SPS experiments were designed to study the effect of specific design features on pavement performance. Each SPS experimental test site consists of multiple test sections, each of which is 152 m in length. The SPS experiments that were studied in this research project were experiments 1, 2, 5 and 6. New pavements were constructed for SPS-1 and SPS-2 experiments, and profile data were collected on these pavements after construction. Thereafter, these test sections have been profiled at regular intervals. For these sections, the roughness of the pavement when it was opened to traffic, as well as roughness data collected at approximately annual intervals are available. The SPS-5 and SPS-6 experiments study the effect of different rehabilitation treatments on asphalt concrete and jointed concrete pavements, respectively. For these two experiments, profile data were collected at the test sections prior to and after rehabilitation, and thereafter at approximately annual intervals.

ROUGHNESS STUDIES

Several research projects that used LTPP data to study roughness progression have been performed during the past several years. The first ever comprehensive analysis of roughness progression at LTPP sections was performed by Perera et al. (4). This research project investigated the time-sequence roughness data at GPS test sections to study trends in development of roughness, and developed models to predict roughness. An evaluation of roughness data collected for the SPS-1, -2, -5 and –6 experiments were also performed in this study. Khazanovich et al. (5) used LTPP data to investigate common characteristics of good and poorly performing PCC pavements. They grouped jointed plain concrete (JPC), jointed reinforced concrete (JRC) and continuously reinforced concrete (CRC) pavements into three groups (poor, normal and good) based on time vs IRI relationships, and examined factors contributing to differences in pavement performance. Owusu-Antwi et al. (6) and Titus-Glover et al. (7,8) used LTPP data to analyze the performance of PCC pavements. They determined design features and construction practices that enhance pavement performance, and developed models to predict roughness. Simpson et al. (9) performed a sensitivity analysis of IRI data at the GPS sections. Very few time-sequence IRI values were available when this study was performed. Byrum (10) analyzed profile data collected at GPS-3 and 4 sections and developed a curvature index to quantify slab shape from profile elevation data, and showed that slab curvature was related to PCC pavement performance. An analysis of pavement performance trends for test sections in SPS-5 and SPS-6 projects was performed by Daleiden et al. (11). In this study, a comparison of performance trends of different test sections were made to evaluate the effect of different rehabilitation treatments. The parameters studied in this research were pavement distress (e.g., fatigue cracking, longitudinal cracking, transverse cracking), roughness, rutting, and deflection data.

In the NCHRP project 10-47 (12), variations in roughness statistics due to distress, lateral wander of traveled path, and temperature differential in PCC slabs were studied. This study utilized roughness data collected from test sections that were established on in-service roads specifically for this study, as well as data collected at LTPP sites. Paterson (13) utilized data from a study that was conducted in Brazil to develop models to predict roughness. Von Qunitus et al. (14) used LTPP data to study the relationship between changes in pavement surfaces distress of flexible pavements to incremental changes in IRI.

ROUGHNESS DEVELOPMENT OF AC PAVEMENTS

In investigating roughness characteristics of GPS sections, Perera et al. (4) found a strong relationship between pavement performance and environmental factors. When they performed this study, each GPS sections had been profiled an average of four times. When roughness progression for test sections in each GPS experiment was plotted for each of the four environmental zones (i.e., wet-freeze, wet no-freeze, dry-freeze, and dry no-freeze), there were distinct trends in roughness progression between the regions. The observed roughness development trends in GPS-1 sections seem to indicate that pavement roughness remains relatively constant over the initial life of the pavement, and then after a certain point show a rapid increase. The IRI plots show several sections that were over 15 years old, but had low IRI values. An analysis of these sections indicated they have carried a relatively low cumulative traffic volume when compared to the theoretical cumulative traffic volume the section was capable of carrying. A preliminary analysis of the sections that were showing a high increase in roughness over the monitored period indicated that these sections were close to or have exceeded their design life based on equivalent axle loads.

ROUGHNESS DEVELOPMENT OF PCC PAVEMENTS

A comprehensive analysis of IRI trends of GPS-3, GPS-4 and GPS-5 pavements was performed by Perera et al. (4). At the time this study was performed, these sections had an average of four time-sequence IRI values. This analysis indicated distinct IRI trends for each of these experiments.

Perera et al. (4) found that for JPC pavements (i.e., GPS-3), there were distinct differences in IRI progression between doweled and non-doweled pavements. Generally, the non-doweled pavements showed higher rates of increase in roughness when compared to doweled pavements. For both doweled and non-dowelled pavements, higher IRI values were generally indicated for pavements located in areas that received higher precipitation, had higher freezing indices, and had a higher content of fines in the subgrade. In the non-freeze regions, pavements located in areas that had a higher number of days above 32°C had lower IRI values for both doweled and non-doweled pavements. Pavements that had higher modulus values for PCC had higher IRI values. These observations indicate that mix design factors and the type of aggregate used may influence the performance of the pavements from a roughness point of view.

Khazanovich et al. (5) analyzed roughness trends in JPC (i.e., GPS-3) sections by dividing the sections into three groups based on IRI vs. time performance. The three groups were classified as poor, normal and good. The performance of a pavement section was classified to be good if the IRI satisfied the following condition: $IRI < 0.631 + 0.0631 * Age$, where IRI is in m/km, and age is the pavement age in years. The performance of a pavement section was classified to be poor if the IRI satisfied the following condition: $IRI > 1.263 + 0.0947 * Age$, where IRI is in m/km, and age is the pavement age in years. Pavement sections falling between the good and poor cut-off limits were considered to be performing normally. Of the poor performing sections, approximately 71 percent were located in wet-freeze region, 24 percent in dry-freeze region, and 6 percent in wet no-freeze region. None of the poorly performing sections were located in dry no-freeze regions. Higher IRI values were related to high freeze index values, higher freeze thaw cycles, and higher annual days below 0 °C. They also found that the presence of increased moisture over an extended period of time, characterized by the average number of wet days per year, caused higher roughness. Pavements in warmer climates generally had lower IRI values. They also found a strong relationship between pavement performance and subgrade type. Approximately 67 percent of sections constructed on fine-grained subgrade had a poor IRI performance, while only 33 percent of sections on coarse-grained soils had a poor IRI performance. No trend between traffic and IRI was found. Sections with stabilized bases had lower IRI compared to sections with granular bases. In the poor performance group, 82 percent of the sections had granular bases while 18 percent of the sections had stabilized bases. Sections with asphalt stabilized bases had significantly lower IRI than all other bases. They used linear regression to backcast an estimate of the initial as-constructed roughness and to obtain a rate of increase of roughness. They found that poor performing sections had the highest average rate of increase of roughness, while good performing sections had the lowest rate. They also found that poor performing sections had higher backcasted initial roughness when compared to normal and good sections.

Perera et al. (4) found that for JRCP (i.e., GPS-4) pavements, higher IRI values were associated with higher precipitation, higher moisture content in subgrade, thicker slabs, longer joint spacing, lower water cement ratios, and higher modulus values for PCC. Khazanovich et al. (5) performed an analysis of JRCP sections using an approach similar to that used in the analysis of GPS-3 sections. They determined JRCP constructed on coarse-grained soil performs better than JRCP constructed on fine grained subgrade. All JRCP rated as poor were constructed on fine grained subgrade, while no JRCP rated as poor was constructed on coarse grained soil. They indicated where poor subgrade soil exists, the specification of a thick granular layer will be beneficial. They did not find any specific trends between IRI and traffic, but observed JRCP in good IRI performance category carried much higher ESALs than those in the poor or normal group. Higher IRI values were associated with thicker slabs, which indicated thicker slabs were constructed rougher than thinner slabs. Pavements in areas having a greater annual precipitation or a higher number of wet days had a higher IRI. There were no significant differences in IRI between granular and stabilized bases. They used a linear regression on the time-sequence IRI data to backcast the initial roughness value and obtain a rate of increase of IRI. This analysis indicated that both the initial IRI and rate of increase of IRI over time were greater for the JRCP rated as poor when compared to the normal and good performing category. They found that the mean backcasted initial IRI of JRCP rated as poor was 2.38 m/km, while the sections that were rated as good had a mean backcasted initial IRI of 1.10 m/km. The sections that were rated as poor had an IRI increase per year that was twice as high for JRCP rated as good. They also found that on average, sections with higher k-values had lower IRI values.

Perera et al. (4) analyzed roughness trends of CRCP pavements and observed that CRCP pavements appear to maintain a relatively constant IRI over the monitored period. The IRI behavior pattern was observed to be similar for new as well as old pavements. They report that there were many sections that are over 15 years old, but are still very smooth (IRI < 1.5 m/km). Lower IRI values were associated with higher percentage of longitudinal steel and higher water cement ratios for PCC mix, while higher IRI values were associated with higher values of PCC modulus. In non-freezing areas, higher IRI values were noted for pavements in areas that had higher number of days above 32°C. Khazanovich et al. (5) analyzed roughness trends in CRCP pavements by dividing the LTPP sections into three groups based on time vs IRI performance. The three groups were classified as poor, normal and good. They found higher percentage of steel reinforcement resulted in smoother pavements. They indicate in general, pavements constructed over coarse grained subgrade performed better than those constructed over fine grained subgrade. Of all poorly performing sections, 63 percent were located on fine grained subgrade while 37 percent was located on coarse grained subgrade. They did not find any trends between IRI and traffic, but found that sections that were in the good category had higher traffic volumes.

EFFECT OF SLAB CURVATURE ON ROUGHNESS

Analysis of LTPP data by Byrum (10) as well as data analyzed for the NCHRP Project 10-47 (12) have shown that jointed PCC pavements can take a shape where the slab is curled upwards or downwards. Figure 1 shows an example of a PCC slab that is curled downwards, where the joints are at a lower elevation with respect to the center of the slab. This PCC pavement has a joint spacing of 9 m, which can be seen in this figure. Figure 2 shows an example of a PCC slab that is curled upward with respect to the center of the slab, which has a

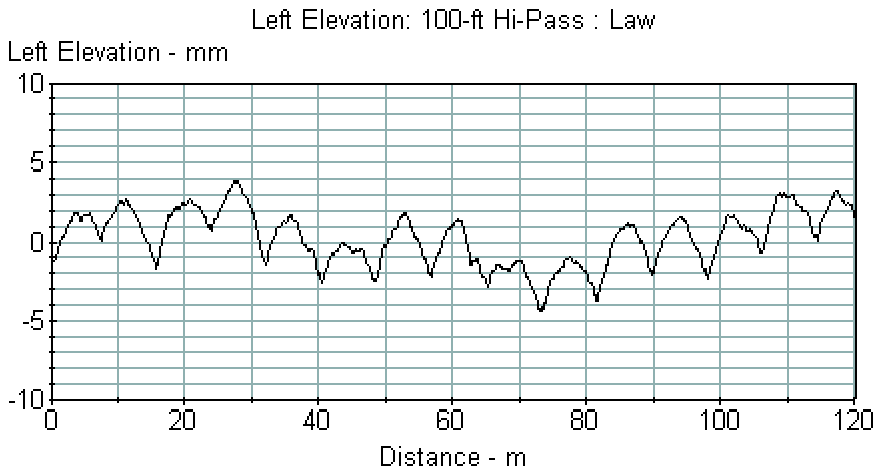


Figure 1. Slab with joints curled down.

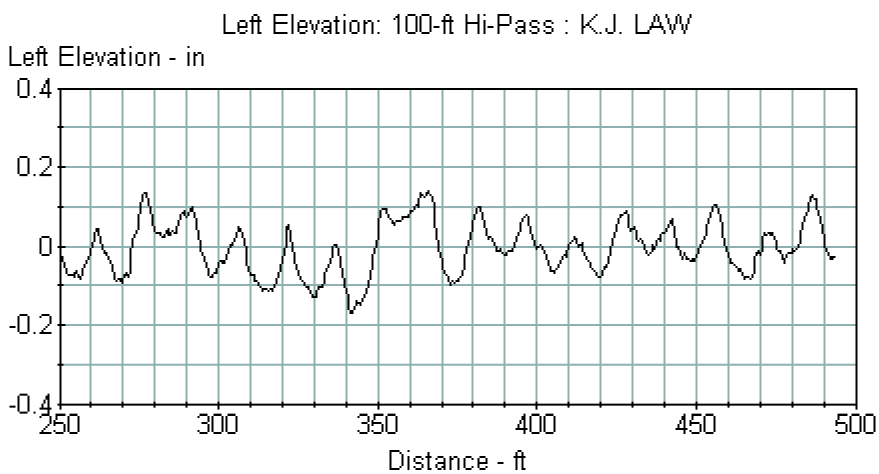
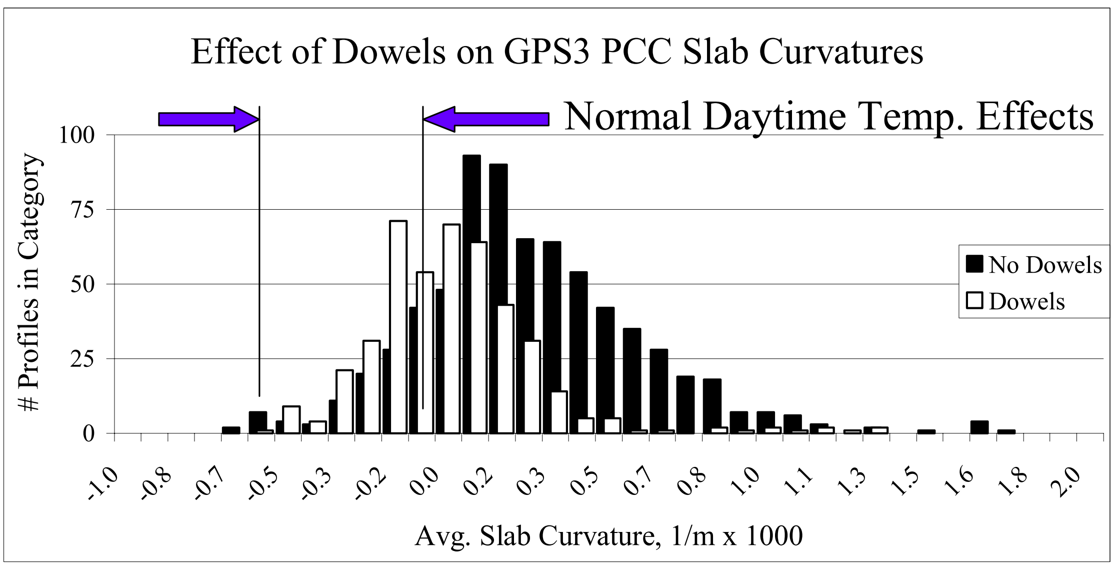


Figure 2. Slab with joints curled up.

joint spacing of 15 feet. These shapes were observed during a period when the temperature differential between the top and bottom of the slab was low, and therefore the shapes were not a result of the temperature gradient. These curvatures are a result of the locked-in curvature in the slab that occurs because of construction conditions or are related to moisture variations in the slab. Pavements that are severely curved downwards as shown in figure 1 can suffer from excessive mid panel deflections and premature cracking that begin at the bottom of the slab. Pavements that are severely curved upward as shown in figure 2 suffer excessive joint deflections, resulting in premature faulting and spalling, and may experience mid-panel transverse cracking that starts from the top of the slab.

Byrum (10) used LTPP profile data to develop a curvature index for JPC slabs. This index presents a measure of the curvature that is present in PCC pavements. A complex interaction of temperature, moisture, and material creep that occurs early in the pavement life can apparently result in the development of large locked-in slab curvature. Byrum showed that the curvature index of PCC slabs was related to performance of PCC pavements. He showed that JPC pavements without dowels have more curvature than PCC pavements without dowels. The distribution of the curvatures that were observed for doweled and non-doweled pavements is shown in figure 3.



No Dowels	LCI	RCI
Minimum	-0.68	-0.68
Maximum	1.62	1.70
Average	0.21	0.22
Std Dev	0.34	0.35
Count	351	351

Dowels	LCI	RCI
Minimum	-0.54	-0.61
Maximum	1.30	1.28
Average	-0.03	-0.02
Std Dev	0.27	0.26
Count	218	218

Figure 3 Average slab curvatures for GPS-3 pavements along the wheel paths (10).

ROUGHNESS CHARACTERISTICS OF OVERLAID PAVEMENTS

Perera et al. (4) investigated roughness characteristics of SPS-5 projects that deal with the performance of selected asphalt concrete rehabilitation treatment factors. The study found that irrespective of the roughness before overlay of a section, the roughness after overlay of the sections for a specific project would fall within a relatively narrow band. They also analyzed IRI data from the GPS-6B and GPS-7B pavements for which IRI before and after the overlay was available. The analysis indicated that a relatively thin overlay could reduce the IRI of a pavement dramatically. For example, a 100 mm thick AC overlay reduced the IRI of a flexible pavement from 3.15 to 0.63 m/km. Similarly, a 84 mm thick AC overlay reduced the IRI of a PCC pavement from 2.68 to 0.87 m/km. Sufficient time-sequence IRI data were not available for the GPS-6B and GPS-7B experiments to see the how the rate of IRI development is affected by the IRI before the overlay.

SPS EXPERIMENTS

An initial evaluation of the SPS-1, SPS-2, SPS-5 and SPS-6 projects has been performed under the FHWA data analysis contract (14, 15, 16, 17). These studies looked at available data for the projects, summarized construction deviations and construction problems in each project, and performed an initial evaluation of the data trends. Daleiden et al. (11) performed an analysis of performance trends at SPS –5, -6 and –7 sites. They presented time-sequence plots for cracking, rutting, deflection and roughness data. However, no clear conclusions on pavement performance could be obtained from this study, as limited monitoring data were available at the time this study was conducted.

TRANSVERSE VARIATIONS, SEASONAL VARIATIONS AND DAILY VARIATIONS OF IRI

Several experiments were conducted using an inertial profiler for NCHRP project 10-47 (12) to investigate the effect of lateral variations of the profiled path on IRI. A shift in the wheel path of 0.3 m typically caused variations of IRI ranging from 5 to 10 percent. In this project, IRI values from LTPP seasonal sites were analyzed to study variations in IRI due to seasonal effects. (12). Also, data from PCC seasonal sites were used to study daily variations in IRI. The report prepared for the project (12) describes the seasonal variations in roughness that was observed at the LTPP seasonal monitoring sites. When daily variations in IRI at the seasonal monitoring sites were analyzed, it was noted for slabs that were curled downwards, the roughness of the pavement increased in the afternoon when compared to the morning. The roughness of slabs that are curled upwards decreased in roughness from morning to afternoon. The magnitude of this change in roughness observed during the day due to temperature effects was generally less than 0.1 m/km for most sections.

MODELS TO PREDICT ROUGHNESS DEVELOPMENT

Perera et al. (4) developed models to predict the development of roughness for GPS experiments 1 through 4 using an optimization technique. These models predict the initial IRI of the pavement with the use of subgrade properties and structural properties of the pavement, and then predict a growth rate as a function of time, traffic, subgrade properties, and pavement structure. Models to predict roughness that were developed using LTPP data for PCC pavements are presented by Titus-Golver (7,8). Paterson (13) used data from Brazil to develop models to predict roughness based on traffic, structural parameters of pavement and distress data. The incremental change in roughness was modeled through three groups of components, dealing with structural, surface distress, and environmental-age-condition factors. Von Quintus et al. (18) studied relationships between changes in pavement surface distress in flexible pavements to incremental changes in IRI using LTPP data. Models to predict IRI based on pavement distress was developed in this study.

CHAPTER 3

SELECTION OF DATA ELEMENTS AND DATA SYNTHESIS

IMS DATABASE

The data collected for the LTPP program are stored in the LTPP Information Management System (IMS) database. This data can be divided into the following categories: inventory, maintenance, climatic, monitoring, traffic, materials testing, and rehabilitation. A brief description of the data elements contained in each category follows.

Inventory Data: Inventory tables contain information related to the location of the section, historical information about the section, and material characteristics of the pavement obtained from State transportation agency archives.

Maintenance Data: Data tables in this category record maintenance activities that have been performed on the test sections.

Climatic Data: The climatic data at the GPS sections are derived from weather data collected by the National Oceanic and Atmospheric Administration and the Canadian Climatic Center (for Canadian test sections). Data collected from five weather stations that are close to each GPS section are used in deriving the climatic data. Weather stations have been installed at SPS sites and site specific weather data are collected.

Monitoring Data: Data tables in this category contain data that are obtained by monitoring activities that are performed at the test sections. These include profile data, deflection data, friction data, surface distress data, and transverse profile data.

Traffic Data: Traffic data tables contain historical traffic estimates provided by State highway agencies as well as monitored traffic data collected by weigh in motion equipment.

Materials Testing: Laboratory test data for pavement and subgrade materials are contained in these tables. Samples were obtained from pavement layers and subgrade at each tests section from just outside the section limits. Extensive laboratory tests were performed on these samples to characterize pavement layer and subgrade properties.

Rehabilitation Data: Major improvements that are made at test sections are documented in the data tables that are in this category.

The data for the test sections are uploaded into the IMS database after undergoing quality control checks. The data that have satisfied the quality control checks are referred to as "Level E" data

IDENTIFICATION OF DATA ELEMENTS

The objectives of this research was to determine the effect of factors such as design and rehabilitation parameters, climatic conditions, traffic levels, material properties, and extent and severity of distress that cause changes in pavement smoothness. The first step in the study was to identify variables that are available in the LTPP database that could be related to the development of roughness.

The information obtained from the literature review was used to identify data elements that have been identified in past research projects as having an effect on roughness development. The data tables in IMS that contain the data elements were then identified. All data tables in the IMS were reviewed, and data elements that could have an impact on roughness development that were not identified previously were selected based on engineering judgment. Table 3 presents the data elements that were identified to be included in the analysis database that was built for this research study.

Table 3. Identified data elements.

Description	Data Element	Applicable Experiment	
		GPS	SPS
Section Data			
LTPP experiment number	EXPERIMENT_SECTION	1-7	1,2,5,6
Construction date	INV_AGE	1-7	1,2,5,6
Drainage type	INV_GENERAL	1-7	5, 6
Drainage location	INV_GENERAL	1-7	5, 6
Roughness			
IRI Value	MON_PROFILE_MASTER	1-7	1,2,5,6
Profile Data	MON_PROFILE_DATA		Selected Sections
Pavement Layer Data			
AC Thickness	TST_L05B	1,2,6,7	1,5,6
PCC thickness	TST_L05B	3,4,5,7	2,6
Base thickness	TST_L05B	1-7	1,2,5,6
Subbase thickness	TST_L05B	1-7	1,2,5,6
Material code for base/subbase	TST_L05B	1-7	1,2,5,6
Base gradation	TST_SS01_UG01_UG02	1-7	1,2,5,6
Base moisture content	TST_SS01_UG01_UG02	1-7	1,2,5,6
Base/subbase liquid limit	TST_UG04_SS03	1-7	1,2,5,6
Base/subbase plastic limit	TST_UG04_SS03	1-7	1,2,5,6
Base/subbase plasticity index	TST_UG04_SS03	1-7	1,2,5,6
Traffic Data			
Historical ESALs	TRF_EST_ANL_TOT_GPS_LN	1-7	1,2,5,6
Monitored ESALs	TRAFFIC_MONITOR_BASIC_INFO	1-7	1,2,5,6
Subgrade Data			
Subgrade plastic limit	TST_UG04_SS03	1-7	1,2,5,6
Subgrade liquid limit	TST_UG04_SS03	1-7	1,2,5,6
Subgrade plasticity index	TST_UG04_SS03	1-7	1,2,5,6
Subgrade gradation properties	TST_SS01_UG01_UG02	1-7	1,2,5,6
Subgrade moisture content	TST_SS01_UG01_UG02	1-7	1,2,5,6
Subgrade material code	TST_L05B	1-7	1,2,5,6
Subgrade resilient modulus	TST_SS07	1-7	1,2,5,6
% Greater than 2mm	TST_SS02_UG03	1-7	1,2,5,6
% Coarse sand	TST_SS02_UG03	1-7	1,2,5,6
% Fine sand	TST_SS02_UG03	1-7	1,2,5,6
% Silt	TST_SS02_UG03	1-7	1,2,5,6
% Clay	TST_SS02_UG03	1-7	1,2,5,6

Table 3. Identified data elements (Continued).

Description	Data Element	Applicable Experiment	
		GPS	SPS
Shoulder Information			
Shoulder surface type	INV_SHOULDER	1-7	5, 6
Shoulder width	INV_SHOULDER	1-7	5, 6
Shoulder paved width	INV_SHOULDER	1-7	5, 6
Shoulder base type	INV_SHOULDER	1-7	5, 6
Shoulder surface thickness	INV_SHOULDER	1-7	5, 6
Shoulder base thickness	INV_SHOULDER	1-7	5, 6
Rehabilitation Data			
Overlay thickness	RHB_IMP, INV_MAJOR_IMP	6,7	5, 6
Overlay date	RHB_IMP	6,7	5, 6
Climatic Data			
Annual precipitation	CLM_VWS_PRECIP_ANNUAL	1-7	1,2,5,6
Intense precipitation days	CLM_VWS_PRECIP_ANNUAL	1-7	1,2,5,6
Number of wet days/yr	CLM_VWS_PRECIP_ANNUAL	1-7	1,2,5,6
Average maximum temperature	CLM_VWS_TEMP_ANNUAL	1-7	1,2,5,6
Average minimum temperature	CLM_VWS_TEMP_ANNUAL	1-7	1,2,5,6
Number of days below 0° C/yr	CLM_VWS_TEMP_ANNUAL	1-7	1,2,5,6
Number of days above 32 °C/yr	CLM_VWS_TEMP_ANNUAL	1-7	1,2,5,6
Number of freeze thaw cycles/yr	CLM_VWS_TEMP_ANNUAL	1-7	1,2,5,6
Annual Freeze index	CLM_VWS_TEMP_ANNUAL	1-7	1,2,5,6
PCC Test Data			
PCC compressive strength	TST_PC01	3, 4, 5, 7	2, 6,
PCC Poisson's ratio	TST_PC04	3, 4, 5, 7	2, 6,
PCC unit weight	TST_PC05	3, 4, 5, 7	2, 6,
PCC elastic modulus	TST_PC04	3, 4, 5, 7	2, 6,
PCC split tensile strength	TST_PC02	3, 4, 5, 7	2, 6,
PCC density	TST_PC05	N/A	2, 6,
PCC percent voids	TST_PC05	N/A	2, 6,
PCC air content (hardened PCC)	TST_PC08	N/A	2
PCC flexural strength	TST_PC09	N/A	2
PCC Steel Information			
Reinforcing type	INV_PCC_STEEL	4,5	6
Transverse bar diameter	INV_PCC_STEEL	4,5	6
Transverse bar spacing	INV_PCC_STEEL	4,5	6
Longitudinal bar diameter	INV_PCC_STEEL	4,5	6
Longitudinal bar spacing	INV_PCC_STEEL	4,5	6
Longitudinal steel percentage	INV_PCC_STEEL	4,5	6

Table 3. Identified data elements (Continued).

Description	Data Element	Applicable Experiment	
		GPS	SPS
PCC Mix Data			
Entrained air	INV_PCC_MIXTURE	3,4,5,7	6
Coarse aggregate in mix - weight	INV_PCC_MIXTURE	3,4,5,7	2, 6
Fine aggregate in mix - weight	INV_PCC_MIXTURE	3,4,5,7	2, 6
Cement in mix - weight	INV_PCC_MIXTURE	3,4,5,7	2, 6
Water in mix - weight	INV_PCC_MIXTURE	3,4,5,7	2, 6
Curing method	INV_PCC_MIXTURE	3,4,5,7	6
Paver type	INV_PCC_MIXTURE	3,4,5,7	6
Cement type	INV_PCC_MIXTURE	3,4,5,7	2, 6
Slump	INV_PCC_MIXTURE	3,4,5,7	6
PCC mix temperature	TST_FRESH_PCC	N/A	2
Slump	TST_FRESH_PCC	N/A	2
Air content	TST_FRESH_PCC	N/A	2
PCC Joint Information			
Joint spacing	INV_PCC_JOINT	3, 4	6, 7
Load transfer	INV_PCC_JOINT	3	6, 7
Joint skewness	INV_PCC_JOINT	3	6, 7
Dowel diameter	INV_PCC_JOINT	3, 4	6, 7
Dowel coating	INV_PCC_JOINT	3, 4	6, 7
AC Properties			
Extraction results	TST_AC_04	1,2,6,7	1,5,6,8
Resilient modulus	TST_AC07	1,2,6,7	1,5,6,8
Bulk specific gravity	TST_AC02	1,2,6,7	1,5,6,8
Maximum specific gravity	TST_AC03	1,2,6,7	1,5,6,8
Distress Data			
AC manual distress data	MON_DIS_AC_REV	1,2,6	1,5,8
AC automated distress data	MON_DIS_PADIAS_AC	1,2,6	1,5,8
Jointed PCC manual distress data	MON_DIS_JPCC_REV	3,4	2
Jointed PCC automated distress	MON_DIS_PADIAS_JPCC	3,4	2
CRCP manual distress	MON_DIS_CRCP_REV	5	N/A
CRCP automated distress	MON_DIS_PADIAS_CRCP	5	N/A
Faulting data	MON_DIS_JPCC_FAULT_SECT	3,4	2
Rutting data	MON_RUT_DEPTH_POINT	1,2,6,7	1,5,6

Note: N/A – Not Available

BUILDING THE ANALYSIS DATABASE

The analysis database used for this project was built using Access 97. The data tables containing the data elements that were identified were obtained from the IMS. The data obtained corresponded to IMS Release 10.2. Computer programs were written using visual basic programming language to extract the identified data elements from IMS data tables, and to build the analysis database. Several values were available for a test section for some data elements. For example, for a specific section, thickness values were available from several coring locations. A similar situation existed for materials test data, where results were available for multiple samples obtained at different locations for a test section. For cases where multiple data values were available for a data element, the values were averaged to obtain a unique value for the section.

For this research project, the mean IRI of the test section, which is the average IRI of the left and right wheel paths, was used to characterize the roughness. For a specific test date at a test section, the LTPP database generally has five IRI values that have been obtained from five profile runs. The mean IRI values of these multiple profile runs were averaged to obtain the roughness for that specific test date.

TRAFFIC DATA

The IMS database has historical as well as monitored traffic data. Monitored traffic data is available only after the section was accepted into the LTPP program. Weigh-in-Motion (WIM) scales and Automatic Vehicle Classification systems have been installed at some LTPP test sections. At some test sections, portable equipment is used to collect WIM and classification data. As both historical and monitored traffic data are available at a site, a procedure was developed to obtain the cumulative traffic that has been applied at the site. This procedure consisted of developing a traffic growth curve for the site using both the historical and monitored traffic data. A best fit exponential curve of the following form was fitted to the time sequence historical and monitored traffic data at each site.

$$KESAL(t) = (KESAL_0)\ e^{rt}$$

where,

$KESAL(t)$	=	Kilo Equivalent Axle Loads (KESAL) at time t years
$KESAL_0$	=	Kilo Equivalent Single Axles (KESAL) per year at traffic open date
r	=	Average annual traffic growth rate, percent
t	=	Time, years

In some cases there was good agreement in the trend between the historical and monitored traffic data, while in other cases there were significant differences between the two trends. Figure 4 shows two cases, where for one case the trend between the historical and monitored traffic is poor, while for the other case the two trends show good agreement. Figure 4 shows the individual trends for the historical and monitored data as well as the combined trend for the data that was obtained by fitting an exponential curve to the data.

Once the initial KESAL and the traffic growth rate for a site is known, the cumulative traffic at any point in time can be determined. It should be noted that acceptable curve fits could not be made at several sites because of large variability in the traffic data between the years. For such sites, an appropriate growth curve was assigned based on the traffic data trend at that site.

DATA AVAILABILITY

Backcalculated moduli are not currently available in the IMS. However, a data analysis contractor had performed backcalculation using the deflection data collected at GPS sections. This data had not passed Level E quality checks. This data was obtained through the NCHRP. A review of the data indicated backcalculated moduli were not available for a significant number of sections, and also there appeared to be many outliers in the data. Because of these limitations, backcalculated moduli were not used in the analysis. Resilient moduli values for base and subgrade were also not available for most GPS sections, and therefore were not used in the analysis. Table 4 presents the availability of some key data elements for GPS sections.

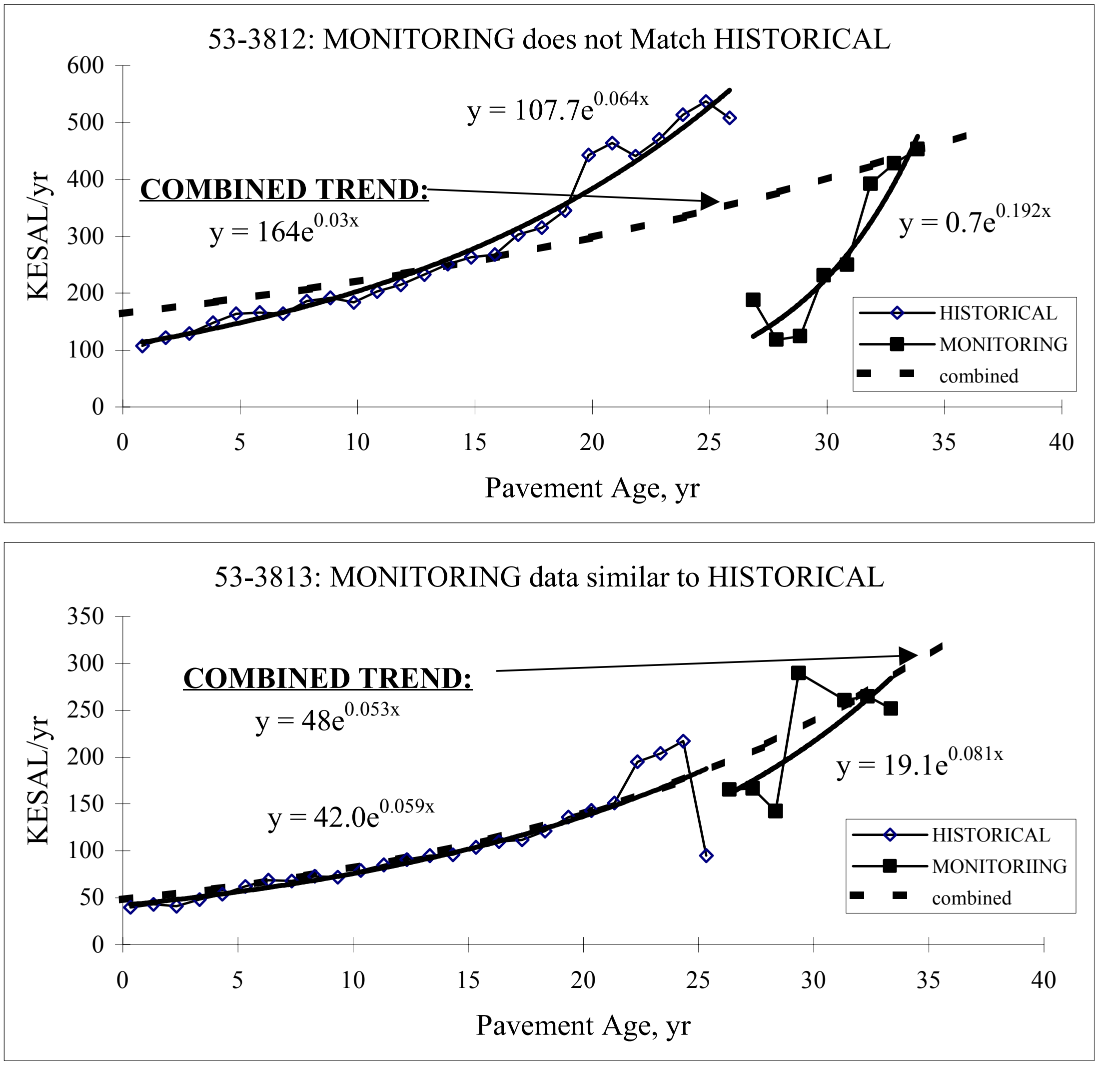


Figure 4. Comparison of historical, monitoring and combined traffic trends.

Generally, sufficient data were available for GPS sections to investigate the effect of design, climatic and materials test parameters on roughness development. Materials test data as well as traffic data were not available for most SPS sections. Therefore, the effect of materials properties and traffic effects on roughness development could not be studied for SPS sections.

Table 4. Data availability at GPS sections

Parameter	Percentage Sections Where Data is Available									
	GPS Experiment									
	GPS 1	GPS 2	GPS 3	GPS 4	GPS 5	GPS 6A	GPS 6B	GPS 7A	GPS 7B	GPS 9
Traffic	98	98	100	100	100	98	100	100	100	88
Construction Date	100	100	100	100	100	100	100	100	100	100
Climatic Data	99	99	100	100	100	100	100	100	100	100
Drainage Information	100	99	100	100	100	100	100	100	100	100
Shoulder Information	100	99	100	100	100	100	100	100	100	96
Surface Thickness	91	98	98	100	99	98	100	100	100	69
Overlay Thickness						75	80	94	81	69
Base Thickness	83	97	92	89	98	74	95	79	90	54
Base Gradation	83		79	94	100	100	98	100	88	100
Base Moisture Content	80		79	91	93	97	98	100	82	100
Base- Plasticity Index	81		81	97	100	100	98	100	88	100
Subgrade Gradation	92	95	91	97	98	100	96	97	100	92
Subgrade Plasticity Index	93	95	91	98	98	100	96	97	100	92
Subgrade Moisture Content	90	95	89	97	98	100	96	97	100	92
Subgrade Hydrometer Analysis	86	90	86	95	93	98	93	94	100	88
Extracted AC Content	89	96				95	97	35	20	
AC - Max. Specific Gravity	89	96				95	97	35	20	
AC - Bulk Specific Gravity	90	96				95	97	35	20	
PCC - Tensile Strength			92	86	95			91	94	88
PCC - Elastic Modulus			92	92	99			100	97	96
PCC - Compressive Strength			85	92	96			97	90	96
PCC - Unit Weight			92	92	99			100	97	96
PCC - Poissons Ratio			91	92	99			100	97	96
PCC - Air Content			84	66	87			44	65	77
PCC - Mix Design			89	80	89			56	65	73
PCC - Curing Method			84	85	90			44	77	69
PCC - Slump			73	51	77			26	42	65
PCC - Steel				58	98			32	45	42
PCC - Joint Spacing			100	100				82	61	85
PCC - Load Transfer			97	97				65	58	73

This page left intentionally blank.

CHAPTER 4

DATA ANALYSIS METHODS

RELATIONSHIP BETWEEN IRI AND DATA ELEMENTS

Once the databases containing the data elements selected for analysis were assembled, statistical procedures such as univariate analysis and bivarate analysis were performed on the data. The univariate analysis consisted of an investigation of the distribution of each data element. The analysis was carried out separately for each GPS and SPS experiment. Data distributions were analyzed using histograms and box-plots. A box-plot is a simple graphical representation showing the center and the spread of the distribution, along with outliers. Figure 5 presents an example of a box plot that shows the distribution of the first IRI value that was obtained at SPS-2 sections that have a lean concrete base. The horizontal line at the interior of the box is located at the median of the data. The height of the box is equal to the interquartile distance (IQD), which is the difference between the third quartile of the data and the first quartile. The whiskers (the lines extending from the top and the bottom of the box) extend to the extreme values of the data or a distance of 1.5 X IQD from the center, whichever is less. For data having a normal distribution, approximately 99.3% of the data falls inside the whiskers. Data points that fall outside the whiskers may be outliers, and are indicated by horizontal lines. For the box-plot shown in figure 5, the median IRI value is 1.34 m/km. The first and the third quartile values that are indicated by the lower and the upper limits of the box are 1.19 m/km and 1.50 m/km, respectively. The horizontal lines above the top whisker show the outliers in the data set.

The bivariate analysis consisted of computing the Pearson's correlation coefficient between the data elements selected for analysis and the median IRI value over the monitored period at a section, and/or the change in IRI value over the monitored period at a section. This analysis was carried out for GPS sections as these sections had sufficient data to carry out this analysis. The correlation coefficient has a value between 1 and -1, with values approaching 1 indicating a strong positive correlation, values near zero indicating no correlation, and values approaching -1 indicating an inverse relationship between parameters. As it is possible to have a strong correlation

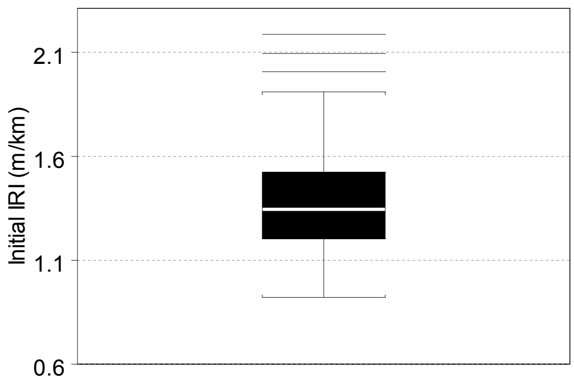


Figure 5. Example of a box-plot.

between two variables simply because of biases in the data distribution and influential observations, scatter plots were used to obtain an insight into the relationship between the two parameters to identify the true trends in the data. Two way scatter plots between each data element and median IRI of the section over the monitored period, and the rate of change of IRI over the monitored period were examined to identify data trends. The scatter plot between the data element and the median IRI provides an insight into the relationship between the data element and the level of IRI, while the scatter plot between the rate of change of IRI and the data elements provides an insight into the effect of the data element on the change in roughness. Scatter plots between the different data elements were also examined to investigate correlations between data elements. This study provided information on which data elements to use in model building, as using data elements that are correlated with each other in model building will not yield accurate models. For each GPS experiment, scatter plots were used initially to examine trends for the whole data set. Thereafter, for each GPS experiment, data trends were examined separately for the different environmental zones. The data elements that were used in the analysis of the GPS sections are shown in table 5.

Four environmental zones were considered in this analysis, and they correspond to the four environmental zones that are used in the LTPP program, which are wet-freeze, wet no-freeze, dry-freeze and dry no-freeze. The boundary between wet and dry regions was taken as 508 mm of annual precipitation, and the boundary between the freezing and non-freezing zones

Table 5. Data elements analyzed.

Parameter	Applicable GPS Experiment
Pavement Age	1-7
Surface Thickness	1-7
Base Thickness	1-7
Total Pavement Thickness	1-7
Structural Number	1,2,6
Overburden Pressure	1-7
AC Bulk Specific Gravity	1,2,6
AC Air Voids	1,2,6
Asphalt Content	1,2,6
Annual Precipitation	1-7
Intense Precipitation Days per year	1-7
Annual Wet Days	1-7
Mean Temperature	1-7
Days with Temperature > 32 °C, per year	1-7
Days with Temperature < 0 °C, per year	1-7
Annual Freezing Index	1-7
Freeze Thaw Cycles per Year	1-7
Plasticity Index Subgrade	1-7
Plastic Limit Subgrade	1-7
Moisture Content Subgrade	1-7
Silt Content in Subgrade	1-7
Clay Content in Subgrade	1-7
Percent Passing No. 200 Sieve, Subgrade	1-7
Moisture Content Base	1-7
Percent Passing No. 200 Sieve, Base	1-7
Joint Spacing PCC	3,4
PCC Elastic Modulus	3,4,5
PCC Compressive Strength	3,4,5
PCC Tensile Strength	3,4,5
PCC Poisson's Ratio	3,4,5
PCC Unit Weight	3,4,5
PCC - Coarse Aggregate, Weight	3,4,5
PCC – Fine Aggregate, Weight	3,4,5
PCC - Cement, Weight	3,4,5
PCC – Water Cement Ratio	3,4,5
PCC – Air	3,4,5
PCC – Slump	3,4,5
Traffic	1-7

was taken as an annual freezing index of 89 °C days. Figure 6 shows the general distribution of the four environmental zones in the United States.

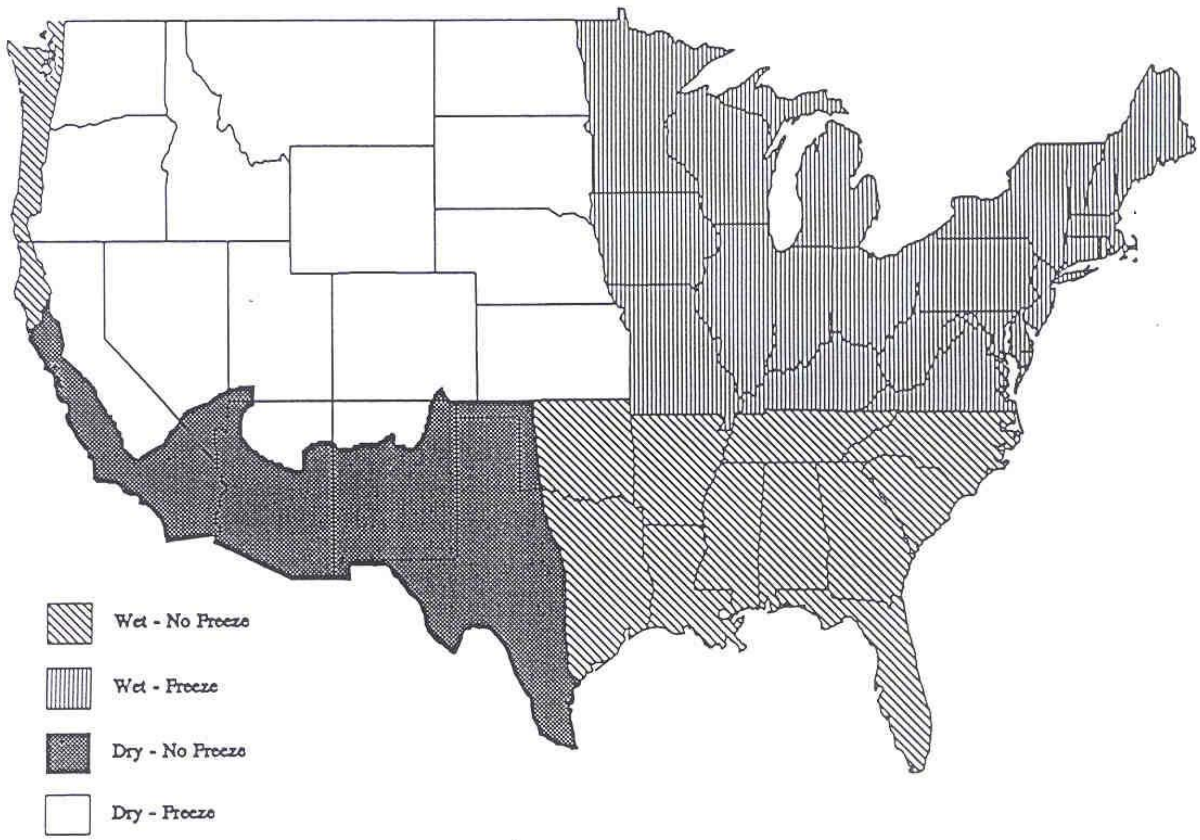


Figure 6. Environmental zones.

MODELING APPROACH

One popular approach that might be considered for modeling roughness development is regression analysis, where each roughness observation is considered to be an independent observation. The IRI or some function of IRI could be taken as the response and the time of measurement and other variables could be used as predictors. However, this approach would not be appropriate for this time-sequence IRI data for two main reasons. First, one of the fundamental assumptions of regression analysis is that the observations are independent. This assumption is clearly violated in this data since observations are grouped by pavement section. The time-sequence IRI values that are obtained for a specific pavement section would be dependent on the past roughness at the section. If this dependence is ignored and regression analysis is used, the typical drawback is that the significance of predictors is overstated. The other main drawback to regression analysis is that it fails to take advantage of the information provided by following a given section over time.

Consider the relationship between IRI and pavement age that is shown in figure 7. If each observation was considered to be an independent variable, the data points will be treated as shown in figure 7(a). Longitudinal data analysis methods take into account the time sequence nature of the IRI values at a section to predict future IRI. Figure 7(b) illustrates the approach that is used in longitudinal data analysis, where the time sequence aspect of IRI values is considered in the analysis. The results that are obtained by such an analysis will be different from an analysis that considers each point as an independent observations as shown in figure 7(a). Longitudinal data analysis is performed by using a mixed effects model analysis.

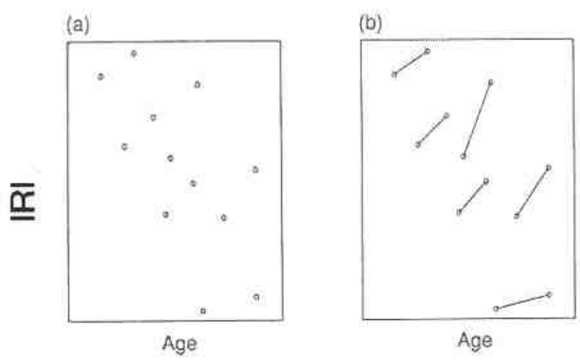


Figure 7. Modeling of time-sequence data.

Mixed effects models are commonly used in the biomedical and social sciences where one might be interested in the progress of a patient or subject over time as a function of treatment and/or environmental factors. This modeling approach is well established and has been found effective in these applications. The data under consideration here are similar to those found in epidemiological applications where subjects are measured at varying time points, where there are a large number of potential predictors and a complex pattern of missing observations. A mixed effects model analysis is similar to a regression analysis in that many of the steps are familiar. Diagnostic plots need to be checked, variables selected and possibly transformed. However, one well-known feature of a regression model that is not found in a mixed model is the R^2. In a regression model, there is only one kind of random variation - the R^2 tells us the relative size of this residual random variation compared to the variation explained by the model. In a mixed model, there are multiple sources of residual variation. At a minimum, there will be the variation in individual IRI measurements and the variation in whole pavement sections. There is no simple equivalent to R^2. Nevertheless, the random effect standard errors do tell how well the model can predict future observations. These models may indicate that a great deal of variation is not explained by the available predictors. This does not mean that the models do not fit well. On the contrary, the mixed effects model does a good job of describing the inherent variation in the data.

The general form of the model that is used in a mixed model analysis is:

$$y_i(t_{ij}) = \beta_0 + \beta_t t_{ij} + \sum_{k=1}^{p} \beta_k \chi_{ik} + \gamma_i + \varepsilon_{ij}$$

$y_i(tij)$	=	Response of pavement section i at time t_{ij}
β_t	=	Coefficient that controls the growth of roughness over time
$\sum_{k=1}^{p} \beta_k \chi_{ik}$	=	The term which indicates how the predictors affect roughness. There are p predictors where x_{ik} denotes the value of predictor k for section i, while coefficients $\beta_1, \dots \beta_p$ controls the size of these effects
γ_i	=	Random effects term drawn from a normal distribution with mean zero and variance to be estimated. This term represents variations among pavement sections not explained by predictors
ε_{ij}	=	Error term. In the simplest form of the model, these errors are independent and normally distributed with a variance to be estimated.

The statistical software package S-Plus (19) contains a procedure for performing mixed model analysis. The mixed model analysis was carried out to build models to predict development of roughness for GPS sections. Selection of data elements to use as predictor variables were made based on the observations from the two-way scatter plots. Models to predict development of roughness for SPS projects could not be carried out as sufficient traffic and materials test data were not available for these projects. However, the mixed model method was used in the analysis of SPS projects to test the significance of available data elements to development of roughness.

This page left intentionally blank.

CHAPTER 5

ROUGHNESS CHARACTERISTICS OF SPS-1 AND SPS-2 PROJECTS

SPS-1 EXPERIMENT: STRATEGIC STUDY OF STRUCTURAL FACTORS FOR FLEXIBLE PAVEMENTS

Introduction

The SPS-1 experiment was developed to investigate the effect of selected structural factors on the long-term performance of flexible pavements that were constructed on different subgrade types and in different environmental regions. New pavements were constructed for the SPS-1 experiment. In the SPS-1 experiment, twelve test sections were constructed at a project location. The twelve test sections in a project were either section numbers 1 through 12, or section numbers 13 through 24. The pavement structure of the test sections in the SPS-1 experiment is shown in table 6. The subgrade types considered in this experiment are classified as fine grained and coarse grained, and the environmental regions considered are the four LTPP environmental regions: wet freeze, wet-no freeze, dry freeze and dry-no freeze. The structural factors considered in this experiment are asphalt thickness, base type, base thickness and drainability (presence or lack of it as provided by an open graded permeable asphalt treated layer and edge drains). Five different base types are used in this experiment: dense graded aggregate base (DGAB), asphalt treated base (ATB), asphalt treated base (ATB) over dense graded aggregate base (DGAB), permeable asphalt treated base (PATB) over dense graded aggregate base (DGAB), and asphalt treated base (ATB) over permeable asphalt treated base (PATB). The test sections are profiled immediately after construction, and thereafter at approximately annual intervals.

Analyzed Projects

A review of the LTPP database indicated profile data were available for sixteen SPS-1 projects. Table 7 presents the SPS-1 projects for which IRI data were available. Table 7 also

Table 6. Structural properties of SPS-1 sections.

Test Section Number	AC Thickness (mm)	Layer 2		Layer 3	
		Material	Thickness (mm)	Material	Thickness (mm)
1	175	DGAB	200	-	-
2	100	DGAB	300	-	-
3	100	ATB	200	-	-
4	175	ATB	300	-	-
5	100	ATB	100	DGAB	100
6	175	ATB	200	DGAB	100
7	100	PATB	100	DGAB	100
8	175	PATB	100	DGAB	200
9	175	PATB	100	DGAB	300
10	175	ATB	100	PATB	100
11	100	ATB	200	PATB	100
12	100	ATB	300	PATB	100
13	100	DGAB	200	-	-
14	175	DGAB	300	-	-
15	175	ATB	200	-	-
16	100	ATB	300	-	-
17	175	ATB	100	DGAB	100
18	100	ATB	200	DGAB	100
19	175	PATB	100	DGAB	100
20	100	PATB	100	DGAB	200
21	100	PATB	100	DGAB	300
22	100	ATB	100	PATB	100
23	175	ATB	200	PATB	100
24	175	ATB	300	PATB	100

Note: DGAB - Dense Graded Aggregate Base, ATB - Asphalt Treated Base, PATB – Permeable Asphalt Treated Base

presents the following information for each project: test section numbers in project, climatic zone, subgrade type, construction date, last profile date in the database, age of project at first profile date, age of project at last available profile date, time difference between first and last profile dates, the number of times the project has been profiled, and the estimated annual ESALs at the site.

Ten projects have been profiled within one year of construction, four projects between one and two years after construction, and one project (in Florida) two years or after construction, and one project (in Alabama) three years after construction. Four SPS-1 projects have been profiled only once. The others were profiled two to seven times.

Table 7. SPS-1 projects.

Location	State Code	Section Numbers in Project	Climatic Zone (Note 1)	Subgrade Type	Construction Date	Last Available Profile Date	Age of Project at First Profile Date (Years)	Age of Project at Last Profile Date (Years)	Time Difference Between First And Last Profile Dates (Years)	Number of Times Profiled	Estimated Traffic (KESAL/Yr)
Alabama	AL	1-12	WNF	Fine	1/1/93	1/27/98	3.0	5.1	2.1	3	237
Arizona	AZ	13-24	DNF	Coarse	8/1/93	12/4/98	0.5	5.3	4.8	5	223
Arkansas	AR	13-24	WNF	Coarse	9/1/94	7/1/97	0.8	2.8	2.0	2	210
Delaware	DE	1-12	WF	Coarse	5/1/96	11/5/98	0.6	2.5	1.9	7	203
Florida	FL	1-12	WNF	Coarse	2/1/95	1/27/97	2.0	2.0	0.0	1	1463
Iowa	IA	1-12	WF	Fine	11/19/92	7/19/99	0.9	6.7	5.8	6	N/A
Kansas	KS	1-12	DF	Fine	11/1/93	3/21/99	0.5	5.4	4.9	5	130
Louisiana	LA	13-24	WNF	Fine	7/1/97	11/17/97	0.4	0.4	0.0	1	524
Michigan	MI	13-24	WF	Fine	11/1/95	6/25/97	1.2	1.6	0.4	3	72
Nebraska	NE	13-24	DF	Fine	8/1/95	5/16/98	0.3	2.8	2.5	4	145
Nevada	NV	1-12	DF	Coarse	9/1/95	8/28/98	1.6	3.0	1.4	3	799
New Mexico	NM	1-12	DNF	Fine	11/1/95	3/11/97	1.4	1.4	0.0	1	393
Ohio	OH	1-12	WF	Fine	1/1/95	11/12/98	1.6	3.9	2.3	4	N/A
Oklahoma	OK	13-24	WNF	Fine	6/1/97	11/19/97	0.5	0.5	0.0	1	393
Texas	TX	13-24	WNF	Fine	4/1/97	4/2/98	0.4	1.0	0.6	2	N/A
Virginia	VA	13-24	WF	Fine	11/28/95	10/29/98	0.4	2.9	2.5	7	N/A

Note 1: DF - Dry Freeze, DNF - Dry No-Freeze, WF - Wet Freeze, WNF - Wet No-Freeze

Analysis of Early Age IRI

An analysis was performed to study the early-age IRI characteristics of SPS-1 projects. The initial IRI values for the SPS-1 projects were obtained at varying times after construction. Therefore, the initial IRI may not necessarily correspond to the IRI that is obtained immediately after construction. Therefore, the term early-age IRI is used in this analysis to differentiate from the initial IRI of the pavement, which is the IRI immediately after construction. All SPS-1 projects on which the IRI was obtained less than two years after construction were used in this analysis. This excluded two projects, Florida and Alabama from the analysis.

Figure 8 shows the average early-age IRI of the test sections within each project, differentiated according to the asphalt concrete thickness. The value shown for a project in figure 8 is the average IRI of six test sections that have an AC thickness of 100 mm or 175 mm. The average IRI values for the 100 mm AC and 175 mm AC sections were close to each other for each project, but for most projects the average IRI of 175 mm thick AC sections was lower than the average IRI of 100 mm sections. The maximum difference between the two thicknesses occurred for the project in Ohio, where the average IRI for the 175 mm thick AC was 0.2 m/km lower than the average IRI for the 100 mm thick AC. The projects in Nebraska and Ohio had the highest early-age IRI values, while the projects in Louisiana, Nevada and New Mexico had the lowest early-age IRI values. The standard deviation of early-age IRI for the SPS-1 projects is shown in figure 9. The project in Ohio had the highest standard deviation in IRI between the test sections.

The frequency distribution of early-age IRI values of the test sections in the SPS-1 projects separated according to the two AC thicknesses is shown in figure 10, while the cumulative frequency distribution is shown in figure 11. The cumulative frequency distribution curve shows that 175 mm AC surfaces have lower IRI values than 100 mm AC surfaces. The curve shows that an IRI value of less than 0.8 m/km was achieved on 40 percent of sections that received a 100 mm AC surface and 55 percent of the sections that received a 175 mm AC surface. An IRI value of less than 1.0 m/km was obtained by 75 percent of the sections that received a 100 mm AC surface and 85 percent of the sections that received a 175 mm AC surface

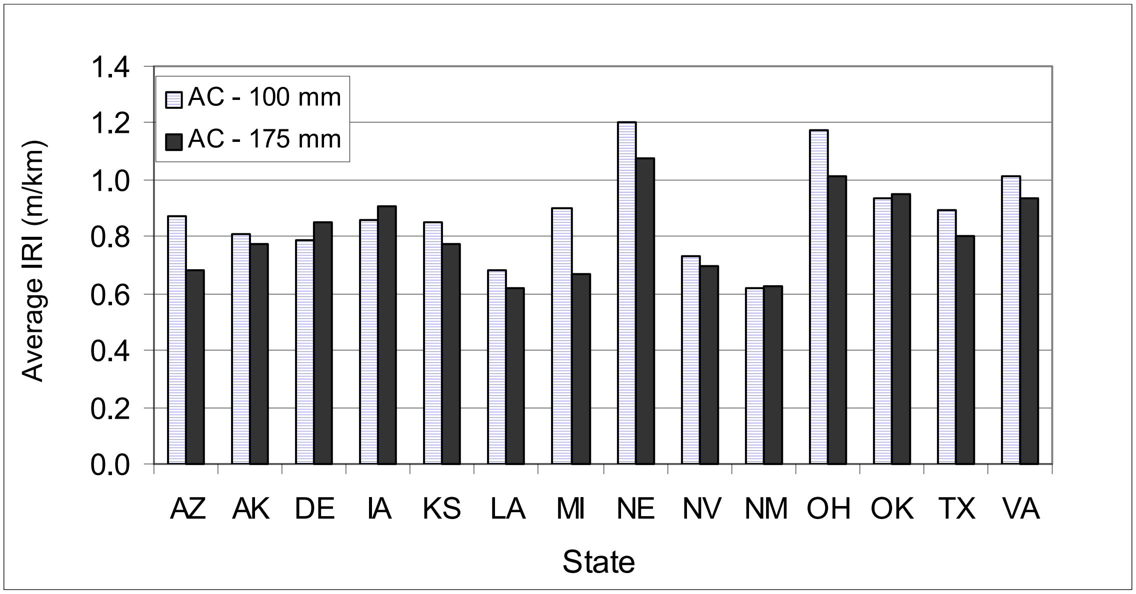


Figure 8. Average early-age IRI of SPS-1 projects.

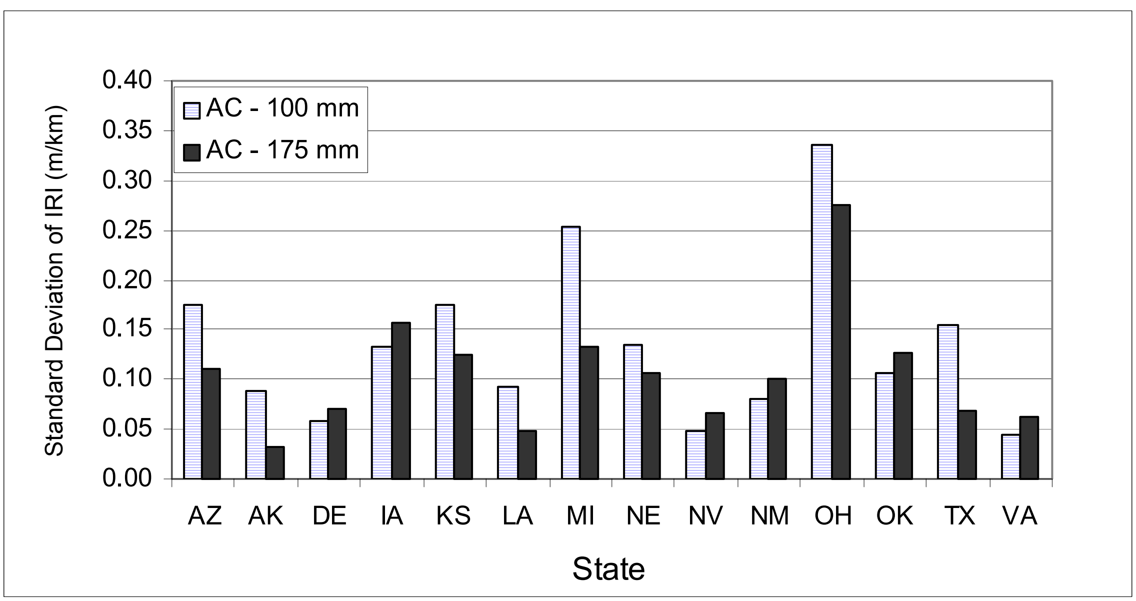


Figure 9. Standard deviation of early-age IRI for SPS-1 projects.

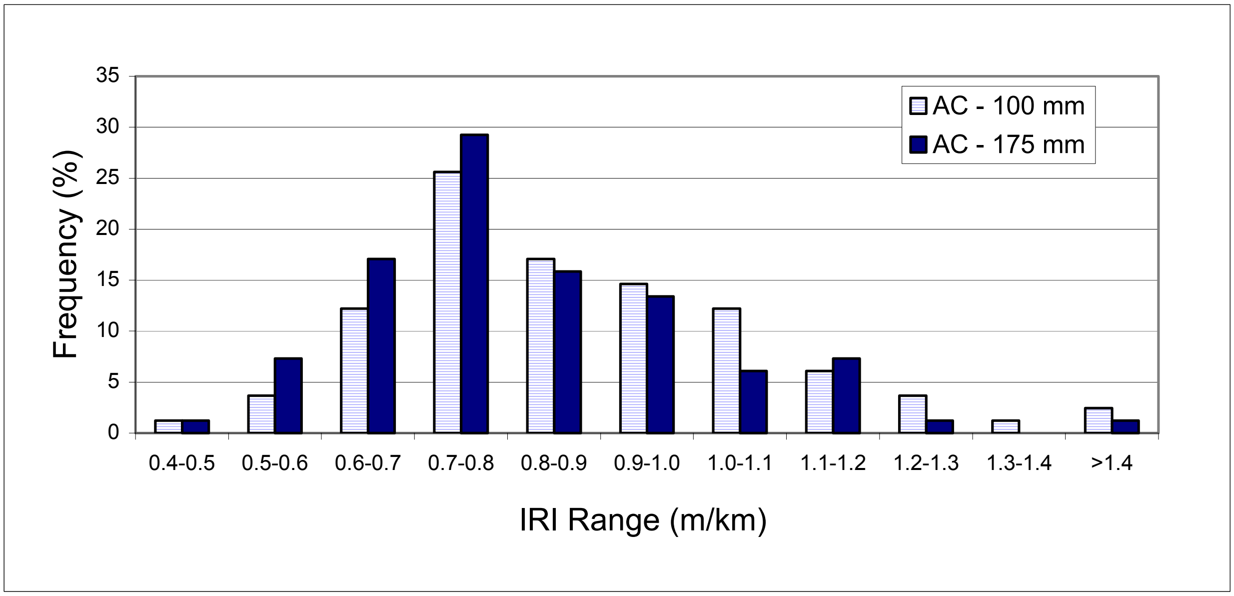


Figure 10. Frequency distribution of early-age IRI.

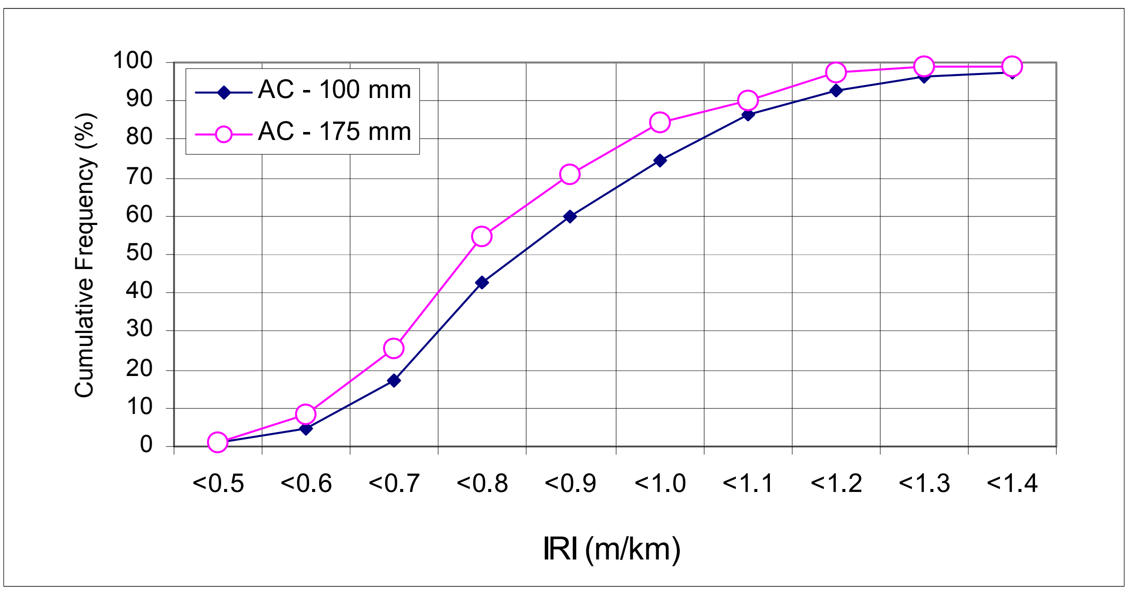


Figure 11. Cumulative frequency distribution of early-age IRI.

The average and the standard deviation of early-age IRI values that were obtained for the 100 mm and 175 mm AC surfaces are shown in table 8. A two-way ANOVA on early-age IRI that was conducted by treating AC Thickness (two levels) and three base types (three levels – DGAB, ATB, PATB) indicated that AC thickness was significant (p-value 0.03).

Table 8. Average and standard deviation of early-age IRI: SPS-1.

AC Thickness (mm)	IRI (m/km)	
	Average	Std. Dev
100	0.88	0.21
175	0.82	0.18

An analysis was performed to study if there was an influence of base type on which the AC surface is placed on the early-age IRI. The AC surfaces in SPS-1 projects were placed on three different base types: dense graded aggregate base (DGAB), asphalt treated base (ATB) and permeable asphalt treated base (PATB). For each project, the AC surface was placed on DGAB for two sections, on ATB for seven sections, and on PATB for three sections. Figure 12 shows a box-plot of the range of early-age IRI values that were obtained for each base type.

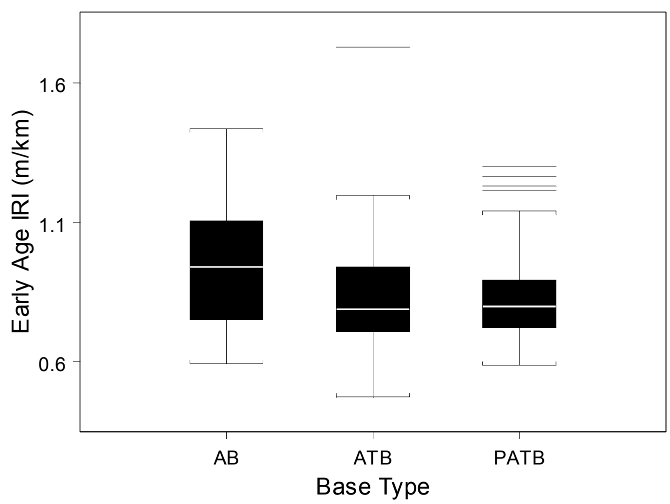


Figure 12. Distribution of early-age IRI for each base type.

The average early-age IRI values for the three base types are: DGAB – 0.94 m/km, ATB – 0.82 m/km, and PATB – 0.84 m/km. A two-way ANOVA on early-age IRI was conducted by treating AC Thickness (two levels) and base types (three levels – DGAB, ATB, PATB). The analysis indicated that base type was significant (p-value 0.02). A multiple comparison of the IRI values indicated that the IRI values obtained on DGAB and ATB were significantly different.

The analysis indicated there were no differences between IRI values obtained on AC surfaces placed on PATB when compared to ATB or DGAB. The box-plot of IRI values showed that there were several PATB sections that had high IRI values, but these are indicated to be outliers when compared to the other PATB sections.

Changes in IRI for SPS-1 Projects

Plots showing the change in IRI over time at individual SPS-1 projects are included in Appendix A. Table 9 presents the changes in IRI for the test sections in each SPS-1 project. The change in IRI was computed by subtracting the IRI at the first profile date from the IRI at the last available profile date. Table 9 also shows the age of project at last profile date, age of the project when it was first profiled, and the time difference corresponding to the change in IRI. In table 9, cases that show an IRI increase of 0.1 m/km or greater are shown in bold. Table 10 presents the percent change in IRI at the test sections, with respect to the IRI at the first profile date. Cases where the percent increase is greater or equal to 10 percent are shown in bold. The average IRI at the first profile date for all SPS-1 test sections was 0.85 m/km. Ten percent of this value is 0.09 m/km, which is a small increase in IRI if the magnitude of the change is considered. A study of transverse profile variations at a new AC pavement by Karamihas et al. (12) indicated IRI variations of 5 percent could occur due to lateral wander of 0.3 m from the wheel path. Therefore, for sections that show a percent change of less than 5 percent, the change in roughness may have been caused by variations in the profiled path.

All sections in the Delaware SPS-1 show a negative percent change in IRI, except for one section that shows no change in IRI. This project has been profiled seven times since construction, with the first profile date being 12/5/96. The IRI of all sections for the first profile date is higher than the subsequent six IRI values that were obtained. This indicates that either there was an error in the profiling equipment at the first profile date or the higher IRI was caused by variations in the pavement profile due to environmental causes (e.g., shrink or swell of subgrade). The difference in IRI between the last profile date and second profile date indicated the change in IRI for the test sections to be between –0.03 and 0.02 m/km, except for one section

Table 9. Changes in IRI for SPS-1 sections.

State	Age of Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Change in IRI (IRI at Last Profile Date - IRI at First Profile Date) (m/km) Section Number											
				1	2	3	4	5	6	7	8	9	10	11	12
Alabama	3.0	5.1	2.1	0.02	0.08	0.01	0.05	0.02	0.07	0.01	-0.01	-0.05	0.07	0.01	0.05
Delaware	0.6	2.5	1.9	-0.10	0.00	-0.09	-0.13	-0.10	-0.08	-0.12	-0.11	-0.13	-0.15	-0.13	-0.15
Iowa	0.9	6.7	5.8	**0.46**	**1.27**	**0.37**	**0.30**	**0.34**	**0.30**	**1.13**	**0.71**	0.07	**0.54**	**0.34**	**0.31**
Kansas	0.5	5.4	4.9	**0.41**	**0.41**	**0.75**	**0.18**	**1.56**	**0.39**	**0.15**	**0.60**	**0.37**	**0.19**	**0.13**	0.00
Nevada	1.6	3.0	1.4	0.02	0.04	0.01	0.00	-0.01	0.01	0.01	-0.01	0.01	0.01	0.01	0.01
Ohio	1.6	3.9	2.3	**2.68**	**0.31**	**2.71**	**0.47**	**0.99**	**0.61**	**0.16**	**0.99**	**0.74**	**0.40**	**0.49**	**0.49**

State	Age of Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Change in IRI (IRI at Last Profile Date - IRI at First Profile Date) (m/km) Section Number											
				13	14	15	16	17	18	19	20	21	22	23	24
Arizona	0.5	5.3	4.8	0.03	0.07	-0.02	0.03	0.00	-0.01	0.09	0.02	0.01	0.05	0.02	0.02
Arkansas	0.8	2.8	2.0	0.02	0.08	0.04	-0.05	0.00	0.00	0.04	0.03	0.00	0.02	-0.01	0.01
Michigan	1.2	1.6	0.4	N/A	N/A	0.02	0.03	0.04	-0.04	N/A	**1.36**	**0.14**	N/A	0.04	0.02
Nebraska	0.3	2.8	2.5	**0.51**	**0.25**	-0.02	**0.17**	-0.03	**0.19**	0.08	**0.12**	**0.14**	**0.31**	**0.18**	0.04
Texas	0.4	1.0	0.6	0.03	**0.15**	**0.12**	-0.02	0.01	-0.06	**0.10**	**0.13**	0.06	0.05	0.05	**0.19**
Virginia	0.4	2.9	2.5	**0.67**	0.05	0.03	0.01	0.01	0.03	0.08	**0.11**	0.09	0.06	0.07	0.07

Note: N/A - Sections not accepted into the LTPP program because of deviations from construction guidelines.

Table 10. Percent change in IRI for SPS-1 sections.

State	Age of Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Percent Change in IRI (Note 1) (m/km)											
				Section Number											
				1	2	3	4	5	6	7	8	9	10	11	12
Alabama	3.0	5.1	2.1	2	8	1	8	4	**11**	2	-1	-7	9	1	7
Delaware	0.6	2.5	1.9	-12	0	-11	-16	-14	-11	-20	-15	-18	-24	-19	-26
Iowa	0.9	6.7	5.8	**28**	**59**	**34**	**27**	**25**	**25**	**100**	**47**	**8**	**36**	**30**	**30**
Kansas	0.5	5.4	4.9	**29**	**26**	**49**	**19**	**64**	**34**	**19**	**44**	**34**	**22**	**15**	0
Nevada	1.6	3.0	1.4	2	6	1	0	-1	1	1	-1	2	2	2	2
Ohio	1.6	3.9	2.3	**66**	**20**	**100**	**39**	**48**	**35**	**11**	**53**	**51**	**25**	**39**	**35**

State	Age of Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Percent Change in IRI (Note 1) (m/km)											
				Section Number											
				13	14	15	16	17	18	19	20	21	22	23	24
Arizona	0.5	5.3	4.8	3	**10**	-3	4	1	-1	9	2	1	5	2	3
Arkansas	0.8	2.8	2.0	2	**10**	5	-6	0	0	5	4	0	3	-2	1
Michigan	1.2	1.6	0.4	N/A	N/A	3	5	5	-4	N/A	**60**	**10**	N/A	8	3
Nebraska	0.3	2.8	2.5	**26**	**18**	-2	**14**	-3	**14**	6	9	**10**	**22**	**16**	3
Texas	0.4	1	0.6	3	**17**	**12**	-2	1	-7	**12**	**15**	7	6	5	**18**
Virginia	0.4	2.9	2.5	**41**	5	3	1	1	2	**7**	9	8	5	7	8

Note 1: Percent Change in IRI = 100 X (IRI Last Profile Date - IRI First Profile Date)/(IRI at First Profile Date)

N/A: Sections not accepted to the LTPP program because of deviations from construction guidelines..

that had a change in IRI of 0.09 m/km. Thus, all sections in this project have more or less maintained their roughness values from the second to the last profile date. Therefore, it appears there was an error in the profiling equipment when this project was profiled the first time.

Most of the SPS-1 projects are still young, and many projects are not showing changes in roughness. Material test data and monitored traffic data for most of the SPS-1 projects are not yet available in the IMS. Therefore, a comprehensive statistical analysis to identify parameters that relate to development of pavement roughness cannot be performed yet.

The projects located in Alabama, Arizona, Iowa and Kansas are between five and seven years old. All sections in the Alabama and Arizona projects have not shown much change in IRI, in fact the highest percentage change in IRI for a test section in the Arizona project is 10 percent and in the Alabama project is 11 percent. The test sections in the Iowa and Kansas projects show an increase in IRI with only one section in each project showing a percent change in IRI that is less than 10 percent. The other sections in these two projects are showing an increase in IRI that range from 15 to 100 percent. The only other project where the majority of the sections show an increase in roughness is the project in Ohio, which is 3.9 years old and the test sections show an increase in IRI ranging from 11 to 100 percent.

An evaluation of the pavement distress at the SPS-1 projects was performed to see if the distresses that contribute to increase in roughness could be identified. The distress survey date that was closest to the last profile date was selected in the database, and the distresses for this date were summarized. Distresses that were evaluated were fatigue cracking, block cracking, longitudinal cracking in the wheel path, number of transverse cracks, and the length of transverse cracking. The summarized distresses are shown in table 11. The number of sections in each project that exhibit fatigue cracking, longitudinal cracking in the wheel path and transverse cracking are shown in table 11. Block cracking was not noted in any SPS-1 project. Table 11 also presents the average distress, which was computed by summing each type of distress for all sections and dividing this value by the number of sections in the project that exhibited that distress. For each distress type all severity levels were combined when computing the average values.

Table 11. Pavement distress at SPS-1 projects.

State	Last Profile Date	Distress Survey Date	Distress Survey Method	Sections Exhibiting Distress			Average Distress (see Note 1)			
				Fatigue Cracking	Longitudinal Cracking in Wheel Path	Transverse Cracking	Fatigue Cracking (m^2)	Longitudinal Cracking Wheel Path (m)	Transverse Cracks (No)	Transverse Cracking (m)
Alabama	1/98	2/00	Pasco	0	1	4	-	1	1	2
Arizona	12/98	2/99	Manual	6	10	1	8	33	1	1
Arkansas	7/97	6/97	Manual	5	4	3	11	4	3	3
Delaware	11/98	6/99	Pasco	1	0	2	2	-	1	1
Florida	1/97	12/96	Manual	0	0	0	-	-	-	-
Iowa	7/99	8/99	Pasco	4	10	12	10	36	11	20
Kansas	3/99	12/99	Manual	12	4	9	158	34	6	17
Louisiana	11/97	5/98	Manual	0	0	0	-	-	-	-
Michigan	6/97	10/98	Manual	2	2	3	14	31	1	2
Nebraska	5/98	10/98	Manual	0	2	2	-	8	2	3
Nevada	8/98	12/98	Manual	3	2	0	4	13	-	-
New Mexico	3/97	5/97	Manual	0	0	0	-	-	-	-
Ohio	11/98	8/99	Pasco	0	0	0	-	-	-	-
Oklahoma	11/97	12/97	Manual	0	0	0	-	-	-	-
Texas	4/98	3/98	Manual	1	1	0	3	61	-	-
Virginia	10/98	10/99	Manual	2	1	1	72	15	1	1

Note 1: Average Distress = Sum of Distresses / Number of Sections With Distress
- Distress type not present.

All sections in the Iowa SPS-1 project exhibited transverse cracking, with ten of these sections also exhibiting longitudinal cracking in the wheel path. The increase in roughness for the sections in this project is attributed to these two distresses. All sections in the Kansas project exhibited fatigue cracking, with nine of these sections also exhibiting transverse cracking, and the increase in roughness for this project is attributed to these two distresses. The project in Ohio also showed a large increase in roughness, but as shown in table 11 none of the sections in this project had any distresses at the last distress survey date. However, this section has non-wheel path longitudinal cracking and rutting. The increase in roughness for this project is attributed to rutting. Table 11 shows that ten sections in the Arizona SPS-1 project have longitudinal cracking in the wheel path that are either low or medium severity, but the sections in the projects have shown no increase in roughness. It is likely that the dry no-freeze environment and the coarse subgrade at this project is helping to prevent differential movement between the two sides of the crack, and therefore not causing an increase in roughness. Some sections in the project in Texas, which is one year old, are showing an increase in roughness. At the time of last profile date, the rutting in the left wheel path of the sections ranged from 0 to 18 mm with an average of 9.8 mm and the rutting in the right wheel path ranged from 0 to 8 mm, with an average of 4 mm. The increase in roughness of this project is attributed to rutting of the sections.

The three projects that showed the largest increase in roughness were located in Iowa, Kansas and Ohio, and all these three projects had test section numbers 1 through 12. All three projects are located on a fine grained subgrade. An ANOVA was conducted using the data from these three projects to see if differences in performance between test sections could be identified. The two-way ANOVA was conducted with state and section number being considered as independent variables, and the percent increase in roughness as the dependant variable. The ANOVA indicated there was no significant difference in IRI between the test sections. That is the statistical analysis did not indicate that stronger pavement sections behaved differently from some of the weaker test sections, or if the provision of drainage caused a difference in IRI of the test sections. Sufficient materials test data are not available in the LTPP database to investigate the cause for the increase in roughness at the projects in Iowa, Kansas, and Ohio, or for the cause of the good performance for the projects in Arizona and Alabama (both of which are older than 5 years).

Summary of Findings

The data from the SPS-1 projects indicated the sections with a 100 mm thick AC surface had an average early-age IRI of 0.88 m/km, and a standard deviation of IRI of 0.21 m/km. The sections with a 175 mm thick AC surface had an average IRI of 0.82 m/km, and a standard deviation of IRI of 0.18 m/km. IRI values less than 0.8 m/km was achieved on 40 percent of the sections that received a 100 mm AC surface and 55 percent of the sections that received a 175 mm AC surface. An IRI of less than 1.0 m/km was obtained by 75 percent of the sections that received a 100 mm AC surface and 85 percent of the sections that received a 175 mm AC surface.

The AC surfaces in SPS-1 projects have been placed on three different types of bases: DGAB, ATB, and PATB. The average IRI of sections placed on base types of DGAB, ATB and PATB were 0.94 m/km, 0.82 m/km, and 0.84 m/km. The statistical analysis indicated that there was a significant difference in early age IRI of pavements placed on DGAB and ATB. The analysis also indicated there was no significant difference between early age IRI 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. In both the Iowa and Kansas projects, all sections except for one are showing a percent increase in IRI between 15 to 100 percent. This increase in IRI occurred in approximately 6 years for the Iowa project, and approximately 5 years for the Kansas project. The increase in IRI at the project in Iowa is attributed to transverse cracking and longitudinal cracking in the wheel path, while the increase in IRI for the project in Kansas is attributed to fatigue cracking and transverse cracking. The percent increase in IRI at the test sections in the Ohio project ranged from 11 to 100 percent, with this increase occurring approximately in 4 years. The increase in IRI at the sections in the Ohio project is attributed to rutting. Some of the test sections in the

Texas project are showing an increase in IRI of over 10 percent within an approximately 6-month period. This increase in IRI is attributed to rutting. A statistical analysis was conducted using the percent increase in roughness of the test sections in Kansas, Iowa and Ohio to see if differences in test sections could be identified. The analysis indicated that there were no significant differences between the test sections.

The average project IRI, which is the average of the IRI of all sections in the project for the projects in Iowa, Kansas and Ohio were 0.88 m/km, 0.81 m/km and 1.10 m/km, respectively. Although the pavements in these projects achieved a smooth pavement initially, many sections, including very thick sections had high increases in roughness during the initial life of the pavement. This demonstrates that achieving a smooth pavement initially does not guarantee that it will remain smooth even during the initial life. Factors such as mix design problems in the AC, inadequate preparation of the subgrade prior to placing the pavement, or other construction problems can cause a pavement that is built smooth initially to increase its roughness within a short time period.

SPS-2 EXPERIMENT: STRATEGIC STUDY OF STRUCTURAL FACTORS FOR RIGID PAVEMENTS

Introduction

The SPS-2 experiment was designed to investigate the effect of selected structural factors on the long-term performance of rigid pavements constructed on different subgrades and in different environmental regions. New pavements were constructed for the SPS-2 experiment. In the SPS-2 experiment, twelve test sections were constructed at a project location. All test sections were doweled jointed PCC with a joint spacing of 4.6 m. The twelve test sections constructed for a project were either section numbers 1 through 12, or section numbers 13 through 24. The pavement structure of the test sections is shown in table 12. The structural factors considered in this experiment are: thickness of PCC layer (200 and 275 mm), flexural strength of the PCC at 14 days (3.8 and 6.2 MPa), base type (lean concrete base, dense graded

Table 12. Structural properties of SPS-2 sections.

Test Section Number	PCC Thickness (mm)	Flexural Strength (MPa)	Lane Width (m)	Layer 2 Material	Layer 2 Thickness (mm)	Layer 3 Material	Layer 3 Thickness (mm)
1	200	3.8	3.66	DGAB	150	-	-
2	200	6.2	4.27	DGAB	150	-	-
3	275	3.8	4.27	DGAB	150	-	-
4	275	6.2	3.66	DGAB	150	-	-
5	200	3.8	3.66	LCB	150	-	-
6	200	6.2	4.27	LCB	150	-	-
7	275	3.8	4.27	LCB	150	-	-
8	275	6.2	3.66	LCB	150	-	-
9	200	3.8	3.66	PATB	100	DGAB	100
10	200	6.2	4.27	PATB	100	DGAB	100
11	275	3.8	4.27	PATB	100	DGAB	100
12	275	6.2	3.66	PATB	100	DGAB	100
13	200	3.8	4.27	DGAB	150	-	-
14	200	6.2	3.66	DGAB	150	-	-
15	275	3.8	3.66	DGAB	150	-	-
16	275	6.2	4.27	DGAB	150	-	-
17	200	3.8	4.27	LCB	150	-	-
18	200	6.2	3.66	LCB	150	-	-
19	275	3.8	3.66	LCB	150	-	-
20	275	6.2	4.27	LCB	150	-	-
21	200	3.8	4.27	PATB	100	DGAB	100
22	200	6.2	3.66	PATB	100	DGAB	100
23	275	3.8	3.66	PATB	100	DGAB	100
24	275	6.2	4.27	PATB	100	DGAB	100

Note: DGAB – Dense Graded Aggregate Base, LCB - Lean Concrete Base, PATB - Permeable Asphalt Treated Base

aggregate base and permeable asphalt treated base over dense graded aggregate base), lane width (3.65 and 4.27 m), and drainability (presence or lack of it as provided by an open graded permeable asphalt treated layer and edge drains). The subgrade types considered in this experiment are classified as fine grained and coarse grained, while the environmental regions considered are the four LTPP regions: wet-freeze, wet no-freeze, dry-freeze and dry-no freeze. The test sections in a SPS-2 project were profiled immediately after construction, and thereafter at approximately annual intervals.

Analyzed Projects

A review of the IMS database indicated that profile data were available for twelve SPS-2 projects. Table 13 presents the following information for each SPS-2 project: section numbers in project, climatic zone, subgrade type, construction date, last available profile date, age of project at first profile date, age of project at last profile date, time difference between first and last profile dates, the number of times the project was profiled, and the traffic volume. Ten of the twelve SPS-2 projects have been profiled within one year after construction, with the other two projects being profiled when their age was between one and two years. The SPS-2 project in Arkansas was profiled only once after construction, while the others were profiled 2 to 9 times.

Analysis of Early Age IRI

The SPS-2 projects were first profiled at varying times after construction. Therefore, the IRI obtained during the first profile date may not necessarily correspond to the IRI that is obtained immediately after construction. Therefore, the term early-age IRI is used in this analysis to differentiate from the initial IRI of the pavement, which corresponds to the IRI immediately after construction. The early-age IRI values of all SPS-2 projects were used in this analysis.

Figure 13 shows the average early-age IRI of the test sections in each project, differentiated according to the PCC thickness. The IRI shown for a project in figure 13 is the average IRI of the six test sections that have a PCC thickness of 200 mm or 275 mm. The average IRI for the 200 mm PCC and 275 mm PCC sections were close to each other for most projects, but for a majority of the projects the average IRI of the 275 mm thick PCC pavements were higher than the average IRI of the 200 mm thick PCC pavement. The maximum difference in the average IRI between the two thicknesses occurred for the project in Delaware, where the average IRI of the 275 mm thick PCC sections were 0.3 m/km greater than the average IRI of the

Table 13. SPS-2 projects.

Location	State Code	Section Numbers in Project	Climatic Zone (Note 1)	Subgrade Type	Construction Date	Last Available Profile Date	Age of Project at First Profile Date (Years)	Age of Project At Last Profile Date (Years)	Time Difference Between First And Last Profile Date (Years)	Number of Times Profiled	Traffic (KESAL/yr)
Arizona	AZ	13-24	DNF	Coarse	10/1/93	12/8/98	0.3	5.2	4.9	5	1094
Arkansas	AR	13-24	WNF	Fine	11/1/95	2/6/97	1.3	1.3	0.0	1	N/A
Colorado	CO	13-24	DF	Coarse	11/1/93	8/14/98	0.4	4.8	4.3	3	487
Delaware	DE	13-24	WF	Coarse	5/1/96	7/28/98	0.6	2.2	1.6	6	N/A
Iowa	IA	13-24	WF	Fine	12/1/94	7/17/99	0.2	4.6	4.4	5	56
Kansas	KS	1-12	DF	Fine	8/1/92	3/15/99	0.0	6.6	6.6	8	639
Michigan	MI	13-24	WF	Fine	11/1/93	4/12/99	0.8	5.4	4.6	9	1600
Nevada	NV	1-12	DF	Coarse	9/1/95	8/28/98	0.8	3.0	2.2	4	N/A
North Carolina	NC	1-12	WNF	Fine	1/1/94	11/4/98	0.2	4.8	4.6	8	728
North Dakota	ND	13-24	DF	Fine	11/1/94	5/14/99	1.8	4.5	2.7	2	N/A
Ohio	OH	1-12	WF	Fine	6/1/96	11/12/98	0.2	2.4	2.2	4	N/A
Washington	WA	1-12	DF	Coarse	11/1/95	5/7/99	0.0	3.5	3.5	4	N/A

Note 1: DF: Dry Freeze, DNF - Dry No-Freeze, WF - Wet Freeze, WNF - Wet No-Freeze

200 mm sections. The standard deviations of early-age IRI for the SPS-2 projects are shown in figure 14. The 275 mm PCC sections in Iowa and Nevada had the highest standard deviation in IRI.

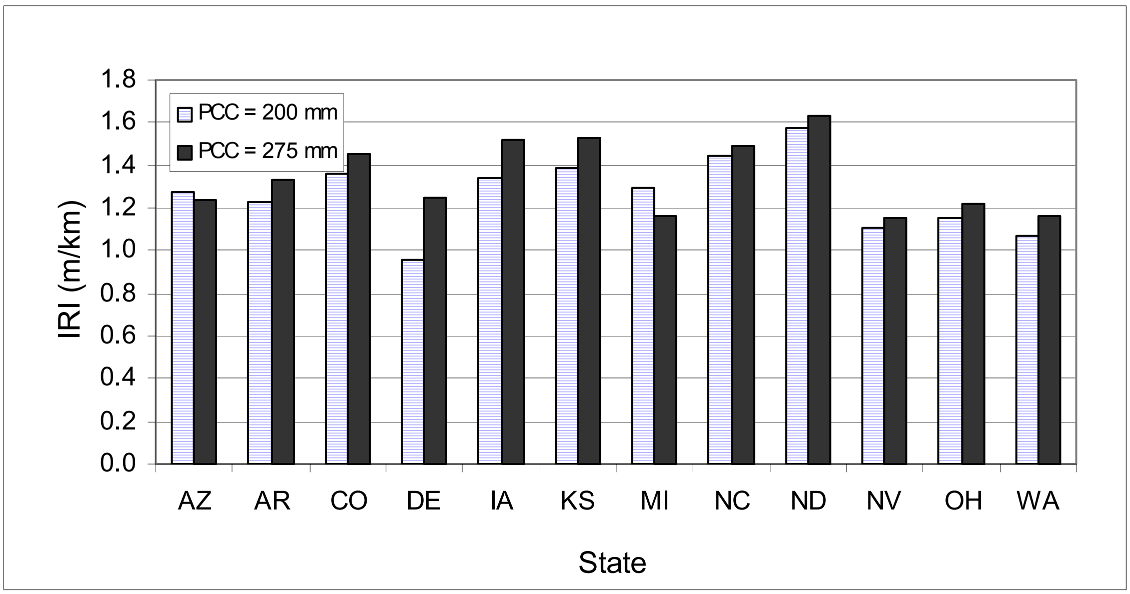


Figure 13. Average early-age IRI of SPS-2 projects.

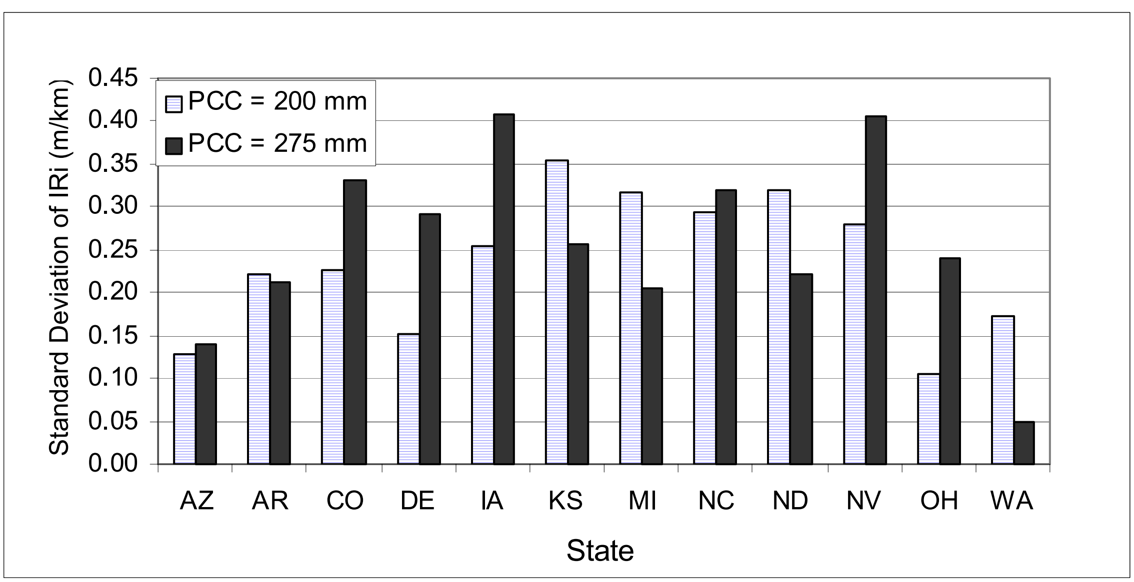


Figure 14. Standard deviation of early-age IRI for SPS-2 projects.

The frequency distribution of the early-age IRI values of the test sections in the SPS-2 projects separated according to the two PCC thicknesses is shown in figure 15, while the cumulative frequency distribution is shown in figure 16.

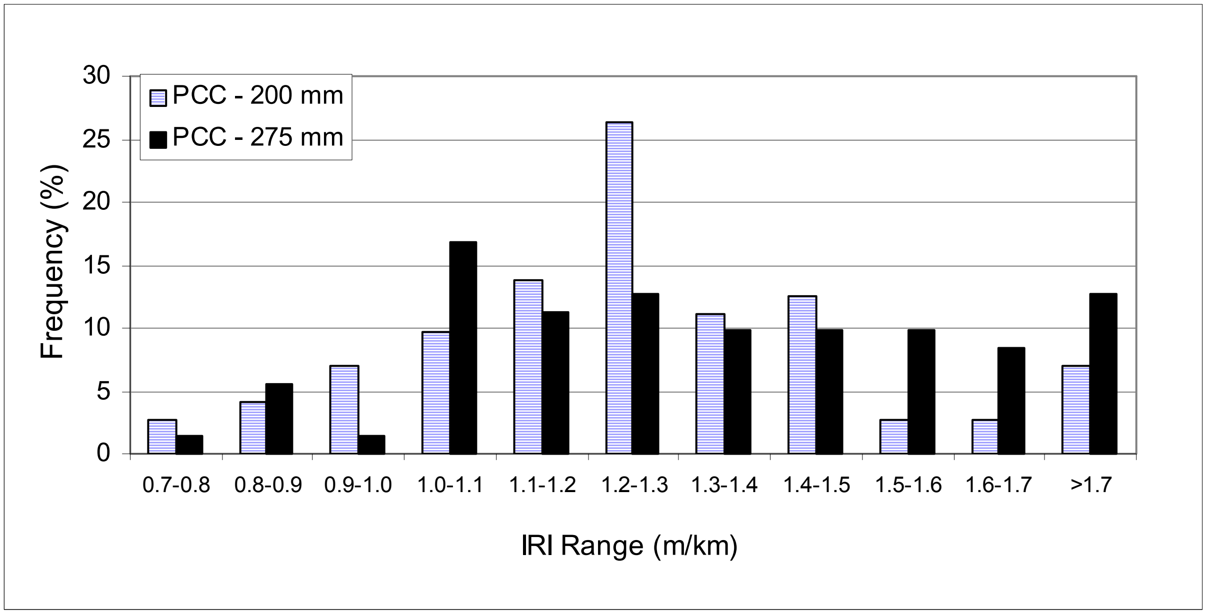


Figure 15. Frequency distribution of early-age IRI.

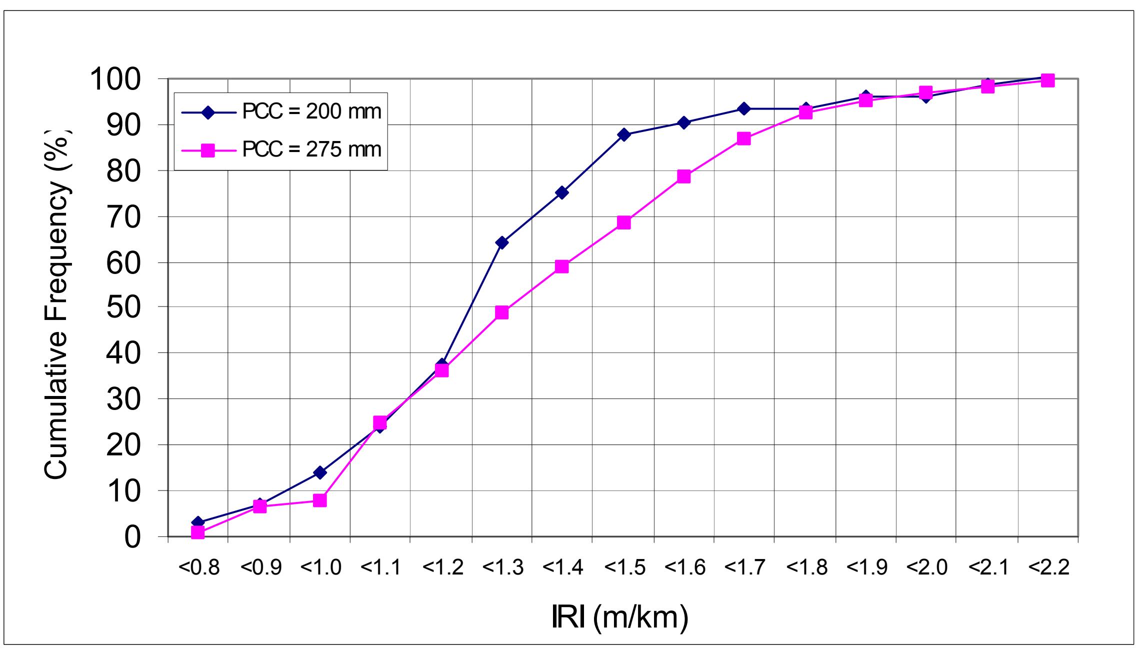


Figure 16. Cumulative frequency distribution of early-age IRI.

About 40 percent of the sections had an IRI of less than 1.2 m/km for both 200 mm and 275 mm thick PCC pavements. About 90 percent of 200 mm PCC sections and 70 percent of the 275 mm PCC sections had an IRI of less than 1.5 m/km.

The average and the standard deviation of early-age IRI values for the two different PCC thicknesses are shown in table 14.

Table 14. Average and standard deviation of early-age IRI: SPS-2 .

PCC Thickness	IRI (m/km)	
(mm)	Average	Std. Dev
200	1.27	0.28
275	1.30	0.30

An analysis was performed to study if the early-age IRI depended on the base type, PCC thickness, and flexural strength of PCC. The PCC surface in SPS-2 projects was placed on three different base types: dense graded aggregate base (DGAB), lean concrete base (LCB), and permeable asphalt treated base (PATB). In a SPS-2 project, four sections were placed on each of the three base types. The two flexural strengths used in the SPS-2 projects were 3.8 and 6.2 Mpa, while the two PCC thicknesses are 200 mm and 275 mm.

A three way ANOVA was conducted using the early-age IRI as the dependant variable, and PCC thickness, base type, and flexural strength as independent variables. The ANOVA indicated PCC thickness and flexural strength were not significant while the base type was significant (p-value = 0.02). A multiple comparison indicated IRI values of PCC pavements on LCB was significantly different than PATB. Figure 17 shows a box-plot of the distribution of the early-age IRI values categorized according to the base type. As shown in the box-plot, the pavements placed on LCB had the highest median IRI value. Three sections placed on LCB had IRI values greater than 2.0 m/km, and are considered to be outliers. Two sections placed on PATB also had early-age IRI values that were greater than 2.0 m/km, and are also considered to be outliers. Table 15 presents the average, standard deviation, and 15th and 85th percentile early-IRI values classified according to base type. As shown in table 15, the highest early-age IRI values were obtained on PCC pavements that were placed on LCB bases.

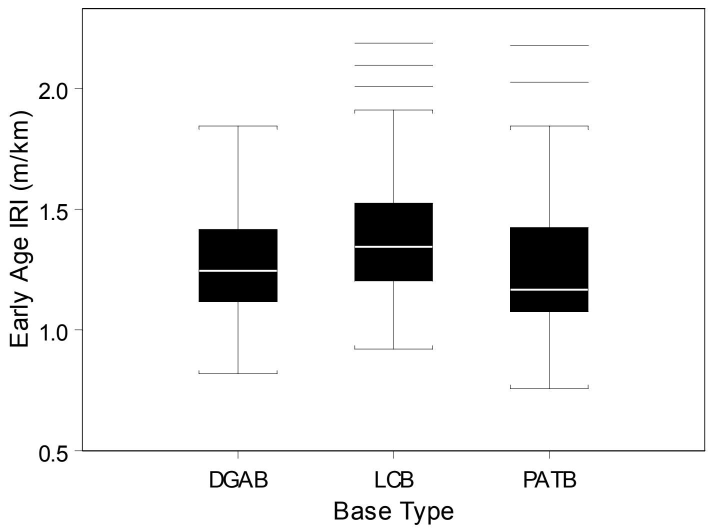


Figure 17. Box-plot of early-age IRI.

Table 15. Average and standard deviation of early-age IRI classified according to base type.

Base Type	Early Age IRI (m/km)			
	Average	Standard Dev	15th Percentile	85th Percentile
Aggregate Base	1.27	0.24	1.02	1.49
Lean Concrete Base	1.40	0.29	1.10	1.60
Permeable Asphalt Treated Base	1.25	0.32	0.98	1.55

Changes in IRI for SPS-2 Projects

Plots showing the changes in IRI of the test sections in individual SPS-2 projects are included in Appendix B. The SPS-2 projects are still young, with 27 percent of the projects being between 5 and 7 years old, and 63 percent of the projects being less than five years old. Table 16 presents the changes in IRI for the test sections in each SPS-2 project relative to the IRI at the first profile date. Table 16 also shows the age of the project when it was first profiled, age of project at last profile date, and the time difference corresponding to the change in IRI. In table

Table 16. Change in IRI at SPS-2 sections.

State	Age of Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Change in IRI (IRI at Last Profile Date - IRI at First Profile Date) (m/km)											
				Section Number											
				1	2	3	4	5	6	7	8	9	10	11	12
Delaware	0.6	2.2	1.6	**0.17**	0.07	-0.03	-0.01	0.05	-0.29	0.04	0.05	0.01	**0.13**	0.04	0.03
Kansas	0.0	6.6	6.6	**0.30**	-0.23	0.06	**0.11**	0.04	-0.48	0.03	**0.12**	-0.06	**0.12**	-0.05	-0.07
N. Carolina	0.2	4.8	4.6	**0.31**	**0.39**	**0.21**	**0.13**	0.06	**0.19**	0.09	**0.23**	**0.18**	0.02	**0.10**	**0.12**
Nevada	0.8	3.0	2.2	**0.51**	**0.61**	**0.24**	**0.54**	**0.19**	**0.10**	**0.44**	**0.12**	**0.39**	**0.37**	**0.50**	N/A
Ohio	0.2	2.1	1.9	**0.21**	**0.20**	0.01	0.09	**0.10**	0.09	-0.14	-0.21	**0.13**	-0.06	-0.04	-0.10
Washington	0.1	3.5	3.5	**0.14**	0.03	0.02	0.03	0.01	**0.54**	**0.11**	**0.13**	**0.10**	**0.23**	0.00	**0.10**

State	Age of Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Change in IRI (IRI at Last Profile Date - IRI at First Profile Date) (m/km)											
				Section Number											
				13	14	15	16	17	18	19	20	21	22	23	24
Arizona	0.3	5.2	4.9	**0.31**	-0.23	**0.28**	-0.05	-0.09	-0.35	**0.12**	-0.12	**0.11**	-0.10	**0.21**	0.04
Colorado	0.5	4.8	4.4	-0.04	-0.08	0.00	-0.06	0.05	-0.06	0.07	0.08	-0.08	0.00	-0.17	-0.06
Iowa	0.2	4.6	4.4	0.01	0.06	0.06	**0.10**	**0.26**	**0.11**	-0.10	0.02	**0.12**	0.06	-0.15	-0.11
Michigan	0.9	5.5	4.7	**1.15**	**0.22**	**0.73**	-0.12	**2.15**	**0.36**	**0.18**	0.02	0.07	-0.07	-0.02	-0.08
North Dakota	1.8	4.5	2.7	-0.24	-0.11	-0.04	0.06	-0.10	N/A	-0.01	0.03	-0.11	-0.15	-0.05	-0.13

Note: N/A - IRI at first profile date incorrect, therefore change not computed.

16, cases that show an IRI increase of 0.10 m/km or greater are shown in bold. Table 17 shows the percent change in IRI at the test sections, with respect to the IRI at the first profile date. Cases where the percent increase is greater than or equal to 10 percent are shown in bold.

In the Nevada project, all sections except for two sections showed 21 to 64 percent increase in IRI, which occurred during 2.2 years. Several sections in the Michigan project show a high increases in roughness. Section 17 in the Michigan project that shows an IRI increase of 214 percent failed because of pumping occurring in the LCB layer. In the North Carolina project, 66 percent of the sections show an increase in IRI of greater than 10 percent within a 4.6 year period. All projects that have section 1 show an increase in IRI that is greater than 10 percent, with the age of the sections ranging from 2.1 to 6.6 years. There are five projects that have section 13 that has similar characteristics as section 1 except that the lane width is 4.27 m. Two of these projects are showing an increase in IRI that is greater than 10 percent. No other trends can be seen for individual projects, or for similar sections across the projects. There are sections that are showing a decrease in roughness from the initial IRI, and some projects have sections that are showing large increases in roughness when compared to other sections in that project.

It is not possible to do a comprehensive analysis on the performance of the SPS-2 projects because of the lack of materials testing data. However, some of the specific observations that were described previously can be investigated to determine if the cause of the change in IRI can be identified from the profile data.

Investigation of Specific Cases

Based on the findings from the analysis of changes in IRI at SPS-2 sections, the following cases were identified for analysis.

1. Investigate the cause for the high increase in roughness at some sections.
2. Investigate the cause of the high reduction in roughness at some sections.
3. Investigate the cause of the increase in roughness in the North Carolina project.

Table 17. Percent change in IRI at SPS-2 sections.

State	Age of Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Percent Change in IRI (Note 1)											
				Section Number											
				1	2	3	4	5	6	7	8	9	10	11	12
Delaware	0.6	2.2	1.6	**16**	8	-3	0	5	-28	4	3	1	**14**	4	2
Kansas	0.0	6.6	6.6	**24**	-19	4	8	3	-23	2	6	-5	9	-4	-4
N. Carolina	0.2	4.8	4.6	**23**	**29**	**12**	**11**	3	**13**	5	**13**	**15**	2	8	11
Nevada	0.8	3.0	2.2	**56**	**41**	**29**	**35**	**21**	7	**45**	7	**48**	**33**	**64**	N/A
Ohio	0.2	2.1	1.9	**17**	**17**	1	**10**	8	7	-10	-14	**13**	-5	-3	-9
Washington	0.1	3.5	3.5	**11**	3	2	2	1	**52**	9	**11**	8	**29**	0	**9**

State	Age of Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Percent Change in IRI (Note 1)											
				13	14	15	16	17	18	19	20	21	22	23	24
Arizona	0.3	5.2	4.9	**21**	-18	**20**	-4	-7	-26	**10**	-10	**10**	-9	**18**	4
Colorado	0.5	4.8	4.4	-4	-7	0	-6	3	-4	5	5	-5	0	-10	-3
Iowa	0.2	4.6	4.4	1	5	3	8	**20**	9	-6	2	9	3	-7	-8
Michigan	0.9	5.5	4.7	**95**	**12**	**83**	-8	**214**	**24**	**15**	2	7	-6	-2	-7
North Dakota	1.2	3.2	2.0	-16	-9	-2	3	-7	N/A	-1	2	-8	-9	-4	-6

Note 1: Percent Change in IRI = 100 X (IRI Last Profile Date - IRI First Profile Date)/(IRI at First Profile Date)

Note: N/A - IRI at first profile date incorrect, therefore change not computed.

4. Investigate the cause for the increase in roughness in the Nevada project.
5. Investigate the cause of high variability in roughness over the years at some sections.

These investigations were carried out by analyzing profile data of the selected sites. The profile data in the IMS database has been filtered with a 100 m cut-off filter. It is difficult to see specific features that affect roughness by viewing these profiles. A closer look at the features that affect roughness was performed by using different filters on the profile data. Band-pass filters that look at specific wavelengths were extensively used in this analysis. All plots in this section that are indicated to have been band-pass filtered have been filtered using with a band pass filter that will keep only the wavelengths between 1.6 and 8 m. Power spectral density (PSD) plots were also extensively used in this analysis. The application of PSD plots for pavement profiles is described by Sayers and Karamihas (20). The PSD plots were used to analyze the distribution of the wavelengths that are contained in a profile. In the PSD plots, the x-axis shows the wavenumber, which is the inverse of wavelength.

Investigate Cause for High Increase in Roughness at Some Sections

Some sections in some SPS-2 projects have shown a high increases in roughness when compared to other sections in that project. Three of the test sections (section 13 in Arizona, section 13 in Michigan, and section 6 in Washington) were selected for detailed investigation.

Arizona – Section 13

Section 13 in Arizona has shown an increase in IRI of 21 percent from the first measured IRI within a period of 4.9 years. This section has a 200 mm PCC slab that rests on a DGAB base, and has a slab width of 4.27 m. This section is showing the highest increase in IRI of all test sections in the Arizona SPS-2 project. The profile date, profile time and the IRI values for this section are shown in table 18. The mean IRI at this section decreased from 1.44 m/km at the first profile date (1994) to 1.20 m/km at the second profile date (1995), and increased to 1.75 m/km at the last profile date (1998).

Table 18. IRI values – section 13 – Arizona.

Profile Date	Profile Time	IRI (m/km)
1/25/94	6:10 AM	1.44
3/5/95	11:21 AM	1.20
12/4/97	11:06 AM	1.71
12/8/98	10:28 AM	1.75

The band-pass filtered profile plots of the right wheel path for the first, second and last profile dates are shown in figure 18. As seen from this plot, the curvature of the slabs is less pronounced at second profile date (date = 1995, IRI = 1.20 m/km) when compared to the first profile date (date = 1994, IRI = 1.44 m/km), but the curvature increases significantly at the last profile dates (date = 1998, IRI = 1.75 m/km).

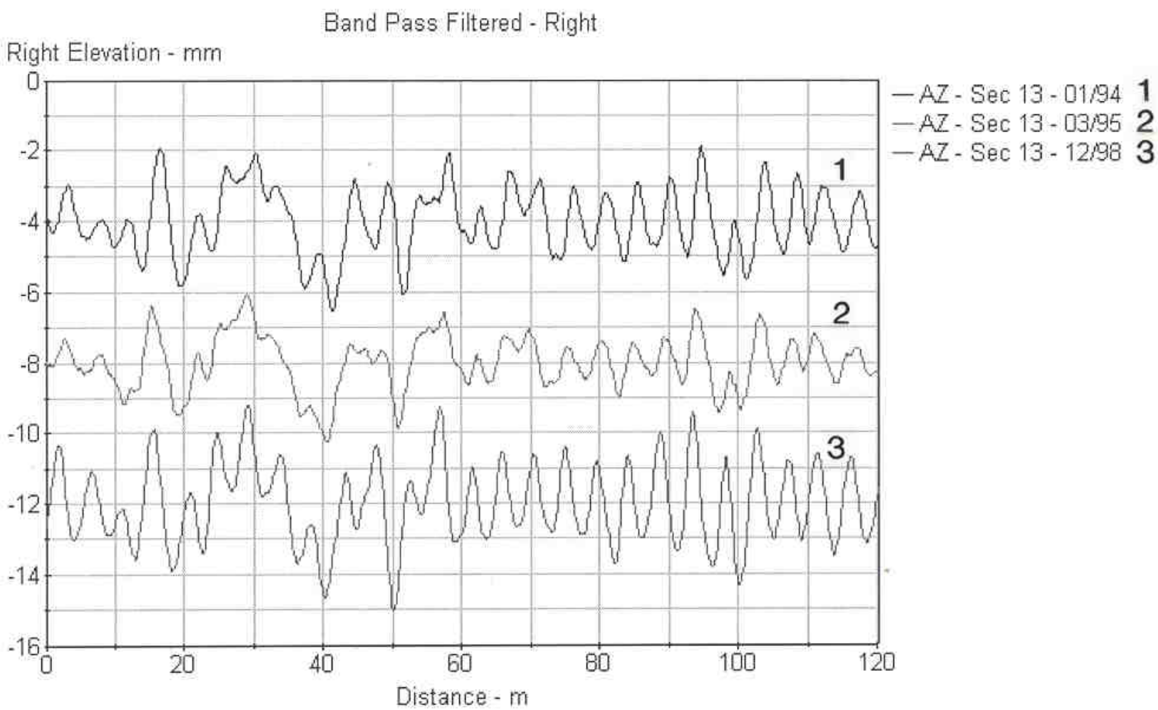


Figure 18. Band pass filtered profile plots – section 13 – Arizona.

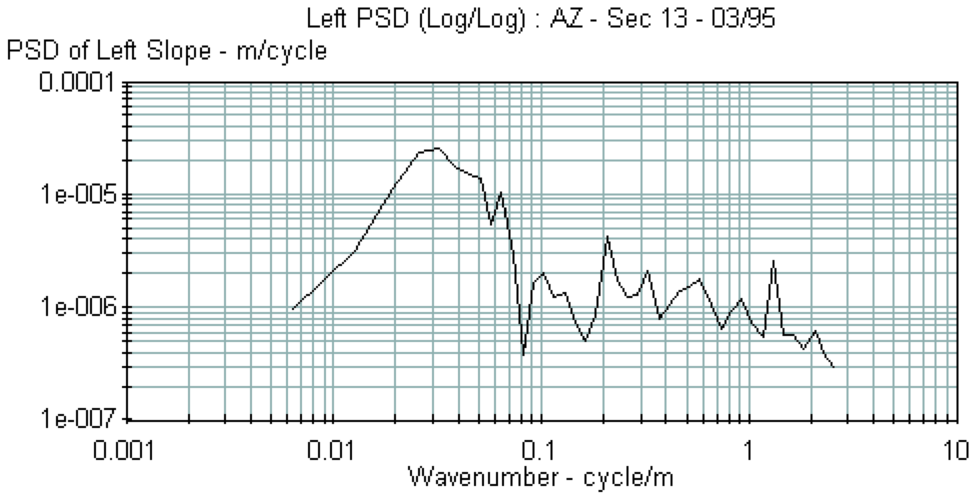
The slabs at this section are curved upwards, where the edges of the slab are at a higher elevation when compared to the center of the slab. The distance between two-peaks in the plot corresponds to 4.6 m, which is the joint spacing of the PCC slabs. Generally, for slabs showing this type of curvature, the curling of the joints is maximum in the early morning when the temperature differential between the top and the bottom of the slab is maximum. The cause for the higher IRI at the first profile date when compared to the second profile date is attributed to this, as this site was first profiled in the early morning. The curvature of the slab at the last profile date when the site was profiled in the mid morning is more pronounced than the curvature at the first profile date when the slab was profiled early in the morning. From these observations it appears that the curled-up shape of the slab is increasing over time, with moisture variations in the slab having a possible effect on slab curling.

The PSD plots for the second profile date (IRI = 1.20 m/km) and last profile date (IRI = 1.75 m/km) are shown in figure 19. A comparison of these two figures show a pronounced peak at a wavenumber of 0.21 cycles/m for the last profiling date of 12/98 when compared to the second profiling date of 03/95. The wavenumber of 0.21 corresponds to a wavelength of 4.6 m, which is the slab length. Therefore, these plots clearly show that the slab curvature is affecting the wavelength distribution in the profile.

Michigan – Section 13

Section 13 in the Michigan SPS-2 project has shown an increase in IRI of 95 percent from its initial IRI. This section has a 200 mm PCC slab that rests on a DGAB base, and has a slab width of 4.27 m. The profile dates, profile times, and IRI values for this section are shown in table 19. The mean IRI at this section has ranged between 1.10 m/km and 1.38 m/km for the first six profile dates from 9/94 to 4/97. From 4/97 to 4/99, the IRI has increased from 1.10 m/km to 2.36 m/km.

The band-pass filtered profile plots of the right wheel path for 1994 (IRI = 1.21 m/km) and 1998 (1.72 m/km) are shown in figure 20. The slabs at this section are curved upwards. The curvature of the slabs in 1998 is much more pronounced than the curvatures in 1994. The site was profiled around noon in 1994 and in the afternoon in 1998, so the profile shapes seen are not influenced by early morning curling behavior. The increase in roughness at this site is attributed to the increase in upward curvature of the slabs.



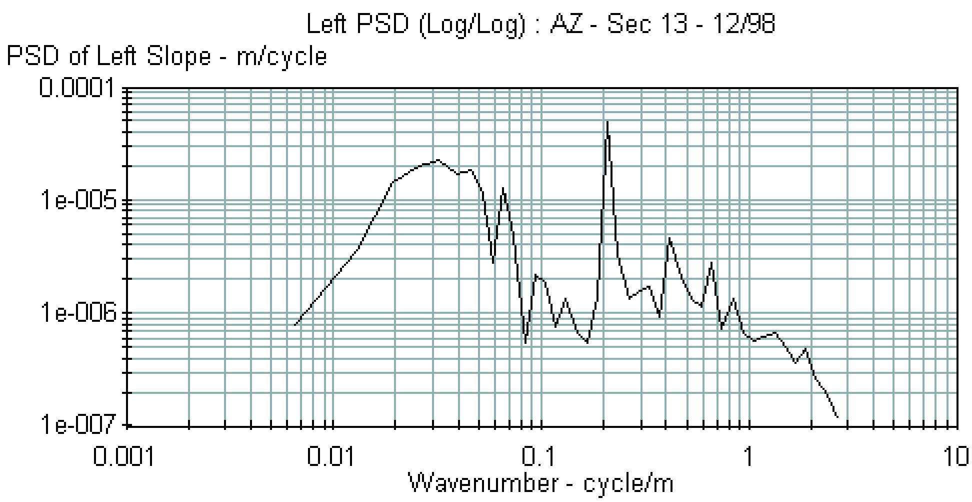


Figure 19. PSD plots – left wheel path profiles – section 13 – Arizona.

Washington – Section 6

Section 6 in the Washington SPS-2 project showed the highest increase in IRI for that project. The profile dates, profile times and IRI for this section are shown in table 20. The IRI of this section increased from 1.04 m/km in 1995 to 1.58 m/km in 1999, an increase of 52 percent that occurred in about three and a half years.

Table 19. IRI values – Section 13 – Michigan.

Profile Date	Profile Time	IRI (m/km)
9/6/94	11:30 AM	1.21
8/11/95	9:35 AM	1.11
1/8/96	4:34 PM	1.23
4/9/96	10:11 AM	1.38
12/29/96	10:28 AM	1.23
4/15/97	1:05 PM	1.10
7/1/97	9:23 AM	1.47
11/5/98	4:35 PM	1.72
4/12/99	12:15 PM	2.36

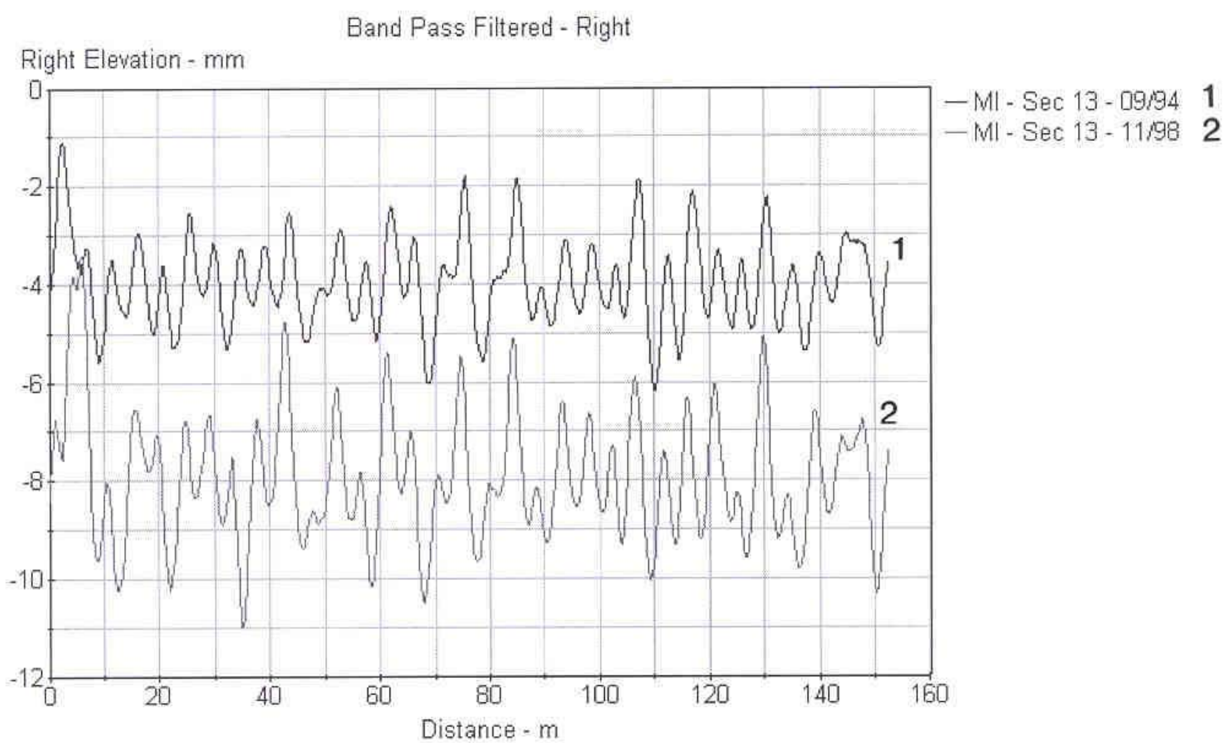


Figure 20. Band pass filtered profile plots – section 13 – Michigan.

Table 20. IRI values – Section 6 – Washington.

Profile Date	Profile Time	IRI (m/km)
11/18/95	1:18 PM	1.04
10/6/97	4:48 PM	1.28
5/15/98	2:27 PM	1.44
5/7/99	1:02 PM	1.58

The 100-m filtered right wheel path profile plots for this section for the four profiling dates are shown in figure 21. As seen from this figure, the slabs take a pattern where the middle of the slab is at a higher elevation compared to the joints. This pattern is clearly seen at the beginning of the section where the humps occur at 4.6 m intervals.

The band-pass filtered right wheel path profiles for this section are shown in figure 22. This figure shows the downward curvature has increased from the first profile date (1995) to the last profile date (1999). The increase in the downward curvature of this site over time contributed to the increase in roughness at this site. The increase in the downward curvature for this section may have been due to changes in moisture conditions within the slab.

Investigate the Cause of High Decrease in Roughness

Test sections in some SPS-2 projects have shown a high decrease in roughness when compared to other test sections in that project. Three test sections that showed a high decrease in roughness, section 18 in Arizona, section 6 in Delaware, and section 6 in Kansas were selected to investigate if the cause of the reduction in roughness could be identified.

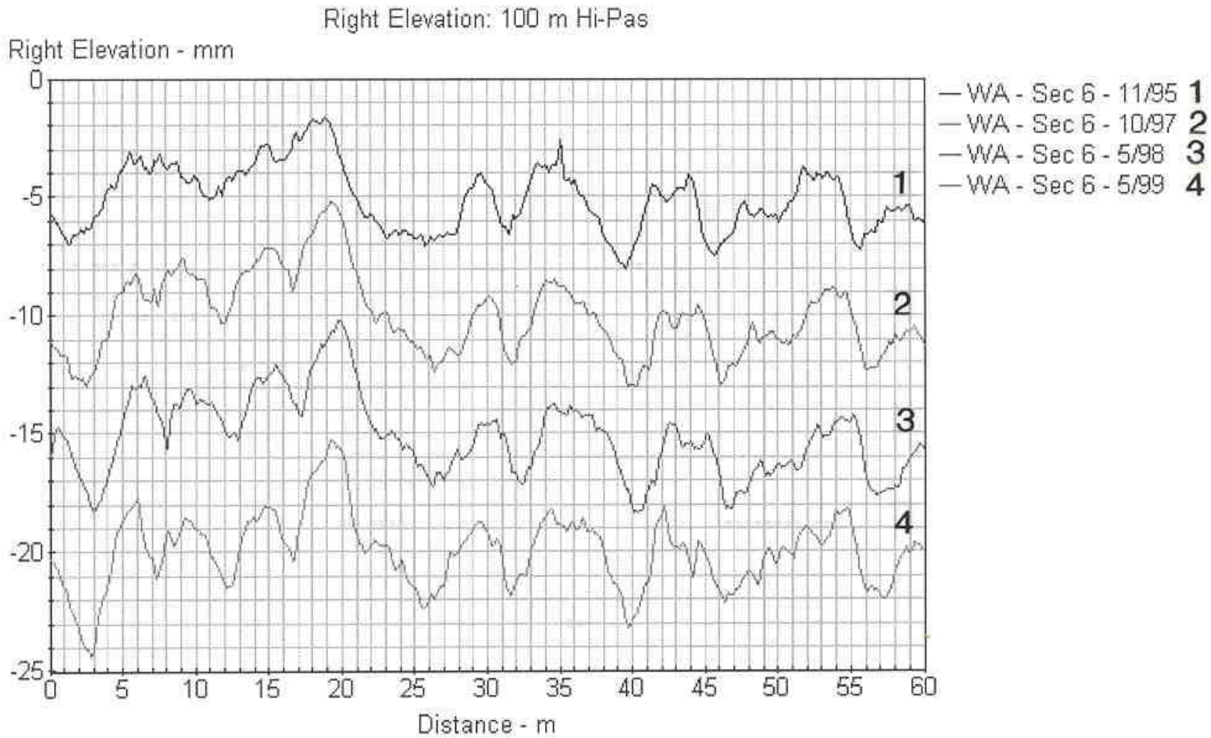


Figure 21. High pass filtered (100 m) right wheel path profile – section 6 – Washington.

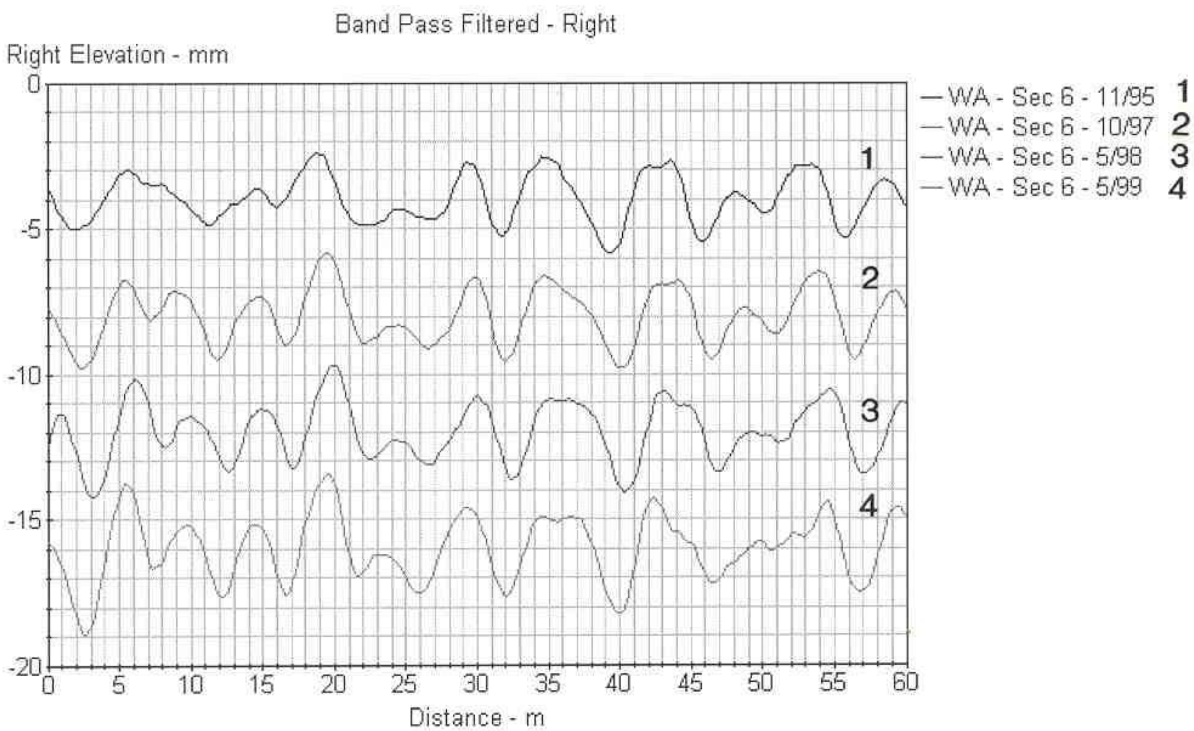


Figure 22. Band pass filtered profile plots – section 6 – Washington.

Arizona – Section 18

Section 18 in the Arizona SPS-2 project has shown a decrease in IRI of 26 percent when compared to the IRI at first profile date. This section has a 200 mm PCC slab with a LCB base, and has a slab width of 3.66 m. The profile dates, profile times and IRI for this section are shown in table 21. The IRI of this section at the first profile date was 1.36 m/km, and thereafter for the other three profile dates the IRI ranged between 0.95 to 1.01 m/km.

Table 21. IRI values – section 18 – Arizona.

Profile Date	Profile Time	IRI (m/km)
1/25/94	6:10 AM	1.36
3/5/95	11:21 AM	0.95
1/27/97	11:22 AM	0.98
12/4/97	11:06 AM	1.01
12/8/98	10:28 AM	1.01

The band-pass filtered left wheel path profile for test dates in 1994, 1995 and 1998 are shown in figure 23. The slabs at this section are curled upwards. The plot for the first profile date shows a high curvature, when compared to the profile plots for the other two profile dates. The site was profiled very early in the morning on the first profile date, and for other dates it was profiled between approximately 10:30 A.M. and 11:30 AM. The cause for the higher curvature for the first profile date is attributed to curling of the slab due to the temperature differential between the top and the bottom of the slab, that is maximum during the early morning. This effect can be clearly seen in a PSD plot by the high peak that occurs at a wavenumebr of 0.23 cycles per meter, which corresponds to a wavelength of 4.6 m. The decrease in roughness noted at the site was therefore caused by the higher initial IRI that was caused by curling due to temperature effects.

Figure 24 shows a bar graph of the IRI values of the test sections in the Arizona SPS-2 project. This figure shows the difference in IRI values between the first profile date when the site was profiled very early in the morning and the second profiling date when the site was profiled

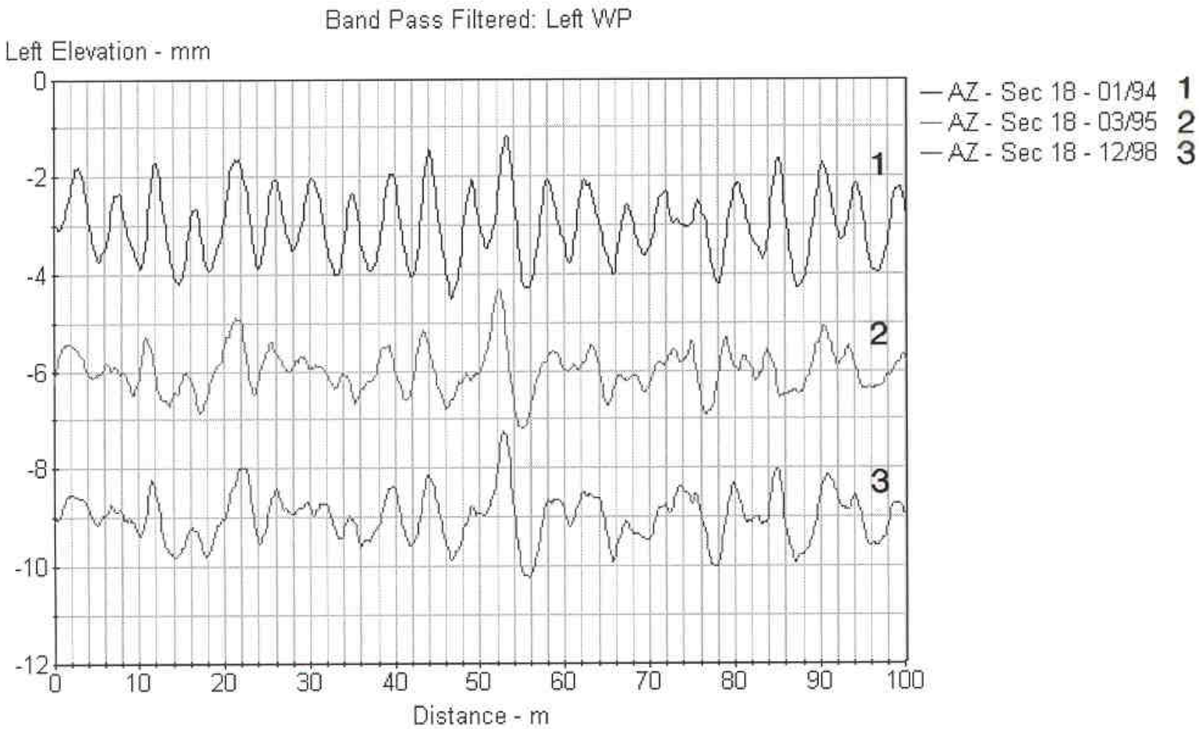


Figure 23. Band pass filtered profile plots – section 18 – Arizona.

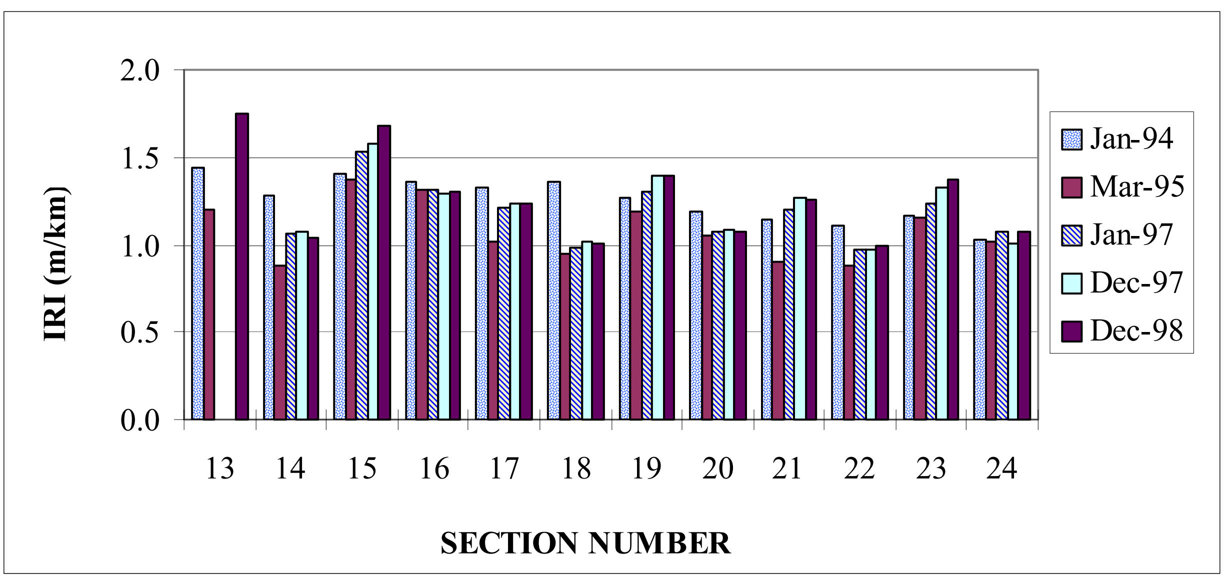


Figure 24. IRI values at SPS-2 project in Arizona.

approximately at 11:20 AM varied from section to section. This difference was minimal for sections that had 275 mm thick PCC slabs (sections 15, 16, 19, 20 23, 24), when compared to the other sections that had 200 mm thick PCC slabs.

Delaware – Section 6

Test section 6 in the Delaware SPS-2 project had a decrease in IRI of 28 percent when compared to the IRI at the first profile date. This section has a 200 mm PCC slab, which is on a DGAB base, and has a slab width of 4.27 m. The only other section in this project to show a decrease in IRI was section 3, which showed a decrease in IRI of 3 percent. The profile dates, profile times and IRI for this section are shown in table 22. The IRI of section 6 decreased from 1.04 m/km at the first profile date (1996) to 0.75 m/km at the last profile date (1998).

Table 22. IRI values – Section 6 – Delaware.

Profile Date	Profile Time	IRI (m/km)
12/6/96	10:53 AM	1.04
6/10/97	10:30 AM	0.74
9/29/97	10:57 AM	0.86
2/25/98	8:26 AM	0.90
5/13/98	1:33 PM	0.71
7/28/98	10:24 AM	0.75

The band-pass filtered left wheel path profile plots for three profile dates, first (12/96, IRI = 1.04 m/km), third (9/97, IRI = 0.86 m/km), and last (7/98, IRI = 0.75 m/km) are shown in figure 25. The profile plot shows the decrease in the curvature effect over the years, which contributed to the decrease in IRI over the years. As all three profiles were taken around 10:30 in the morning, the changes in curvatures should not be related to early morning curling effects.

Kansas - Section 6

Test section 6 in the Kansas SPS-2 project showed a decrease in IRI of 23 percent, when compared to the IRI obtained at the first profile date. This section has a 200 mm PCC slab, which is on a DGAB base, and has a slab width of 4.27 m. The profile dates, profile times and IRI for

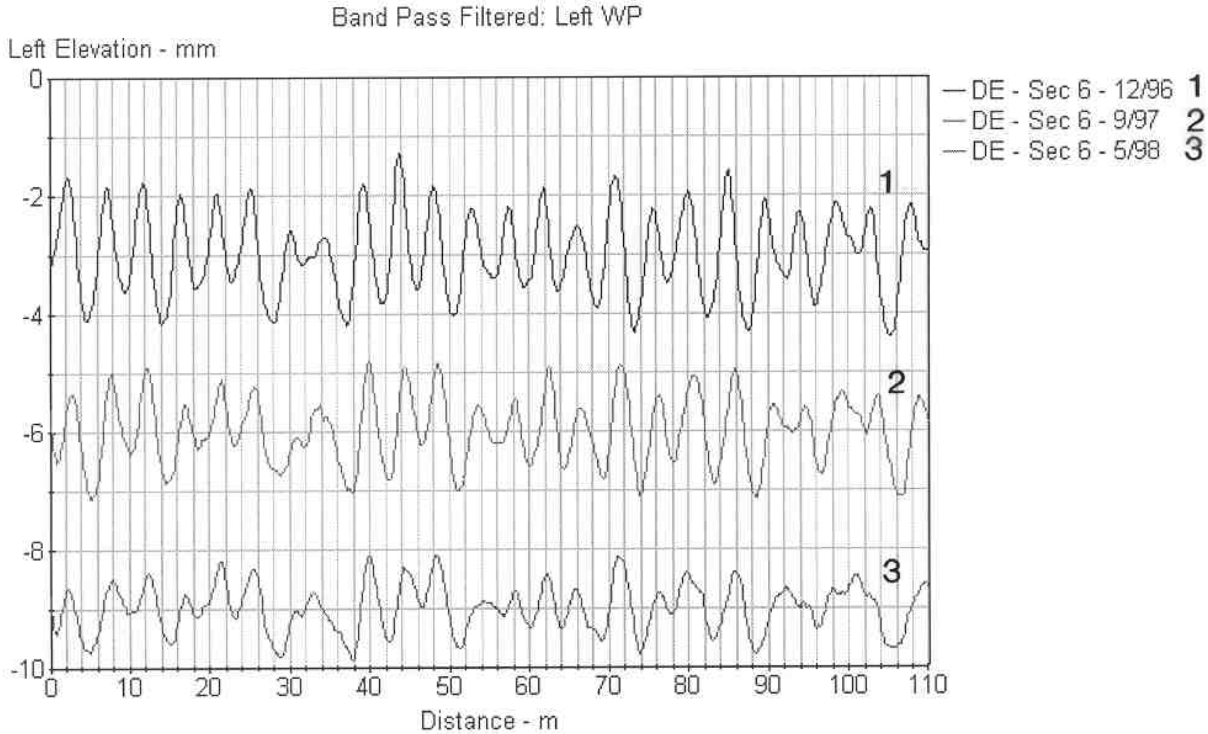


Figure 25. Band pass filtered profile plots – section 6 – Delaware.

this section are shown in table 23. The IRI of this section decreased from 2.09 m/km at the first profile date (1992) to 1.62 m/km at the last profile date (1999). The band pass filtered left wheel path profiles for the first (date = 1992, IRI = 2.09 m/km) and last profile dates (date = 1999, IRI = 1.62 m/km), as well as the profile for 1995 (IRI = 1.39 m/km) are shown in figure 26. The highest curvature in the profile plots is seen for the first profile date (1992) that had the highest IRI, while the least curvature is seen for 1995 when the IRI was the lowest. The effect of curvature is clearly seen in the last part of the profile. The variation in the IRI at this section can be attributed to curvature variations of the slabs. The highest IRI at the section occurred at the first profile date, when the section was profiled past noon. Therefore, the high IRI for this date cannot be attributed to early morning slab curling effects. The profiles in 1995 and 1999 were obtained relatively early in the morning, but yet the IRI at these two dates were less than the IRI obtained at the first profile date. The variation of slab curling observed at the site may be related to moisture variations in the PCC slab.

Table 23. IRI values – Section 6 – Kansas.

Profile Date	Profile Time	IRI (m/km)
8/14/92	1:52 PM	2.09
3/10/93	11:26 AM	1.51
5/15/94	11:00 AM	1.38
2/18/95	9:12 AM	1.39
4/20/96	1:31 PM	1.41
3/3/97	11:40 AM	1.70
5/15/98	10:26 AM	1.57
3/15/99	8:34 AM	1.62

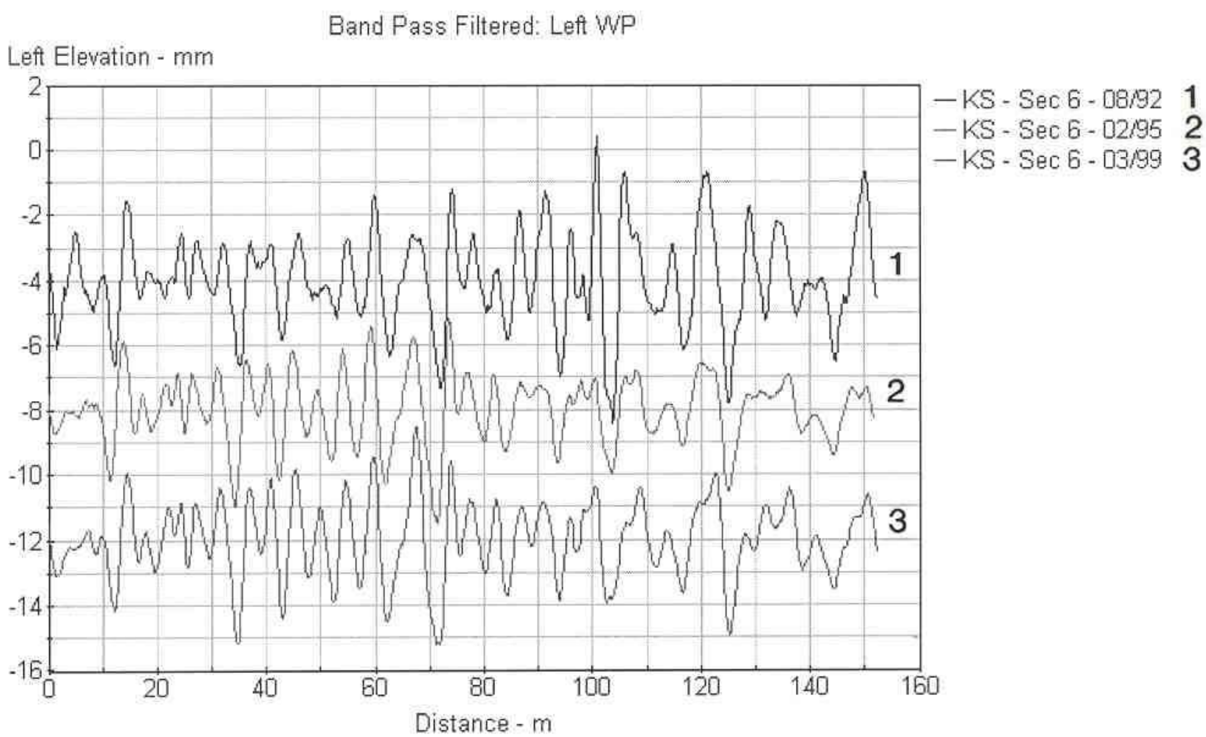


Figure 26. Band pass filtered profile plots – section 6 – Kansas.

Investigate Cause of Increase in Roughness at Test Sections in the North Carolina Project

All sections in the North Carolina project show an increase in IRI when compared to the IRI at the first profile date, with seven test sections showing an increase in IRI of over 10 percent. Section 2 in the project showed the highest percentage increase in IRI of 29 percent. This section has a 200 mm PCC slab, has a 150 mm thick DGAB base, and a slab width of 4.27 m. The profile dates, profile times, and IRI values for this test section is shown in table 24. The IRI of this section from the first profile date (3/94) to the seventh profile date (7/98) ranged from 1.37 m/km to 1.44 m/km. However, at the last (eighth) profile date, the IRI suddenly increased to 1.77 m/km.

Table 24. IRI values – section 2- North Carolina.

Profile Date	Profile Time	IRI (m/km)
3/30/94	10:28 AM	1.37
1/6/96	5:46 AM	1.42
2/28/96	10:43 AM	1.44
10/7/97	1:12 PM	1.41
2/18/98	1:57 PM	1.41
5/19/98	10:36 AM	1.38
7/24/98	11:31 AM	1.42
11/4/98	8:45 AM	1.77

The band pass filtered right wheel path profile plots for 10/97 (IRI = 1.41 m/km), 7/98 (IRI = 1.42 m/km), and last profile date of 11/98 (IRI = 1.77 m/km) are shown in figure 27. The slabs in this section are curled upwards. The plot shows that the slab curvature for the profile date of 11/98 is more than that of the other two dates. The increase in roughness at this test section at the last profile date was due to the increase in slab curvature. The IRI at the last profile date was obtained at 8:45 A.M. in the morning, and the increase in IRI that occurred for that date are attributed to early morning curling effects in the slab.

The PSD plots for profile dates of 10/97 (IRI = 1.41 m/km) and 11/98 (IRI = 1.77 m/km) are shown in figure 28. Note the difference in the plots at a wavenumber of 0.2, where there is a pronounced peak in the profile plot for the profile date of 11/98. The wave number of 0.2

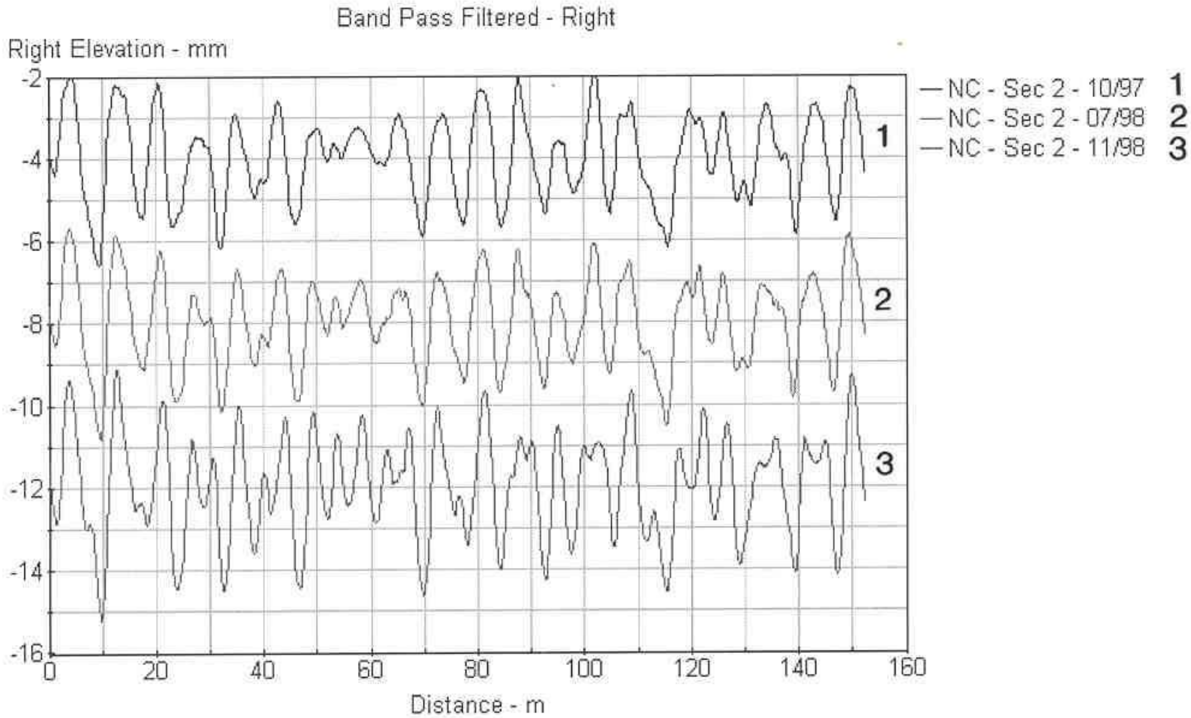


Figure 27. Band pass filtered profile plots – section 2 – North Carolina.

corresponds to a wavelength of 4.6 m, which is the slab length. The curling of the slabs, that are 4.6 m in length, caused a contribution to the IRI and made the IRI to increase at the last profile date.

The IRI values of all test sections in the North Carolina project for the different profiling dates are shown in figure 29. All sections in this project, except for section 10 show a sudden increase in IRI at the last profile dates. This is caused by early morning curling of the slabs. For this project, the effect of curling is seen for both slab thicknesses of 200 and 275 mm.

Investigate Cause for Increase in Roughness at the Nevada Project

All sections in the Nevada project show an increase in IRI when compared to the first profile date, with 5 sections showing an increase in IRI of over 40 percent. This increase in IRI occurred within a 2.2 year period. Section 7 in the Nevada project, which showed an increase in

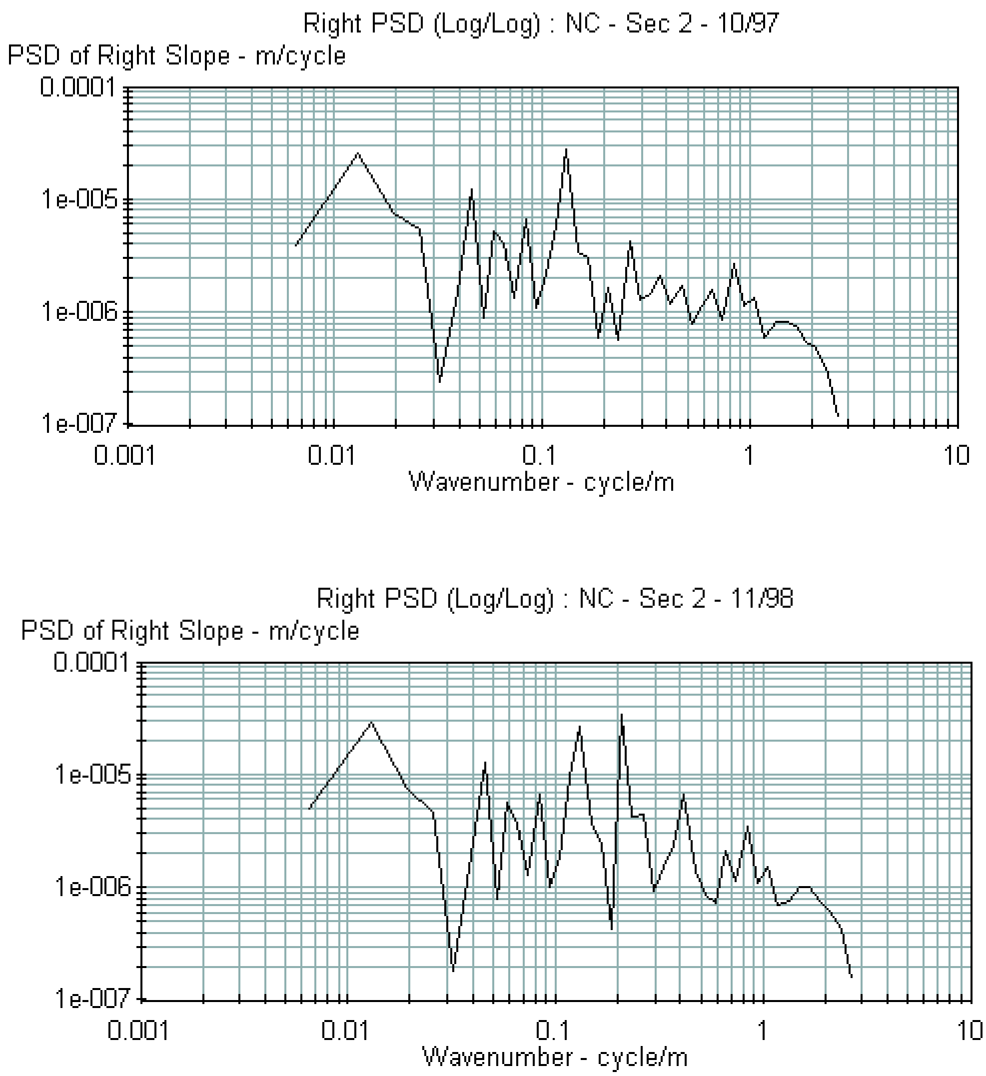


Figure 28. PSD plots – left wheel path profiles – section 2 – North Carolina.

IRI of 45 percent, was analyzed. This section has a 275 PCC slab, with a 150 mm LCB base, and a slab width of 4.27 m. The profile date, profile time and IRI values of test section 7 in the Nevada SPS-2 project are shown in table 25.

The IRI of this section at the first profile date was 1.0 m/km, but within 1.5 years it increased to 1.43 m/km at the third profile date, and remained close to this value at the last profile date. The band pass filtered right wheel path profile plots for the four profile dates are shown in figure 30. The slabs in this section are curled upwards. The profile plots show that the

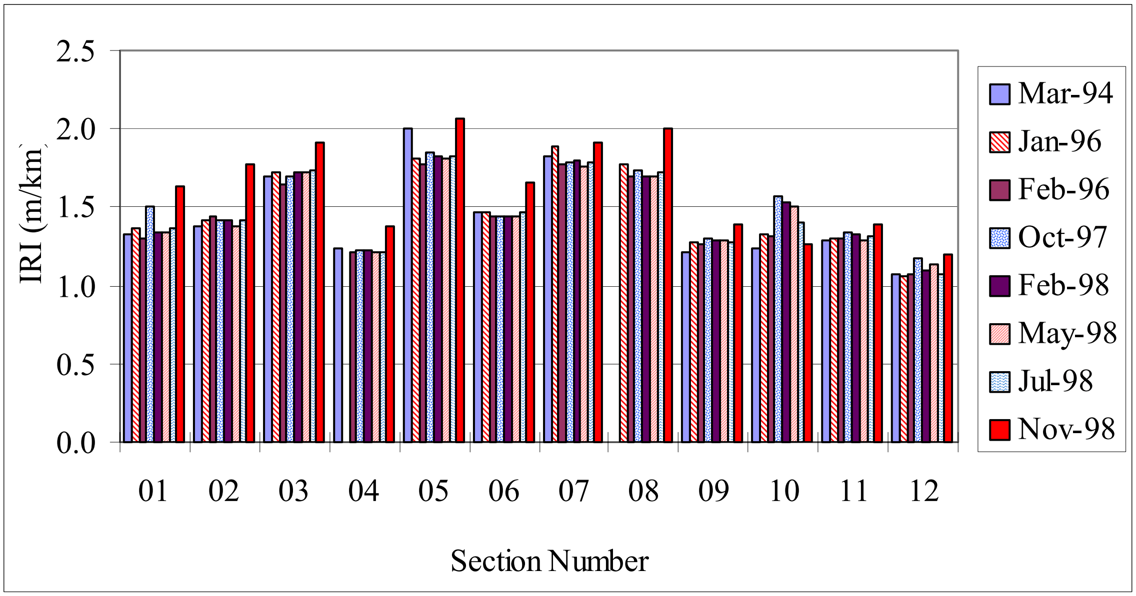


Figure 29. IRI values for North Carolina SPS-2 project.

Table 25. IRI values – Section 7- Nevada.

Profile Date	Profile Time	IRI (m/km)
6/28/96	1:01 PM	1.00
4/22/97	9:51 AM	1.12
11/18/97	2:47 PM	1.43
8/28/98	10:14 AM	1.44

slab curvature for the third and fourth profile dates are much more pronounced than the curvatures for the first and second profile dates. The increase in roughness at this section is attributed to the increase in curvature of the slabs. The other sections in this project that are showing a large increase in IRI also show a similar increase in slab curvature.

The construction report for this project indicated that construction difficulties were encountered during the construction of this project as the contractor had difficulties in working with the PCC mixes which were different than the mixes used in Nevada (15). Extensive transverse cracking had occurred throughout this project at a very early age.

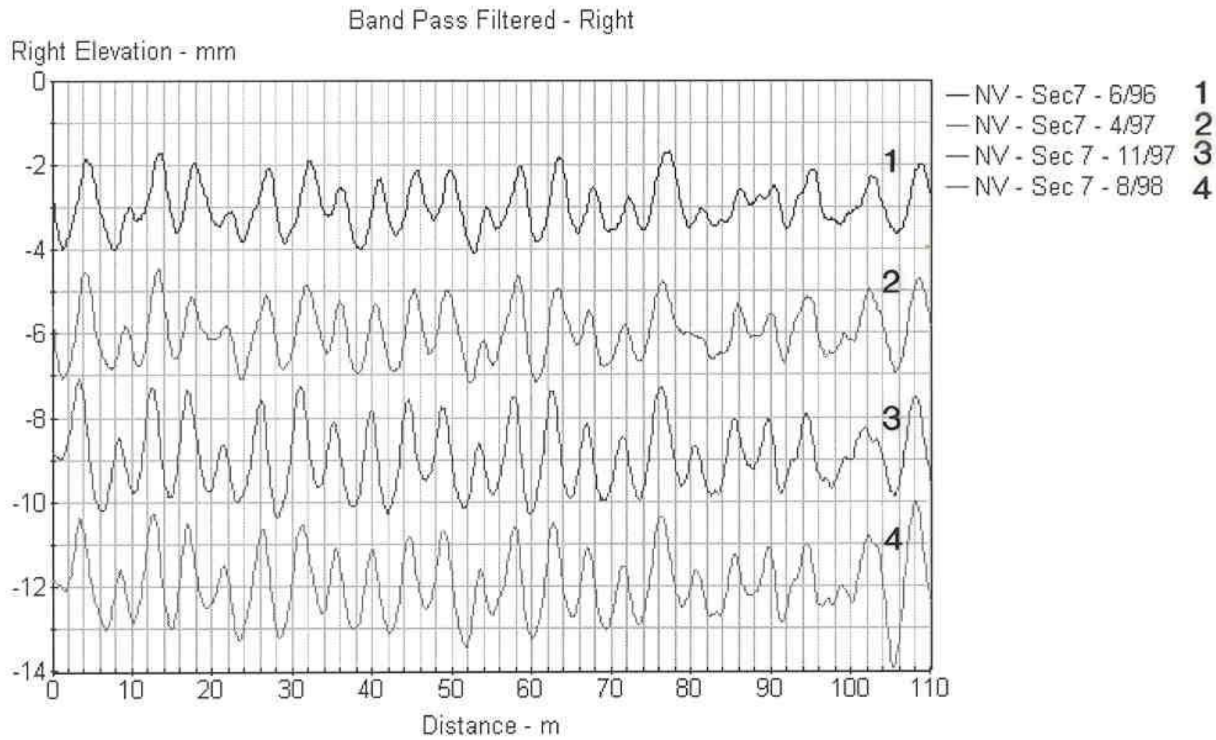


Figure 30. Band pass filtered profile plots – section 7 – Nevada.

Investigate Cause of High Variability in Roughness Over the Years

Section 14 in the Michigan SPS-2 project has shown highly variable IRI values over the years. This section has 200 mm PCC slab that is on a DGAB base, and has a 3.66 m slab width. The profile date, profile time and IRI values of this section are shown in table 26.

The IRI of this section at the first profile date in 1994 was 1.84 m/km, while the IRI at the last profile date in 1999 was 2.06 m/km. But between these years, the IRI of this section has ranged from a low of 1.22 m/km to a high of 2.71 m/km. Figure 31 shows the right wheel path band pass filtered profiles for 1994 (IRI = 1.84 m/km), 1997 (1.62 m/km), and 1998 (2.71 m/km). The variations in curvature for these three test dates are clearly seen in the plot, with increasing curvature corresponding to increasing roughness. The lowest IRI for this section

Table 26. IRI values – section 14 – Michigan.

Profile Date	Profile Time	IRI (m/km)
9/6/94	11:30 AM	1.84
8/11/95	9:35 AM	1.22
1/8/96	4:34 PM	2.17
4/9/96	10:11 AM	2.10
12/29/96	10:28 AM	2.03
4/15/97	1:05 PM	1.60
7/1/97	9:23 AM	1.62
11/5/98	4:35 PM	2.71
4/12/99	12:15 PM	2.06

occurred in 1995, with that profile being obtained at 9:35 AM. The IRI values shown in table 26 show that the IRI is not being influenced by early morning temperature effects. The curling effects seen at this section may be related to moisture variations within the PCC slab.

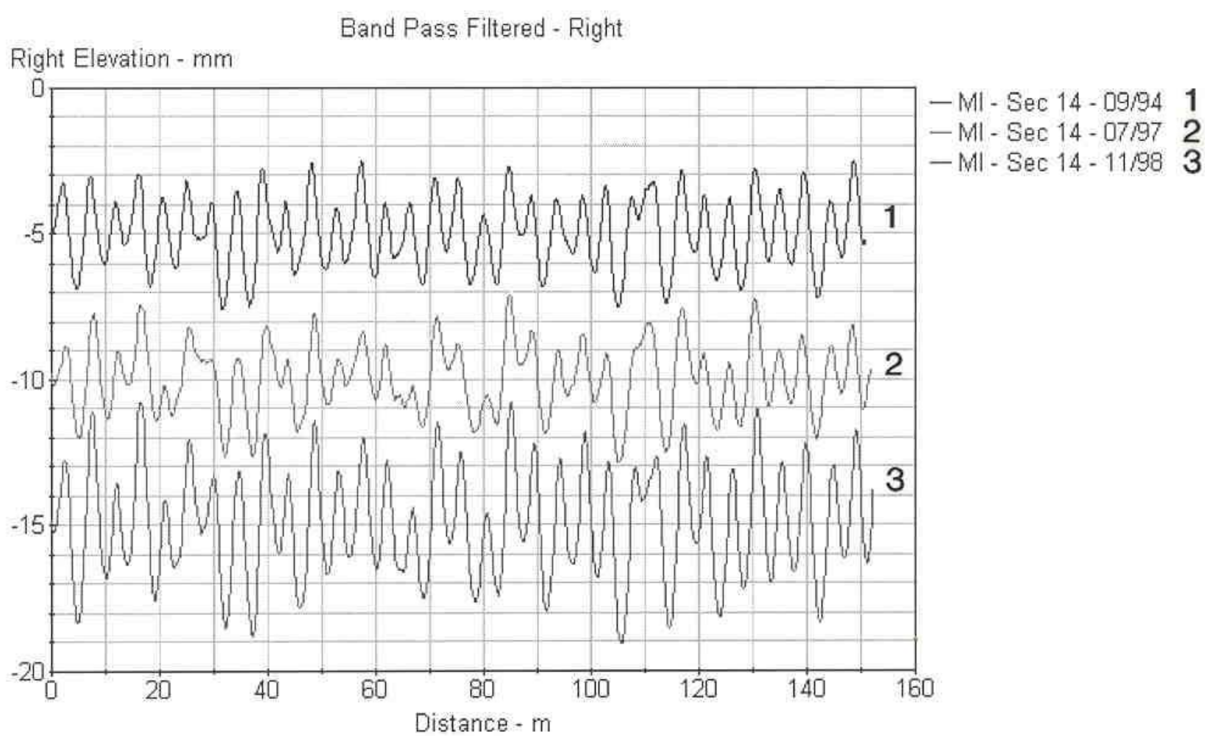


Figure 31. Band pass filtered profile plots – section 14 – Michigan.

Relationship Between Changes in IRI and Distress

Byrum (21) has shown that distresses in PCC slabs such as faulting is related to slab curvature. The sections that show high curvature or high variations in curvature are expected to show pavement distress early in the life. The distress data in the LTPP database was reviewed to see if the sections showing a high increase in IRI or a high reduction in IRI had higher distress when compared to the other sections. However, a clear relationship between IRI changes and pavement distress was not found. The only exception was the project in Nevada, where transverse cracking was noted at most sections in the Nevada project.

Summary of Findings

The data from the SPS-2 projects indicated the average early-age IRI of the 200 mm thick PCC pavements to be 1.27 m/km, with the standard deviation of IRI being 0.28 m/km. For the 275 mm thick PCC pavements, the average IRI was 1.30 m/km, with the standard deviation of IRI being 0.30 m/km. A statistical test on the early-age IRI considering the two PCC thicknesses of 200 mm and 275 mm indicated that PCC thickness was not significant. The PCC surface in the SPS-2 projects have been placed on three different base types: DGAB, LCB and PATB. The average early-age IRI values for PCC pavements placed on DGAB, LCB, and PATB were 1.27 m/km, 1.40 m/km, and 1.25 m/km, respectively. The highest early-age IRI was obtained for PCC surfaces placed on LCB.

An evaluation of the changes in roughness that had occurred over the monitored period at the SPS-2 sections indicated several distinct patterns: (1) some sections in some of the projects showed high increases in roughness, (2) some sections in some projects showed a reduction in roughness, (3) most of the sections in the Nevada project showed very high increases in roughness, (4) most of the sections in the North Carolina project showed an increase in roughness in excess of 10 percent, (5) Some sections in some projects showed very high variability in roughness between the years. An investigation was carried out using the profile data to determine if the cause for the changes in roughness that have occurred for the cases described previously could be found. In all of the investigated cases, it was found that the changes in roughness that had occurred could be related to changes in curvature of the PCC slabs.

Some of the sections that have shown a high increase in roughness are: (1) section 13 in the Arizona project that showed an increase in IRI of 21 percent over a 5 year period, (2) section 13 in the Michigan project that showed an increase in IRI of 95 percent over a 4.5 year period, and section 6 in Washington that showed an increase in roughness of 52 percent over a 3.5 year period. The cause for the increase in roughness of the sections in the Arizona and Michigan projects was the increase in upward slab curvature over time. For the section in the Washington project, the increase in roughness was caused by the increase in downward slab curvature over time. These changes were not due to temperature variations, and may have been caused by changes in moisture conditions within the PCC slab over time.

Some of the sections that have shown a decrease in roughness are: (1) section 18 in Arizona that showed a decrease in IRI of 26 percent, (2) Section 6 in Delaware that showed a decrease in IRI of 26 percent, (3) Section 6 in Kansas that showed a decrease in IRI of 23 percent. The cause for the decrease in IRI for the project in Arizona was because the section was first profiled early in the morning, when temperature related curling was present in the slab. For the sections in Delaware and Kansas, the reduction in roughness was not due to a temperature effect, but caused by a decrease in slab curvature over time, which may have been caused by variations in the moisture conditions in the PCC slab over time.

Most of the sections in the project in North Carolina showed a sudden increase in roughness at the last profile date, with some sections showing an increase as much as 25 percent. The cause for this increase in roughness was because the section was last profiled early in the morning when slab curling due to temperature was present, and this resulted in an increase in roughness. Most of the sections in the Nevada project showed very high increases in roughness with 5 sections showing an increase in IRI of over 40 percent that occurred with a 2 year period. The cause for the increase in roughness of these sections was the increase in slab curvature that occurred over time. A few of the SPS-2 sections showed large variations in roughness over the monitored period. Section 14 in Michigan had an IRI of 1.84 m/km the first time it was profiled, and an IRI of 2.06 m/km after five years. In between these two dates, the section had been profiled seven times, with the IRI ranging between 1.22 m/km and 2.71 m/km during this period. The variations in IRI at this section were caused by changes in slab curvature, which was not caused by temperature affects.

In the Nevada project, large changes in IRI occurred at both 200 mm and 275 mm thick PCC slabs. However, in other project large changes in roughness generally occurred on sections that had 200 mm thick PCC slabs. Test section 1 in all projects showed an increase in IRI of over 10 percent. Test section 1 has a 200 mm thick PCC slab that had a 14-day flexural strength target of 3.8 Mpa, and rests on a DGAB base. Section 1 in all projects showed an increase in curvature over the years, and the cause for the increase in IRI is attributed to the increase in curvature. It appears that this particular pavement section is more susceptible to changes in curvature than the rest of the pavement sections in the SPS-2 experiment.

CHAPTER 6

ROUGHNESS CHARACTERISTICS OF SPS-5 AND SPS-6 PROJECTS

SPS-5 EXPERIMENT: REHABILITATION OF ASPHALT CONCRETE PAVEMENTS

Introduction

The specific pavement studies SPS-5 experiment was developed to investigate the performance of selected AC rehabilitation treatment factors. The rehabilitation treatment factors include overlay mix type (recycled and virgin), overlay thickness (50 mm and 125 mm), and surface preparation of the existing AC surface prior to overlay (minimal and intensive preparation). Nine test sections are included in each SPS-5 project, with eight sections being experimental sections and one section being the control section. The overlay thickness, type of AC used for the overlay (virgin or recycled) and the type of surface preparations that is carried out on the test sections prior to placing the AC overlay are shown in table 27.

Table 27. Treatments applied to SPS-5 test sections.

Section Number	Surface Preparation	Type of AC	Overlay Thickness (mm)
1	Routine Maintenance	-	0
2	Minimum surface preparation	Recycled	50
3	Minimum surface preparation	Recycled	125
4	Minimum surface preparation	Virgin	125
5	Minimum surface preparation	Virgin	50
6	Intensive surface preparation	Virgin	50
7	Intensive surface preparation	Virgin	125
8	Intensive surface preparation	Recycled	125
9	Intensive surface preparation	Recycled	50

Table 28 presents a description of the types of surface preparation activities that were carried out at the sections prior to placing the AC overlay. Section 1 is designated as a control section, which receives only limited routine-type maintenance. Repair activities on the control

Table 28. Surface preparation activities for SPS-5 test sections.

Test Section Details	Surface Preparation								
Treatment Options		Minimal				Intense			
Section Number	1	2	3	4	5	6	7	8	9
Overlay Thickness (mm)	0	50	125	125	50	50	125	125	50
Overlay Material	-	R	R	V	V	V	V	R	R
Patching	X	X	X	X	X	P	P	P	P
Crack Sealing	X	-	-	-	-	P	P	P	P
Leveling	-	A	A	A	A	-	-	-	-
Milling	-	F	F	F	F	X	X	X	X
Seal Coat	B	-	-	-	-	-	-	-	-

R - Recycled Hot Mixed Asphalt Concrete
V - Virgin Hot Mixed Asphalt Concrete
X - Perform
A - If ruts are > 12 mm
B - Not permitted in first year of study
P - Perform after milling as required
F - Milling permitted only to remove open graded friction courses

section are limited to those maintenance activities needed to keep the section in a safe and functional condition. Repair activities on this section were carried out according to the guidelines of the State highway agency. The minimal level of surface preparation applies to test sections 2 through 5, and consists primarily of patching of severely distressed areas and potholes and placement of a leveling course in ruts that are greater than 12 mm. The intensive level of preparation applies to test sections 6 through 9, and includes milling of the existing AC surface, patching of distressed areas, and crack sealing after milling. Milling of the surface is the primary difference between the minimal and intensive preparation levels in this experiment. Milling was performed in the intensive surface preparation sections to a depth of 38 to 50 mm, and the depth of material removed by milling was replaced with an equal thickness of AC overlay material. This material is a virgin mix on test sections 6 and 7 and a recycled mix on test sections 8 and 9. The depth of replacement material is not counted as a part of the overlay thickness specified in the experiment. The recycled AC that is used consisted of 30 percent recycled asphalt mix.

Analyzed Projects

A review of the IMS database indicated that profile data were available for seventeen SPS-5 projects. Table 29 presents the following information for each SPS-5 project: state located, climatic zone, subgrade type, if pre-rehabilitation IRI and distress data are available for the project, rehabilitation date, age of project at first profile date, age of project at last available profile date, number of times the project has been profiled after rehabilitation, pre-rehabilitation IRI of the project, and the annual ESALs at the site. The pre-rehabilitation IRI of the project was computed by averaging the pre-rehabilitation IRI of all test sections in the SPS-5 project.

Figure 32 shows the pre-rehabilitation IRI of the SPS-5 projects. The pre-rehabilitation IRI was computed by averaging the pre-rehabilitation IRI of all test sections in a SPS-5 project. Out of the fifteen projects for which pre-rehabilitation IRI values were available, 53 percent of the projects had an IRI over 1.5 m/km, while 47 percent of the projects had an IRI of less than 1.5 m/km. Considering that an IRI value of 1.5 m/km corresponds to a present serviceability rating of 3.4 (22), 47 percent of the projects were in a fairly good condition from a roughness point of view when rehabilitation was performed. In fact the projects in Alabama, Florida, Georgia and Maine had project IRI values between 1.0 and 1.2 m/km, which are very low IRI values.

Figure 33 presents the pre-rehabilitation standard deviation of IRI of the test sections that are contained in each SPS-5 project. There were large differences in the variability of IRI values between the test sections for the different projects. The standard deviation of IRI between the projects ranged from a low of 0.11 m/km (Georgia) to a high of 0.56 m/km (Colorado).

Table 30 presents the average distress per section at the SPS-5 projects prior to rehabilitation for the following distress types: fatigue cracking, block cracking, longitudinal cracking, transverse cracking and patching. The average distress per section for a specific distress type in a project was computed by averaging the distresses present in all test sections. For each distress type, all severity levels were combined in computing the average. Table 30 also

Table 29. SPS-5 projects.

State	State Code	Climatic Zone (Note 1)	Subgrade Type	Availability of Pre-Rehabilitation Data		Rehab. Date	Age of Project After Rehabilitation at First Profile Date (yr)	Age of Project at Last Profile Date (yr)	Number of Times Profiled After Rehabilitation	Pre-Rehab. Project IRI (m/km)	Traffic KESAL (per year)
				IRI	Distress						
Alabama	AL	WNF	Coarse	Yes	Yes	12/19/91	0.3	4.3	3	1.2	N/A
Alberta	AB	DF	Coarse	Yes	Yes	10/3/90	0.0	8.6	9	1.9	N/A
Arizona	AZ	DNF	Coarse	Yes	Yes	4/20/90	0.4	8.6	6	1.9	206
California	CA	DNF	Coarse	Yes	Yes	4/25/92	0.8	6.9	5	2.1	1591
Colorado	CO	DF	Fine	Yes	Yes	10/3/91	0.1	7.8	8	1.9	438
Florida	FL	WNF	Coarse	Yes	Yes	4/5/95	0.6	2.4	2	1.2	N/A
Georgia	GA	WNF	Coarse	Yes	No	6/7/93	2.9	5.9	2	1.0	N/A
Maine	ME	WF	Coarse	Yes	Yes	6/20/95	2.2	3.0	2	1.2	N/A
Manitoba	MB	DF	Fine	No	Yes	9/1/89	0.1	9.9	9	N/A	N/A
Maryland	MD	WF	Fine	Yes	Yes	4/1/92	0.2	6.4	6	1.6	N/A
Minnesota	MN	WF	Fine	Yes	Yes	9/15/90	0.8	8.0	6	2.8	57
Mississippi	MS	WNF	Fine	Yes	No	9/24/90	0.1	8.6	5	2.3	676
Montana	MT	DF	Coarse	Yes	Yes	9/11/91	0.2	7.7	9	1.4	N/A
New Jersey	NJ	WF	Coarse	Yes	Yes	8/18/92	0.2	6.0	5	1.9	347
New Mexico	NM	DNF	Coarse	No	No	9/11/96	0.5	0.5	1	N/A	N/A
Oklahoma	OK	WNF	Fine	Yes	No	7/8/97	0.5	0.5	1	1.9	N/A
Texas	TX	WNF	Fine	Yes	No	9/1/91	0.4	5.8	4	1.5	N/A

Note 1: DF - Dry Freeze, DNF - Dry No-Freeze, WF - Wet Freeze, WNF - Wet No-Freeze

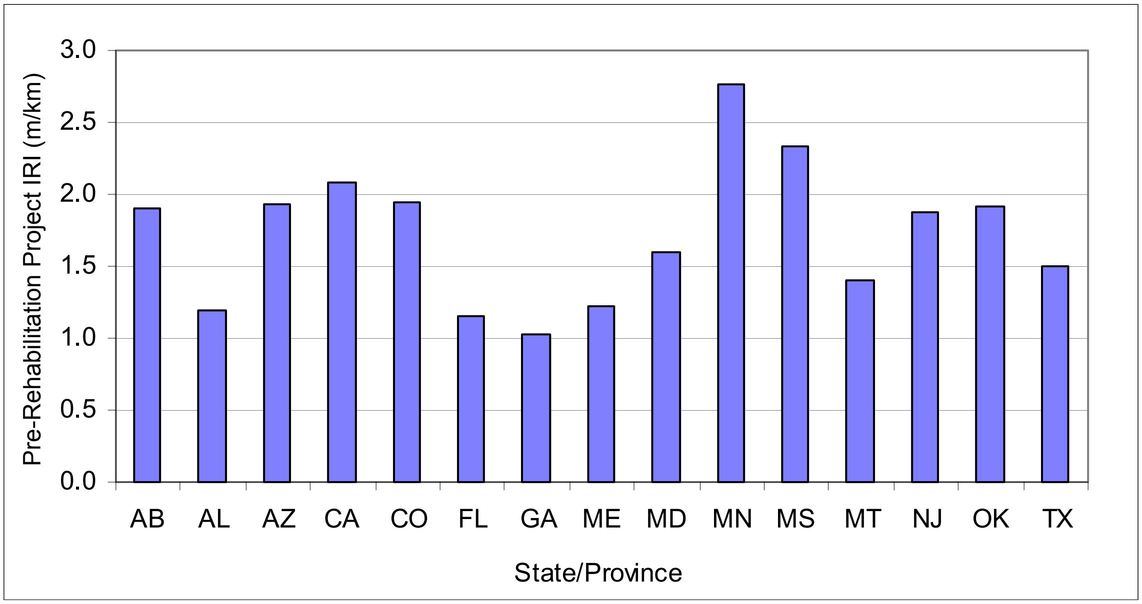


Figure 32. Pre-rehabilitation project IRI of SPS-5 projects.

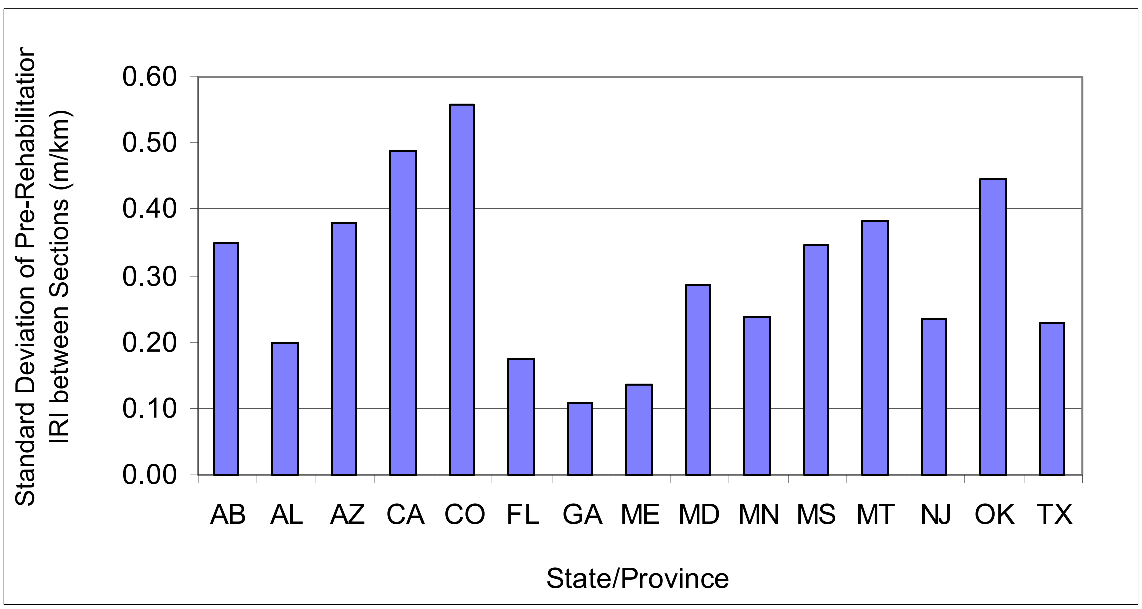


Figure 33. Standard deviation of pre-rehabilitation IRI of test sections in SPS-5 projects.

presents the average pre-rehabilitation IRI for each project. The project in Florida has a very low IRI, but is exhibiting a significant amount of distress. Table 31 presents the standard deviation of distress for the test sections in each project for fatigue cracking, block cracking, longitudinal

Table 30. Average distress and pre-rehabilitation IRI for SPS-5 projects.

State	Average Value For a Section							
	Pre-Rehab. IRI (m/km)	Fatigue Cracking (m^2)	Block Cracking (m^2)	Longitudinal Cracking (m)	Transverse Cracks (No)	Transverse Cracks (m)	Patches (No)	Patches (m^2)
Alabama	1.2	24	0	0	0	0	3	1
Alberta	1.9	1	0	1	0	1	0	0
Arizona	1.9	74	0	142	130	277	0	0
California	2.1	37	0	87	70	117	0	0
Colorado	1.9	12	0	144	16	20	1	8
Florida	1.2	183	156	138	66	34	0	0
Georgia	1.0	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Maine	1.2	0	0	275	20	6	0	0
Manitoba	N/A	5	0	11	2	3	1	41
Maryland	1.6	73	5	51	11	33	1	3
Minnesota	2.8	0	0	152	37	129	0	0
Mississippi	2.3	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Montana	1.4	131	10	46	44	72	0	0
New Jersey	1.9	77	179	19	11	19	2	5
New Mexico	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Oklahoma	1.9	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Texas	1.5	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Note: N/A – Data not available.

cracking and transverse cracking. The data shows that there is variability in the amount of distresses present at the test sections in a specific project. The largest standard deviation in distress was observed for the projects in Florida and New Jersey. The large standard deviation in block cracking and fatigue cracking observed in these two projects may have been caused by inconsistent classification of these two distress types between the sections.

IRI After Rehabilitation

The average IRI of the SPS-5 projects after rehabilitation is presented in figure 34, while the standard deviation of IRI values for the test sections within a project is presented in figure 35. The average and the standard deviation values were computed using the IRI values for sections 2 through 9, all of which received an AC overlay. The post-rehabilitation project IRI ranged from

Table 31. Standard deviation of pre-rehabilitation distress for test sections in SPS-5 projects.

State	Average Pre-Rehab. IRI (m/km)	Standard Deviation of Distress				
		Fatigue Cracking (m^2)	Block Cracking (m^2)	Longitudinal Cracking (m)	Transverse Cracks (No)	Transverse Cracks (m)
Alabama	1.2	11	-	-	-	-
Alberta	1.9	4	-	1	1	1
Arizona	1.9	60	-	87	50	118
California	2.1	13	-	63	45	85
Colorado	1.9	16	-	17	10	13
Florida	1.2	207	208	235	114	59
Georgia	1.0	N/A	N/A	N/A	N/A	N/A
Maine	1.2	-	-	35	22	6
Manitoba	N/A	13	-	16	2	4
Maryland	1.6	48	11	43	3	9
Minnesota	2.8	-	-	9	7	28
Mississippi	2.3	N/A	N/A	N/A	N/A	N/A
Montana	1.4	51	28	20	23	34
New Jersey	1.9	150	119	25	6	12
New Mexico	N/A	N/A	N/A	N/A	N/A	N/A
Oklahoma	1.9	N/A	N/A	N/A	N/A	N/A
Texas	1.5	N/A	N/A	N/A	N/A	N/A

Note: N/A Data not available
- Distress type not present

0.49 m/km (New Mexico) to 1.50 m/km (Mississippi). The project in New Mexico had the lowest standard deviation in IRI (0.04 m/km) with the project in Manitoba having the highest standard deviation in IRI (0.26 m/km). The post-rehabilitation IRI values for each test section in the SPS-5 projects are shown in table 32.

Relationship Between IRI Before and After Rehabilitation

Figure 36 shows the relationship between IRI prior to overlay and after overlay for the test sections in the SPS-5 experiment, differentiated according to the overlay thickness. Data from fifteen SPS-5 projects are shown in this figure. As there are 8 test sections in each SPS-5 project that received an overlay, in figure 36 there are 60 data points each for 50 mm and 125

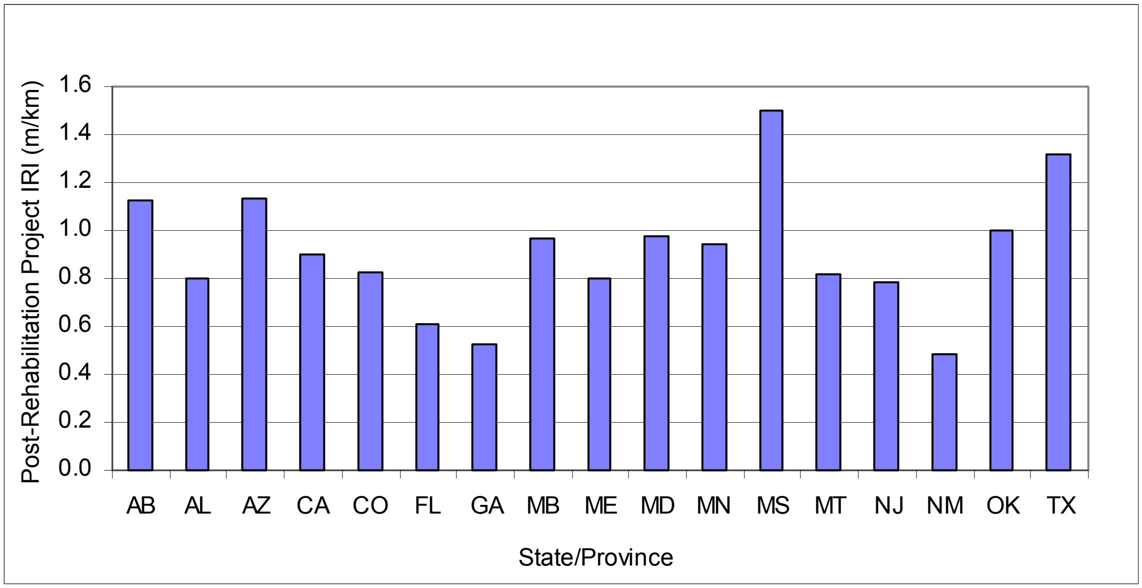


Figure 34. Post-rehabilitation project IRI for SPS-5 projects.

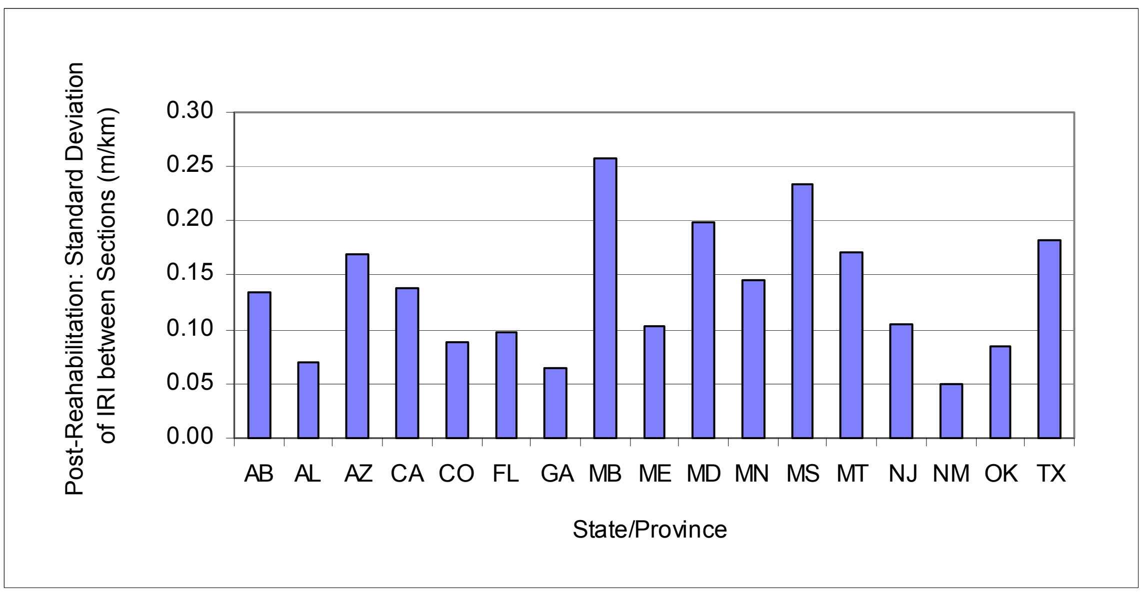


Figure 35. Post-rehabilitation standard deviation in IRI for SPS-5 projects.

mm overlays. As shown in this figure, there were several test sections that had low IRI values (i.e., close to 1.0 m/km) prior to rehabilitation. The IRI of these sections generally reduced by a small amount after rehabilitation. For sections that had an IRI of less than 1.5 m/km prior to overlay, the IRI after overlay was less than 1.0 m/km for approximately 80 percent of the

Table 32. Post-rehabilitation IRI values for test sections in SPS-5 projects.

State	Pre-Rehab IRI of Project (m/km)	IRI After Rehabilitation (m/km)								
		Test Section								Average (m/km)
		2	3	4	5	6	7	8	9	
Alabama	1.2	0.76	0.77	0.83	0.85	0.82	0.70	0.93	0.81	0.81
Alberta	1.9	1.04	1.05	1.29	1.14	1.06	1.38	1.04	1.01	1.12
Arizona	1.9	1.36	0.95	1.20	1.27	1.02	1.30	0.94	1.03	1.13
California	2.1	0.95	1.08	1.02	0.72	0.81	0.83	0.75	1.01	0.90
Colorado	1.9	0.94	0.78	0.83	0.78	0.93	0.70	0.78	0.91	0.83
Florida	1.2	0.68	0.74	0.64	0.49	0.50	0.55	0.72	0.57	0.61
Georgia	1.0	0.52	0.53	0.48	0.56	0.47	0.47	0.66	0.52	0.53
Maine	1.2	1.39	0.68	0.88	0.84	0.67	0.79	0.82	0.75	0.85
Manitoba	N/A	1.20	0.79	0.79	1.08	1.45	0.69	0.81	0.99	0.97
Maryland	1.6	1.39	1.03	0.91	1.00	0.74	0.88	0.79	1.03	0.97
Minnesota	2.8	0.85	0.76	1.12	1.08	1.08	0.85	1.00	0.78	0.94
Mississippi	2.3	1.41	1.80	1.20	1.72	1.41	1.26	1.41	1.78	1.50
Montana	1.4	0.89	1.00	0.69	0.68	0.69	1.14	0.75	0.71	0.82
New Jersey	1.9	0.99	0.67	0.72	0.89	0.73	0.78	0.74	0.75	0.78
New Mexico	N/A	0.45	0.59	0.45	0.44	0.47	0.44	0.51	0.49	0.48
Oklahoma	1.9	1.32	1.14	1.01	1.07	1.02	1.00	0.88	0.94	1.05
Texas	1.5	1.23	1.11	1.54	1.36	1.52	1.45	1.06	1.24	1.32
Average		**1.02**	**0.91**	**0.92**	**0.94**	**0.91**	**0.89**	**0.86**	**0.90**	

projects. Figure 36 shows that the IRI after overlay for most projects that had an IRI prior to overlay of greater than 1.5 m/km fell within a relatively narrow band of between 0.8 and 1.2 m/km. The data shows that overlays that are 50 mm thick are capable of reducing the IRI by a large amount. For example, such overlays were capable of achieving an IRI of less than 1.0 m/km for sections that had an IRI of 2.5 m/km prior to overlay.

A statistical analysis was conducted to determine if there was a relationship between the pre-rehabilitation IRI and the IRI after rehabilitation. The effect of overlay thickness, milling and AC type were also investigated in this analysis. The statistical analysis was performed by fitting a linear model to predict the IRI after rehabilitation by considering the following factors as independent variables: State, pre-rehabilitation IRI, overlay thickness (two levels), milling (yes or no), and AC type (virgin or recycled). The interaction terms between pre-rehabilitation IRI, overlay thickness, milling and AC type were also considered in the model. The factors State,

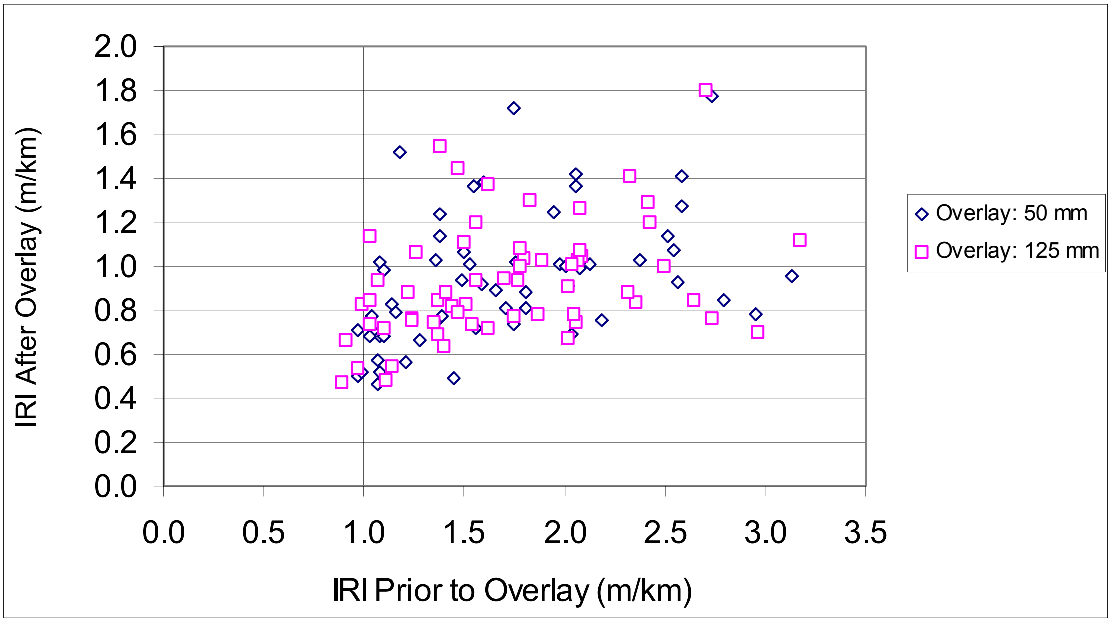


Figure 36. Relationship between IRI prior to overlay and IRI after overlay.

overlay thickness, milling and AC type are qualitative variables, while IRI after rehabilitation and pre-rehabilitation IRI are quantitative variables. The linear model function in S-plus software was used to fit the model. After fitting a model an ANOVA was performed and the main effects and the interactions were checked for significance. Only the factor State was significant, although there was evidence of a weak effect of milling (p-value = 0.07). The results from the statistical analysis indicated that the IRI after overlay did not depend on pre-rehabilitation IRI, overlay thickness, if milling was performed or not prior to overlay, or on the type of AC (virgin or recycled). These results are in agreement with the average IRI value for each section shown in table 32, where the average IRI of the sections are close to each other.

As many of the SPS-5 projects had pre-rehabilitation project IRI values that were less than 1.5 m/km, a similar analysis as described previously was performed to see if a different result would be obtained if only the projects that had an IRI greater than 1.5 m/km were considered. The projects considered for this analysis were: Alberta, Arizona, California, Colorado, Maryland, Minnesota, Mississippi, New Jersey and Oklahoma. This analysis indicated that factors State and milling were significant. Therefore, the analysis indicated for projects that have an IRI of greater than 1.5 m/km, milling of the surface prior to overlay does result in a smoother pavement.

Figure 37 shows the relationship between IRI prior to overlay and IRI after overlay for sections that received a 50 mm overlay, with data points differentiated between sections that did and did not receive milling prior to overlay. Figure 38 shows a similar plot for sections that received a 125 mm overlay. For sections that had an IRI of greater than 1.5 m/km, the sections with milling prior to overlay generally had a lower IRI than sections that were not milled, which confirms the results of the statistical analysis.

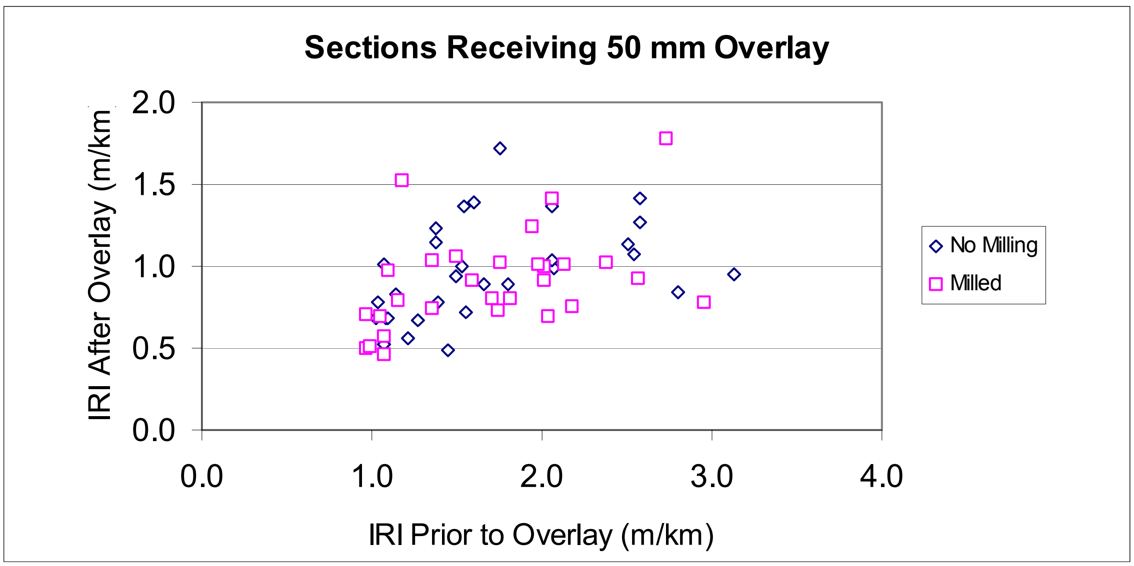


Figure 37. Relationship between IRI prior to and after overlay for 50 mm overlays.

The cumulative frequency distribution of IRI after rehabilitation for test sections in the projects that had an IRI greater than 1.5 m/km is presented in figure 39. This figure shows an IRI of less than 1.0 m/km was obtained for 50 percent of sections that received either a 50 mm or 125 mm overlay, but were not milled prior to overlay. For sections that were milled prior to overlay, 60 percent of sections with 50 mm overlays and 70 percent of sections with 125 mm overlays obtained IRI values that were less than 1.0 m/km.

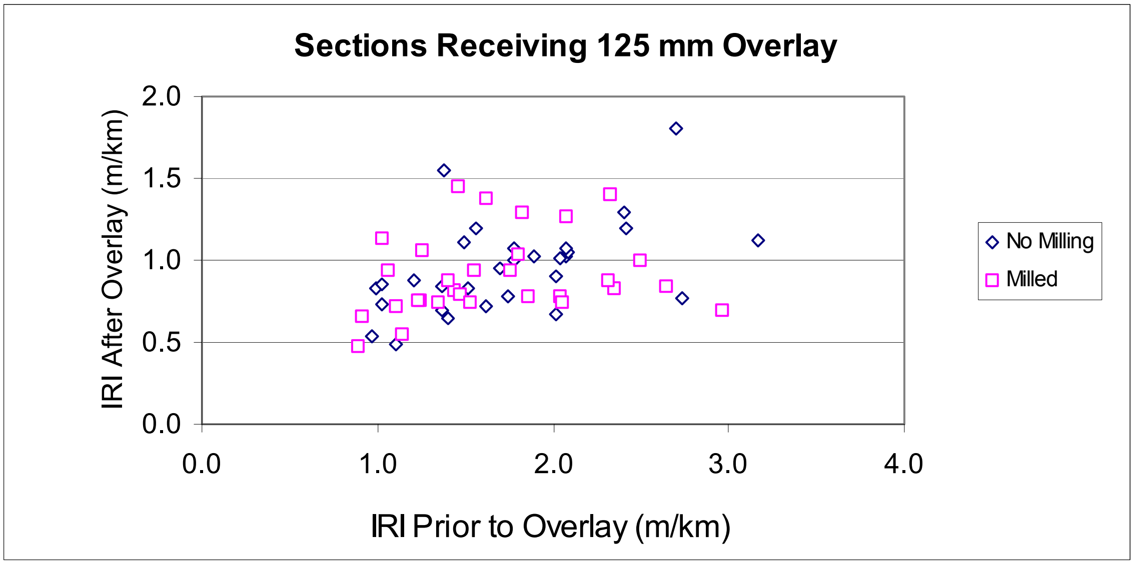


Figure 38. Relationship between IRI prior to and after overlay for 125 mm overlays.

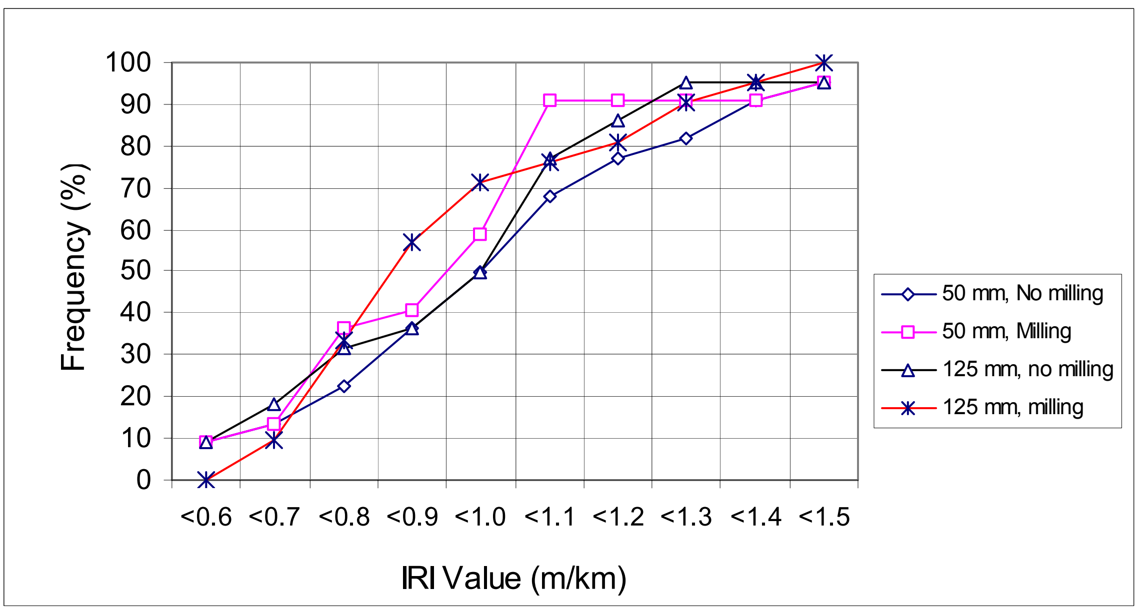


Figure 39. Cumulative frequency distribution of IRI after overlay – Projects with pre-rehabilitation IRI > 1.5 m/km.

The average IRI values obtained for several different scenarios are presented in table 33. When projects that have a pre-rehabilitation IRI of greater than 1.5 m/km are considered, the sections that received milling prior to overlay had an IRI that was 0.07 m/km less than the IRI obtained for projects that did not receive milling prior to overlay. Although statistically it was shown that milling does make a difference in IRI values for projects that have a pre-rehabilitation IRI of greater than 1.50 m/km, as shown in table 33 in terms of magnitude the difference in IRI values for the two cases is small.

Table 33. Average IRI values for different scenarios.

Case	Overlay Thickness (mm)	Milled Prior to Overlay ?	IRI After Overlay (m/km)	
			Average	Standard Deviation
All Projects	50	No	0.98	0.30
All Projects	50	Yes	0.91	0.31
All Projects	125	No	0.91	0.29
All Projects	125	Yes	0.88	0.26
Pre-Rehabilitation IRI > 1.5 m/km	50	No	1.11	0.25
Pre-Rehabilitation IRI > 1.5 m/km	50	Yes	1.04	0.27
Pre-Rehabilitation IRI > 1.5 m/km	125	No	1.06	0.27
Pre-Rehabilitation IRI > 1.5 m/km	125	Yes	0.99	0.24

The cause for the milled sections to have a lower IRI can be attributed to two reasons. Milling the surface prior to placing the surface provides a more uniform surface for paving, which will result in a lower IRI. Also, as the milled thickness is replaced in addition to placing the overlay, the number of lifts used in placing the AC thickness for milled sections may have been more when compared to non-milled sections.

The relationship between pre and post-overlay IRI values for test sections in three SPS-5 projects are shown in figure 40. The pre-overlay project IRI, which is the average IRI of the eight test sections in the project that received an overlay is shown on top of each graphs. The pre-overlay project IRI for the three projects shown in figure 40 is 1.8, 2.3 and 1.9 m/km. These figures also show that there appears to be no relationship between pre and post overlay IRI values. It can be seen that for a specific SPS-5 project, the IRI values for all the test sections in the project tend to fall within a relatively narrow band of IRI values, irrespective of the IRI value prior to overlay of the test sections. This observation was generally noted for all SPS-5 projects.

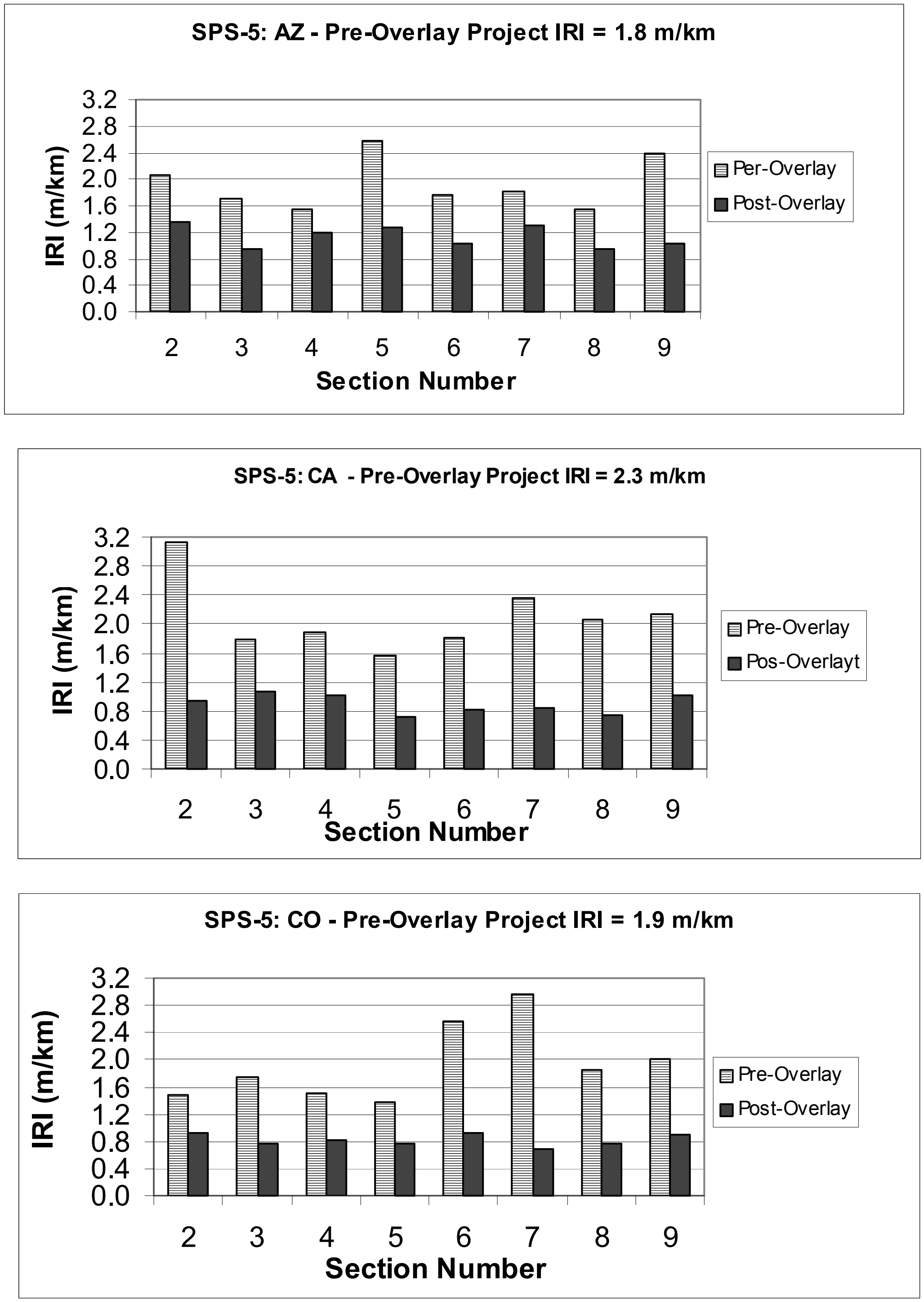


Figure 40. Relationship between pre and post overlay IRI values for three SPS-5 projects.

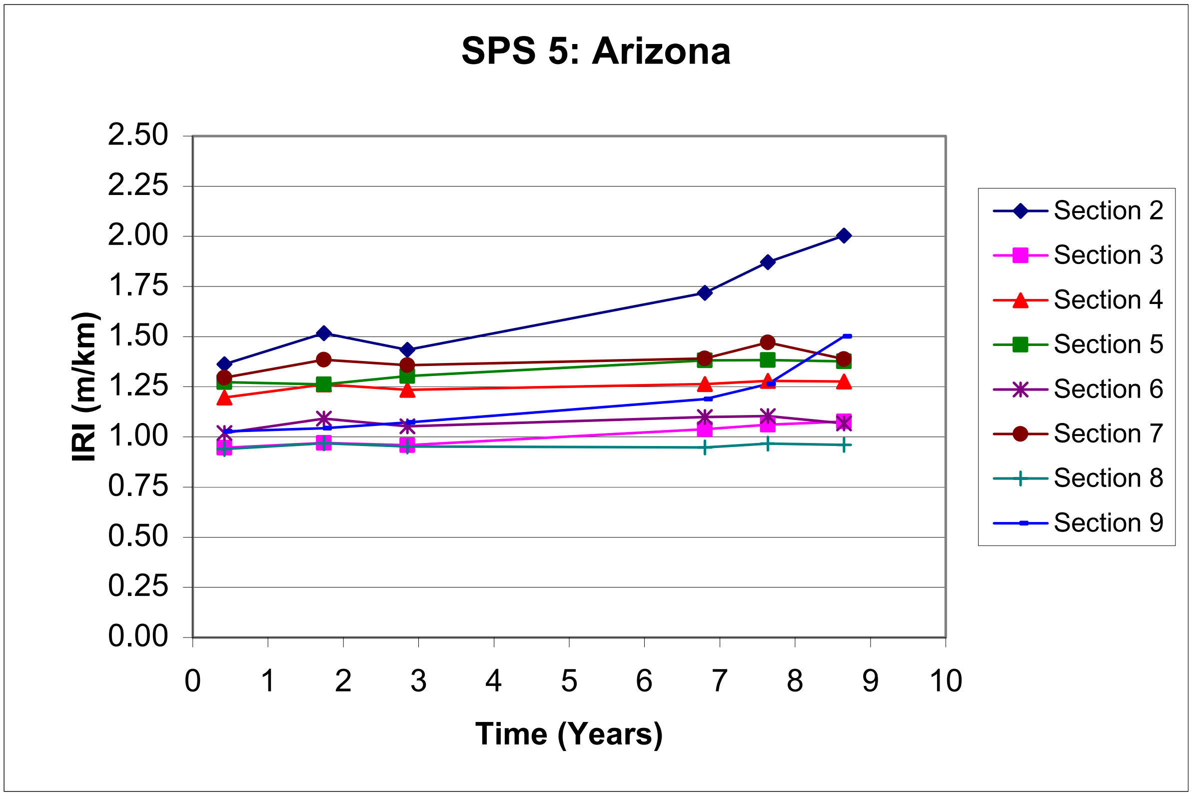
Changes in IRI for SPS-5 Projects

The changes in IRI over time for the SPS-5 projects in Arizona and Minnesota are shown in figure 41. Similar plots for all SPS-5 projects are included in Appendix C.

For SPS-5 projects that had at least three time-sequence IRI values, a linear regression was performed between IRI and time for each section to obtain the rate of development of roughness. Projects in Florida, Georgia and Maine had two time sequence IRI values after rehabilitation, but the time duration between these two profile dates was approximately 1, 2, and 3 years, respectively. Based on the review of IRI values, it was determined that a realistic rate of development of roughness could not be obtained from two time-sequence IRI values that were less than 2 years apart. The project in Georgia had two time-sequence IRI values that were approximately 3 years apart, and a rate of development of roughness was computed for sections in this project based on the two IRI values.

Figure 42 shows a box plot of the distribution of the rate of development of roughness at the test sections in the SPS-5 projects. The sections that received a 50 mm overlay without milling (sections 2 and 5) show a higher range between the first and third quartile ranges as well as for the overall range when compared to the two sections that received a 50 mm overlay after milling (sections 6 and 9). When compared to sections that received a 50 mm overlay, all sections that received a 125 mm overlay had lower ranges for rate of development of roughness between the first and third quartile, as well as a lower overall range.

The rate of development of IRI values that were computed for the test sections in each project were used to compute an average rate of development of roughness for each test section. That is for a specific test section, the rate of development of IRI obtained for that test section in all projects was averaged. The computed average rate of development of roughness values for the eight treatment types are shown in table 34.



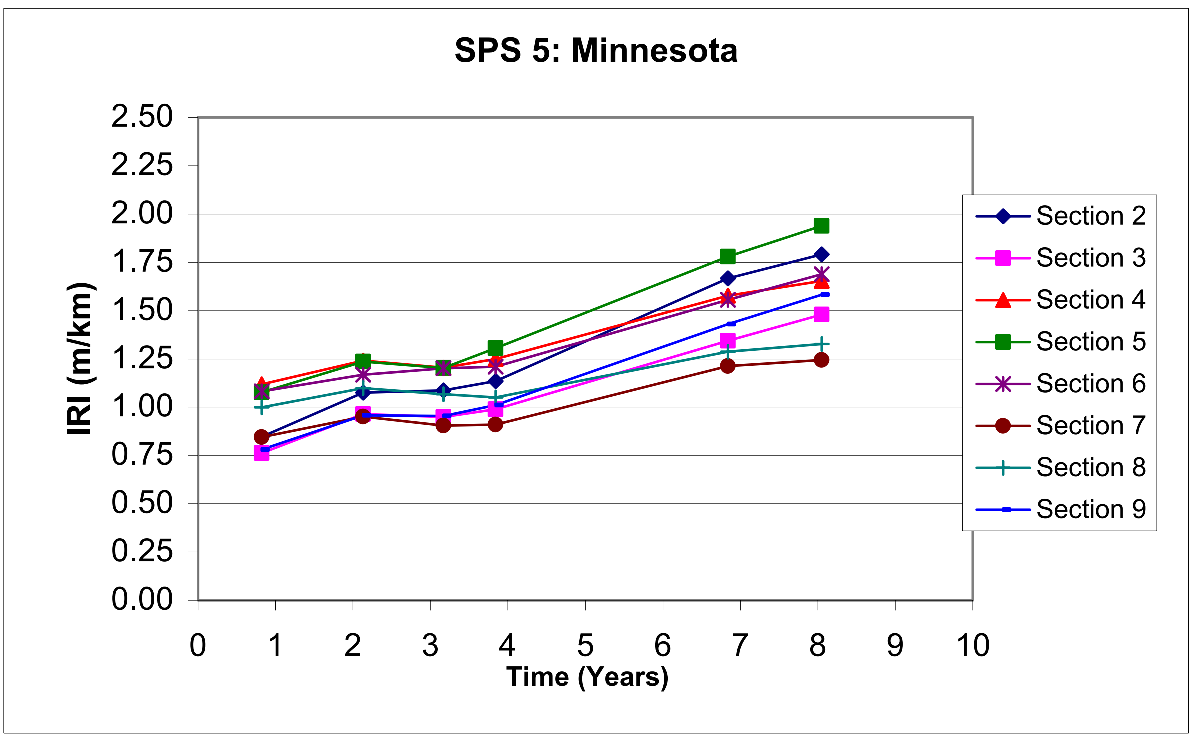


Figure 41. Change in IRI at SPS-5 projects in Arizona and Minnesota.

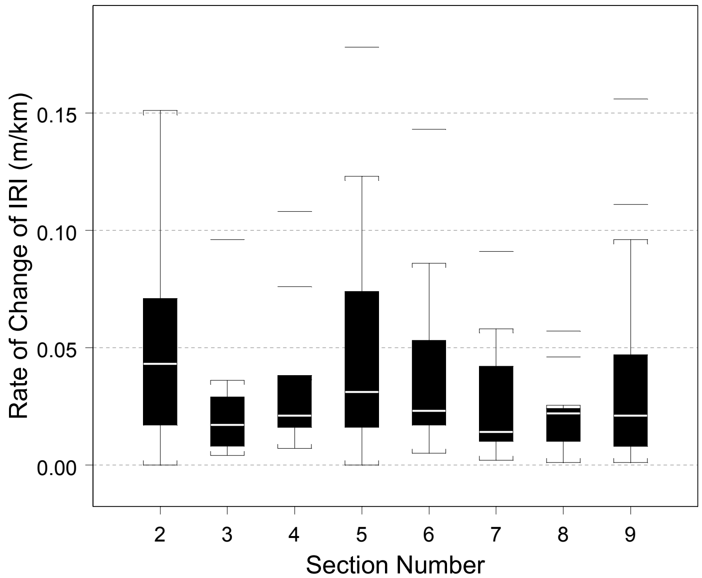


Figure 42. Box-plot of rate of development of IRI

Table 34. Average rate of development of IRI

Section Number	Surface Preparation	Type of AC	Overlay Thickness (mm)	Rate of Change of IRI (m/km)
2	Minimum	Recycled	50	0.049
3	Minimum	Recycled	125	0.023
4	Minimum	Virgin	125	0.032
5	Minimum	Virgin	50	0.051
6	Intensive	Virgin	50	0.039
7	Intensive	Virgin	125	0.028
8	Intensive	Recycled	125	0.022
9	Intensive	Recycled	50	0.044

The percent change in IRI for each test section in all SPS-5 projects is shown in table 35. The age of the projects shown in table 35 range from 2.4 to 9.9 years, with an average age of 7 years. Generally, projects that had higher pre-rehabilitation IRI show a higher increases in IRI. A statistical analysis was performed to investigate the factors that affect the increase of roughness

Table 35. Percent change in IRI at SPS-5 sections.

State	Pre-Overlay Project IRI (m/km)	Age of Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Percent Change in IRI (Note 1) Section Number								
					2	3	4	5	6	7	8	9	Average
Alabama	1.2	0.3	4.3	4.0	6	5	9	11	13	22	8	11	11
Alberta	1.9	0.0	8.6	8.6	39	25	24	10	1	4	21	42	21
Arizona	1.9	0.4	8.6	8.2	47	14	7	8	5	7	2	46	17
California	2.1	0.8	6.9	6.1	107	11	10	162	42	40	43	103	65
Colorado	1.9	0.1	7.8	7.7	32	7	10	8	118	53	10	17	32
Florida	1.2	0.6	2.4	1.8	0	3	1	11	-2	0	0	-5	1
Georgia	1.1	2.9	5.9	3.0	8	4	13	4	13	13	11	4	9
Maine	1.2	2.2	3.0	0.8	0	-1	-2	-1	-5	0	-3	-2	-2
Manitoba	N/A	0.1	9.9	9.8	36	40	44	59	13	39	20	35	36
Maryland	1.6	0.2	6.4	6.2	-23	13	68	41	23	14	7	17	20
Minnesota	2.8	0.8	8.0	7.2	111	94	48	80	56	47	33	103	72
Mississippi	2.3	0.1	8.6	8.5	20	-3	20	2	26	4	11	-7	9
Montana	1.4	0.2	7.7	7.5	72	18	22	77	45	2	23	99	45
New Jersey	1.9	0.2	6.0	5.8	9	8	11	0	15	8	4	7	8
Texas	1.5	0.4	5.8	5.4	9	14	1	10	4	-3	11	3	6
AVERAGE		0.6	6.7	6.0	32	17	19	32	24	17	13	31	23

Note 1: Percent Change in IRI = 100 X (IRI Last Profile Date - IRI First Profile Date)/(IRI at First Profile Date)
N/A - Data not available.

by considering the following factors: pre-overlay IRI, milling (yes or no), overlay thickness (two levels), AC type (virgin or recycled), and time.

This analysis was performed by fitting a model to the time-sequence IRI values for all sections, and testing for significance of factors. The fitted model was of the following form:

$Log(IRI_{tij})$ = $State_j$ + Log (Pre-Overlay IRI_{ij}) + Main effects and interaction with time for Milling, Overlay Thickness, AC Type appropriate for $Section_{ij}$ + Time * Log (Pre-Overlay IRI_{ij})

Where,

IRI_{tij} = IRI at time t for section i in State j

Pre-Overlay IRI_{ij} = Pre-Overlay IRI for section i in State j

$Section_{ij}$ = Section i in State j, where I ranges from 2 to 9

In this model Milling (Yes or No), Overlay thickness (50 mm and 125 mm) and AC type (Virgin or Recycled) are qualitative factors. The model was fitted using the mixed model function of S-plus, and then the main effects and interactions were tested for significance. The test indicated State, interaction of time and overlay thickness, and interaction of time and Pre-overlay IRI were significant. The analysis indicated that the factors that are related to the future roughness of a section are pre-overlay IRI of section, overlay thickness, and time.

Figures 43 and 44 show the relationship between the rate of change of IRI and the pre-rehabilitation IRI for sections with 50 mm and 125 mm thick overlays, respectively. These figures confirm the result of the statistical analysis, which is that sections that had higher pre-overlay IRI show a higher rate of development of roughness.

The IRI vs. time plots of the different test sections in each SPS-5 project were generally parallel to each other (see figures in Appendix C). When we consider each overlay thickness separately, for a specific SPS-5 project the sections that had a lower initial IRI show a trend of maintaining a lower IRI value over time when compared to sections that had a higher initial IRI

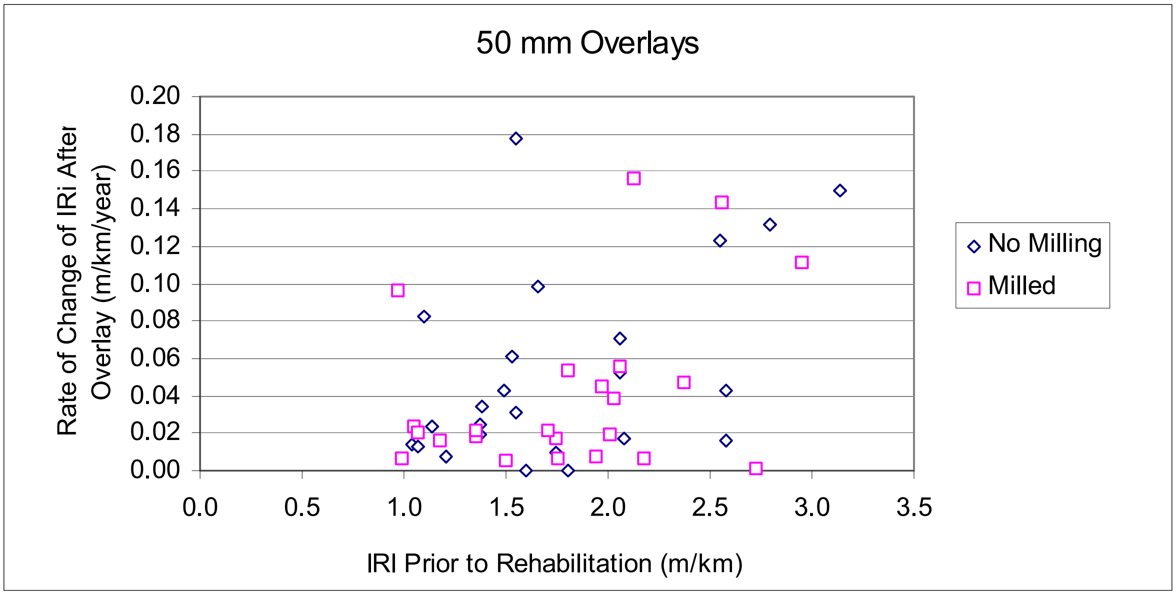


Figure 43. Rate of change of IRI vs. IRI prior to rehabilitation - 50 mm overlay sections.

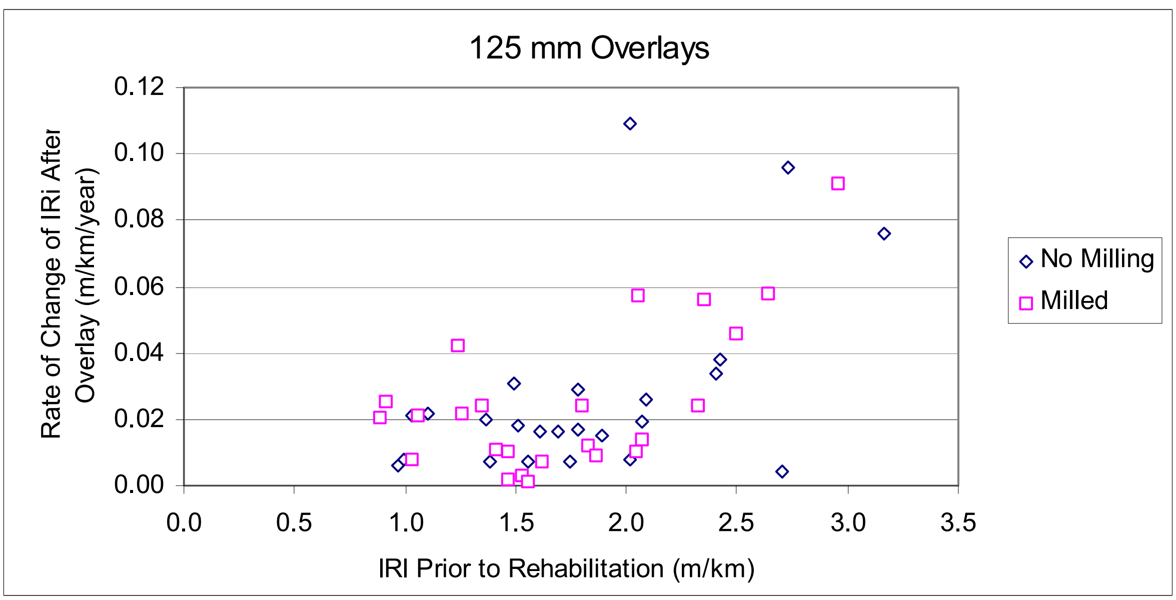


Figure 44. Rate of change of IRI vs. IRI prior to rehabilitation – 125 mm overlay sections.

value. The rate of change of IRI values shown in table 34 show the average value of the rate of change of IRI for sections that received a 125 mm overlay (sections 3, 4, 7, 8) were close to each other. Based on this data it can be seen that a section that has a lower initial IRI will maintain a lower IRI over time when compared to a section that had a higher initial IRI. These results are based on observations and analysis that were made on projects that had an average age of seven years. The milled and non-milled sections that received a 125 mm overlay do not show a difference in the rate of change of IRI yet. However, as the pavements age, these trends may change. For sections that received a 50 mm overlay, the milled sections show a lower rate of change of IRI when compared to non-milled sections. If a comparison of roughness values over time is made between a non-milled and a milled section, where the non-milled section has a lower initial IRI, at some point in time the roughness values of these two sections will become equal (because the rate of change of IRI of non-milled section is higher that that of the milled section).

The three projects that had the highest lengths for transverse cracking were Arizona, California and Minnesota (see table 30). The highest average increase in roughness for SPS-5 projects are seen for the California and Minnesota projects (see table 35). The Arizona site also had a high number of transverse cracks, but did not show a higher percent change in IRI. This data indicates there may be a relationship between changes in roughness after overlay and transverse cracking at a section prior to overlay, but factors such as environment, subgrade conditions and traffic may have interaction effects with transverse cracking on the development of roughness.

Traffic data as well as key material testing data are not available yet in the LTPP database for many SPS-5 projects. Also, the age of the SPS-5 projects vary from 2.4 to 9.9 years, and the younger projects are not yet showing much change in IRI. Because of these limitations, a comprehensive analysis of the data to build models to predict development of roughness cannot be carried out yet. Because of the non-availability of traffic data, the influence of traffic on changes in roughness cannot be studied. It was also seen that the pre-rehabilitation IRI as well as pavement distress prior to rehabilitation varied between the test sections in many projects. This introduces an additional confounding factor to the analysis. In spite of these limitations, the existing data does reveal trends in roughness development at the SPS-5 projects.

Summary of Findings

Pre-rehabilitation IRI values of test sections were available for 15 SPS-5 projects. The pre-rehabilitation project IRI, which is the average IRI of all the test sections in the project, was less than 1.5 m/km for 7 of the projects. Considering that an IRI of 1.5 m/km corresponds to a serviceability rating of 3.4 (22), 7 of the analyzed SPS-5 projects were in a fairly good condition from a roughness point of view when rehabilitation was performed.

For sections that had an IRI of less than 1.5 m/km prior to overlay, the IRI after overlay was less than 1.0 m/km for 80 percent of the sections. For sections that had an IRI of over 1.5 m/km prior to overlay, the IRI after overlay for most sections fell between 0.8 and 1.2 m/km. It was seen that 50 mm overlays were capable of achieving a large reduction in roughness, with some of the 50 mm overlays reducing the IRI of the pavement from 2.5 m/km to 1.0 m/km

An analysis of all the data for the SPS-5 projects indicated that the IRI after overlay did not depend on the pre-rehabilitation IRI, overlay thickness, if milling was performed or not prior to overlay, or the type of AC. An analysis of the data from the projects that had an IRI of greater than 1.5 m/km indicated that milling prior to placing an overlay results in a smoother pavement. For projects having an IRI greater than 1.5 m/km, milling prior to placing an overlay resulted in a pavement that on average had an IRI of less than 0.07 m/km when compared to a non-milled section. Generally, for each SPS-5 project, the IRI of all test sections in the project fell within a relatively narrow band of IRI values, irrespective of the IRI prior to overlay of the test sections.

A statistical analysis indicated that the progression of the roughness over time of the overlaid pavements depend on the pre-overlay IRI of the section and overlay thickness. The statistical analysis did not indicate milling prior to overlay or AC type as being significant factors that affect the progression of roughness of overlaid pavements.

When all projects were considered, the following average rate of increase of roughness were observed: 50 mm overlays with milling prior to overlay – 0.042 m/km/year, 50 mm overlays without milling prior to overlay – 0.050 m/km/year, 125 mm overlays with milling prior to overlay – 0.025 m/km/year, 125 mm overlays without milling prior to overlay – 0.028 m/km/year. It should be noted that these results are based on analysis that were made on projects that that had an average age of seven years. These results as well as observation of time-sequence data for the SPS-5 projects indicate that for a specific overlay thickness, a lower initial IRI results in a lower IRI over the service life of the pavement

SPS-6 EXPERIMENT: REHABILITATION OF JOINTED CONCRETE PAVEMENTS

Introduction

The SPS-6 experiment was developed to investigate the effect of different rehabilitation techniques performed on jointed concrete pavements. In this experiment, preparation and/or restoration of the existing pavement are classed into three levels: minimal, intensive, crack and seat or break and seat. The treatments minimal and intensive are applied with and without an AC overlay. The rehabilitation treatments applied to the test sections in the SPS-6 test sections are presented in table 36.

Table 36. Treatments applied to SPS-6 test sections.

Section Number	Surface Preparation	AC Overlay Thickness (mm)
1	Routine Maintenance	0
2	Minimum Restoration	0
3	Minimum Restoration	100
4	Minimum Restoration (saw and seal joints inn AC)	100
5	Intensive Restoration	0
6	Intensive Restoration	100
7	Crack/Break Seat	100
8	Crack/Break Seat	200
Note: In Section 4, after the placement of the AC overlay, the AC surface is sawed and sealed over the joints and working cracks of the PCC		

A detailed description of the surface preparation that is applied to the test sections is presented in table 37. Each SPS-6 project consists of seven test sections and a control section.

The control section designated as section 1 receives only maintenance activities that are needed to keep the section in a safe and functional condition in accordance with the standard procedure of the State agency where the project is located. The monitored portion of test sections 2 and 5 is 305 m, while that of the other sections is 152 m.

Table 37. Surface preparation activities for SPS-6 test sections.

	Surface Preparation							
Test Section Details and Treatment Options		Minimal			Intensive		Crack & Seat	
Section number	1	2	3	4	5	6	7	8
Section length (m)	152	305	152	152	305	152	152	152
Overlay thickness (mm)	0	0	100	100	0	100	100	200
Joint sealing	X	X	N	N	R&R	N	N	N
Crack sealing	X	X	N	N	R&R	N	N	N
Partial depth patch	N	X	X	X	R&R	R&R	N	N
Full depth patch/joint repair	N	X	X	X	R&R	R&R	N	N
Load transfer restoration	N	N	N	N	B	B	N	N
Full surface diamond grinding	N	X	N	N	A	N	N	N
Undersealing	N	N	N	N	X	X	N	N
Subdrainage	N	N	N	N	A	A	A	A
Crack/break and seat	N	N	N	N	N	N	A	A
Saw and seal	N	N	N	A	N	N	N	N

X - Apply treatment as warranted
R&R - Remove and replace existing and apply additional as warranted
N - Do not perform
B - Full depth doweled patch or retrofit dowels in slots.
A - Apply treatment regardless of condition or need.

Analyzed Projects

A review of the IMS database indicated profile data were available for ten SPS-6 projects. Table 38 presents the following information for each SPS-6 project: state located, climatic zone, subgrade type, if pre-rehabilitation IRI and distress data are available for the project, rehabilitation date, age of project at first profile date, age of project at last available profile date, number of times the project has been profiled after rehabilitation, pavement type (plain or reinforced), pre-rehabilitation IRI of the project, and the estimated annual ESALs at the

Table 38. SPS-6 projects.

State	State Code	Climatic Zone (Note 1)	Subgrade Type	Availability of Pre-Rehabilitation Data		Rehab. Date	Age of Pavement After Rehabilitation at First Profile Date (Yr)	Age of Project at Last Profile Date (Yr)	Number of Times profiled After Rehabilitation	Pavement Type	Pre-Rehab Project IRI (m/km)	Estimated Traffic KESAL (per year)
				IRI	Distress							
Arizona	AZ	DNF	Coarse	Yes	Yes	8/5/90	1.1	8.6	8	JPCP	1.9	1591
California	CA	WNF	Coarse	Yes	Yes	8/10/92	0.7	5.7	3	JPCP	3.2	N/A
Illinois	IL	WF	Fine	Yes	Yes	6/11/90	1.5	7.7	4	JRCP	2.3	723
Indiana	IN	WF	Fine	Yes	Yes	8/15/90	0.3	8.3	6	JPCP	1.8	317
Iowa	IA	WF	Fine	No	No	8/16/89	0.8	9.9	8	JRCP	N/A	490
Michigan	MI	WF	Fine	Yes	Yes	5/15/90	0.6	8.9	7	JRCP	2.1	360
Missouri	MO	WF	Fine	Yes	Yes	8/10/92	0.6	6.5	5	JRCP	2	N/A
Oklahoma	OK	WNF	Fine	Yes	No	8/27/92	0.6	6.8	3	JRCP	1.8	731
Pennsylvania	PA	WF	Fine	Yes	Yes	9/30/92	0.2	5.7	5	JRCP	2.5	N/A
South Dakota	SD	DF	Fine	Yes	No	9/25/92	1.1	6.6	5	JPCP	2.8	59

Note 1: DF - Dry Freeze, DNF - Dry No-Freeze, WF - Wet Freeze, WNF - Wet No-Freeze
N/A - Data not available

site. The pre-rehabilitation IRI of the project was computed by averaging the pre-rehabilitation IRI of the test sections in the SPS-6 project.

Figure 45 shows the pre-rehabilitation IRI for the nine SPS-6 projects for which pre-rehabilitation IRI values were available. The values shown in figure 45 were computed by averaging the pre-rehabilitation IRI of the test sections for each project. Six projects had a pre-rehabilitation IRI that was between 1.5 and 2.5 m/km, and three projects had IRI values exceeding 2.5 m/km.

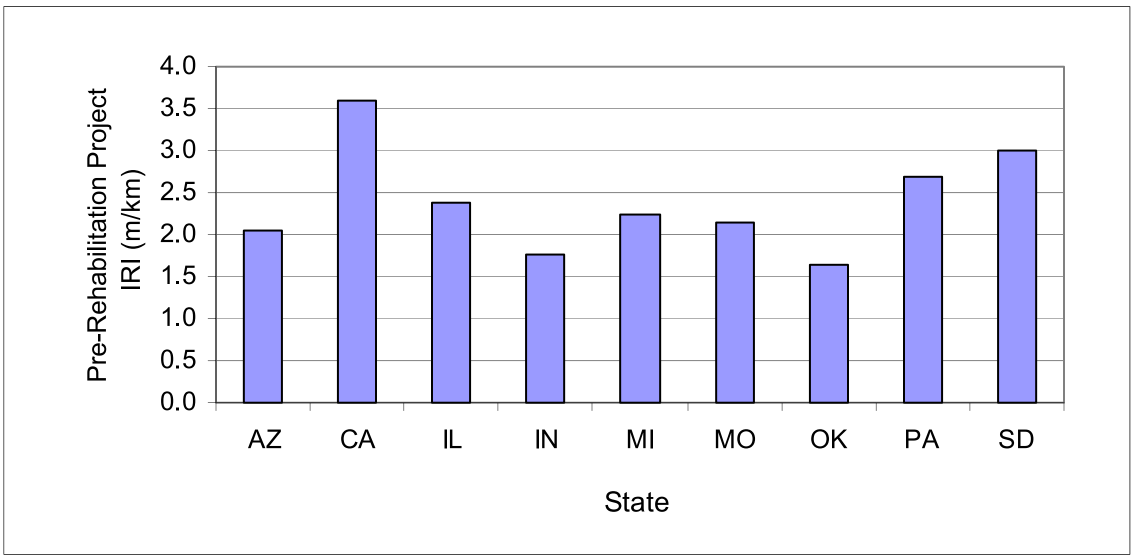


Figure 45. Pre-rehabilitation project IRI of SPS-6 projects.

Figure 46 presents the pre-rehabilitation standard deviation of IRI of the test sections that are contained in each SPS-6 project. There were large differences in the variability of IRI values between the test sections for the different projects. The sections in Indiana showed the lowest variability (standard deviation of IRI = 0.2 m/km), while the sections in Arizona showed the largest variability (standard deviation of IRI = 0.6 m/km).

Table 39 presents the average distress per section prior to rehabilitation for the seven projects for which pre-rehabilitation distress data were available. The average distress per section for a specific distress type in a SPS-6 project was computed by averaging the distresses (all severity levels) present in all test sections for that SPS-6 project. Table 39 also presents the pre-rehabilitation IRI for the project.

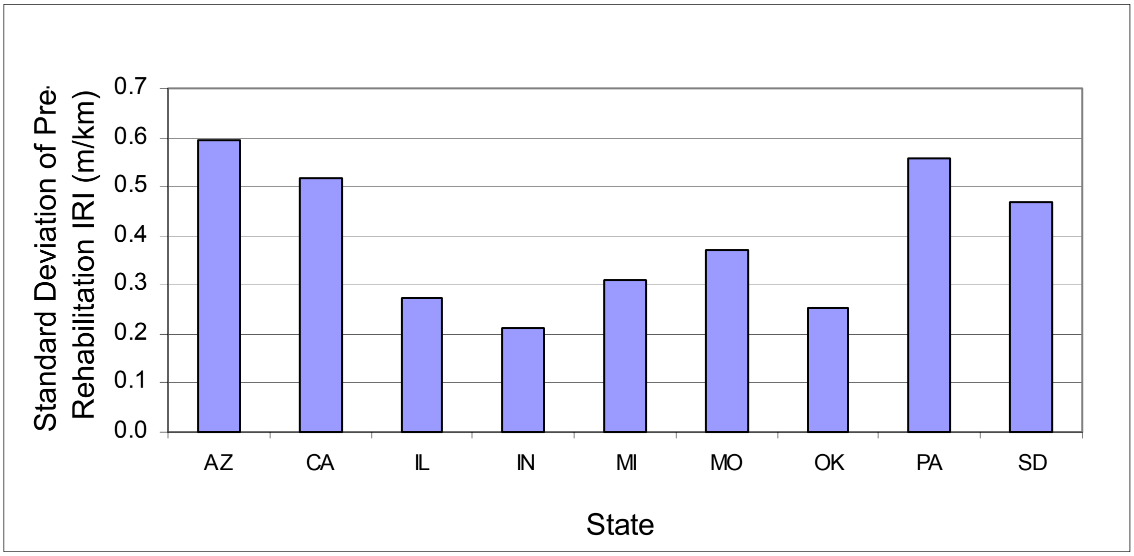


Figure 46. Standard deviation of pre-rehabilitation IRI of test sections in SPS-6 projects.

Table 39. Average distress per section and pre-rehabilitation IRI for SPS-6 projects.

Distress Type	Average Value per Section						
	State						
	AZ	CA	IL	IN	MI	MO	PA
Corner Breaks, Number	0	7	1	0	0	0	1
D. Cracking, Area (m^2)	0	0	0	0	0	6	0
Longitudinal Crackling, Length (m)	28	35	0	0	0	0	3
Transverse Cracks, Number	14	33	17	2	27	17	4
Transverse Cracks, Length (m)	46	89	56	9	112	45	13
Longitudinal Spalling, Length (m)	2	0	3	13	16	0	2
Transverse Spalls, Number	31	6	3	22	2	0	1
Transverse Spalls, Length (m)	98	2	2	52	6	0	1
Flexible Patches, Number	0	1	0	39	9	0	7
Flexible Patches, Area (m^2)	0	3	0	19	6	0	3
Rigid Patches, Number	0	0	1	0	3	3	1
Rigid Patches, Area (m^2)	0	0	13	0	28	45	10
Pre-Rehabilitation IRI (m/km)	1.9	3.2	2.3	1.8	2.1	2.0	2.5
Note: Pre-rehabilitation distress data not available for IA, OK and SD							

IRI After Rehabilitation

The post-rehabilitation IRI value for each test section in the SPS-6 projects is shown in table 40. Section 5 for projects in Michigan and Indiana were not diamond ground after repairs and the post-rehabilitation IRI values for these two sections are not shown in table 40.

Table 40. Post-rehabilitation IRI values for SPS-6 projects.

State	Pre-Rehab Average IRI of Project (m/km)	IRI After Rehabilitation (m/km)						
		Test Section						
		2	3	4	5	6	7	8
Arizona	1.9	3.5	0.9	0.9	1.5	1.0	0.8	0.9
California	3.2	1.4	0.9	0.8	1.1	0.9	1.0	0.9
Illinois	2.3	2.2	1.0	1.1	0.8	1.1	1.2	1.1
Indiana	1.8	3.6	0.9	0.9	N/A	0.9	1.0	0.9
Iowa	N/A	1.2	0.9	1.1	1.5	0.9	1.0	1.2
Michigan	2.1	2.1	1.3	1.2	N/A	0.9	1.1	0.9
Missouri	2.0	N/A	1.1	1.1	N/A	1.1	1.3	1.3
Oklahoma	1.8	1.1	0.7	0.9	0.8	0.9	1.1	1.3
Pennsylvania	2.5	2.1	1.1	1.1	1.4	1.1	1.0	1.0
South Dakota	2.8	1.0	1.1	1.3	0.9	1.0	1.0	0.8
Average (m/km)		**1.9**	**1.0**	**0.9**	**1.1**	**1.0**	**1.1**	**1.0**
Standard Deviation (m/km)		**1.0**	**0.2**	**0.4**	**0.3**	**0.1**	**0.1**	**0.2**
N/A - IRI values not available. For section 5 in Michigan and Indiana values are omitted because the sections were not diamond ground								

The pre- and post-rehabilitation IRI values for section 2 that received minimal surface preparation are shown in table 41. Some states diamond ground this section, while others did not (Table 37 indicates that the States were given the option of carrying out diamond grinding of this section.) As shown in table 41, the IRI value after rehabilitation for sections that were diamond ground ranged from 1.03 to 1.36 m/km. The post-rehabilitation IRI values for section 2 in Arizona and Indiana (that were not diamond ground) showed a large increase in IRI after repairs. The increase in IRI value for section 2 in Arizona and Indiana after repairs was 1.03 m/km and 2.00 m/km, respectively from the pre-rehabilitation IRI. The repairs performed on the Arizona section consisted of joint sealing, crack sealing and partial depth patches, while at the section in Indiana full depth patches were performed. These repair activities resulted in an increase in IRI.

Table 41. Pre- and post-rehabilitation IRI values for section 2.

State	IRI (m/km)		Diamond Ground ?
	Pre-Rehabilitation	Post-Rehabilitation	
Arizona	2.43	3.46	No
California	3.44	1.36	Yes
Illinois	2.05	2.17	No
Indiana	1.64	3.64	No
Iowa	N/A	1.22	Yes
Michigan	2.04	2.08	No
Missouri	1.94	1.09	Yes
Oklahoma	2.10	1.09	Yes
Pennsylvania	2.22	2.05	No
South Dakota	3.05	1.03	Yes
Note: N/A - Value not available			

Section 3 through 8 in the SPS-6 projects received AC surface, except for section 5 that was diamond ground. The post-rehabilitation average IRI value for sections 3 through 8 for each SPS-6 project is shown in figure 47, while the standard deviations of IRI for test sections 3 through 8 for each project is presented in figure 48. The post-rehabilitation project IRI ranged from 0.93 m/km (Indiana) to 1.12 m/km (Pennsylvania). The project in Indiana had the lowest standard deviation in IRI (0.04 m/km) with the project in Arizona having the highest standard deviation in IRI (0.24 m/km).

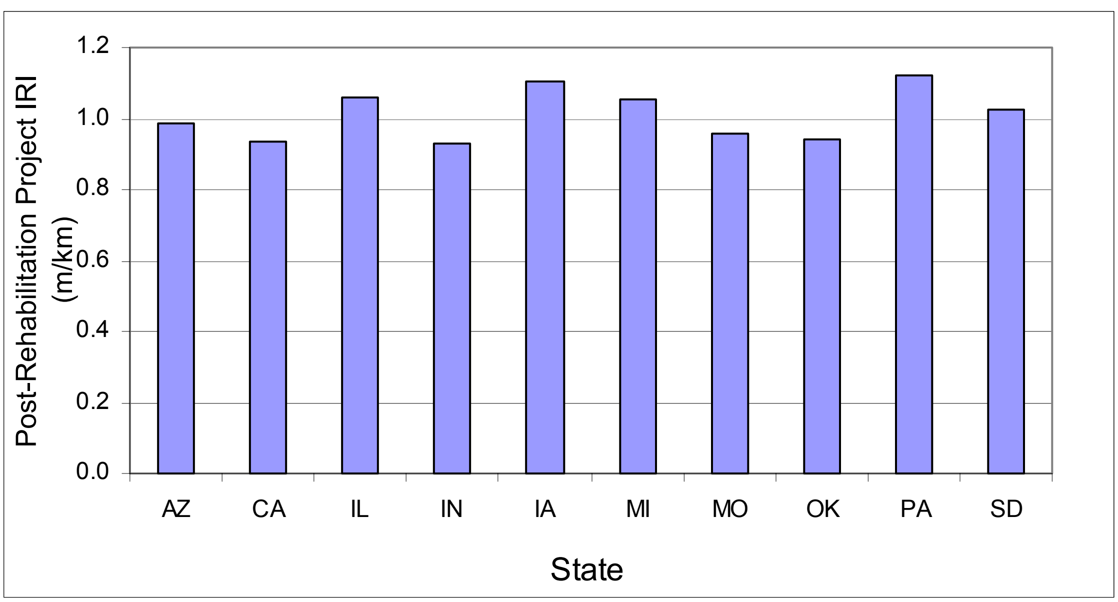


Figure 47. Average post-rehabilitation IRI of sections 3 through 8.

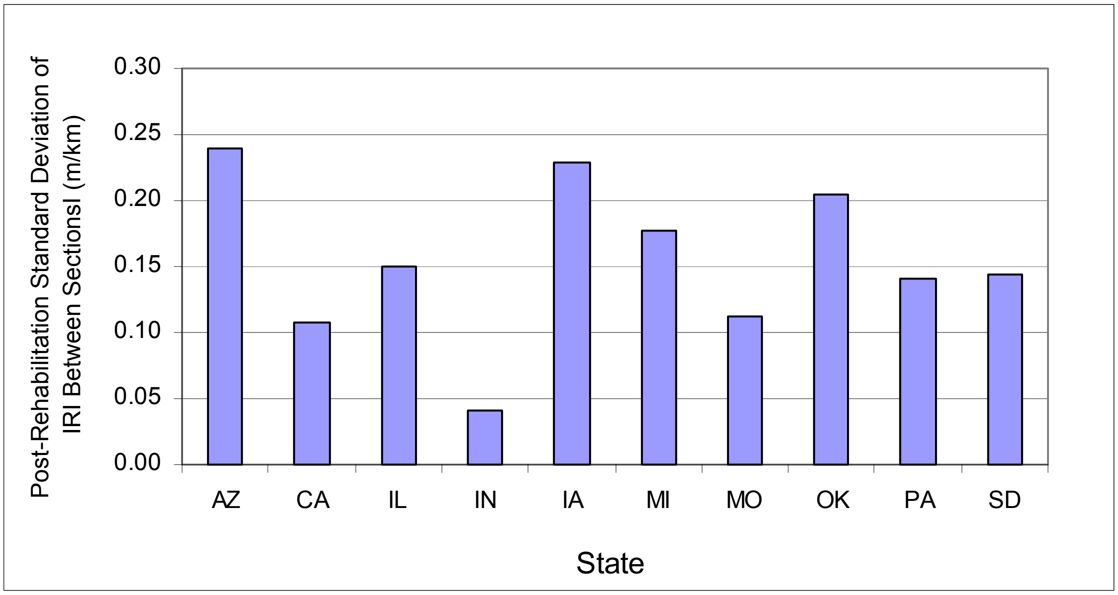


Figure 48. Post-rehabilitation standard deviation in IRI for sections 3 to 8.

Relationship Between IRI Before and After Rehabilitation

Figure 49 shows the relationship between IRI prior to rehabilitation and IRI after rehabilitation for section 2, which received minimum restoration. For section 2 (minimum restoration), the construction guidelines gave the States the option of diamond grinding the section if warranted (see table 37). In some SPS-6 projects, the minimum restoration section was diamond ground, while in others it was not. In figure 49, the sections that have post-rehabilitation values of less than 1.40 m/km are the projects that received diamond grinding. The data show in this figure show that diamond grinding can reduce the IRI of a pavement by a significant amount. As shown in figure 49, the pre-rehabilitation IRI of sections that were diamond ground ranged from 1.5 to 3.5 m/km, while the post-rehabilitation IRI ranged from 0.8 to 1.4 m/km. Generally, the sections that had lower pre-rehabilitation IRI values obtained lower IRI values after diamond grinding.

Figure 50 shows the relationship between IRI prior to rehabilitation and IRI after rehabilitation for section 5 that received intensive surface preparation followed by diamond

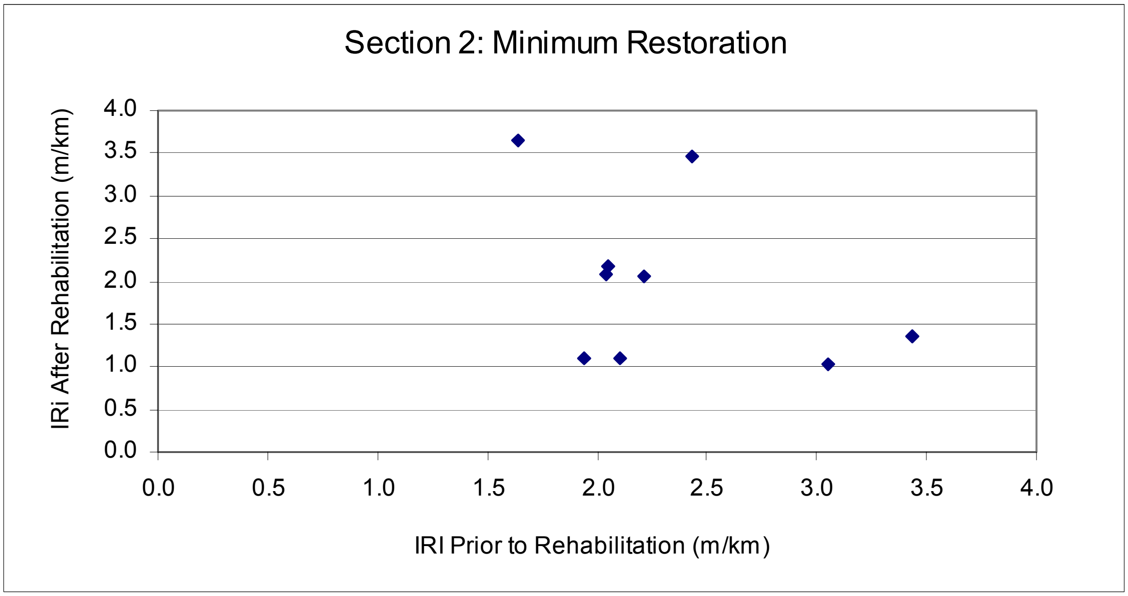


Figure 49. Relationship between IRI prior to and after rehabilitation for section 2 (minimal surface preparation).

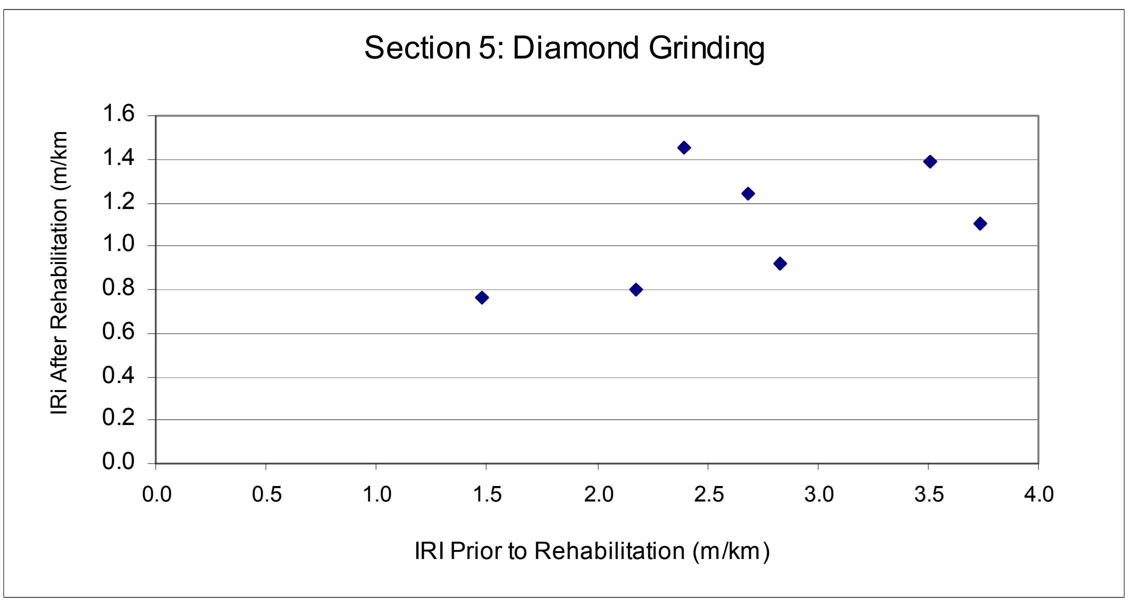


Figure 50. Relationship between IRI prior to and after rehabilitation for section 5 (intensive surface preparation followed by diamond grinding)

grinding. The pre-rehabilitation IRI of sections that were diamond ground ranged from 1.5 to 3.7 m/km, while the post-rehabilitation IRI ranged from 0.8 to 1.5 m/km. Generally, the sections that had lower pre-rehabilitation IRI values obtained lower IRI values after diamond grinding.

Figure 51 shows the relationship between IRI prior to rehabilitation and IRI after rehabilitation for sections 3, 4 and 5 all of which received a 100 mm AC overlay, with section 3 and 4 receiving minimum restoration prior to overlay, and section 5 receiving intensive surface preparation prior to overlay. Figure 51 show that AC overlays can reduce the roughness of a section significantly. There are several sections that had pre-rehabilitation IRI values that ranged from 2.9 to 3.8 m/km, but after the 100 mm AC overlay, the IRI of these sections ranged from 0.8 to 1.3 m/km.

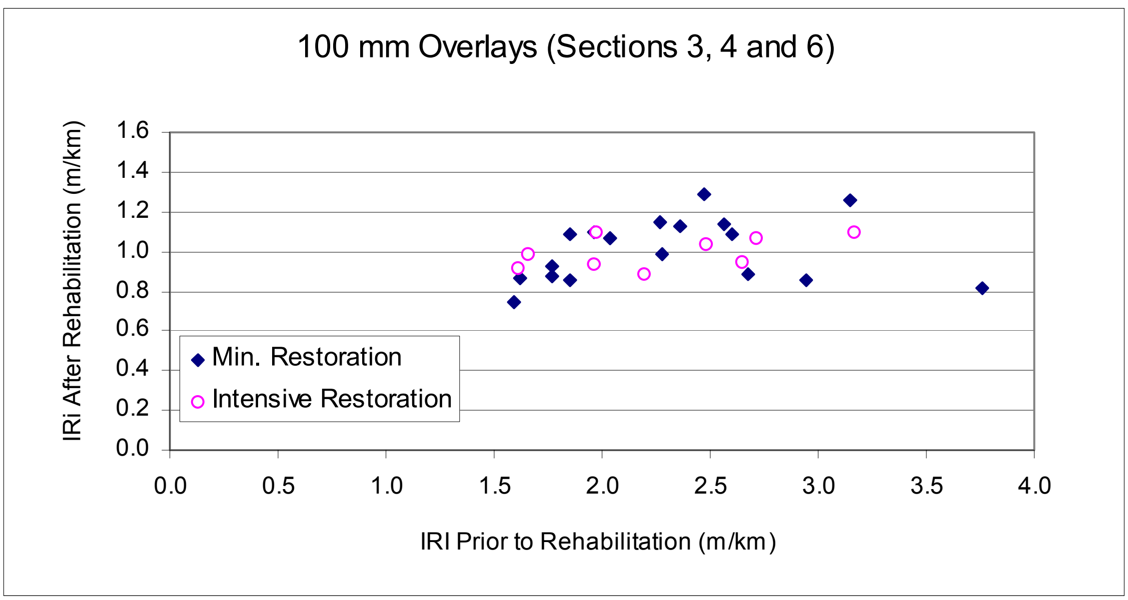


Figure 51. Relationship between IRI prior to and after rehabilitation for Sections 3, 4 and 6 (100 mm overlay)

Figure 52 shows the relationship between IRI prior to rehabilitation and IRI after rehabilitation for sections 7 and 8, that were crack/break seated and received a 100 mm and a 200 mm AC surface, respectively. As shown in figure 52, the post-rehabilitation IRI of crack/break seat sections ranged from 0.8 to 1.3 m/km.

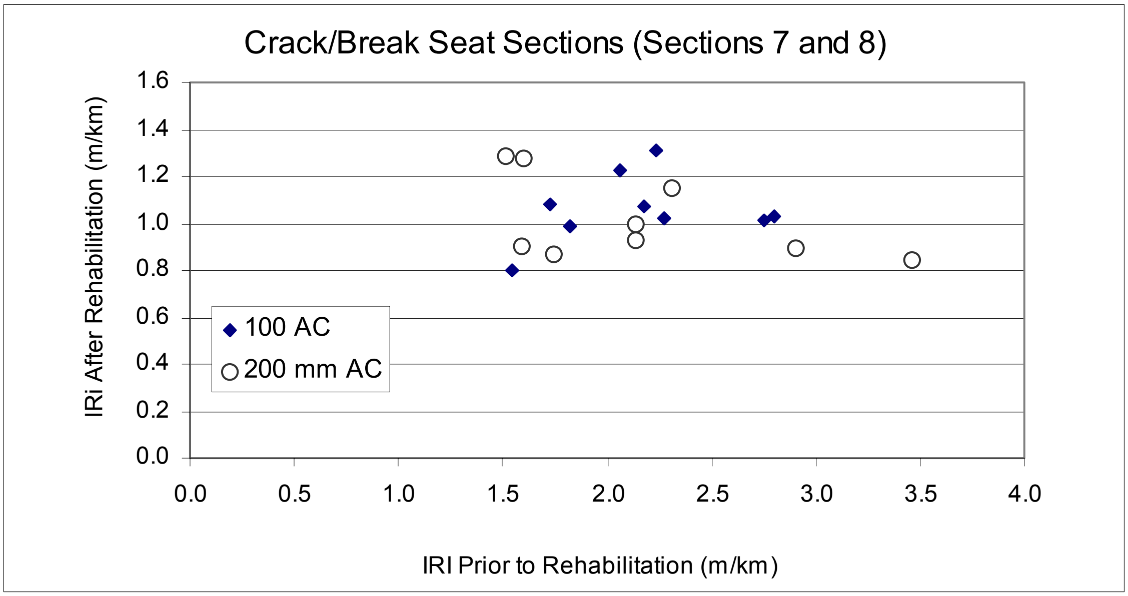


Figure 52. Relationship between IRI prior to and after rehabilitation for sections 7 and 8 (crack/break seated)

The relationship between pre and post-overlay IRI values for test sections in three SPS-6 projects (Arizona, Illinois and California) are shown in figure 53. The pre-rehabilitation project IRI, which is the average pre-rehabilitation IRI of the test sections in the project is shown on top of each graph. The pre-rehabilitation project IRI for the three projects shown in figure 53 range from 1.9 to 3.2 m/km. Data shown in figure 53 indicate that for a specific SPS-6 project, the IRI for all test sections except for section 2, tends to fall within a relatively narrow band of IRI values, irrespective of the IRI prior to rehabilitation of the test sections. This observation was generally noted for all SPS-6 projects that were analyzed. The pre- and post-rehabilitation IRI for test section 2 for the three projects shown in figure 53 show different trends. For the section in California, the IRI showed a large reduction, which was because the section was diamond ground. The section in Arizona showed a large increase in IRI after repairs, while section 2 in Illinois showed a small increase in IRI after repairs.

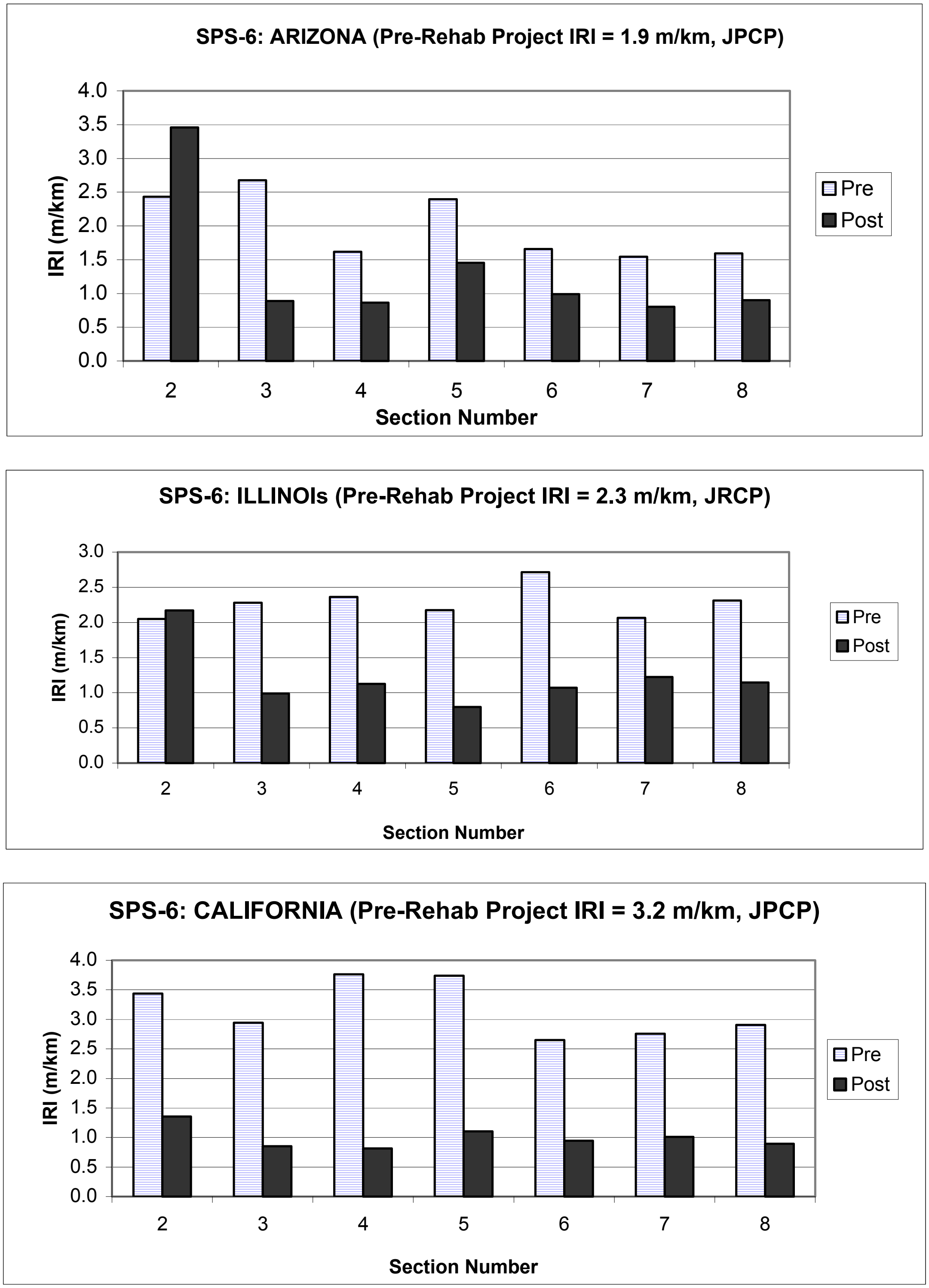


Figure 53. IRI before and after overlay for the different treatment factors for three SPS-6 projects.

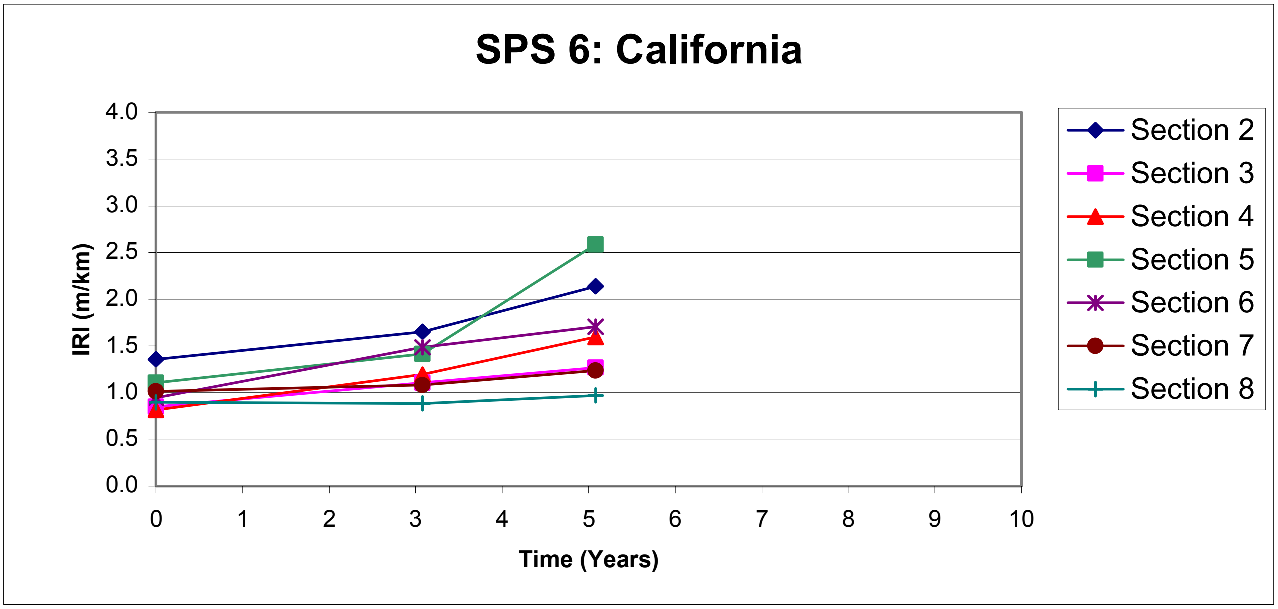
An ANOVA was performed to see if the IRI values after rehabilitation for the seven different treatment methods were different from each other. The ANOVA indicated there were differences in IRI values between the sections (p-value < 0.001). A multiple comparison using statistical analysis indicated section 2 was different from other sections. In some SPS-6 projects section 2 had been diamond ground, while in others it had not been diamond ground. The cause for this section being significantly different than other sections was due to the higher IRI values for sections that had not been diamond ground. Another ANOVA was performed by considering sections 3 through 8, which indicated that there was no significant difference in IRI values between the sections. The interpretation of this result is that the IRI values that are obtained after diamond grinding, and AC overlay of 100 mm (minimal and intensive surface preparation), and crack/break seat and an AC surface (100 mm and 200 mm), were similar.

An analysis similar to that performed for SPS-5 by fitting a model to see if the post-rehabilitation IRI values depended on pre-rehabilitation IRI values was not carried out for the SPS-6 projects. This was because the post-rehabilitation IRI values for the sections that were subjected to crack/break seat are not expected to depend on the pre-rehabilitation IRI value. Elimination of these sections, as well as elimination of section 2 that had different treatments in different States from a model fitting analysis would not leave an adequate data set to carry out such an analysis.

Change in IRI for SPS-6 Projects

The change in IRI over time for the SPS-6 projects in California and Oklahoma are shown in figure 54. Similar plots for all SPS-6 projects are included in Appendix D.

For SPS-6 projects that had at least three time-sequence IRI values, a linear regression was performed to obtain the rate of development of roughness. An ANOVA was performed to determine if there was a difference in rate of development of roughness between the seven different rehabilitation types. The rate of development of roughness was taken as the dependant variable, and State and treatment type was taken as the independent variables. The ANOVA



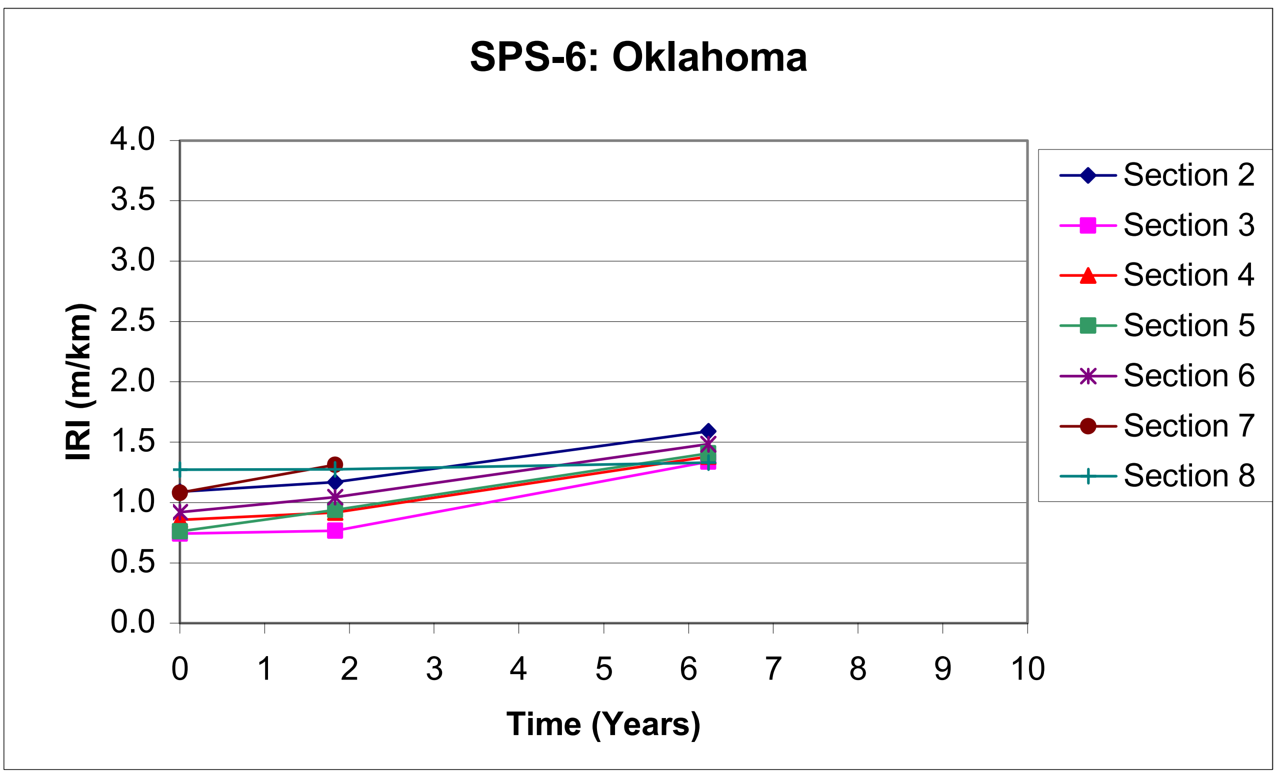


Figure 54. Changes in IRI for SPS-6 projects in California and Oklahoma.

indicated that the rehabilitation type was significant ($p < 0.001$). A multiple comparison analysis showed that the rate of change of roughness for section 5 (diamond grinding) was statistically different from all other types of treatment, and also section 2 (minimum restoration) was different from section 8 (crack/break seat and 200 mm AC surface). As different treatment types were performed at section 2 for different projects, another ANOVA was performed by omitting the data for section 2. This analysis also indicated that rate of change of IRI for section 5 (diamond grinding) was statistically different from all other sections.

Figure 55 shows a box-plot that shows the distribution of the rate of change of IRI at the different sections. This figure shows the highest values for the rate of change of roughness was obtained at section 5, which received diamond grinding.

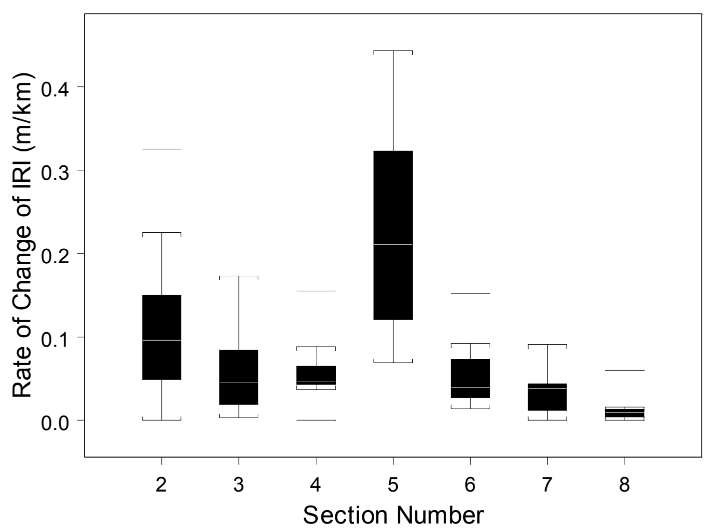


Figure 55. Box plot of rate of development of IRI.

The average and the standard deviation values for the rate of development of roughness for each treatment type are presented in table 42. The values shown in this table for a specific treatment type was obtained by using the rate of change of IRI values obtained for each project.

Table 42. Average and standard deviation of rate of change of IRI

Section Number	Surface Preparation	AC Overlay Thickness (mm)	Rate of Change of IRI (m/km/yr)	
			Average	Std. Dev.
2	Minimum Restoration	0	0.114	0.101
3	Minimum Restoration	100	0.058	0.051
4	Minimum Restoration (saw and seal joints)	100	0.057	0.042
5	Intensive Restoration	0	0.200	0.137
6	Intensive Restoration	100	0.054	0.042
7	Crack/Break Seat	100	0.032	0.033
8	Crack/Break Seat	200	0.013	0.017

The relationship between the rate of change of IRI and pre-rehabilitation IRI are shown in figures 56 through 59 for the different rehabilitation types. For the minimum restoration sections (see figure 56), the sections that received diamond grinding generally show a high rate of change of IRI. A clear relationship between rate of change of IRI and pre-rehabilitation IRI cannot be observed in figure 57, that shows the data for sections that received a 100 mm overlay. For sections that received diamond grinding (see figure 58), sections that had higher IRI prior to rehabilitation have a higher rate of change of IRI. Figure 59 shows that the majority of crack/break seat sections that received a 100 mm AC surface have a higher rate of change of IRI than the sections that received a 200 mm AC surface.

The percent change in IRI for the test sections (from the IRI value after rehabilitation) for all SPS-6 projects are shown in table 43. The values presented in this table are consistent with the results obtained from the ANOVA. The lowest changes in IRI values were observed at section 8, but two States had percent changes in IRI that exceeded 20 percent for this section.

An evaluation of the distresses observed on the diamond ground section (section 5) was performed to see if the distresses that were contributing to the high rate of increase of roughness seen at this section could be identified. Section 5 in projects shown in table 44 were selected for analysis. Table 44 gives the following information for the diamond ground section: IRI before diamond grinding, IRI after diamond grinding, IRI at last profile date, if pre-rehabilitation

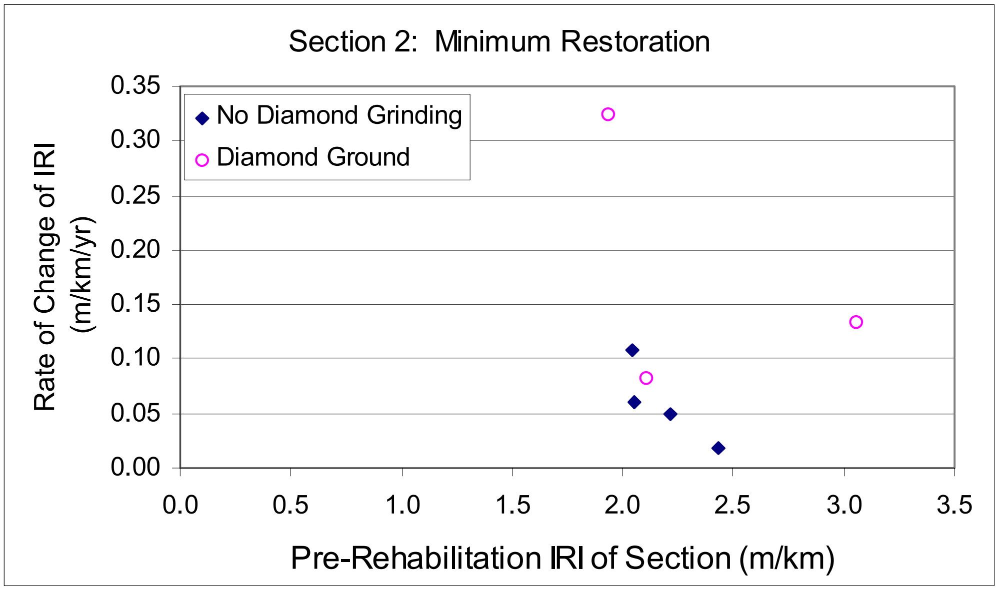


Figure 56. Relationship between rate of change of IRI and pre-rehabilitation IRI: Section 2 (minimum restoration)

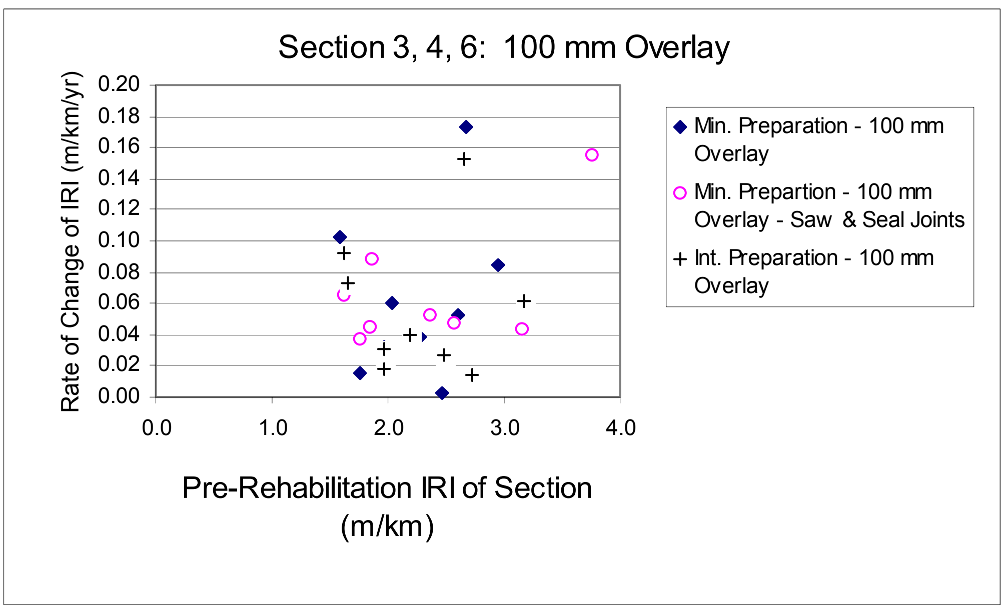


Figure 57. Relationship between rate of change of IRI and pre-rehabilitation IRI: Section 3, 4 and 6 (sections receiving 100 mm overlay)

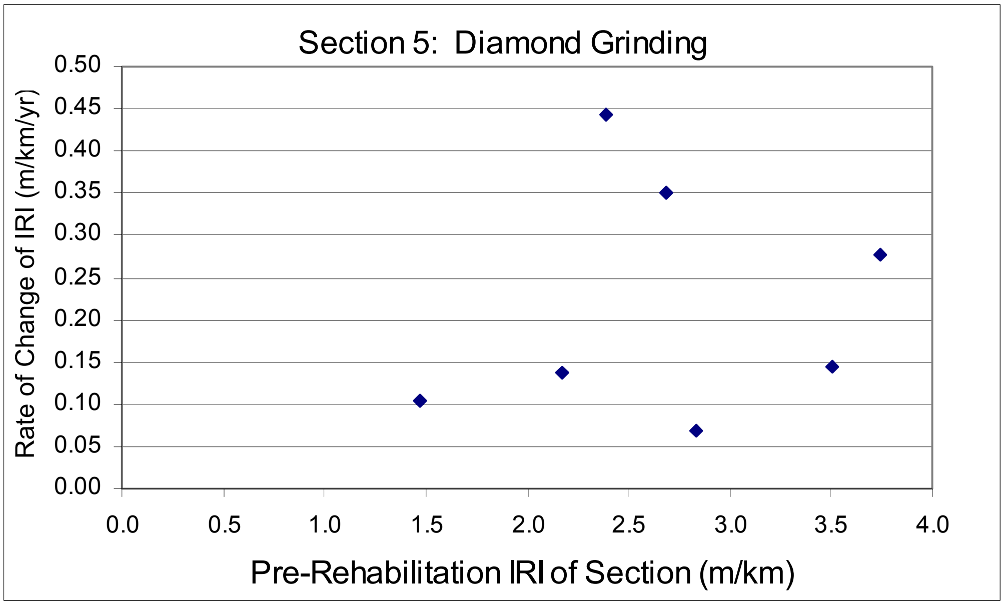


Figure 58. Relationship between rate of change of IRI and pre-rehabilitation IRI; Section 5 (diamond grinding)

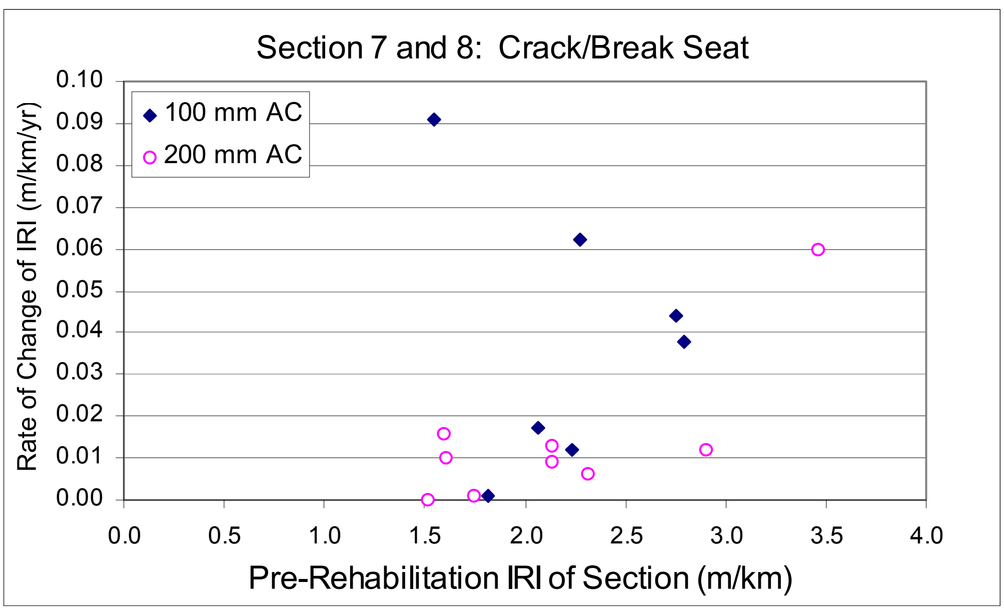


Figure 59. Relationship between rate of change of IRI and pre-rehabilitation IRI; Section 7 and 8 (crack/break seat and AC surface)

Table 43. Percent change in IRI at sections in SPS-6 projects.

State	Age When Project at First Profile Date (Yrs)	Age of Project at Last Profile Date (Yrs)	Time Difference for IRI Change (Years)	Percent Change in IRI (Note 1)						
				Section Number						
				2	3	4	5	6	7	8
Arizona	1.1	8.6	7.5	3	146	49	43	56	92	21
California	0.7	5.7	5.1	58	49	96	134	80	22	8
Illinois	1.5	7.7	6.2	17	23	27	103	9	8	4
Indiana	0.3	8.3	8.0	-1	17	35	N/A	18	4	8
Iowa	0.8	9.9	9.1	64	18	44	68	38	45	1
Michigan	0.6	8.9	8.3	37	-1	-9	N/A	37	-10	-2
Missouri	0.6	6.5	5.9	223	15	22	242	15	15	-4
Oklahoma	0.6	6.8	6.2	46	81	61	85	61	21	4
Pennsylvania	0.2	5.7	5.5	13	31	23	57	34	20	7
South Dakota	1.1	6.6	5.6	70	21	17	47	13	33	31
Average	0.7	7.5	6.7	53	40	37	97	36	25	8

Note 1: Percent Change in IRI = 100 X (IRI Last Profile Date - IRI First Profile Date)/(IRI at First Profile Date)
N/A - Data not available.

Table 44. Diamond ground sections evaluated.

SPS-6 Project	IRI (m/km)			Last Profile Date	Distress Survey Date (Note 1)	Pre-Rehabilitation Distress Available ?	Comment
	Before Diamond Grinding	After Diamond Grinding	Last Profile Date				
Arizona	2.39	1.45	2.08	2/12/93	9/25/91	Yes	Note 2
California	3.74	1.10	2.58	5/6/98	7/28/99	No	Note 3
Illinois	2.18	0.80	1.62	3/4/98	9/14/98	Yes	
Iowa	N/A	1.51	2.53	11/30/93	4/21/93	No	Note 2
Missouri	2.68	1.24	3.01	2/10/99	10/6/98	Yes	
Oklahoma	1.47	0.76	1.41	6/9/99	11/18/98	No	
Pennsylvania	3.51	1.39	2.18	5/28/98	7/21/99	Yes	
South Dakota	2.83	0.92	1.35	5/15/99	8/6/98	No	

N/A - Data not available
Note 1: Distress survey date that is closest to last profile date
Note 2: Appears to be rehabilitated after last profile date
Note 3: Pre-rehabilitation distress data available for most sections in SPS-6, but no data for section 5

distress data were available for the project, and the distress survey date that was closest to the last profile date.

Table 45 shows the distresses noted at the diamond ground section prior to rehabilitation, as well as the distresses for the date closest to the last profile date. The distress quantities shown in table 45 for each distress type is the sum of the distresses for all severity levels. The distress survey type is also indicated in this table. For the Pasco distress surveys, the distresses are obtained from photographic images, while in the manual surveys the distresses are recorded by a surveyor. An evaluation of the data tables in the IMS was performed to obtain the total faulting at the section corresponding to the distress survey date, or at a date closest to this date. However, the faulting data available was for much earlier survey dates, and therefore were not included in the analysis. The most prevalent distress noted at the diamond ground sections was transverse cracking. It is not known if faulting occurring at these cracks, in addition to faulting occurring at the joints are contributing to the increase in roughness. Some of the sections have large number of rigid patches. It is not clear how these patches have performed, and if these patches have tilted or are rocking under traffic and are contributing to the increase in roughness.

Table 45. Distresses at diamond ground sections.

State	Distress Survey Date	Distress Survey Type	Case (Note 1)	IRI (m/km) (Note 2)	Corner Breaks (No)	Long. Cracking Length (m)	Trans. Cracks (No)	Trans. Crack Length (m)	Long. Spalling Length (m)	Trans. Spalls (No)	Trans. Spall Length (m)	Flexible Patches (No)	Flexible Patches Area (m2)	Rigid Patches (No)	Rigid Patches Area (m2)
Arizona	11/21/89	Pasco	Pre-Rehab	2.39	0	8	12	41	0	33	111	0	0	0	0
Arizona	9/25/91	Manual	Last Profile	2.08	5	16	24	80	49	14	18	9	3	52	17
California	7/28/99	Manual	Last Profile	2.58	2	0	33	82	0	0	0	2	35	0	0
Illinois	5/6/90	Pasco	Pre-Rehab	2.18	0	0	13	48	1	1	1	0	0	4	54
Illinois	9/14/98	Manual	Last Profile	1.62	0	0	46	215	5	3	4	1	1	9	68
Iowa	4/21/93	Pasco	Last Profile	2.53	0	1	34	144	9	2	5	57	47	36	242
Missouri	8/7/91	Manual	Pre-Rehab	2.68	0	0	24	0	0	1	0	0	0	6	130
Missouri	10/6/98	Manual	Last Profile	3.01	0	7	34	215	1	21	11	3	1	27	389
Oklahoma	11/18/98	Manual	Last Profile	1.41	0	26	13	33	0	8	14	1	0	7	40
Pennsylvania	7/24/90	Manual	Pre-Rehab	3.51	1	4	5	12	4	0	0	9	5	3	33
Pennsylvania	7/21/99	Manual	Last Profile	2.18	0	3	10	25	147	12	8	27	8	9	145
South Dakota	8/6/98	Manual	Last Profile	1.35	4	24	4	4	5	1	0	2	1	25	62

Note 1: Pre-Rehab: Distresses recorded prior to rehabilitation.
Last Profile - Distresses recoded at a date that was closess to the date the section was last profiled

Traffic data as well as material testing data are not yet available in the LTPP database for many SPS-6 sections. Because of these limitations, a comprehensive analysis of the data to build models to predict development of roughness cannot be carried out yet. It was also seen that the pre-rehabilitation IRI as well as the pavement distress prior to rehabilitation varied between the test sections in individual SPS-6 projects. This introduces an additional confounding factor to the analysis. In spite of these limitations, the existing data does reveal trends in roughness development at SPS-6 sections.

Summary of Findings

Section 2 in a SPS-6 project was subjected to minimum restoration. Minimum restoration consisted of joint sealing, crack sealing, partial depth patching, and full depth patching. Each agency was also allowed to diamond grind this section as warranted. In some projects this sections was diamond ground, while in others it was not. At section 2 in Arizona, joint sealing, crack sealing and partial depth patching was performed, that resulted in the IRI of the section increasing from 2.43 to 3.46 m/km. Full depth patches were performed at section 2 in Indiana, that caused the roughness to increase from 1.64 m/km to 3.64 m/km. These results show that if repairs are not performed correctly in PCC pavements, they can result in an increase in roughness of the pavement. For the sections that were subjected to minimal restoration, and were diamond ground, the post-rehabilitation IRI of the sections ranged from 1.03 to 1.36 m/km.

A statistical analysis indicated there were no differences in IRI values obtained immediately after rehabilitation for sections 3 through 8. That is the analysis indicated applying the following treatments on a PCC pavement result in similar IRI levels: (1) minimum restoration of existing pavement followed by a 100 mm AC overlay (section 3), (2) minimum restoration of existing surface followed by a 100 mm AC surface, with sawing and sealing over joints (section 4), (3) Intensive restoration of existing surface that includes diamond grinding (section 5), (4) Intensive restoration of existing surface followed by a 100 mm AC overlay (section 6), (5) crack/break seat of PCC with a 100 mm AC surface (section 7), (6) crack/break seat of PCC with a 200 mm AC overlay (section 8). An investigation of the IRI before and after rehabilitation for the SPS-6 projects indicated that for a specific SPS-6 project, the IRI after rehabilitation for sections 3 through 8 all fell within a relatively narrow band.

An analysis of the rate of increase of IRI for the different treatment types indicated the following average values for rate of increase of roughness: (1) Section 3, minimum restoration and 100 mm overlay: 0.058 m/km/year, (2) Section 4, minimum restoration and 100 mm overlay with sawing and sealing of joints: 0.057 m/km/year, (3) Section 5, intensive restoration with diamond grinding: 0.200 m/km/year, (4) Section 6, intensive restoration with 100 mm overlay: 0.054 m/km/year, (5) Section 7, crack/break seat with 100 mm AC surface: 0.032 m/km/year, and (6) Section 8, crack/break seat with 200 mm AC surface: 0.013 m/km/year. A statistical analysis indicated the rate of increase of IRI of section 5 (diamond grinding) was statistically different from the rate of increase of IRI of the other sections. Generally, the rate of change of IRI at diamond ground sections was higher for sections that had higher IRI values prior to rehabilitation.

This page left intentionally blank.