Achieving a High Level of Smoothness in Concrete Pavements Without Sacrificing Long-Term Performance

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Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101-2296

Foreword

This report contains guidance on how highway agencies and contractors can achieve smooth, long-lasting portland cement concrete (PCC) pavements. The report: (1) assesses whether high initial smoothness necessarily results in better long-term performance, (2) identifies design features and material properties that can cause an initially smooth PCC pavement to exhibit detrimental long-term performance, (3) provides guidance on materials properties, design features, and construction procedures to avoid these detrimental effects, (4) investigates how the smoothness of a PCC pavement measured immediately after construction can change over the short term, and (5) looks at data collection issues related to lightweight inertial profilers.

This report should be of interest to those involved in the design and construction of concrete pavements. Sufficient copies of this report are being distributed to provide 10 copies to each FHWA Resource Center, 8 copies to each FHWA Division, and a minimum of 12 copies to each State highway agency. FHWA Division offices will distribute documents directly to State highway agencies. Additional copies for the public are available from the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, VA 22161.

Gary L. Henderson
Director, Office of Infrastructure
Research and Development

Notice

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16. Abstract

In a PCC pavement, it is important to achieve both a high level of smoothness during construction, as well as a satisfactory long-term performance. It is not acceptable to construct a pavement with a high initial smoothness that will give poor long-term performance. The design features and material properties of the PCC pavement should be conducive to yielding satisfactory long-term performance. Smoothness measurements for construction acceptance are usually performed shortly after paving is completed. The results from the smoothness measurements are used to judge whether the pavement has achieved the specified smoothness level. However, it is unclear whether the smoothness of a pavement measured immediately after it is paved truly reflects the initial smoothness of the pavement because the smoothness may undergo changes over the short term (e.g., within 3 months) due to curling or warping effects. This report: (1) assesses whether high initial smoothness translates into better long-term performance, (2) identifies design features and material properties in PCC pavements that can cause an initially smooth pavement to exhibit detrimental long-term performance, (3) provides guidance on materials properties, design features, and construction procedures to avoid these detrimental effects, (4) investigates how the smoothness of a PCC pavement measured immediately after construction can change over the short term (within the first 3 months), and (5) looks at data collection issues related to lightweight inertial profilers.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

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

APPROXIMATE CONVERSIONS FROM SI UNITS

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

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

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ACRONYMS

AASHO	American Association of State Highway Officials
ACPA	American Concrete Pavement Association
ASTM	American Society for Testing and Materials
CI	Curvature Index
CTE	coefficient of thermal expansion
DOT	Department of Transportation
ESAL	equivalent single axle loads
FHWA	Federal Highway Administration
GPS	General Pavement Studies
I	Interstate
ICC	International Cybernetics Corporation
IMS	Information Management System
IRI	International Roughness Index
JPC	jointed plain concrete
LTPP	Long-Term Pavement Performance
MPR	mean panel ratings
NCHRP	National Cooperative Highway Research Program
PCC	portland cement concrete
PI	Profile Index
PSD	power spectral density
PSI	Present Serviceability Index
RN	ride number
SHA	State highway agency
UMTRI	University of Michigan Transportation Research Institute

CHAPTER 1. INTRODUCTION AND PROJECT OBJECTIVES

INITIAL SMOOTHNESS OF CONCRETE PAVEMENTS

In 1996, the Federal Highway Administration (FHWA) commissioned a national survey of frequent highway users that asked them what they wanted in their highway "product." The highway users clearly stated that their top priority was quality of road conditions, followed by safety and the need to reduce congestion.(1) In an effort to provide highway users with a smooth ride, highway agencies have implemented smoothness specifications for new pavements. A smoothness specification indicates the acceptable range of smoothness that contractors must achieve to obtain full payment. Several highway agencies are giving bonuses to contractors who obtain better smoothness than the minimal requirement, and they are assessing penalties for those who deliver unacceptable smoothness.

A survey conducted in 2000 indicated that 86 percent of the highway agencies in the United States currently use a smoothness specification for new portland cement concrete (PCC) pavements.(2) Those agencies that do not use a PCC smoothness specification are believed to be the ones that construct few or no PCC pavements. The number of highway agencies using a smoothness specification for the construction of PCC pavements has shown a sharp increase during the last decade. In addition, the smoothness level specifications standards in highway agencies have increased over the years as the contractors became familiar with the smoothness specification.

For a PCC pavement, it is important to achieve both a high level of smoothness during construction, as well as a satisfactory long-term performance. It is not acceptable to construct a pavement with high initial smoothness that will give poor long-term performance. The design features and material properties of the PCC pavement should be conducive to yielding satisfactory long-term performance.

Smoothness measurements for construction acceptance usually are performed shortly after paving is completed, using either a profilograph or a lightweight inertial profiler. The results from the smoothness measurements are used to judge whether the pavement has achieved the specified smoothness level. These results are also used to determine whether contractors receive bonuses or whether they will be assessed penalties.

However, it is unclear whether the smoothness of a pavement measured immediately after it is paved truly reflects the initial smoothness of the pavement, because the smoothness can undergo changes over the short term (e.g., within 3 months) due to curling or warping effects. In other words, a pavement can have a very high smoothness immediately after construction, followed by a decrease in smoothness over a short time period because of changes in slab shape that occur with curling and warping. Another concern with achieving high levels of initial smoothness relates to whether paving contractors are using construction procedures or making changes to the PCC mix design that result in a high initial smoothness but are detrimental to long-term pavement performance.

PROJECT OBJECTIVES

The objectives of this research project were to:

- Assess whether high initial smoothness translates into better long-term performance.
- Identify design features and material properties in PCC pavements that can cause an initially smooth pavement to exhibit detrimental long-term performance.
- Provide guidance on adjustments that can be made to materials properties, design features, and construction procedures to avoid these detrimental effects.
- Investigate how the smoothness of a PCC pavement measured immediately after construction (typically 1 day after construction) can change over the short term (within the first 3 months).

RESEARCH APPROACH

The roughness data collected at Long-Term Pavement Performance (LTPP) program test sections were used to study the roughness progression of jointed plain PCC sections. The initial and long-term smoothness of the test sections were evaluated and compared to determine what effect the mix design properties, material properties, construction procedures, and design features have on pavement performance.

The changes in smoothness that occur over the short term on PCC pavements were investigated by collecting profile data on test sections established on new PCC pavements. The test sections were typically profiled 1 day, 3 days, 7 days, and 3 months after paving. This investigation evaluated the short-term smoothness changes of a PCC pavement.

ORGANIZATION OF THE REPORT

A literature review was performed to compile information related to these topics:

- Methods used to measure the initial smoothness of a PCC pavement for construction acceptance.
- State highway agency (SHA) concrete smoothness specifications and practices.
- Smoothness indices used to judge pavement smoothness.
- Design, construction, and mix design properties that can affect the initial smoothness of a PCC pavement.

As part of the literature review, SHA and American Concrete Pavement Association (ACPA) personnel were asked to provide their views on how the concrete mix design may affect the smoothness of the as-built pavement.

Chapter 2 presents the results of the literature survey on procedures used to determine pavement smoothness for construction acceptance and smoothness indices used to judge pavement smoothness. Chapter 3 presents the literature survey results describing design, construction, and mix design properties that can affect the initial smoothness of a PCC pavement.

Chapter 4 presents an overview of the analysis procedures. Chapter 5 presents findings from the analysis of LTPP data, which were used to study roughness progression at LTPP test sections. The LTPP data were also used to determine factors causing pavements to show poor long-term performance and to identify factors allowing pavements to retain their smoothness over the service life. Chapter 6 presents the findings from investigations of short-term changes in the smoothness of PCC pavements. This chapter also presents findings from the analysis of data collected from five paving projects at 1 day, 3 day, 7 day, and 3-month intervals after paving.

Chapter 7 presents conclusions from this research. Chapter 8 presents recommendations and guidelines for design features, PCC material properties, and construction procedures to achieve pavements with both high initial smoothness and good long-term performance. Guidelines and recommendations on how to conduct smoothness testing are also presented in this chapter.

CHAPTER 2. PAVEMENT SMOOTHNESS MEASUREMENTS

EQUIPMENT FOR SMOOTHNESS MEASUREMENT

The most common equipment currently used to measure smoothness of new PCC pavements for construction acceptance is the profilograph.[2] The trace obtained from the profilograph is used to compute the Profile Index (PI) of the pavement.

For pavement management purposes, SHAs use inertial profilers to collect profile data on their pavement network and then compute the International Roughness Index (IRI) using this data. Currently, many SHAs use PI to judge the smoothness of pavements for construction acceptance and then use IRI to monitor the performance of the pavement.

Recently, several SHAs have started using inertial profilers to measure the smoothness of new PCC pavements. Inertial profilers are capable of recording profile features of the road that affect ride quality. The collected data are then used to compute IRI to judge the smoothness of the pavement for construction acceptance. (A profilograph simulation can also be carried out on the inertial profiler data to obtain a profilograph trace. This trace can then be used to compute the PI of the roadway.) Several SHAs that have been using the PI for construction acceptance have converted to using IRI, and other SHAs currently using profilographs are looking into the possibility of adopting IRI as the construction acceptance index for smoothness.

Profilographs

The most common profilograph used today is the truss-type California profilograph (see figure 1). The profilograph consists of a rigid frame with a system of support wheels at each end and a center wheel for profile measurement. The distance between the centers of the supporting wheel systems in the profilograph is 7.6 meters (m) (25 feet (ft)). The support wheels establish a datum from which the deviations of the center wheel can be evaluated. The center wheel is linked to a strip chart recorder or a computer that records the movement of the center wheel from the established datum. The profilograph is pushed along the pavement, and 3 to 5 kilometers (km) (1.9 to 3.1 miles (mi)) of the pavement can be measured in 1 hour.

Most profilographs in use today are computerized to electronically record the data. Mechanical profilographs used before computerized profilographs recorded data on a strip chart recorder. Computerized profilographs use a computer program to compute the PI of the pavement. The strip chart recorder output from mechanical profilographs is evaluated either manually or electronically. Using a manual method, a technician evaluates the profilograph output to determine the PI. Using an electronic method, the output of the strip chart recorder is scanned, and a computer program performs the data reduction.

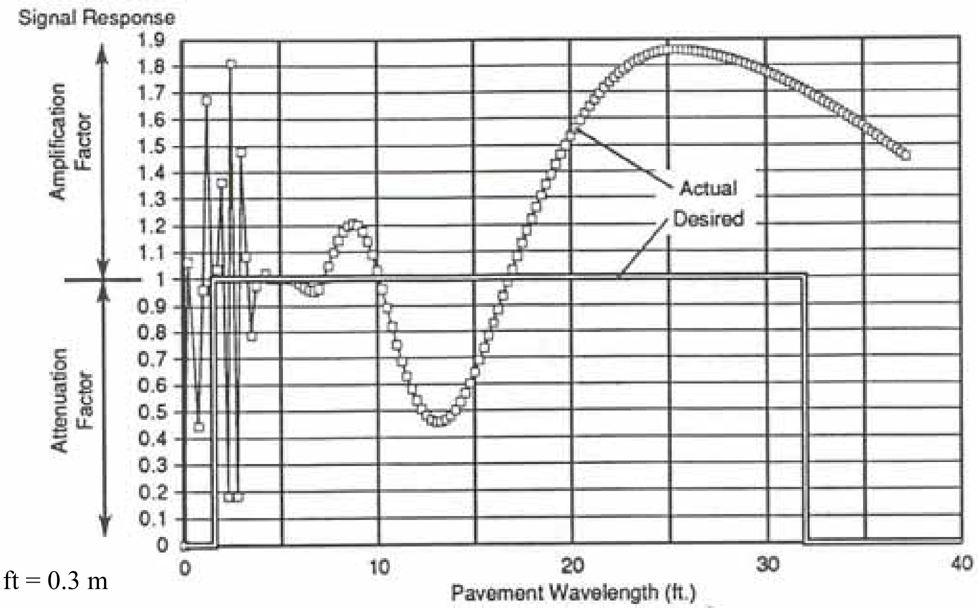
Figure 1. Truss-type California profilograph.

There have been questions about the effectiveness of profilographs in measuring wavelengths related to ride quality. Kulakowski and Wambold reported that profilographs vary in how they respond to wavelengths present on roadways.(3) According to the authors, profilographs correctly measure some wavelengths, amplify some wavelengths, and hardly measure some wavelengths. Figure 2 shows the actual and desired frequency response of a 12-wheel California profilograph. As shown in this figure, the California profilograph gives a poor measurement for wavelengths between 3 to 4.6 m (10 to 15 ft), and amplifies the response for wavelengths between the 6.1- to 12.2-m (20- to 40-ft) range by as much as two times.

Because profilographs are known to amplify and attenuate the true pavement surface profile, concerns have been raised about the suitability of using profilograph data to judge the smoothness of new pavements for construction acceptance.

Inertial Profilers

Inertial profilers are able to measure elevation features of the roadway that affect ride quality. The first high-speed inertial profiler was developed by Spangler and Kelley.(4) Most SHAs use high-speed profilers to collect roughness data on their highway networks. Figure 3 shows a photograph of a high-speed profiler. Currently, new PCC pavements usually are profiled immediately after they are paved; therefore, van-based high-speed profilers cannot be used to collect profile data on these pavements, because the pavement is not yet strong enough to support the weight of a van. Lightweight inertial profilers (see figure 4) based on a utility vehicle can be used to profile new PCC pavements several hours after the pavement is placed. The profiling system in a lightweight profiler is similar to the profiling system in a high-speed inertial profiler.



1 ft = 0.3 m

Figure 2. Desired and actual frequency response of 12-wheel California style profilograph.[3]

Figure 3. High-speed profiler.

Figure 4. Lightweight profiler.

A schematic diagram of a high-speed inertial profiler is shown in figure 5. The principal components of an inertial profiler are the height sensor(s), accelerometer(s), a distance measuring system, and computer software and hardware.

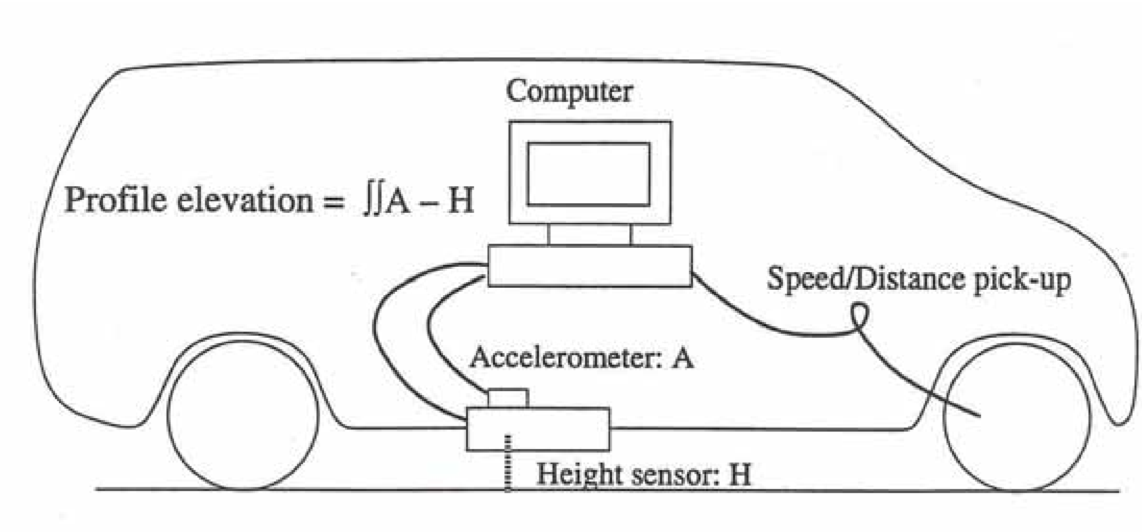


Figure 5. Components of an inertial profiler.[5]

The height sensor records the height to the pavement surface from the vehicle. The accelerometer located on top of the height sensor records the vertical acceleration of the vehicle. The acceleration is mathematically converted to vertical displacement. The distance measuring system keeps track of the distance with respect to a reference starting point. Data from the height sensor and the accelerometer are combined to compute the profile of the pavement, and the computed profile is recorded in the computer at a specified distance interval. Laser sensors are the most common height sensors currently used in profilers.

SMOOTHNESS INDICES

Three smoothness indices currently in use are the PI, IRI, and ride number (RN). The trace obtained from the profilograph is used to obtain PI (PI can be simulated from measurements obtained from an inertial profiler). The data obtained from an inertial profiler must be used to compute IRI and RN. A description of these three indices is presented in this section.

Profile Index

There is no universal standard for reducing profilograph traces. Each State agency has its own standardized procedures. Therefore, comparisons of the PI values between States may not be meaningful. California has had extensive experience with the use of profilographs and uses California Test Method 526 for reducing profilograph traces.[6] The general procedures used in reducing the profilograph trace to obtain PI are described in this section. A computer program performs these procedures for computerized profilographs and when scanned data from mechanical profilographs are analyzed.

Outline Trace

The purpose of outlining the trace is to average out spikes and minor deviations caused by rocks, texture, dirt, or transverse grooving. Outlining consists of drawing a new profile line through the midpoint of the spikes of the field trace as shown in figure 6. It is assumed that this was an enhancement adopted by many agencies to reduce variability and expedite trace reduction.[7] Outlining the trace is currently not included in the California Test Method 526.[6]

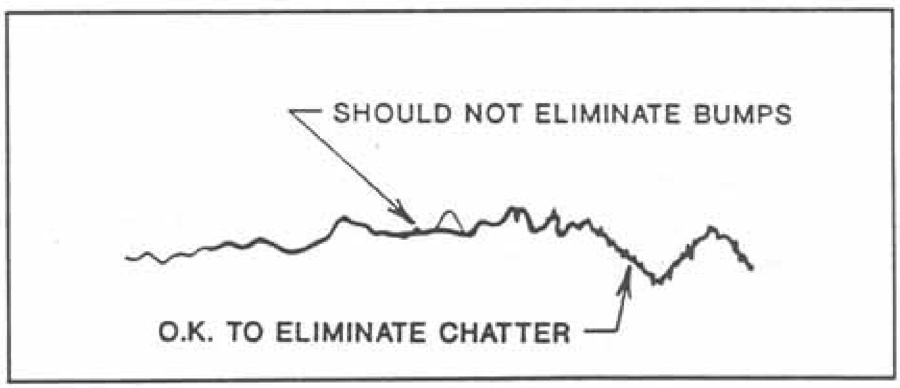


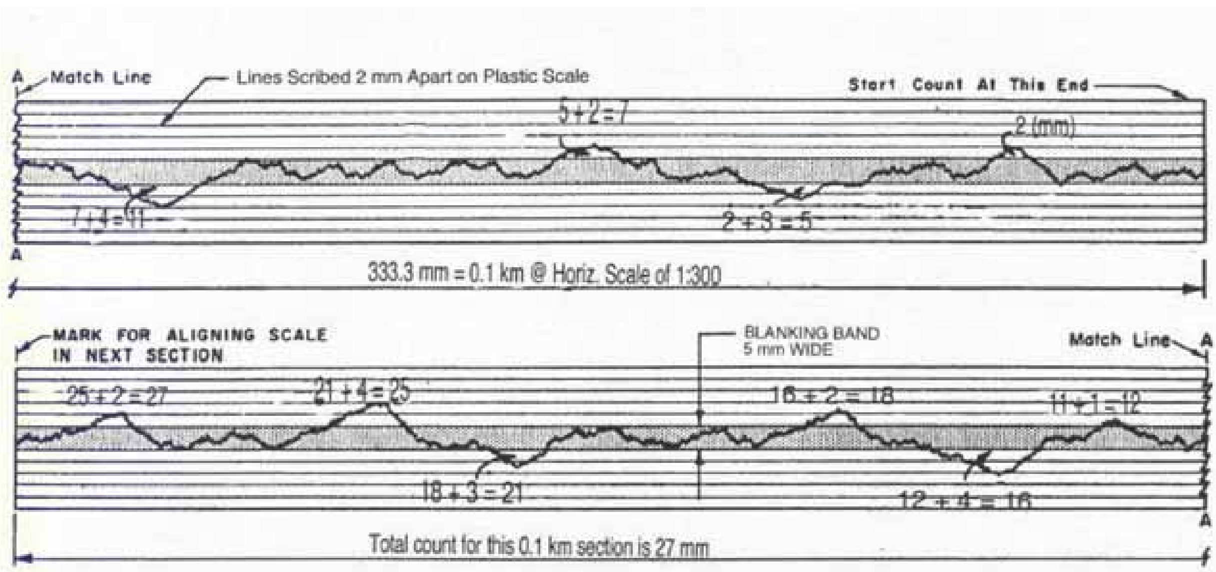
Figure 6. Example of an outlined trace.[7]

Position Blanking Band

The next process in trace reduction is to place the blanking band on the profile trace. The procedure for positioning the blanking band is described in California Test Method 526.[7] The scale used to evaluate the profilograph trace is made of plastic; it is 40 millimeters (mm) (1.6 inches) wide and 333.3 mm (13.1 inches) long, representing a pavement length of 0.1 km (0.06 mi). At the center of the scale, there is an opaque band 5 mm (0.2 inches) wide, which extends the full length of the scale. In the horizontal direction, the scale represents a scale of 1:300; the scale is a true scale (1:1) in the vertical direction. Parallel to the opaque band on both sides are five scribed lines that are at 2-mm (0.08-inch) intervals. The blanking band is placed over the profile trace so that the 5-mm (0.2-inch)-wide center band blanks out as much of the profile as possible. When this is done, the deviations above and below the opaque band will be approximately balanced. The common blanking bands in use today are the 5-mm (0.2-inch) and zero blanking bands. The zero blanking band just has a reference line. The length of the plastic scale can vary according to the length of the pavement used to evaluate the PI. The scale dimensions described previously are applicable when the evaluated length of the pavement is 0.1 km (0.06 mi). Some highway agencies are using a pavement length of 161 m (528 ft) to evaluate the PI. In these cases, the length of the scale will be different from the value described previously.

Determine Profile Index

The California Test Method 526 describes the procedure for computing the PI.[7] Starting at the right end of the scale, the heights of the scallops appearing both above and below the blanking band are measured to the nearest 1 mm (0.04 inch) and totaled. The excursions are evaluated against the five parallel lines scribed on both sides of the blanking band. However, unless the feature projects 0.6 mm (0.02 inch) or more and extend longitudinally for 0.6 m (2 ft) or more on the pavement (2 mm (0.08 inch) on the profilograph trace), they are not counted. The sum of the recorded heights within a given segment will be the PI for that segment. The PI is expressed in terms of millimeters per kilometer (mm/km) or inches per mile (inches/mi). Figure 7 shows an example of a profilograph trace and how the PI is computed.



1 mm = 0.039 inch
1 km = 0.621 mi

Figure 7. Determining PI from a profilograph trace.[6]

International Roughness Index

IRI was developed in a 1982 study performed to establish a correlation and calibration standard for roughness measurements.[8] IRI is defined as a property of the true profile, and therefore it can be computed from the profile measured with any valid profiler. IRI was mainly developed to match the response of passenger cars, but subsequent research has shown good correlation with light trucks and heavy trucks. Specifically, IRI is highly correlated to three vehicle response variables that are of interest: road meter response (for historical continuity), vertical passenger acceleration (for ride quality), and tire load (for vehicle controllability and safety).

The computation of IRI is based on a mathematical model called a quarter car model. The quarter car is simulated on the measured profile to calculate the suspension deflection. This simulation is performed for a speed of 80 kilometers per hour (km/h) (50 miles per hour (mi/h)). The mathematical simulation is carried out by a computer program shown schematically in figure 8. The quarter car model used in the IRI algorithm is just what its name implies: a model of one corner (a quarter) of a car. As shown in figure 8, the quarter car is modeled as one tire that is represented with a vertical spring, the mass of the axle supported by the tire, a suspension spring and damper, and the mass of the body supported by the suspension for that tire. The absolute values of the suspension motions obtained from the simulation are summed and then divided by the simulation length to obtain the average suspension motion over the simulated length. The value computed is the IRI, which has units of slope, with the most common units being inches per mile or meters per kilometer.

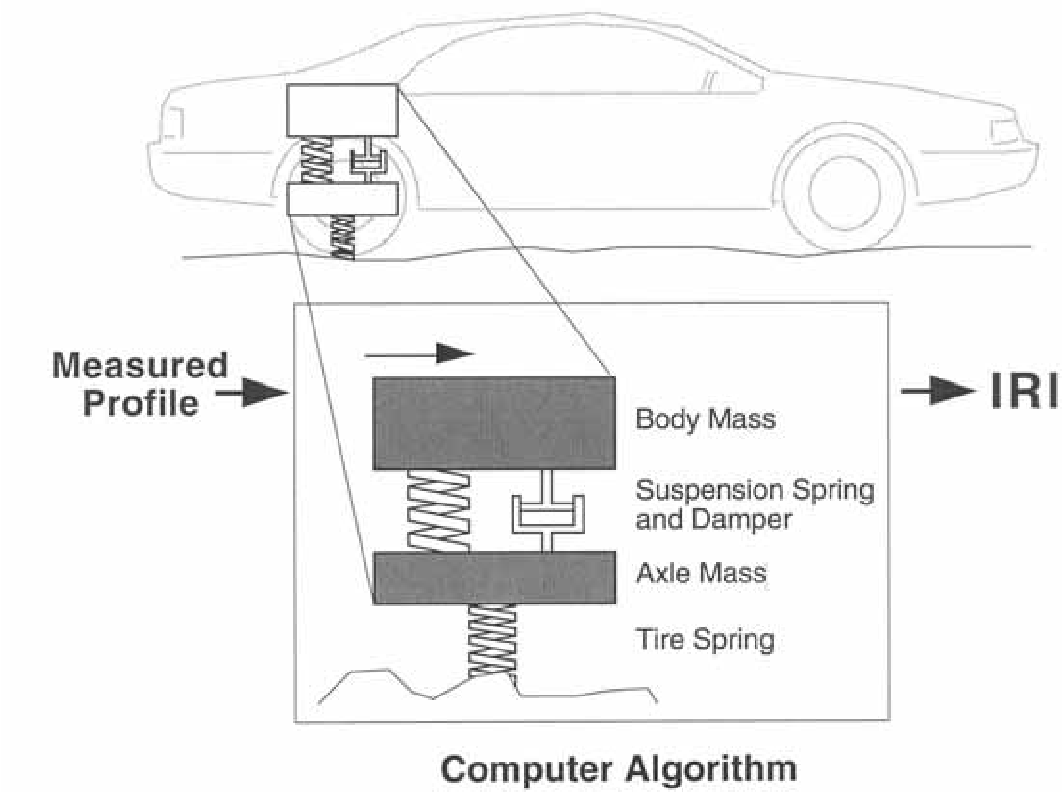


Figure 8. Illustration of computer algorithm used to compute IRI.[5]

A computer program is used to calculate IRI from profile data. The American Society for Testing and Materials (ASTM) Standard E1926, "Standard Practice for Computing International Roughness Index of Roads from Longitudinal Profile Measurements" presents the computer program used to compute IRI.[9] IRI is calculated for a single profile. As most profilers collect data along the two wheel paths, the IRI computation should be carried out separately for each wheel path. The average of the IRI values obtained along the two wheel paths is referred to as the mean IRI and this number is frequently used as a measure of roughness of the road.

The response of the IRI quarter car filter to different wavelengths is shown in figure 9. IRI is mostly influenced by wavelengths ranging from 1.2 to 30.5 m (4 to 100 ft). However, there is still some response for wavelengths outside this range. The IRI filter has maximum sensitivity to sinusoids with a wavelength of 2.4 m (8 ft) and 15.4 m (51 ft).

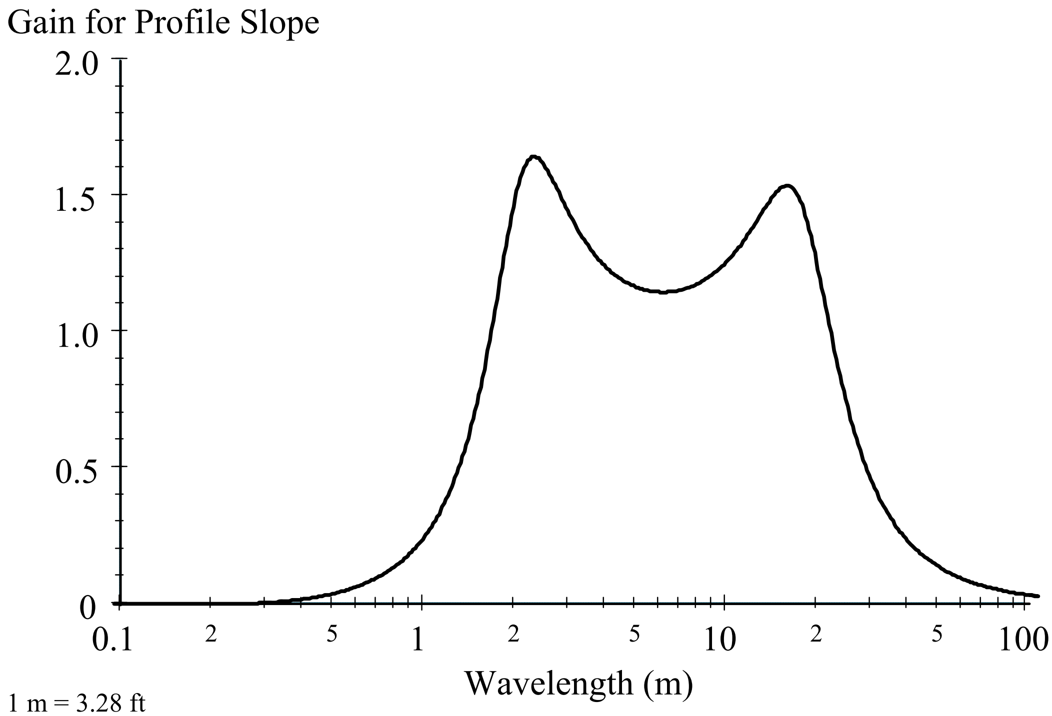


Figure 9. Response of IRI filter.(5)

The shape of slabs in a PCC pavement can vary because of curling and warping. Slab curling refers to the change in the PCC slab shape that occurs because of daily variations in temperature. Slab warping refers to the change in slab shape that occurs from moisture variations in the PCC slab, which occur over the long term. It is believed that in the early morning hours when the top of the slab is cooler than the bottom of the slab, the slab has an upward curl, where the slab joint is at a higher elevation with respect to the center of the slab. In the afternoon hours, when the top of the slab is warmer than the bottom, the slabs are believed to have a downward curl, where the center of the slab is at a higher elevation than the slab joints. However, research has shown that some pavements will not change the slab shape from an upward curl to a downward curl because of daily temperature variations.(10,11) These pavements are permanently in an upward curled position, and although the amount of curling will change due to temperature variations, the slabs will not attain a flat position or a downward curl.

Curling or warping present in PCC slabs will affect IRI. Higher amounts of curling or warping will cause IRI to increase. Most jointed PCC pavements have a joint spacing between 4.6 and 6.1 m (15 and 20 ft). The presence of curling or warping in the PCC slab will cause an increase in the wavelength content at a wavelength that is equal to the joint spacing, which in turn will cause an increase in IRI.

Ride Number

For decades, highway engineers have been interested in obtaining the opinion of the traveling public on the roughness of roads. The Present Serviceability Index (PSI) scale from the American Association of State Highway Officials (AASHO) Road Test has been of interest to engineers since its introduction in the 1950s. PSI ranges from 0 to 5, with 5 representing a perfectly smooth road and 0 representing a road that is almost impassable. RN is a PI intended to indicate rideability on a scale similar to PSI. The longitudinal profile measurements taken with a profiler are processed using a computer program to obtain RN.

The National Cooperative Highway Research Program (NCHRP) sponsored two research projects in the 1980s that investigated the effect of road surface roughness on ride comfort.(12,13) The objective of that research was to determine how features in road profiles were linked to subjective opinion about the road from members of the public. During two studies, spaced at about a 5-year interval, mean panel ratings (MPR) were determined experimentally on a 0 to 5 scale for test sites in several States. Longitudinal profiles were obtained for the left and right wheel paths of the lanes that were rated. Profile-based analyses were developed to predict MPR. A method was developed by which power spectral density (PSD) functions were calculated for the two longitudinal profiles measured along the wheel paths and reduced to provide a summary statistic called PI. (No relationship exists between PI used in RN computations and PI obtained from the reduction of profilograph traces.) The PI values for the two profiles were then combined in a nonlinear transform to obtain an estimate of MPR. The mathematical procedure developed to calculate RN is described in NCHRP Report 275.(12) Software for computing RN with this method was never developed for general use.

In 1995, some of the data from these two NCHRP projects and a panel study conducted in Minnesota were analyzed by the University of Michigan Transportation Research Institute (UMTRI) for a pooled-fund study initiated by FHWA.(14) The objective of this analysis was to develop and test a practical mathematical process for obtaining RN. The profile data in the original NCHRP research were obtained from several instruments. Most measurements were made with a K.J. Law profiler owned by the Ohio Department of Transportation (DOT) and were thought to be accurate. A few other test sites were profiled with instruments whose validity has been questioned. The new analysis was limited to 138 test sites that had been profiled with the Ohio system and the data from the Minnesota study. Based on analysis of this data, a new profile analysis method to compute RN was developed.(14) This procedure predicts MPR slightly better than previously published algorithms. The software was tested on profiles obtained from different systems on the same sites, and similar values of RN were obtained.

RN uses a scale from 0 to 5. This scale was selected because it is familiar to the highway community. RN is a nonlinear transform of a statistic called PI, which is computed from profile data. PI ranges from 0 (a perfectly smooth profile) to a positive value proportional to roughness. PI is transformed to a scale that goes from 5 (perfectly smooth) to 0 (the maximum possible roughness).

Figure 10 shows the sensitivity of PI for a slope sinusoid. When a sinusoid is given as an input, the PI filter produces a sinusoid as the output. The amplitude of the output sinusoid is the amplitude of the input, multiplied by the gain shown in the figure. The maximum sensitivity occurs for a wavelength of about 6.1 m (20 ft). The content of a road profile that affects RN is different from the content that affects IRI. IRI has high sensitivity to sinusoids with a wavelength of 2.4 to 15.4 m (8 to 51 ft). Figure 10 shows that RN has a low sensitivity to a wavelength of 15.4 m (51 ft) and even lower sensitivity for longer wavelengths. IRI is primarily influenced by wavelengths between 1.2 and 30.5 m (4 and 100 ft). RN is primarily influenced by wavelengths between 0.5 and 11 m (1.6 and 36 ft). IRI and RN will not always correlate the same way and do not have the same meaning. Thus, they each provide unique information about the roughness of the road.

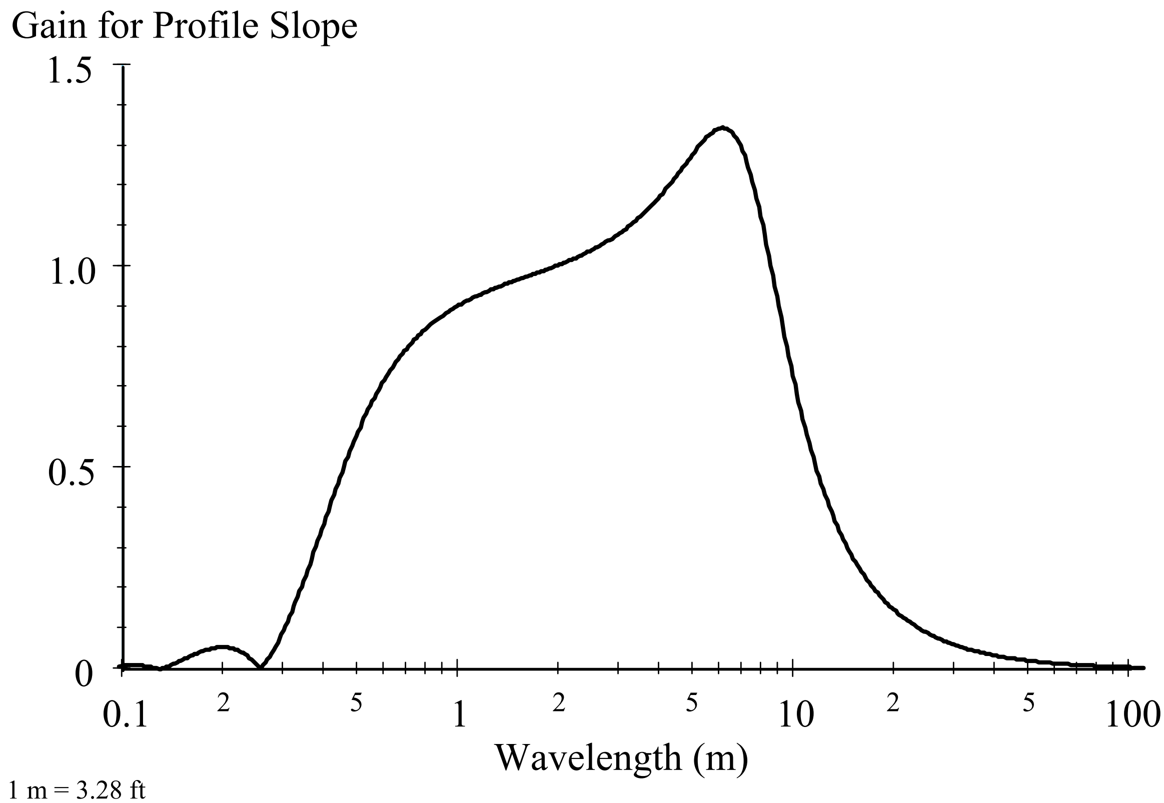


Figure 10. Sensitivity of PI for a slope sinusoid.(5)

The ASTM Standard E 1489, "Standard Practice for Computing Ride Number from Longitudinal Profile Measurements Made by an Inertial Profile Measuring Device," presents the computer program that should be used to compute RN.(9)

SMOOTHNESS SPECIFICATIONS

Rizzo performed a survey in 2000 that found the PI obtained from profilograph measurements was the most common method used in the United States to measure the smoothness of new PCC pavements for construction acceptance.(2) The most common blanking band in use was 5 mm (0.2 inch), but some SHAs were using a 2.5-mm (0.1-inch) or a zero blanking band. The survey also found that some SHAs specify pay incentives for obtaining a smoothness value that is better than the required value while others do not. The full pay PI range varied according to the blanking band that was specified in the smoothness specification. Even for a specific blanking band, the full pay PI ranges were different among States. For example, the full pay PI range for a zero blanking band for two highway agencies varied from between 236 and 315 mm/km (15 to 20 inches/mi) and 394 and 552 mm/km (25 to 35 inches/mi).

Highway agencies continually are using their experiences on paving projects to refine their smoothness specification. Hancock and Hossain presented an overview of how the PCC smoothness specification in Kansas evolved over the years.[15] The first PCC pavement with a smoothness specification was built in Kansas in 1985, and the Kansas DOT (KDOT) adopted the first standard specification in 1990. This specification was based on reducing the output from a 7.6-m (25-ft) California profilograph using a 5-mm (0.2-inch) blanking band. In 1990, KDOT noted a high frequency vibration on a PCC reconstruction project on I-70. Careful review of the profilograph trace revealed a sine-wave oscillation of about 2.4-m (7.9-ft) spacing with a 5.1-mm (0.2-inch) amplitude. However, most of the surface deviations were covered up by the 5-mm (0.2-inch) blanking bandwidth during the trace reduction that resulted in an acceptable PI. A project on Interstate (I)-470 indicated oscillation waves that were spaced at about 9.1 m (30 ft) with an amplitude of 5 mm (0.2 inch), which were again covered by the 5-mm (0.2-inch) blanking band.

These results prompted KDOT to study the effects of blanking bandwidth on trace reduction. Based on this review, KDOT adopted a zero blanking bandwidth. A zero blanking band is merely a reference line placed approximately at the center of the trace. The change in the blanking bandwidth resulted in a new PCC smoothness specification that was adopted in 1992. Further refinements in the smoothness specification occurred between 1992 and 1996. Before 1996, the smoothness specification was based on percentage of bid item. The smoothness specification was designed this way because concrete pavements were usually bid as unit item per square meter. In 1996, KDOT adopted a dollar value incentive scheme where a specific dollar amount was assigned as incentive for each 0.16 km (0.1 mi) section per lane depending on the obtained PI.

As indicated previously, most highway agencies use PI to judge the smoothness of a pavement for construction acceptance. Thereafter, they use a roughness statistic such as IRI to monitor the roughness of their pavement network. With these different profile indices, it is not possible to relate the roughness of the pavement at some point in time with its as-constructed smoothness. Currently, some SHAs are moving toward adopting a consistent measure of pavement smoothness that can be used throughout the life cycle of a roadway. This measure would involve using the same index to measure the pavement smoothness for construction acceptance as well as to monitor the pavement for pavement management purposes. Several SHAs have adopted IRI as this index. Using the same smoothness index to measure the smoothness for construction acceptance and thereafter to monitor the roughness over time will enable highway agencies to monitor the performance of a pavement from cradle to grave.

CHAPTER 3. DESIGN AND CONSTRUCTION FACTORS THAT AFFECT INITIAL SMOOTHNESS

INTRODUCTION

Factors that can influence the initial smoothness of a PCC pavement surface generally can be separated into the following categories:

- Pavement design factors.
- Concrete mix design.
- Construction operation.

The effect of each factor on pavement smoothness is described separately in the following sections.

PAVEMENT DESIGN FACTORS

Design features such as base type and base width, horizontal alignment, and embedded items in the pavement (dowels, steel reinforcement) can have an impact on the initial smoothness of a PCC pavement. The effect of each factor is described separately.

Base/Subbase

One of the most important design considerations for PCC smoothness is the provision of a stable and smooth track line.[16] The track line is the path that wheeled tracks of the slipform paving machine will follow while paving. Providing an even and stable track line is essential for constructing a smooth concrete pavement. Irregularities in the track lines cause the profile pan of the paving machine to continuously adjust its position relative to the machine frame and can cause bumps or dips on the pavement surface.

The simplest way to provide a stable and even track line is to design the base layer 1 m (3.3 ft) beyond the edges of the PCC slab.[17] For concrete overlay projects, special provisions may be necessary to stabilize the track lines as part of the preoverlay preparation. Subbase material stability is another important consideration. Materials stabilized with cement or asphalt and dense-graded granular materials create firm support for construction equipment. Unstabilized permeable layers have caused some placement and performance problems. An important balance must be met between the degree of drainage and the stability of an unstabilized subbase layer.

Horizontal Alignment

It is more difficult to construct a smooth surface for PCC pavements along horizontal curves than those on tangents because of the transitions for superelevation. Generally, roughness is more prevalent in transitions and superelevated portions of a horizontal curve than on tangents. In the transition sections, the profile pan must adjust to meet the varied cross slope requirements of the curve. As with an uneven track line, the constant adjustments of the paving machine can adversely affect the smoothness of the pavement.

As the horizontal curvature increases, the potential for roughness within the curve increases. When the degree of curvature exceeds 6 degrees (or the radius of curvature falls below 300 m (984 ft)), the contractor must focus increased attention to the machine operation and the stringline-staking interval. It has been suggested that when the curvature exceeds 7 degrees, it is virtually impossible to construct the surface to the same specified tolerance desirable for a tangent section because of the significant corrective adjustments necessary by the equipment.(17,18) Pavement smoothness on horizontal curve sections can be improved by reducing the distance between staking rods.

Embedded Items

Plain jointed concrete pavements carrying truck traffic are usually equipped with dowel bars. There are two procedures for incorporating dowel bars into a PCC pavement: inserting the dowel bar into the plastic concrete during construction or placing dowel baskets on the base before placing the concrete. Steel reinforcements are used during construction of jointed reinforced concrete pavements and continuously reinforced concrete pavements. The use of embedded items such as dowel bars and reinforcements can affect the smoothness of the pavement. According to ACPA, four main conditions can cause roughness because of the use of dowel bars and reinforcement.(17) These conditions are lack of consolidation, reinforcement ripple, springback, and damming:

- Lack of consolidation: The presence of dowel baskets may result in a lack of consolidation of concrete in the dowel basket. If this occurs, the concrete may settle over the dowels, creating a rough surface.

- Reinforcement ripple occurs when concrete is restrained by the reinforcing bars, resulting in a ripple on the surface, with the surface slightly lower near each bar than between bars. The degree of surface rippling generally depends upon the finishing techniques and depth of cover to the reinforcement, with less cover producing more prominent rippling.

- Springback occurs when the dowel basket assembly deflects and rebounds after the profile pan of the slipform paver passes over the dowel basket. This action results in a slight hump in the concrete surface just ahead of the dowel basket. It is thought that this effect is more pronounced where agencies require dowel basket spacer wires to be cut before paving, which weakens the assembly.

- Damming typically occurs when paving down steep grades or on lesser grades with a low-friction paving surface. In these cases, the dowel basket assembly or transverse steel can act as a dam on the grade, causing bumps to form at the basket or transverse steel.

For local roads, additional embedded items may include utility boxouts, cast-in-place fixtures, traffic signal handholds, and drainage structures. These items may affect the pavement smoothness because they require extra handwork vibration and finishing efforts to blend them into the surrounding pavement surface. Ideally, in-pavement objects should be in position before placing the concrete to minimize any handwork.

CONCRETE MIX DESIGN

Concrete mix design can have a significant effect on the smoothness of concrete pavements. (See references 17, 19, 20, and 21.) The ACPA publication *Constructing Smooth Concrete Pavements* states: "The concrete mixture should be proportioned to assure proper consolidation without excessive vibration. This is achieved through optimization programs that develop mixtures containing well-graded aggregates. These mixtures are not harsh and unworkable, and they flow easily when vibrated, and consolidate well around embedded fixtures and reinforcement."[17]

When evaluating concrete mix design and quality, many elements need to be considered. These elements include uniformity, workability, finishability, strength, durability, economics, and time required for initial set. The elements having a direct impact on the as-constructed pavement smoothness are uniformity, workability, finishability, and time required for initial set. Strength and durability will influence the roughness progression over time. The following sections describe the ways in which these factors related to mix design could influence pavement smoothness.

Mix Design and Proportions

The concrete mix design can affect the smoothness achieved during pavement construction. The concrete mix design influences the workability and slump of concrete, which have a direct impact on the ease of concrete placement and finishing. The concrete mix design will influence how well the slab can be extruded by the paver into its final configuration.

Harsh concrete mixes slow down the paving operation and require extra effort for finishing. The equipment needs to work harder to spread and extrude harsh concrete mixes. The pavement will require more finishing, resulting in a rougher surface. A harsh and unworkable mixture can override other attempts by a contractor to attain a smooth surface.

A quality concrete mix must have the proper blend of good quality aggregates, appropriate water to cement ratio, and proper air entrainment system for durability. Materials in the mix must be uniform in nature. In addition, to achieve better smoothness, concrete mixes should contain enough fines to relieve harshness. Some contractors have suggested that concrete mixes with a slump between 50 to 65 mm (2.0 to 2.6 inches) work best to construct smooth pavements.[19]

In the 1990s, distresses appeared in some newly constructed concrete airfield pavements. After investigating these pavements, the Air Force Civil Engineering Support Agency concluded that in general, the concrete designed with well-graded combined aggregates performed better than that with gap-graded or poorly graded aggregates. Consequently, the U.S. Air Force has specified that Shilstone-type mix design should be used for rigid airfield pavement construction.[22]

The use of well-graded aggregates can reduce water demand and improve concrete workability, finishability, and strength. In the Shilstone-type mix design procedure, the gradation of the combined aggregate is evaluated graphically to see whether the gradation is acceptable.[23,24] This procedure is illustrated in figure 11. The percentage retained for each reporting sieve size (Y-axis) is plotted versus the considered sieve size (X-axis). The resulting line should have a relatively smooth transition between coarse and fine aggregates. The dotted lines in the figure represent the maximum and minimum percent retained on each sieve size that is used as a guide by the U.S. Air Force.[22] Figure 11 shows two lines representing satisfactory and unsatisfactory combined aggregate gradations.

Figure 12 shows the plot of workability factor versus coarseness factor, another criterion that is evaluated in the Shilstone mix design procedure.[24] The coarseness factor for a combined aggregate gradation is determined by dividing the amount retained above the 9.5-mm (0.4-inch) sieve by the amount retained above the No. 8 sieve, and multiplying the ratio by 100. The workability factor is the percentage of combined aggregate finer than the No. 8 sieve. The plot has five zones; the unique characteristics of each are:

- Zone I—This zone represents a coarse gap-graded aggregate with a deficiency in intermediate particles (passing the 9.5-mm (0.4-inch) sieve and nominally retained on the No. 8 sieve). The aggregate with a gradation in this zone has a high potential for segregation during concrete placement.

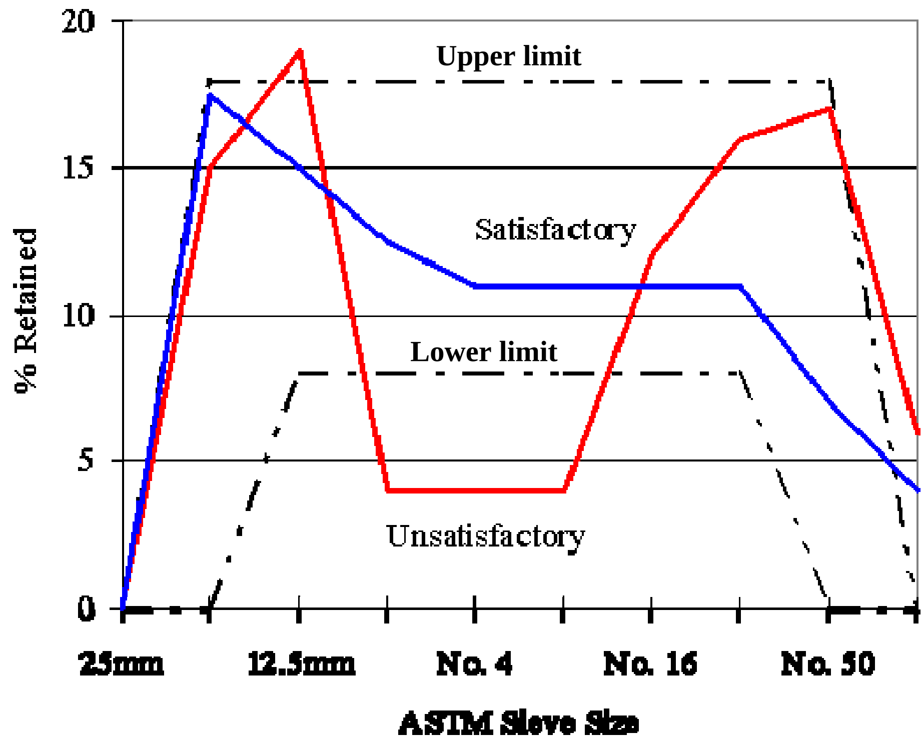
- Zone II—This is the optimum zone for concrete mixtures with nominal maximum size from 38 mm (1.5 inches) through 19 mm (0.75 inch).

- Zone III—This is an extension of the Zone II mix for finer mixtures with nominal maximum size less than 19 mm (0.75 inch).

- Zone IV—Concrete mixtures in this zone generally contain excessive fines, with high potential for segregation during consolidation and finishing.

- Zone V—This is a mixture with too much coarse aggregate, which makes the concrete unworkable.

The unlabeled zone on top of Zone V is referred to as the trend bar. It is the transition zone between Zone V and the other zones on top of the trend bar.



1 mm = 0.039 inch

Figure 11. Percent of combined aggregate retained.(22)

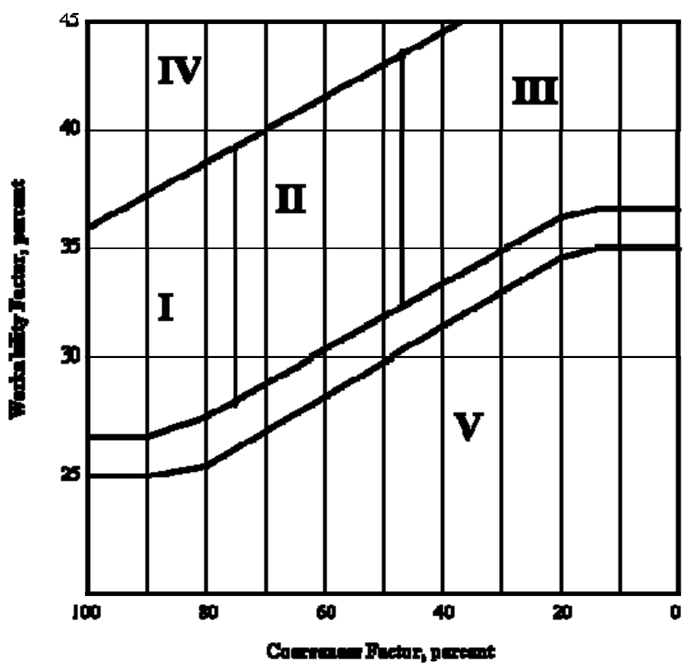


Figure 12. Workability factor versus coarseness factor chart.(24)

Aggregate

Aggregates play a major role in affecting the workability of a concrete mixture and thus affect pavement smoothness. Aggregate factors that affect pavement smoothness include gradation, particle shape, stockpiling and handling of aggregate, and moisture in the aggregate.

It is generally recognized that consistent gradation of aggregate is essential for producing concrete mixes with better smoothness. Variability in gradation can cause a change in water demand, resulting in inconsistent workability of the mix from batch to batch. An inconsistent concrete mix affects the paving operation directly because the paver must make constant adjustments, resulting in a rougher pavement. Poor aggregate gradation decreases mix workability and often causes segregation of the concrete mix.

Aggregate particle shape can also influence the workability characteristics of concrete mixtures, such as consolidation, flow, and the finishability. Natural (rounded) aggregates produce more workable concrete mixes, while angular manufactured aggregates can produce harsh mixes, and thus result in rougher surfaces.

Attention to aggregate stockpiling and handling is also important for producing good quality concrete mixes. Improper stockpiling and handling of aggregate can cause segregation in the concrete mix, which makes it impossible to produce uniform concrete. In addition, slump, yield, and air content can also be affected.

Moisture content of the aggregate plays an important role in obtaining a uniform and quality concrete mix. Uncontrolled aggregate moisture can lead to a loss of concrete uniformity. Variable moisture content in the aggregate can produce concrete mixes with variable slump. If not accounted for in the mix design, excessive moisture in the aggregate will likely reduce concrete strength and increase the shrinkage cracking potential because of the high water-to-cement ratio.

Admixtures

Admixtures in concrete mixes have a direct impact on concrete quality and properties and thus can influence pavement smoothness. Experience has shown that different types of fly ash and the use of water reducing and air-entraining agents affect the workability of concrete mixes, and hence the smoothness of the pavement. Fly ash in concrete can improve the workability of the mix because of the smaller size and spherical shape of fly ash particles. Although fly ash usually slows the early strength gain of concrete, it can ultimately increase the long-term concrete strength.

An air-entraining agent improves the durability of the concrete and has a positive effect on pavement smoothness because it increases the slump (workability) of the concrete. In general, air content in the 5 to 7 percent range helps improve the workability. Concrete mixes with low air content tend to bleed, whereas mixes with high air content will make the concrete mix sticky and unworkable.

The use of a water reducer in the concrete mix also has a positive impact on pavement smoothness because it will result in a lower water-to-cement ratio and better mix consistency. The use of retarder admixtures is also beneficial to the concrete mix because the retarders can keep the mix workable in hot and/or windy weather or when long-distance transportation is required.

Influence of Ride Specification

Since the mid-1980s, most State DOTs have developed and implemented a ride quality specification for concrete pavements. This study investigated whether modifications made to concrete mix designs improved smoothness yet had a detrimental influence on other concrete properties such as strength and durability. A literature search was conducted, then State DOT personnel and industry organization personnel were contacted to explore whether changes in concrete mix designs have been necessary after the implementation of a ride specification.

The literature search did not identify any documents that directly address the issues of pavement smoothness related to concrete mix design. The following organizations were asked for their input on this issue: Florida DOT (FDOT), Colorado DOT (CDOT), Virginia DOT (VDOT), Iowa DOT, Northeast Chapter of ACPA, and Western Pennsylvania ACPA. Most respondents said they believed that no modifications have been required in the concrete mix design to achieve higher smoothness. They also reported that no pavement performance problems have been encountered because of the implementation of a smoothness specification. However, it was mentioned that better mix designs with a more uniform aggregate gradation might improve the constructability of the concrete mix, and hence result in higher smoothness.

A representative from FDOT indicated that Florida has implemented a smoothness specification for concrete pavement construction. After FDOT personnel approve the mix design for a project, it cannot be changed without the approval from the FDOT engineers. No major performance problems have been reported on recently constructed PCC projects.

A CDOT representative indicated that as long as the concrete mix design meets the strength specification, it should not require any modifications for smoothness specification. It was suggested that smoothness is mostly affected by the paving equipment and the construction process.

A representative from VDOT indicated that Virginia has implemented a ride quality specification for concrete pavement construction. In the past, contractors have not consistently achieved the specified degree of smoothness (in terms of IRI) during construction. However, improved pavement designs and concrete placement practices have resulted in more projects meeting or exceeding the smoothness criteria. In Virginia, contractors generally have the flexibility of proposing the concrete mix design for paving operations. The proposed mix design requires approval from VDOT personnel. The State utilizes performance-based specifications for its concrete mixes, using strength and air content as the acceptance criteria.

The Executive Director and Chief Operating Officer of the Northeast Chapter of ACPA stated that concrete mixes with more uniform combined aggregate gradation, such as those proposed by Shilstone, have improved workability and other properties that promote better pavement smoothness. Using this type of mix design has these advantages over traditional concrete mix designs:

- It produces more stable concrete mixes.

- The concrete will have less shrinkage and hence may reduce moisture warping of the concrete slabs in the early stages. Less shrinkage can reduce the built-in pavement roughness that occurs due to warping and curling.

- Compared with conventional mix designs, this concrete mix responds better to vibration and requires less energy to consolidate.

- Concrete mixes with a more uniform aggregate gradation will hold up better after vibration.

- Well-graded mixes are more cost effective because they minimize the paste (the most expensive part of the mix) and maximize the aggregate (the least expensive part).

- Based on the previously described factors, well-graded mixes should be stronger and have better long-term durability.

The Director of Paving Services for Western Pennsylvania ACPA agreed that using more uniform aggregate gradation in concrete mix design might reduce shrinkage and therefore slab warping. The better aggregate gradation will also improve workability (not slump) of the concrete mix. However, the impact of these improvements on pavement smoothness has not been studied and therefore cannot be determined with much confidence.

Although not directly related to the use of ride specification for concrete pavement construction, input from Iowa DOT indicated that this State has adopted concrete mix design procedures based on the Shilstone approach since 2000. Some past mix designs might have been produced with a high sand content or with gap-graded aggregate. By using more uniformly graded aggregates in the mix design, the concrete mixture is more workable, requiring less water in the mix, and resulting in reduced shrinkage. The mixtures also require less vibration energy to achieve the desired consolidation. For example, the combined aggregate gradation of the concrete mix for a concrete pavement with poor performance is shown in figure 13, which indicates that the aggregate gradation is in the unsatisfactory category (i.e., percentage retained for some sieve sizes fall within the lower band). When the coarseness factor and the workability factor of the combined aggregate for this concrete mix are plotted, the point falls within Zone I shown in figure 12. This zone is associated with gap-graded mixes, which have a high potential for segregation. Similar plots were also observed on several other concrete projects exhibiting poor performance.

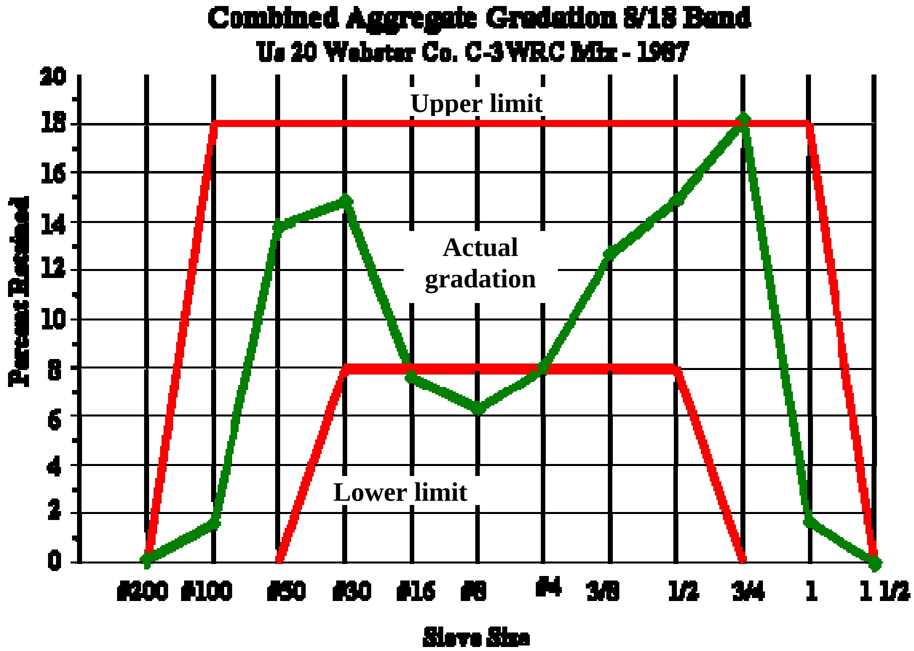


Figure 13. Aggregate gradation for a concrete project with poor performance.

CONSTRUCTION OPERATIONS

The key construction factors that can influence pavement smoothness are:

- Stringline setup and maintenance.
- Grade preparation.
- Concrete consistency.
- Concrete delivery.
- Construction equipment.
- Slipform paver operation.
- Finishing, texturing, curing, and headers.
- Dowels baskets and reinforcement.
- Vertical grades and curves.
- Skilled and motivated crew.

Stringline Setup and Maintenance

A stringline is used to provide an accurate reference for elevation and alignment control of the subgrade surface, placing the base layer, and concrete paving. The stringline is the primary guidance system for a slipform paver operation. The paver's elevation sensing wand rides beneath the string, and the alignment-sensing wand rides against the inside of the stringline. Accurate elevation of the stringline is essential because the sensors controlling the profile pan adjust based on the location of the stringline. Therefore, close attention to the stringline setup and maintenance is required to achieve a smooth pavement surface. Neither the elevation sensing wand nor the alignment sensing wand should measurably deflect the stringline.(17)

The interval between stringline stakes is also important. On tangent sections, a maximum staking interval of 7.6 m (25 ft) typically produces excellent results. Decreasing this interval in horizontal and vertical curves is usually necessary to construct a smooth pavement.(17) Guidelines for staking intervals on horizontal and vertical curves are presented in reference 17.

Associated stringline factors such as stringline material, staking interval, splices, and repositioning frequency can affect the pavement smoothness. Acceptable stringline materials include wire, cable, woven nylon, or polyethylene rope.(17) Air temperature and relative humidity variations during the day can cause sag in the stringline between the stakes.(17) More tension in the stringline allows less sag to occur, even with substantial changes in weather conditions. The staking system normally includes hand winches placed at intervals not more than 300 m (984 ft), allowing the line to be tightened to prevent sagging between stakes.

The splices in the stringline need to be clean and tight. Stakes for securing the stringline should be long enough to be able to firmly embed the stake below the subgrade surface. The above-grade stake length must be adequate to allow adjustment of the stringline to the desired height above the subgrade profile.

Care must be taken to avoid tripping over or disturbing the stringline. If the stringline is disturbed, it must be checked immediately and repositioned to avoid bumps or dips in the pavement. Regular visual inspection of the stringline is recommended. If problems are detected during this inspection, surveying equipment should be used to check the grade of the stakes and stringline.(25) Stringline setup should be kept to a minimum during a project. A reduced number of stringline setups can lead to better smoothness control.

In many paving operations, stringline guidance is setup on both sides of the paver. Advantages offered by using dual stringlines include the ability to see problems before paving, control of yield loss, and the assurance of greater accuracy of the finished grade.(26) ACPA also reports the use of dual stringlines can be beneficial for wide sections with a uniform cross slope. In these cases, small deviations in a single stringline can propagate into large variations in the surface elevation on the other side of the paving machine.(17)

Grade Preparation

Base course placement is considered a very important element affecting concrete pavement smoothness. Good uniformity and stability of the base course is essential for achieving smooth pavements. Because concrete pavements are usually built in one layer, there is only one opportunity to average out roughness from the base surface. It is critical that the base surface be as true to grade as possible before placing the concrete.

For a granular base layer, automated fine-grading equipment guided by a stringline will create the best base surface possible. Fine-grading equipment is easily capable of meeting specifications within a tolerance of ±12 mm (±0.5 inch), when controlled by a stringline. Close control is also required for constructing stabilized base to facilitate pavement surface smoothness. However, instead of using fine-grading equipment, an operator can use skill and experience, which often contributes significantly to meeting surface tolerances for stabilized base materials.

Concrete Consistency

The importance of consistent concrete in ensuring a smooth concrete pavement must not be underestimated. Good batch-to-batch consistency of the concrete mixture improves the quality of the finished pavement because it affects how paving equipment performs. The main goal is to avoid alternating wet and dry concrete batches, which would induce constant equipment adjustment and make it difficult to produce a smooth pavement surface.(19)

Stationary (ready-mix) plants, onsite batching and mixing plants, and truck mixing operations are all capable of producing concrete with consistent properties. However, to achieve consistency between batches, it is important to follow quality control procedures during batching, mixing, hauling, placing, and finishing. A regularly scheduled review of each phase in the operation can identify problems.

Another important item for consideration in producing consistent concrete mixes is sound moisture management. A concrete mixture is sensitive to increases in moisture, especially from free moisture in the fine aggregate. Normally, the moisture content of the sand has more impact on the concrete mix than the moisture content of the coarse aggregate. A mix adjustment to account for the free water in the sand may be required to maintain the water-to-cement ratio and workability.

According to ACPA, the following are important considerations that can help control batch-to-batch variations in concrete consistency:(17)

- Remove free water from drums or dump beds after washing or rainfall.

- Maintain a consistent, clean operation of the front-end loaders and other heavy equipment moving aggregate at the batch plant. Remove aggregate consistently from the stockpiles to maintain consistent aggregate moisture.

- Water coarse aggregates stockpiles as necessary. Some coarse aggregates used in concrete are highly absorptive, requiring a significant quantity of water to create the saturated surface dry condition assumed in the mixture design.

- Measure moisture content. Moisture sensors mounted on the mixing plant's aggregate bins can detect the moisture content for the plant's computer. The computer can then make corrections to the added mix water and attain better uniformity between batches. In the absence of an automated moisture sensing system, moisture tests must be performed on the fine and coarse aggregate on a daily basis.

Concrete Delivery

Once the concrete has been mixed, uniform delivery of the concrete to the job site has a direct impact on the smoothness of the finished concrete pavement.(27) A slowdown or stoppage in the paving process due to a lack of concrete results in bumps or dips on the pavement surface. Consistent delivery of concrete to the paving project is essential for keeping a steady operation. Keeping a consistent delivery schedule is particularly challenging in urban areas; it requires careful evaluation to predetermine whether traffic delays will hamper concrete delivery.

Feeding concrete to the paving machine consistently requires an adequate number of delivery trucks. The delivery of concrete dictates the slipform paver speed. The entire cycle of mixing, discharging, traveling, and depositing concrete must be coordinated for the mixing plant capacity, hauling distance, and paving machine capability.

In a study performed in Argentina, pavement smoothness measurements were obtained on eight concrete pavement sections using an inertial profiler.(28) IRI computed from the profile data was used to judge the smoothness of the pavement. The variables evaluated in this study included the type of paver, the rate of concrete placement, concrete delivery, and concrete plant type. One conclusion from the study was that concrete production and delivery had an effect on concrete pavement smoothness. The pavement sections with shorter concrete delivery distances showed lower IRI.

Construction Equipment

The use of proper paving equipment is also a requirement for constructing smooth concrete pavements. Attention must be paid to function, weight, and size in selecting the most appropriate equipment for the job.(29)

On many mainline paving projects, contractors use a paving train that includes a placer/spreader in front of the slipform paver. A paving train is required when constructing a continuously reinforced concrete pavement. A placer/spreader is beneficial for attaining smoothness. A placer/spreader provides a consistent amount of concrete in front of the paver and allows the paver to maintain a constant speed.

Clean and well-maintained equipment is also key to achieving an efficient and quality paving operation, which in turn affects pavement smoothness. Not only does this refer to paving equipment, but also to mixing and delivery equipment. A very high correlation has been established between equipment condition and pavement smoothness. Loose parts or connections result in poor paver response to changes in trackline and base conditions and can result in poor pavement smoothness. Poorly maintained equipment is also more prone to breakdown, resulting in unexpected stops.

All other things being equal, a heavier paver generally produces a smoother pavement because it is less affected by surges of concrete coming into the paver. In a study conducted in Argentina, eight concrete pavement sections were compared on three highway and airport facilities that were constructed using slipform pavers with different weights. One of the study's conclusions is that pavement sections constructed with the heavier pavers had lower IRI values.[28]

A heavier paver is generally stiffer and more resistant to lifting/buoyancy forces from the concrete. The increasing trend of using end result strength specifications is resulting in stiffer concrete mixes. It is therefore important that the weight of the paver is adequate for placing a stiffer concrete mix. Paver size may also have an effect on concrete pavement smoothness. A larger paver of the same capacity will provide a smoother pavement because it can handle concrete more easily and respond more precisely.

Slipform Paver Operation

A slipform paver spreads and consolidates the concrete as it moves forward. Many factors can affect the operation of the slipform equipment and influence the pavement smoothness. Controlling concrete consistency (mix control) and delivery will result in a steady operation and a smoother surface. The paver cannot produce adequate results if it must stop often or push a large pile of concrete. Set out below are the essential factors that must be followed for constructing smooth pavements. (See references 17, 28, 30, and 31.)

Proper Paver Setup

All slipform paving equipment contains molding components, including a profile pan and side forms. The base or subbase is at the bottom of the mold. These elements confine the concrete and create the mold for the pavement shape. A slipform paver also contains tools that help fill the forms and create a uniform shape. These tools include the auger spreader, spreader plow, strikeoff, and/or tamper bar. Setting up the paver is an important factor affecting the smoothness of the finished pavement surface. The proper steps in setting up the paving equipment must be followed. Attention also must be given to ensuring effective adjustments during the paving operation.

Adequate Paver Travel Speed

A constant paver speed is essential for producing a smooth pavement. There are no set standards for the proper paver speed. The paver speed is proper when it can be supplied with a constant supply of concrete without having to slow down or stop during the paving process. It is essential to avoid stopping the paver during the paving operation to reduce the bumps or dips in the pavement surface.

Vibration of Concrete

In slipform paving, the mold is forced through concrete that remains static on the grade. However, vibrators mounted to the slipform machine are essential to fluidize the concrete and make it easier to mold. The slipform paver passes over the fluidized concrete, with its mass keeping the pan and side forms steady to confine and shape the material. In a steady paving operation, vibration of the concrete as it passes through the paver influences the surface and the resulting smoothness. Too much vibration causes pumping and fluffing at the rear of the pan and brings excess grout to the surface. Insufficient vibration creates drag-out and surface voids.

Vibration of concrete is not a cure for other problems in the concrete mix. Vibrators may identify and exacerbate a concrete mixture problem. Adjusting the frequency of vibrators will not overcome poor equipment set up, poor alignment, or poor mixtures. When operating at a very high frequency, vibrators may cause undesirable results such as loss of air entrainment or vibrator trails.

Concrete Head

Maintaining a consistent and adequate head of concrete in front of the paver can improve the smoothness of a pavement. Maintaining a consistent head of concrete will ensure consistency in the speed of paving, vibration, and consolidation. A consistent head of concrete evens the pressure on the paver so that when required, adjustments to the paver can be made more accurately. If the head of concrete gets too high, it might create a pressure surge under the paving machine that can cause the concrete behind the paver's profile pan to rise up and create a bump. If not enough concrete is placed in front of the paver, the concrete head may run out or the grout box may run empty, creating low spots or voids on the pavement surface.

Communication

Effective communication measures between machine operators and between the paving operation and the concrete batch plant must be implemented. Constant communication between the plant and the paving operation allows the slipform operator to match the paver speed to accommodate concrete delivery.

Finishing, Texturing, Curing, and Headers

Finishing

If the paver has been operated properly, only minimal hand finishing is required.[25] In many cases, the finished profile of a pavement is worsened by hand finishing. Hand finishing the pavement surface using bullfloats is only necessary when the surface contains voids or imperfections. Some contractors overuse mechanical longitudinal floats directly behind the slipform equipment. In general, it is best to limit hand and mechanical finishing. If longitudinal floating is the only method to produce an acceptably closed surface, some corrections are needed to the concrete mixture and/or paving equipment.[17]

Checking the surface behind the paver with a 3- to 7.6-m (9.8- to 25-ft)-wide hand-operated straightedge is a recommended procedure to finish the concrete.[17] Successive straightedging should overlap by one half the length of the straightedge to ensure that high spots are removed and low spots are filled.[17] Experienced finishers can use the straightedge to remove or reduce noticeable bumps by using a scraping motion.[17] In some cases, the profile of the finished pavement shows surface waves that were likely induced or augmented by the improper use of a straightedge.

Texturing

The surface texturing operation does not usually affect pavement smoothness. However, transverse tining equipment has been known to cause surface roughness when rakes do not run over the surface evenly.

Curing

In a recent study, profiles were measured on eight test sections on four newly constructed concrete pavements located along I-70 and I-135 in Kansas.[32] The pavements, with concrete slab thicknesses of 275 and 312 mm (10.8 and 12.3 inches), were placed on top of a stabilized drainable base that was on a lime-treated subgrade. The effect of four different parameters on as-constructed pavement smoothness, as represented by IRI, was assessed. The four parameters were lane (driving and passing), wheel path (left and right), curing condition (single or double curing compound applications), and time of paving (morning or afternoon). Various statistical techniques were employed to analyze the IRI data. Results of the study showed that application of curing compound was the most significant factor affecting the IRI value, with lower IRI being obtained for the double application of the curing compound.

Headers

Headers (transverse construction joints formed at the end of a day's work) are one of the most significant contributors to concrete pavement roughness. Most pavers leave a dip in the slab when they lose the head of the concrete. Headers typically require hand finishing, which makes controlling tolerances harder.

Two possible solutions have proven effective in minimizing ride problems at a header. The first method is to avoid forming headers and use a cut back method to create the joint. In this method, the paving operation continues until all the concrete is used. The following morning, a transverse sawcut is made 1.6 m (5.2 ft) or more from the end of the hardened concrete slab. The end material is removed, and dowels are grouted into holes drilled into the smooth face. The second method is to control the elevation of the header very carefully to properly align and finish the header to correct grade. Then, the paver is pulled off slowly to permit the concrete to set in place.

Dowel Baskets and Reinforcement

Dowel baskets and steel reinforcement in a pavement can adversely affect pavement smoothness. Pinning the basket assemblies securely to the grade is essential so that they can withstand the pressure applied by the concrete during placing.[33] The paver must be heavy enough not to ride up over basket assemblies. Proper dumping or placement techniques and adequate vibration frequency and depth can also minimize dowel-related pavement roughness. The use of an automated dowel bar inserter for dowel placement may help prevent dowel-related pavement roughness by eliminating the movement of the dowel baskets due to the extreme pressure. Other problems related to smoothness that can occur with dowel bars and reinforcements were discussed previously in the section dealing with embedded items.

Vertical Grades and Curves

There are no known differences between paving uphill and downhill.(17) It is more difficult to construct smooth pavements on grades exceeding 3 percent than on flatter grades. According to ACPA, the following adjustments may be necessary during paving operations on steep grades:(17)

- The slump of the concrete should be reduced if it exceeds 12 to 25 mm (0.5 to 1 inch) and if it is difficult to maintain a uniform head of concrete in front of the paver.

- The attitude, draft, or angle of attack of the paver may be adjusted when paving on steep grades. On flat grades, most operators position the profile pan parallel with the stringline. When paving up a steeper grade, the pan may be adjusted to about 50 mm (2 inches) below the surface grade. When paving down a steeper slope, the pan may be adjusted to about 50 mm (2 inches) above the surface grade.

- On tangent sections, a maximum staking interval of 7.6 m (25 ft) produces excellent results. A tighter interval is necessary to produce smooth pavements on vertical curves and should be determined based on the rate of change of curvature. Procedures for determining the staking interval for such cases are presented in reference 17. The stringline is set on chords, and the paving elevation is on a semichord when paving on grades for vertical curves.

Skilled and Motivated Crew

Regardless of the equipment and processes, constructing a smooth pavement requires experienced and motivated personnel. Crew training is vital, especially in subjects that directly affect smoothness. Stringline personnel need math skills and a keen eye, and operators need to understand what equipment activities affect pavement smoothness. In a study conducted in Argentina, the IRI values were reported to have decreased chronologically for paving projects.(28) The authors concluded that one reason for the improvement in pavement smoothness was the increased experience and knowledge the crew gained from performing previous jobs.

CHAPTER 4. ANALYSIS PROCEDURES

INTRODUCTION

The analyses performed for this project can be categorized as (1) analysis of LTPP data and (2) analysis performed on data collected at five test sections to determine short-term changes in roughness. This section presents an overview of the analysis approach and procedures that were used.

ANALYSIS OF LTPP DATA

The LTPP program was designed as a 20-year study of pavement performance. One aspect of the LTPP program is the General Pavement Studies (GPS) program, which studies the performance of inservice pavement test sections. Under the GPS program, more than 800 test sections were established on inservice pavements in all 50 States of the United States and in Canada. In the LTPP program, jointed plain concrete (JPC) pavements are studied in the GPS-3 experiment. Profile data are being collected at regular intervals at these test sections using an inertial profiler. Roughness values computed from the profile data provide an excellent source for investigation of roughness progression of JPC sections.

The following analysis approach was used to analyze data from the GPS-3 sections for this study:

1. Obtain the time-sequence IRI values from the LTPP database. Plot time-sequence IRI values with pavement age to examine roughness progression of pavements.
2. Divide pavement sections into different groups based on rate of increase of roughness.
3. Evaluate the effect of slab curvature on roughness values.
4. Compare design and concrete material parameters among the different groups to identify factors contributing to a higher rate of increase of roughness.

The data collected for the LTPP program are stored in the LTPP Information Management System (IMS) database. Data from the following data categories were used in this analysis: inventory, climatic, monitoring, traffic, and materials testing.

- Inventory data tables contain information related to location of the section and historical information about the section. Data elements such as construction date, joint spacing, load transfer type, and paving method are contained in these tables.

- Climatic data at the GPS sections are derived from weather data collected by the National Oceanic and Atmospheric Administration and the Canadian Climatic Center. Data collected from five weather stations close to each GPS section are used to derive these climatic parameters. Climatic data elements used in this analysis were annual precipitation, mean annual temperature, annual freezing index, freeze-thaw cycles per year, number of days in the year when temperature is greater than 32 °C (90 °F), and number of days in a year when temperature is less than 0 °C (32 °F).

- Monitoring data tables include data obtained by monitoring activities performed at the test sections. This analysis used profile data collected at the test sections as well as the IRI values computed from the profile data. Data collected from pavement distress surveys and fault measurements were also used in this analysis.

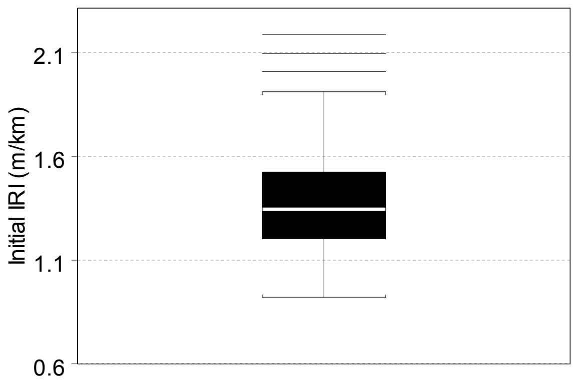
- Traffic data tables contain historical traffic estimates provided by SHAs as well as monitored traffic data collected by weigh in motion equipment. These data were used to obtain cumulative equivalent single axle loads (ESALs) at the test sections.

- Materials testing tables contain laboratory test data for pavement and subgrade materials. The data elements in these tables used for this analysis were elastic modulus of concrete, split tensile strength of concrete, and compressive strength of concrete.

The analysis database used for this project was built using Microsoft® Access 2000. The data tables identified for analysis were obtained from IMS in January 2003. Multiple data values were usually available for material test parameters at each test section. For example, at a test section several cores were obtained for testing and compressive strength values for all tested cores were available in the database. For cases where multiple data values were available for a data element, the values were averaged to obtain a unique value for the test section.

For this analysis, the mean IRI of the test section (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 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.

In the analysis, box plots were used to examine the distribution of data. A box plot is a graphical procedure for illustrating the distribution of data. A box plot shows the center and the spread of the distribution of data as well as outliers. Figure 14 presents a box plot that shows the distribution of the IRI value obtained from a group of test sections. The horizontal line in the interior of the box is located at the median of the data. The top of the box is the 75th percentile value, the bottom of the box is the 25th percentile value, and the difference between these two percentile values is referred to as the interquartile distance. The whiskers (lines extending from the top and bottom of the box) extend to the extreme values of the data, or to a distance of 1.5 times the interquartile distance from the center, whichever is less. For data having a normal distribution, approximately 99.3 percent of the data falls inside the whiskers. Data points that fall outside the whiskers may be outliers, which are indicated by horizontal lines. The box plot shown in figure 14 indicates the median IRI value of the data set to be 1.34 m/km (85 inches/mi). The 25th and 75th percentile values indicated by the lower and the upper limits of the box are 1.19 and 1.50 m/km (47 and 95 inches/mi), respectively. The horizontal lines above the top whisker are outliers.



1 m/km = 63.4 inches/mi

Figure 14. Example of a box plot.

CONCRETE SLAB CURVATURE

The shape of concrete slabs in a pavement can vary depending on the temperature and moisture gradient within the slab as well as the built-in curl in the slab. The concrete slab can take these shapes: (1) curled up, when the joints are at a higher elevation than the center of the slab, (2) curled down, when the center of the slab is at a higher elevation than the joints, or (3) flat. A procedure for determining the curvature of PCC slabs using data collected by inertial profilers was presented by Byrum.[10] In this procedure, a parameter called the Curvature Index (CI), which is calculated from profile data, is used to represent the amount of curvature in PCC slabs. This procedure was used in this analysis to evaluate the effect of slab curvature on roughness. This index not only captures slab curling or warping present on the slabs; it also captures any other curvature that will be present within a slab. A description of CI is presented in appendix A. The sign convention for CI is shown in figure 15; CI will have a positive value if the slabs are curled up and a negative value if the slabs are curled down.

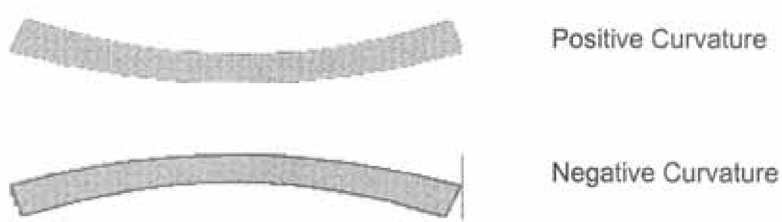


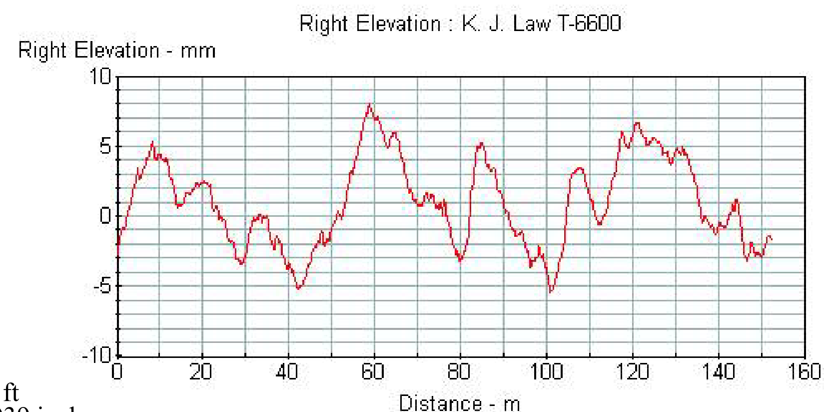
Figure 15. Sign convention for slab curvature.

DATA FILTERING

The profile data available in the LTPP database have been subjected to either a 91-m (300-ft) or a 100-m (328-ft) upper wavelength cutoff. Data collected by the K.J. Law DNC 690 profilers have been subjected to a 91-m (100-ft) upper wavelength cutoff filter, whereas data collected with either K.J. Law T-6600 or International Cybernetics Corporation (ICC) profilers have been subjected to a 100-m (328-ft) upper wavelength cutoff filter. These data need to be filtered further to see details in the profile data.

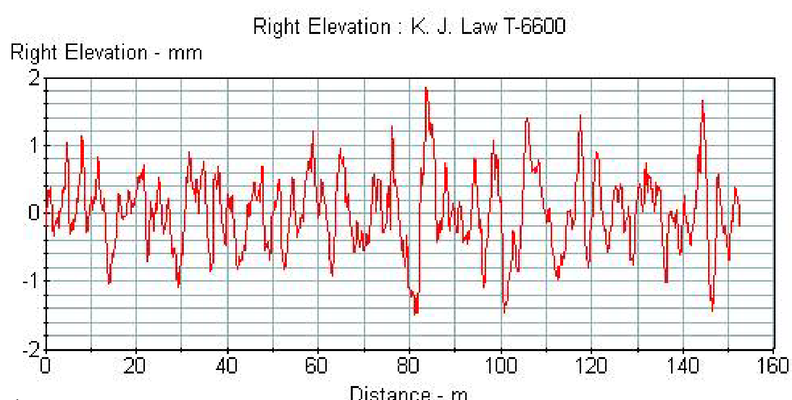
Filters commonly used in profile analysis are high-pass filters, low-pass filters, and band-pass filters. A high-pass filter removes wavelengths greater than a specified value. A low-pass filter removes wavelengths lower than a specified value. A band-pass filter keeps the wavelengths within a specified waveband and removes the other wavelengths.

Figure 16 shows the plot of a typical profile obtained from the LTPP K.J. Law T-6600 profiler. Figures 17, 18, and 19, respectively, show this profile after it has been subjected to a 5-m (16-ft) high-pass filter, a 10-m (33-ft) low-pass filter, and a band-pass filter with a lower wavelength of 5 m (16 ft) and an upper wavelength of 10 m (33 ft). The profile plot shown in figure 17 has all wavelengths that are greater than 5 m (16 ft) removed. The profile plots shown in figure 18 have all wavelengths less than 10 m (33 ft) removed. The plot shown in figure 19 contains only the wavelengths between 5 and 10 m (16 and 33 ft). Filtering techniques can be used to evaluate the presence of curling or warping in slabs and also investigate profile features that may influence roughness. A detailed description of profile data filtering is contained in reference 5.



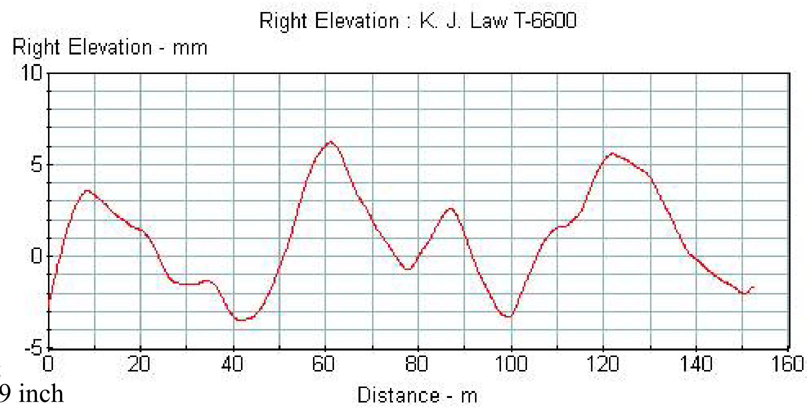
1 m = 3.28 ft
1 mm = 0.039 inch

Figure 16. Profile recorded by profiler.



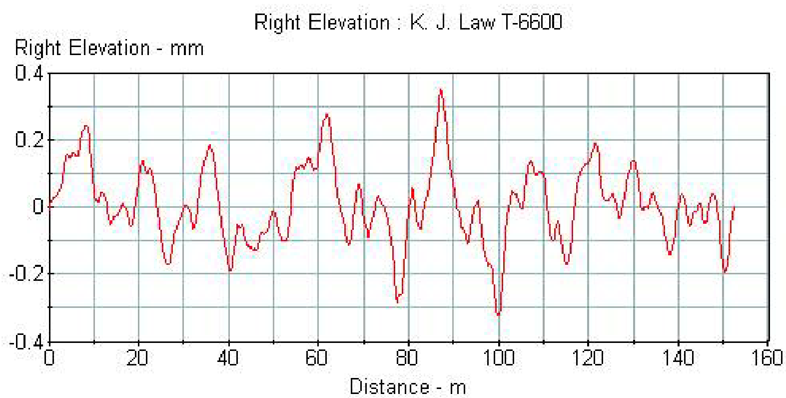
1 m = 3.28 ft
1 mm = 0.039 inch

Figure 17. Profile after being subjected to a 5-m (16-ft) high-pass filter.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 18. Profile after being subjected to a 10-m (33-ft) low-pass filter.



1 m = 3.28 ft
1 mm = 0.039 inch

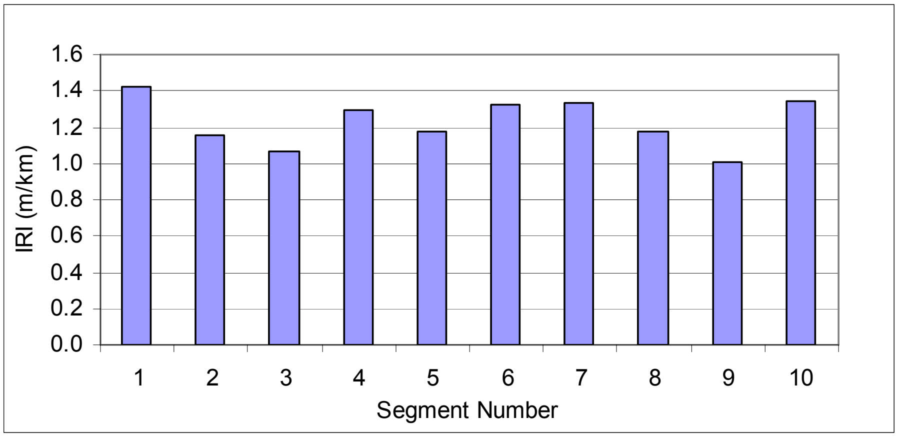
Figure 19. Profile after being subjected to a band-pass filter.

ANALYSIS TO DETERMINE SHORT-TERM CHANGES IN ROUGHNESS

Data collected at test sections established on five paving projects were used in this analysis. The data typically were collected at these times after paving: 1 day, 3 days, 7 days, and 3 months. IRI values obtained at the different time sequences were evaluated to determine short-term changes in IRI. In the analysis, the distribution of the roughness within a section was evaluated using roughness profiles. The profile data were also evaluated using PSD plots to identify wavelengths in the profile that had a significant influence on roughness.

Roughness Profiles

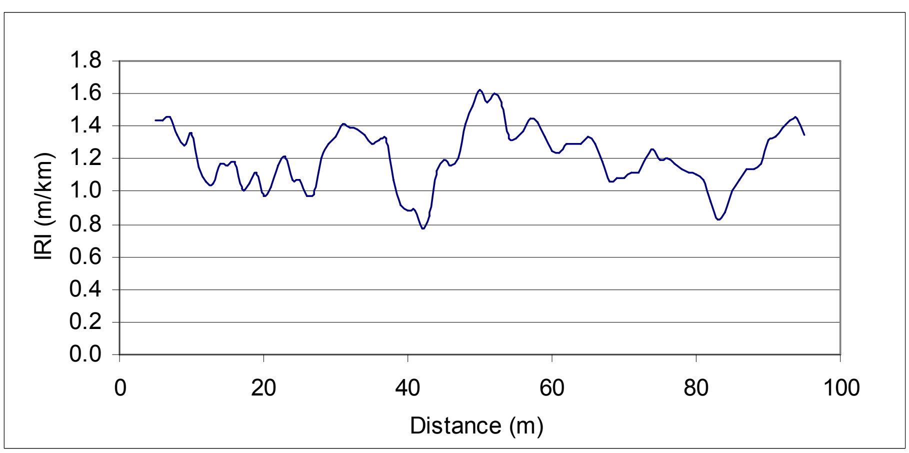
The average roughness of a road segment can be expressed by IRI. However, the roughness within the road segment can be variable. For example, consider a 100-m (328-ft) section of a road that has a roughness of 1.23 m/km (78 inches/mi). This road segment can be divided into 10 equidistant segments, each with a length of 10 m (33 ft). Figure 20 shows the roughness of each of these 10-m (33-ft) segments. As shown, the roughness values for the 10-m (33-ft) segments are variable, with the highest roughness of 1.42 m/km (90 inches/mi) being obtained at segment 1, and the lowest roughness of 1.01 m/km (64 inches/mi) being obtained at segment 9.



1 m/km = 63.4 inches/mi

Figure 20. Roughness of a roadway expressed in 10-m (33-ft) segment lengths.

Instead of using a single value to characterize the roughness of a roadway, a roughness profile can be used to show how roughness varies with distance along the roadway. Figure 21 shows the roughness profile based on a 10-m (33-ft) base length for the same section of roadway whose roughness distribution was shown in figure 20. In figure 21, the roughness value for a specific location is the average roughness over a 10-m (33-ft) length (i.e., base length of roughness profile) centered at that location. For example, the roughness shown at 25 m (82 ft) is the average roughness from 20 to 30 m (66 to 99 ft). The highest roughness value in the roughness profile occurs at 50 m (164 ft), and therefore the 10-m (33-ft) stretch of road with the highest roughness is between 45 and 55 m (148 and 181 ft). A roughness profile can be constructed for any base length. A detailed description of roughness profiles is presented by Sayers.[34]

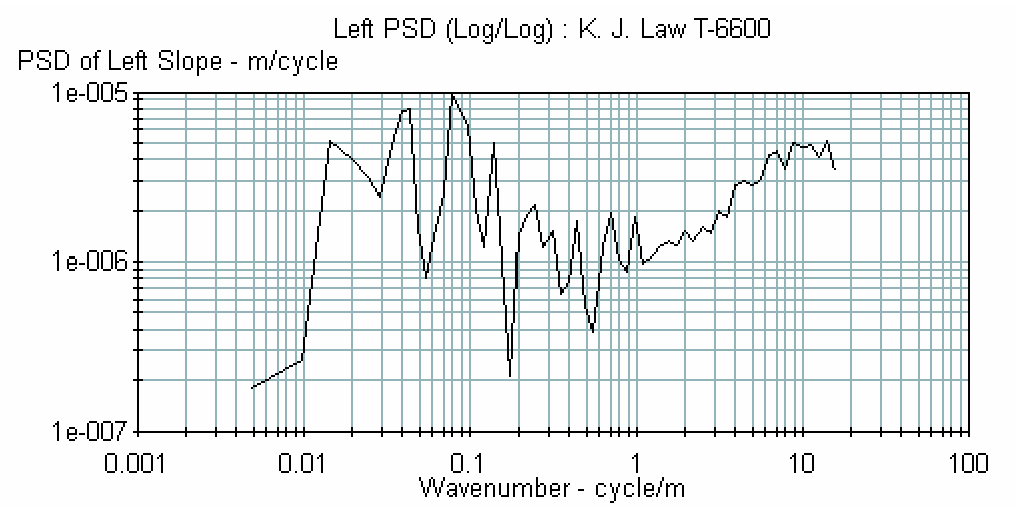


1 m/km = 63.4 inches/mi

Figure 21. Example of a roughness profile.

Power Spectral Density Plots

A road profile encompasses a spectrum of sinusoidal wavelengths. A PSD function is a statistical representation of the importance of various wavelengths contained in the profile.[5] The PSD function of profile slope best shows differences in roughness properties because the basic spectrum of roughness over the wavenumbers is more uniform.[5] In this research project, PSD plots of the profile slope were used in the analyses. Figure 22 shows an example of a PSD plot of a road profile. This plot presents a view of the distribution of the wavelengths contained within the road profile. The X-axis of the PSD plot represents the wavenumber. The wavenumber is the inverse of wavelength. If a prominent wavelength is present in a profile, this wavelength will show up as a spike in the PSD plot.



1 m = 3.28 ft

Figure 22. Example of a PSD plot.

CHAPTER 5. ANALYSIS OF DATA FROM THE LONG-TERM PAVEMENT PERFORMANCE DATABASE

LTPP PROGRAM

The LTPP program was designed as a 20-year study of pavement performance. One aspect of the LTPP program is the GPS program, which monitors the performance of inservice pavement test sections. The length of a test section established for the LTPP program is 152.4 m (500 ft). In the LTPP program, JPC pavements are studied under the GPS-3 experiment.

Profile data collection is being performed at these test sections at regular intervals using an inertial profiler. The profile data collected at the test sections as well as IRI computed from the collected data are stored in the LTPP database. These data are an excellent source for investigating roughness progression of JPC sections.

The GPS-3 sections were established on inservice roads; therefore, the roughness of a test section immediately after construction is not known. The first IRI value available for a test section was obtained when the test section was first profiled after being accepted into the LTPP program.

GPS-3 SECTIONS IN THE LTPP DATABASE

For this study, the data of the GPS-3 test sections were obtained from the LTPP database in January 2003. The construction date of each test section was used to determine the pavement age corresponding to each profile date. The following two criteria were used for selecting GPS-3 sections for this analysis:

- The pavement age had to be younger than 10 years when it was first profiled.
- The test section had to be monitored for at least 9 years.

The first criterion was used because this study's emphasis was to analyze changes in roughness that occurred early in the life of the pavement. The second criterion ensured sufficient time-sequence IRI data were available to evaluate the progression of roughness over time.

Four different environmental zones are considered in the LTPP program: dry freeze, dry no-freeze, wet freeze, and wet no-freeze. The geographical regions corresponding to these environmental zones are shown in figure 23. The boundary between the wet and the dry zone generally corresponds to an annual precipitation of 508 mm (20 inches). The boundary between freezing and nonfreezing zones generally corresponds to an annual freezing index of 89 °Celsius (C) (192.2 °Fahrenheit (F)) days.

The distribution of the GPS-3 sections selected for analysis and classified according to the environmental zones and load transfer type is shown in table 1. Test sections were selected for the LTPP program in the late 1980s. Because there are only five doweled sections in the two dry regions, it appears that most JPC pavements at that time in the dry regions were nondoweled pavements.

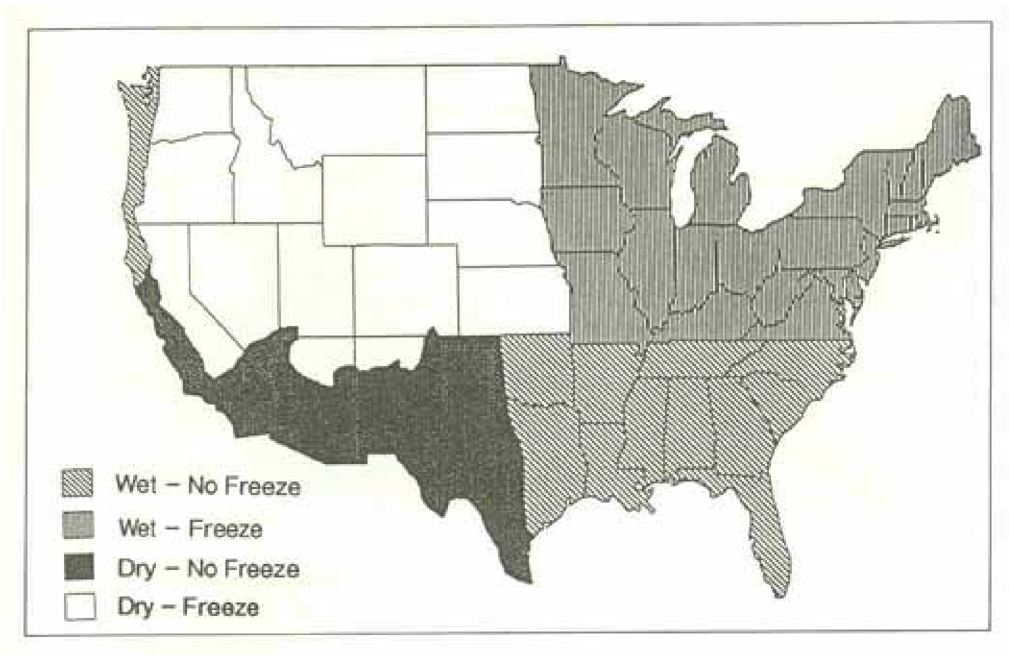


Figure 23. Environmental zones in the LTPP program.

Table 1. Distribution of GPS-3 sections.

Environmental Zone	Number of Sections		
	Doweled	Nondoweled	Total
Dry freeze	3	12	15
Dry no-freeze	2	4	6
Wet freeze	14	16	30
Wet no-freeze	13	1	14
Total	32	33	65

All doweled sections were paved with a slipform paver, except for one section where side forms were used. All nondoweled sections were also paved with a slipform paver, except for four sections where side forms were used.

ROUGHNESS PROGRESSION AT GPS-3 SECTIONS

A linear regression analysis was performed for each GPS-3 section using the time-sequence IRI data to obtain a rate of change of IRI. Based on the rate of change of IRI, the test sections were divided into three data sets:

- Data set 1: Rate of change of roughness less than 0.02 m/km/yr (1.27 inches/mi/yr).
- Data set 2: Rate of change of roughness between 0.02 and 0.04 m/km/yr (1.27 to 2.54 inches/mi/yr).
- Data set 3: Rate of change of roughness greater than 0.04 m/km/yr (2.54 inches/mi/yr).

Roughness trends for each data set were studied separately for nondoweled and doweled pavements.

Nondoweled Sections

Thirty-three nondoweled sections were available for analysis. Table 2 shows the number of test sections in each data set, classified by environmental zone. The GPS section numbers of these sections are shown in table 74, which is in appendix B.

Table 2. Distribution of nondoweled GPS-3 sections.

Data Set	Number of Sections					
	Environmental Zone				Total	Percentage of Total Doweled Sections
	Wet Freeze	Wet No-Freeze	Dry No-Freeze	Dry Freeze		
1	5	1	3	2	11	33
2	4	0	1	6	11	33
3	7	0	0	4	11	33

Roughness progression plots for the sections in each data set are shown in figures 24–26. Each line in a graph represents a pavement section, and the points on a line represent the pavement age and the corresponding roughness of the test section. An evaluation of roughness progression at test sections indicated that some sections showed variability in IRI between the years, and IRI during a particular year sometimes was lower than that for the previous year. This type of condition can occur because of slab curling and warping effects or shrink/swell effects of subgrade that can change the profile of the pavement.

The average IRI of all sections in each data set at the first and the last profile dates as well as the corresponding average pavement ages are shown in table 3. Note that many of these sections are still being monitored, and the last profile date was the one available when data were obtained from the database for this analysis.

Table 3. Average IRI of nondoweled sections at first and last profile dates.

Data Set	Average Pavement Age (Years)		Average IRI (m/km)	
	First Profile Date	Last Profile Date	First Profile Date	Last Profile Date
1	5.6	16.8	1.33	1.40
2	3.8	14.9	1.38	1.69
3	6.0	16.9	1.63	2.69

1 m/km = 63.4 inches/mi

1 m/km = 63.4 inches/mi

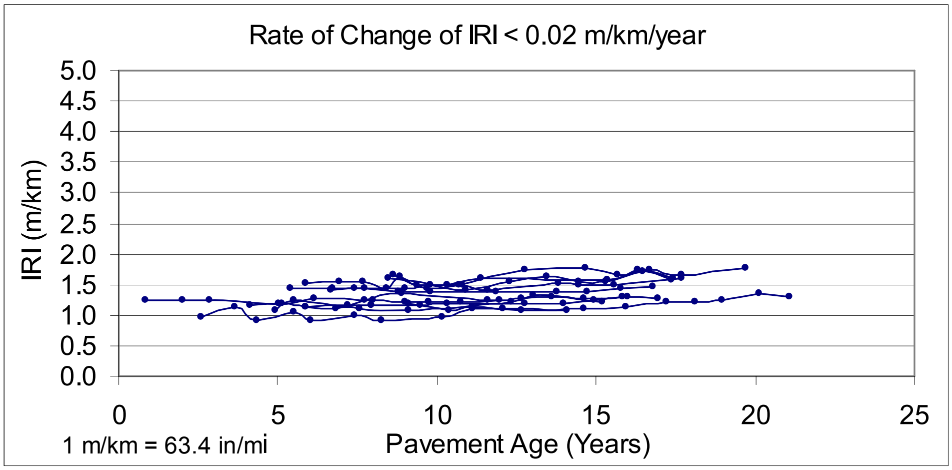


Figure 24. Roughness progression of nondoweled sections, rate of change of IRI < 0.02 m/km/yr.

1 m/km = 63.4 inches/mi

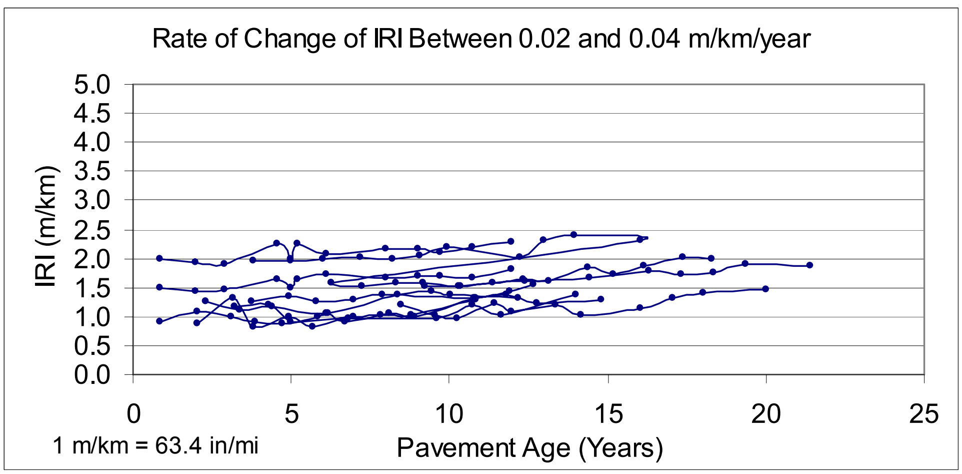


Figure 25. Roughness progression of nondoweled sections, rate of change of IRI between 0.02 and 0.04 m/km/yr.

1 m/km = 63.4 inches/mi

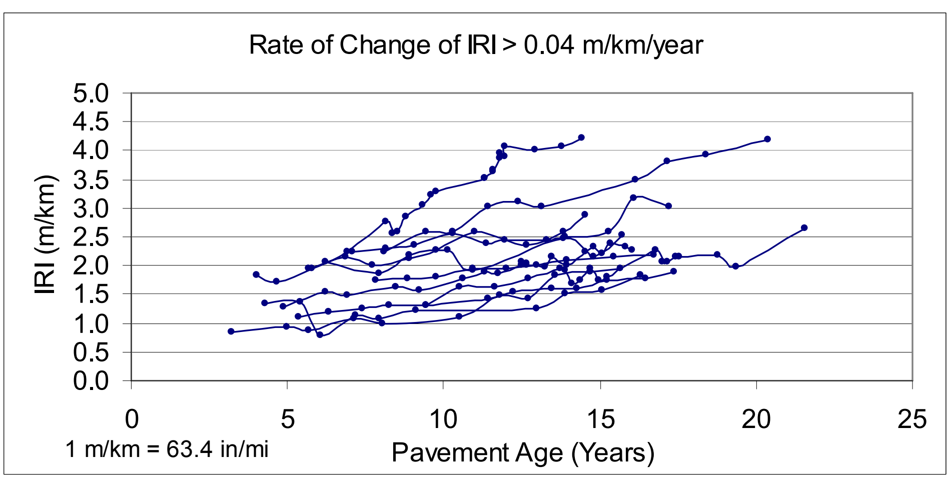


Figure 26. Roughness progression of nondoweled sections, rate of change of IRI > 0.04 m/km/yr.

The average IRI at the first profile date for the pavements in data sets 1 and 2 were close to each other. The average IRI at the first profile date for the pavements in data set 3 was higher than data sets 1 and 2. The pavements that fell into data set 1 have shown very little change in IRI over the monitored period. The average IRI of the pavements in data set 2 have increased from 1.38 to 1.68 m/km (87 to 107 inches/mi) over an 11-year period. At the last profile date, the pavements in data set 3 have an average age of 16.9 years and an average IRI of 2.69 m/km (171 inches/mi), which indicates that these pavements have reached an unacceptable roughness level well before the end of their intended service life.

The average IRI of the pavements that fell into data set 2 at the first profile date was 1.38 m/km (87 inches/mi), and the average age of the pavements at that time was 3.8 years. If a pavement has a rate of increase of IRI of 0.04 m/km/yr (2.54 inches/mi/yr) over a 16-year period, the change in roughness would be 0.64 m/km (41 inches/mi). Based on this rate of increase of roughness, the average IRI of the pavements in data set 2 when the pavements are 20 years old is expected to be 2.02 m/km (128 inches/mi). This is a typical IRI of a pavement after a 20-year life. Hence, pavements that fell into data set 2 are performing as expected, pavements in data set 1 are providing a better performance than expected, and pavements in data set 3 are showing a poor performance.

Doweled Sections

Thirty two doweled sections were available for analysis. Table 4 shows the number of test sections in each data set, classified by environmental zone. The GPS section number of these sections is shown in table 75, which is in appendix B.

Table 4. Distribution of doweled GPS-3 sections.

Data Set	Number of Sections					
	Environmental Zone				Total	Percent of Total Doweled Sections
	Wet Freeze	Wet No-Freeze	Dry No-Freeze	Dry Freeze		
1	8	8	1	1	18	56
2	3	3	1	1	8	25
3	3	2	0	1	6	19

Roughness progression plots for the pavements in each data set are shown in figures 27–29. The average IRI of all sections in each data set at the first and the last profile dates as well as the corresponding average pavement ages are shown in table 5.

The average IRI at the first profile date for the data set 1 pavements was higher than for data set 2. Several sections in data set 1 had high IRI values that caused the IRI at the first profile date to be high. The average IRI of the pavements in data set 3 at the first profile date was higher than for data set 2, but similar to data set 1.

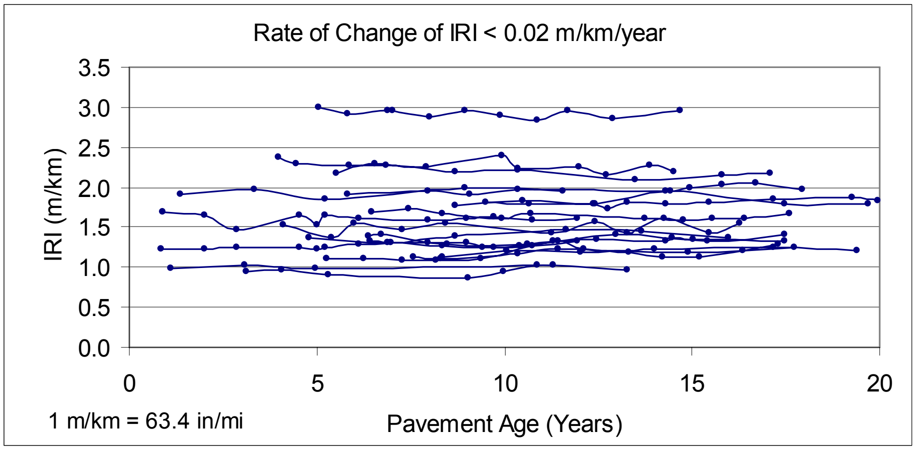


Figure 27. Roughness progression of doweled sections, rate of change of IRI < 0.02 m/km/yr.

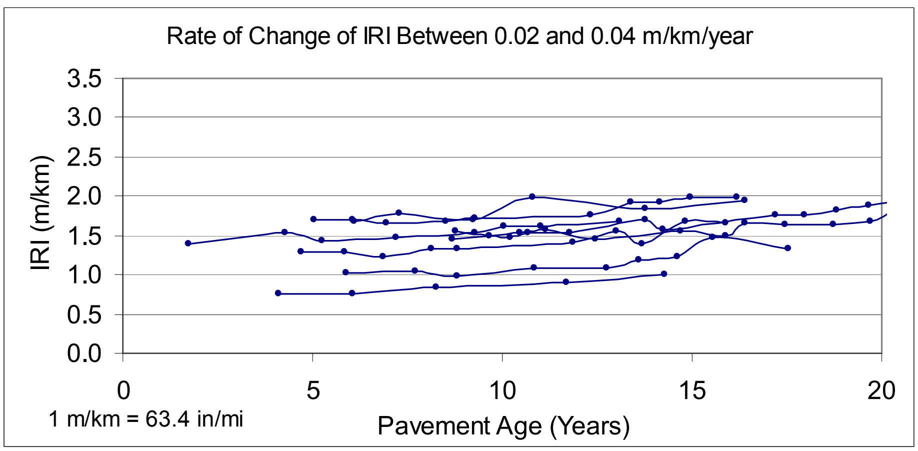


Figure 28. Roughness progression of doweled sections, rate of change of IRI between 0.02 and 0.04 m/km/yr.

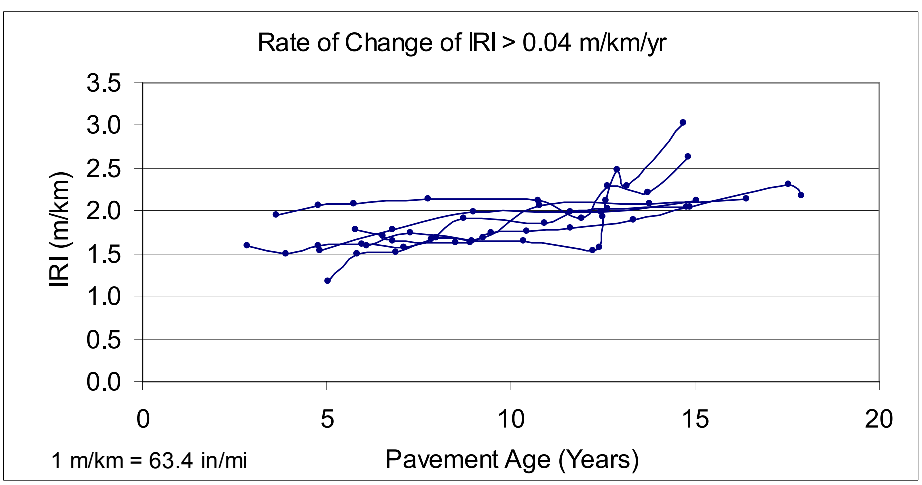


Figure 29. Roughness progression of doweled sections, rate of change of IRI > 0.04 m/km/yr.

Table 5. Average IRI of doweled sections at first and last profile dates.

Data Set	Average Pavement Age (Years)		Average IRI (m/km)	
	First Profile Date	Last Profile Date	First Profile Date	Last Profile Date
1	5.1	16.1	1.64	1.66
2	5.6	16.8	1.36	1.67
3	4.7	15.6	1.60	2.35

1 m/km = 63.4 inches/mi

The pavements in data set 1 have shown very little change in IRI over the monitored period. The average IRI of the pavements in data set 2 have increased from 1.36 to 1.67 m/km (86 to 106 inches/mi) over an 11-year period. At the last profile date, the pavements in data set 3 have an average age of 15.6 years and an average IRI of 2.35 m/km (149 inches/mi), which indicates that these pavements have reached an unacceptable roughness level well before the end of their intended service life. The average IRI of the pavements in data set 2 at the first profile date was 1.36 m/km (86 inches/mi); the average age was 5.6 years. If a pavement has a rate of increase of IRI of 0.04 m/km/yr (2.54 inches/mi/yr) over a 15-year period, the change in roughness would be 0.64 m/km (41 inches/mi). Based on this rate of increase of roughness, the average IRI of the pavements in data set 2 when the pavements are 20 years old is expected to be 2.00 m/km (127 inches/mi). This is a typical IRI of a pavement after a 20-year service life. Hence, the pavements in data set 2 are behaving as expected, the pavements in data set 1 are performing much better than expected, and the pavements in data set 3 have performed poorly.

Comparison Between Doweled and Nondoweled Sections

The percentage of sections for doweled and nondoweled pavements in each data set is shown in table 6. As seen in this table, 56 percent of the doweled sections were in data set 1, compared to 33 percent for nondoweled sections. Nineteen percent of doweled sections fell into data set 3, compared to 33 percent for the nondoweled sections.

Table 6. Comparison between doweled and nondoweled sections.

Data Set	Rate of Change of Roughness (m/km/yr)	Percentage of Sections	
		Doweled	Nondoweled
1	< 0.02	56	33
2	0.02 to 0.04	25	33
3	> 0.04	19	33

1 m/km = 63.4 inches/mi

These data clearly show that doweled and nondoweled pavements have different roughness progression characteristics. Generally, doweled pavements show much less roughness progression compared to nondoweled pavements.

EFFECT OF FAULTING ON ROUGHNESS

Fault measurements at the GPS-3 test sections are obtained at regular time intervals; measurements are obtained along the pavement edge and the right wheel path at each joint and crack. For each test section, the fault measurement date closest to the last profile date was identified in the database. Then, the fault data measured along the right wheel path corresponding to the identified date were extracted from the database.

Faulting data were available for 31 of the 33 nondoweled sections and for all doweled sections. The fault values recorded at joints and cracks were added together to obtain the total faulting for each section. Table 76, which is in appendix C, shows the fault values for all nondoweled sections. This table also shows the following parameters for each test section: last profile date, IRI for last profile date, the fault survey date closest to the last profile date, and the total faulting at the joints and the cracks.

Except for the following few cases, the faulting occurred at the joints and not at the cracks. For nondoweled sections, faulting at the cracks was recorded only at two sections each in data sets 2 and 3. For these four sections, the total faulting within the test section at cracks was less than 5 mm (0.2 inch). For doweled sections, the faulting at cracks was recorded only at two sections. The magnitude of the total faulting within the section at cracks was 2 and 43 mm (0.08 and 1.69 inches) at the two sections.

The relationship between roughness and faulting is discussed separately (below) for nondoweled and doweled pavements.

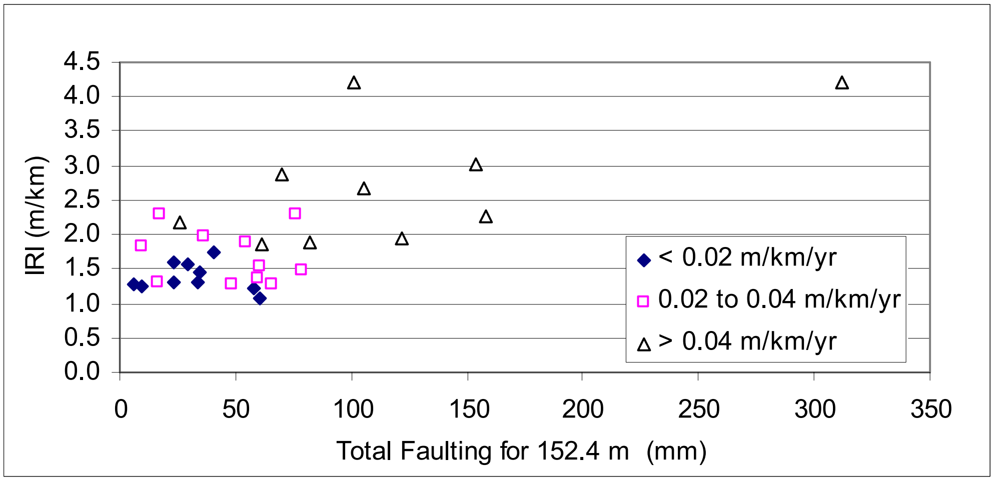
Nondoweled Sections

Figure 30 shows the relationship between IRI at last profile date and the total faulting at the test sections; separate symbols are used for the different data sets. This figure shows a trend of higher roughness that corresponds with increasing faulting. The correlation coefficient between the IRI at last profile date and faulting was 0.70.

The average total faulting for data sets 1 through 3 were 32, 47, and 119 mm (1.26, 1.85, and 4.69 inches), respectively. Pavement sections in data set 3 had high faulting. Faulting data were available for 10 of the 11 sections in data set 3, with 6 of these sections each having total faulting greater than 100 mm (4 inches).

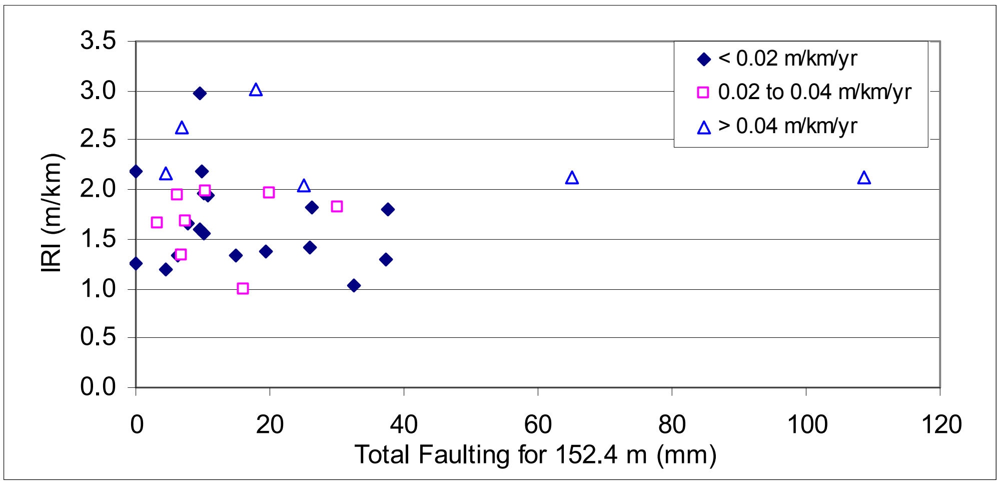
Doweled Sections

Figure 31 shows the relationship between IRI at last profile date and total faulting at the test sections; separate symbols are used for the different data sets. No relationship between IRI and faulting is seen in this figure. The correlation coefficient between IRI at last profile date and total faulting was 0.07.



1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 30. Relationship between IRI and faulting for nondoweled pavements.



1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 31. Relationship between IRI and faulting for doweled pavements.

The average total faulting for data sets 1 through 3 were 15, 13, and 38 mm (0.59, 0.51, and 1.50 inches), respectively. Two sections in data set 3 had total faulting values in excess of 40 mm (1.57 inches), and if the faulting at these two sections were omitted, the average faulting of the sections in data set 3 would be 14 mm (0.55 inches).

Comparison Between Doweled and Nondoweled Sections

A clear relationship between IRI and faulting was observed for nondoweled pavements. This type of relationship was not seen for doweled pavements.

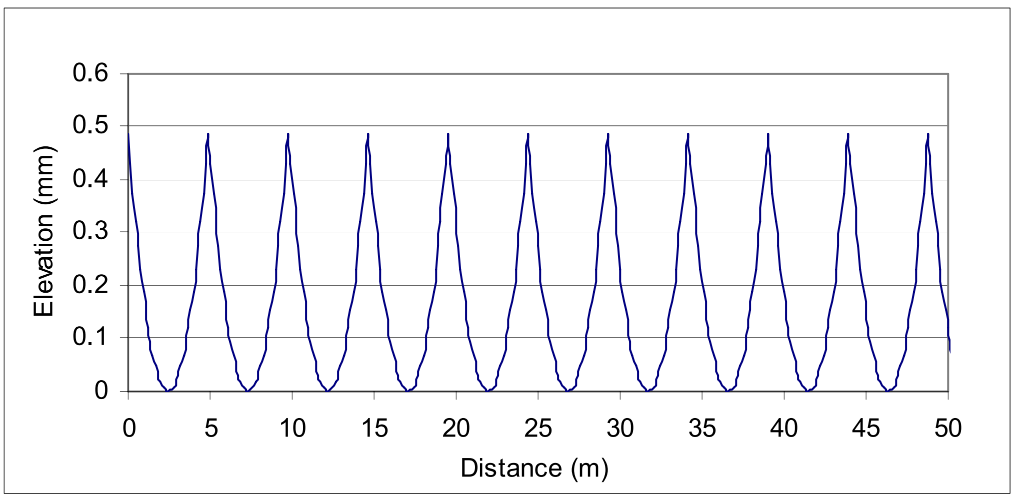
Of the 32 doweled sections, 30 sections (94 percent) had a total faulting that was less than 38 mm (1.50 inches), whereas only 39 percent of the nondoweled sections met this comparison. These data show that dowels served their intended function by providing load transfer between slabs and prevented faulting.

EFFECT OF SLAB CURVATURE ON ROUGHNESS

The LTPP database has profile data for five profiler runs for each profiling date. For each analyzed GPS section, one profile run was selected from the first profile date and last profile date. The CI of the PCC slabs of the selected profile data sets was computed using the method developed by Byrum,[10] described in appendix A. Changes in slab curvatures that have occurred over the monitored period, and the relationship between change in roughness and change in curvature, were examined separately for nondoweled and doweled pavements.

Theoretical Evaluation of Pure Slab Curvature on IRI

A theoretical evaluation of the effect of pure curvature of PCC slabs on IRI was performed by generating profiles having different curvature (where curvature is equal to reciprocal of radius) for three different slab lengths and then computing the IRI values. Figure 32 shows an example of a theoretically generated profile for a PCC pavement with a 4.9-m (16-ft) joint spacing that has a curvature of 0.16×10^{-3} 1/m (0.049×10^{-3} 1/ft).

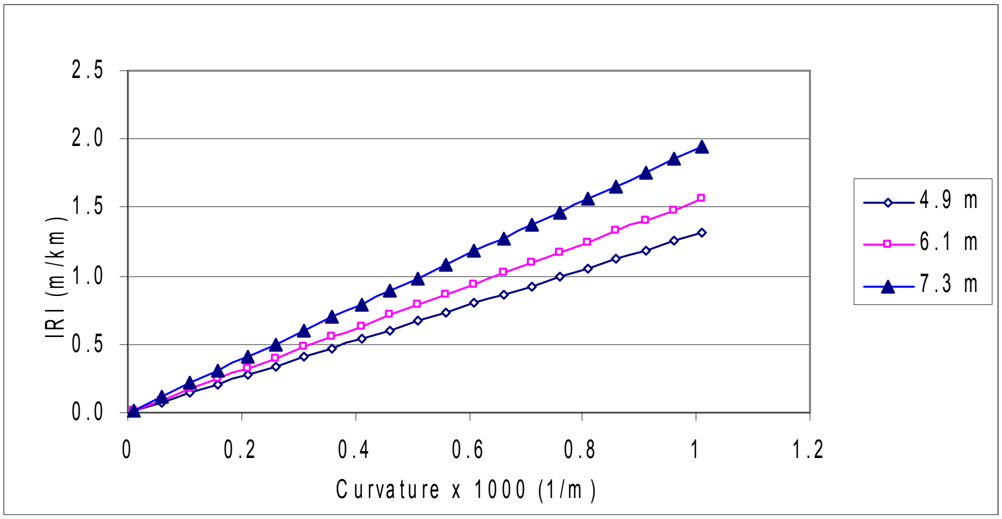


1 mm = 0.039 inch
1 m = 3.28 ft

Figure 32. Theoretically generated slab profiles.

Figure 33 shows the relationship between curvature and IRI for slab lengths of 4.9, 6.1, and 7.3 m (16, 20, and 24 ft) that was obtained from this theoretical evaluation.

The actual curvature behavior of PCC slabs is not expected to follow the shapes generated in this theoretical analysis. In a pavement, the middle portion of the slab will be in contact with the ground with curling being confined to the slab portions close to the joints. However, this example illustrates that slab curvature can have a significant effect on IRI.



1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 33. Relationship between curvature and IRI from theoretical analysis.

This example described the effect of pure curvature on IRI. The CI computed in this study is based on an average curvature computed for all slabs within the section; the average curvature is the average of curvature values computed using different arc lengths (see appendix A). A high value of CI indicates that a high level of slab curling or warping is present in the PCC slabs.

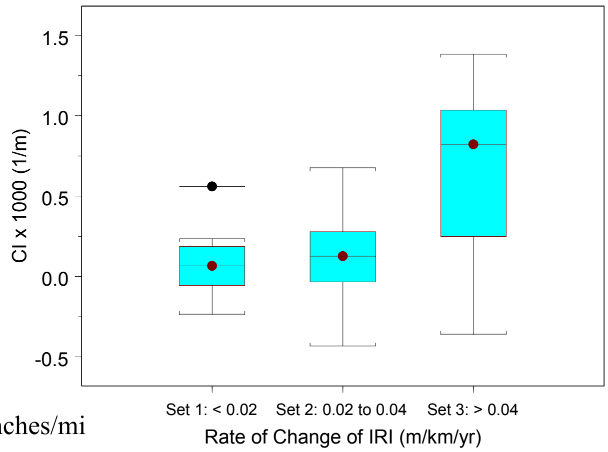
Nondoweled Sections

The CI values for the first profile date and the last profile date for all nondoweled sections are presented in table 78, which is in appendix D. Figure 34 shows a box plot of the distribution of CI values for the last profile date for the three data sets. Table 7 shows the median, 25th percentile, and 75th percentile values of CI at the last profile date. The median CI for all three data sets was positive (slab curved upwards). The CI values of pavements in data set 3 were much higher than those for data sets 1 and 2.

The change in CI that has occurred between the first and last profile dates was examined separately for the three data sets. Figures 35–37 show box plots of the CI at the first and last profile date for data sets 1 through 3.

For data set 1, the median CI for the first and last profile dates were 0.048 x 10^{-3} and 0.066 x 10^{-3} 1/m (0.015 x 10^{-3} and 0.020 x 10^{-3} 1/ft), respectively. The range of CI between the 25th and 75th percentile values for the first and last profile dates was −0.007 x 10^{-3} to 0.141 x 10^{-3} 1/m (−0.002 x 10^{-3} to 0.043 x 10^{-3} 1/ft) and −0.037 x 10^{-3} to 0.173 x 10^{-3} 1/m (−0.011 x 10^{-3} to 0.053 x 10^{-3} 1/ft), respectively. Little change in CI has occurred for the pavements in data set 1.

For data set 2, the median CI for the first and last profile dates were 0.030 x 10^{-3} and 0.127 x 10^{-3} 1/m (0.009 x 10^{-3} and 0.039 x 10^{-3} 1/ft), respectively. The range of CI between the 25th and 75th percentile values for the first and last profile dates was −0.047 x 10^{-3} to 0.135 x 10^{-3} 1/m (−0.014x 10^{-3} to 0.041 x 10^{-3} 1/ft) and 0.009 x 10^{-3} to 0.273 x 10^{-3} 1/m (0.003 x 10^{-3} to 0.083 x 10^{-3} 1/ft), respectively. These data show that the CI range of the pavements in data set 2 has increased over the monitored period.

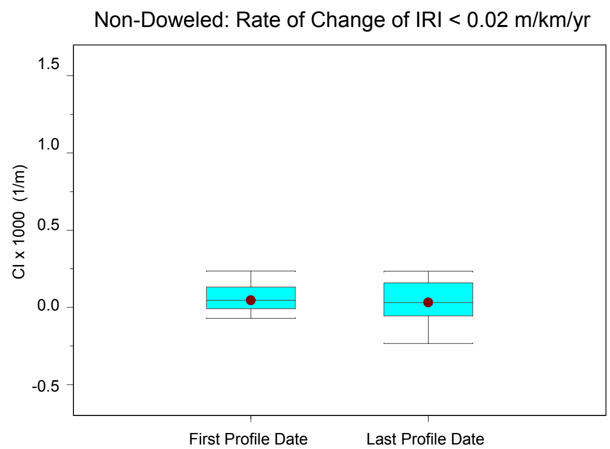


1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 34. Distribution of CI at last profile date for nondoweled pavements.

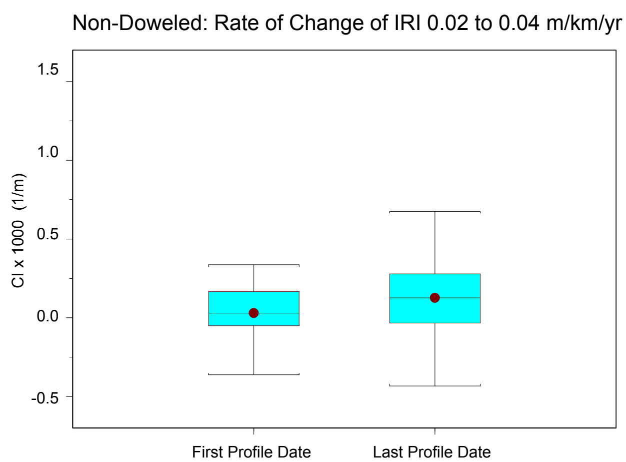
Table 7. Distribution of CI at last profile date for nondoweled sections.

Data Set	Curvature Index x 1,000 (1/m)		
	Median	25th Percentile	75th Percentile
1	0.055	−0.037	0.173
2	0.127	0.009	0.273
3	0.822	0.411	0.962



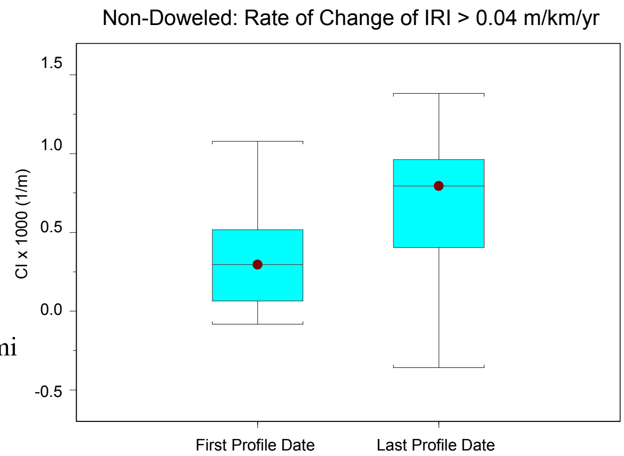
1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 35. Distribution of CI for nondoweled data set 1.



1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 36. Distribution of CI for nondoweled data set 2.

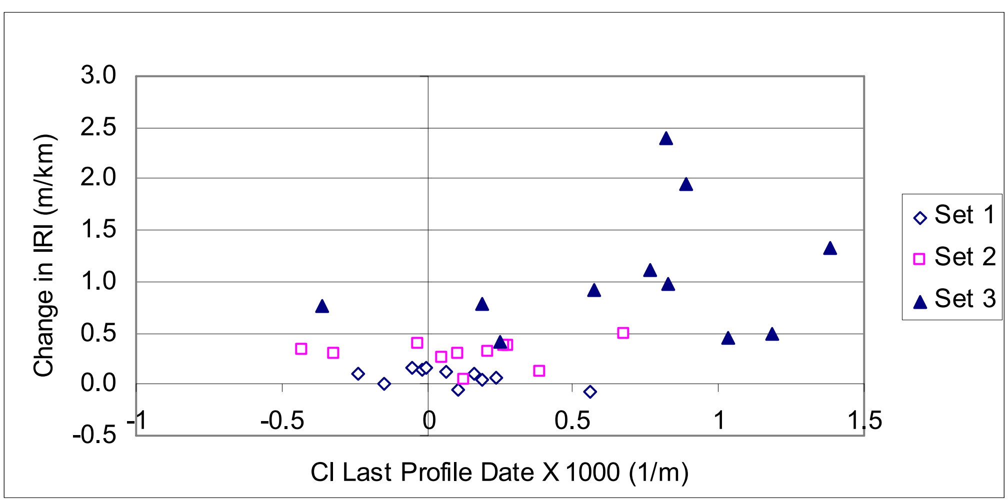


1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 37. Distribution of CI for nondoweled data set 3.

For data set 3, the median CI for the first and last profile dates were 0.287 x 10^{-3} and 0.822 x 10^{-3} 1/m (0.088 x 10^{-3} and 0.251x 10^{-3} 1/ft), respectively. The range of CI between the 25th and 75th percentile values for the first and last profile dates were 0.005 x 10^{-3} to 0.433 x 10^{-3} 1/m (0.002 x 10^{-3} to 0.132 x 10^{-3} 1/ft) and 0.411 x 10^{-3} to 0.962 x 10^{-3} 1/m (0.125 x 10^{-3} to 0.293 x 10^{-3} 1/ft), respectively. These data show that the CI of pavements in data set 3 has greatly increased over the monitored period.

Figure 38 shows the relationship between the change in IRI between the first and last profile dates and CI at the last profile date. This plot shows that 73 percent of sections have a positive curvature at the last profile date. Sections with higher CI (either positive or negative) are generally associated with high changes in IRI.



1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 38. Change in IRI and CI at last profile date for nondoweled pavements.

A correlation analysis was carried out to identify concrete material parameters and environmental parameters that could be related to CI. In addition, scatterplots between the CI at last profile date and each parameter were examined to investigate the relationship between the two factors. This procedure was used to examine whether a relationship existed between the two parameters, because the correlation coefficient can sometimes be significantly influenced by outliers. Based on this investigation, the factors that had the strongest relationship to CI are shown in table 8.

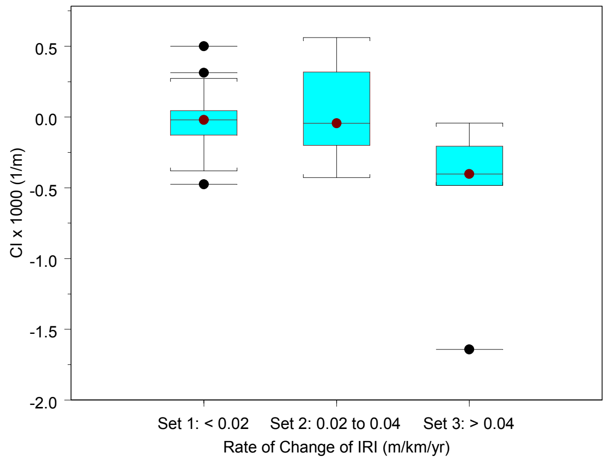
Table 8. Factors with highest correlation to CI for nondoweled pavements.

Factor	Correlation Coefficient
Coefficient of thermal expansion	0.69
Freezing index	0.43
Total faulting at site	0.42
Weight of cement in mix	−0.34
Mean annual temperature	−0.30
Annual precipitation	−0.29
Elastic modulus of concrete	0.28
Number of days below 0 °C (32 °F) per year	0.28
Number of wet days per year	−0.22
Split tensile strength of concrete	0.21
PCC slab thickness	−0.20

Higher values of the following parameters were associated with higher CI values: coefficient of thermal expansion (CTE), freezing index, total faulting at site, elastic modulus of concrete, number of days in a year below 0 °C (32 °F), and split tensile strength of concrete. Higher values of the following parameters were associated with lower CI values: weight of cement in mix, mean annual temperature, annual precipitation, number of wet days per year, and PCC slab thickness.

Doweled Sections

The CI values for the first profile date and the last profile date for all doweled sections are presented in table 79, which is in appendix D. Figure 39 shows a box plot of the distribution of CI values for the last profile date. Table 9 shows the median, 25th percentile, and 75th percentile values of CI at the last profile date.



1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 39. Distribution of CI at last profile date for doweled pavements.

Table 9. Distribution of CI at last profile date for doweled pavements.

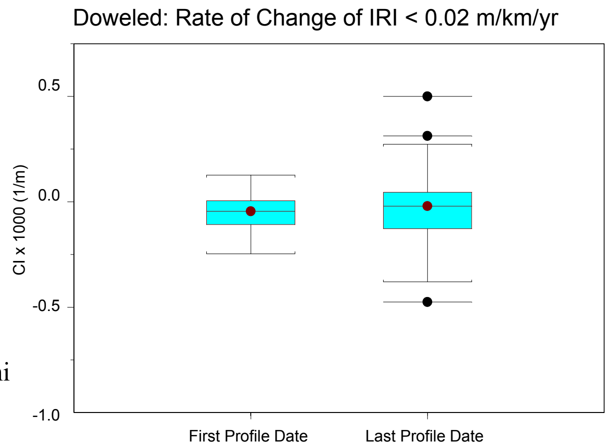
Data Set	Curvature Index x 1,000 (1/m)		
	Median	25th Percentile	75th Percentile
1	−0.020	−0.126	0.045
2	−0.043	−0.181	0.247
3	−0.402	−0.465	−0.253

1/m = 1/3.28 ft

The median CI for all three data sets was negative (downward curvature), with the median CI decreasing for data sets 1 through 3. The median CI for data sets 1 and 2 was close to zero, indicating negligible curvature. Most CI values for data set 1 were within a relatively narrow range. The range of CI for data set 2 was wider than that for data set 1. All sections in data set 3 had negative CI values.

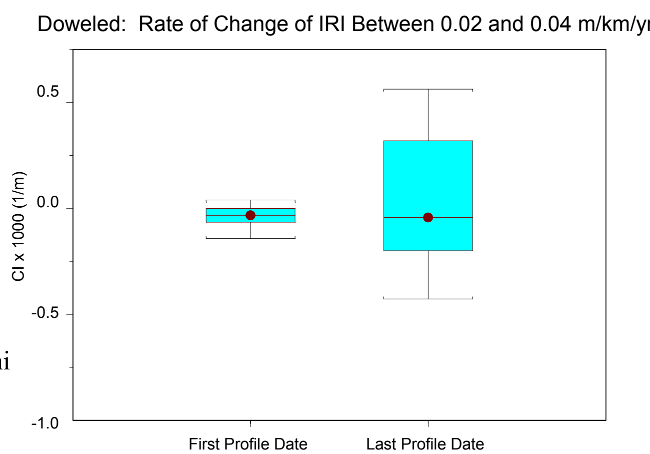
The changes in CI that occurred between the first and last profile dates were examined separately for the three data sets. Figures 40 through 42 show box plots of the CI at the first and last profile date for data sets 1 through 3.

For data set 1, the median CI for the first and last profile dates were -0.045×10^{-3} and -0.020×10^{-3} 1/m (-0.137×10^{-3} and -0.006×10^{-3} 1/ft), respectively. The range of CI between the 25th and 75th percentile values for the first and last profile dates were -0.104×10^{-3} to 0.003×10^{-3} 1/m (-0.032×10^{-3} to 0.001×10^{-3} 1/ft) and -0.126×10^{-3} to 0.045×10^{-3} 1/m (-0.038×10^{-3} to 0.014×10^{-3} 1/ft), respectively. Little change in CI has occurred for the majority of pavements in this data set.



1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

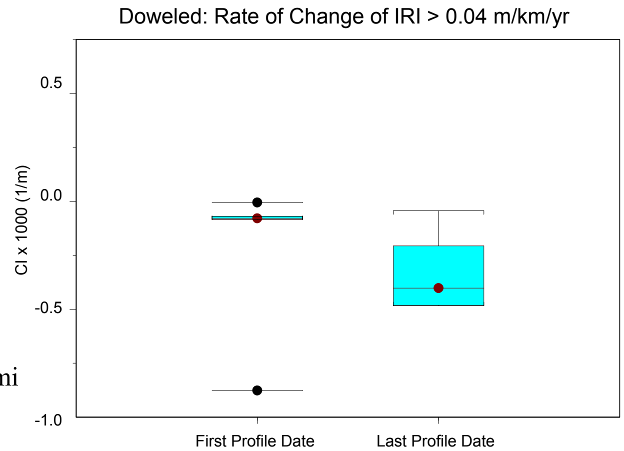
Figure 40. Distribution of CI for doweled data set 1.



1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 41. Distribution of CI for doweled data set 2.

For data set 2, the median CI for the first and last profile dates were −0.033 x 10^{-3} and −0.043 x 10^{-3} 1/m (−0.010 x 10^{-3} and −0.013 x 10^{-3} 1/ft), respectively. The range of CI between the 25th and 75th percentile values for the first and last profile dates were −0.062 x 10^{-3} to −0.011 x 10^{-3} 1/m (−0.019 x 10^{-3} to −0.003 x 10^{-3} 1/ft) and −0.181 x 10^{-3} to 0.247 x 10^{-3} 1/m (−0.055 x 10^{-3} to 0.075 x 10^{-3} 1/ft), respectively. At the first profile date, the CI values encompassed a very narrow range. However, the CI values at the last profile date encompassed a much wider range, with the range increasing both negatively and positively.

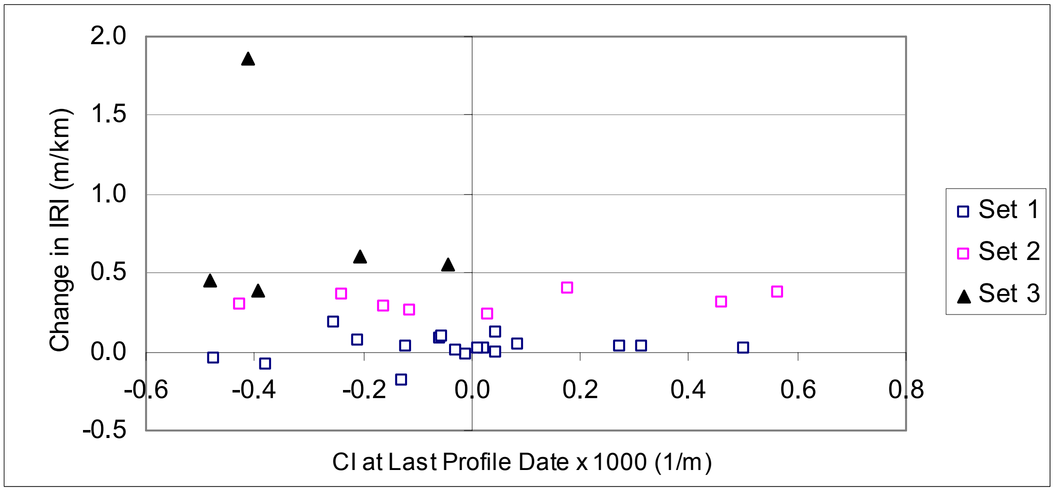


1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 42. Distribution of CI for doweled data set 3.

For data set 3, the median CI for the first and last profile dates were −0.073 x 10^{-3} and −0.402 x 10^{-3} 1/m (−0.022 x 10^{-3} and −0.122 x 10^{-3} 1/ft), respectively. The range of CI between the 25th and 75th percentile values for the first and last profile dates were −0.080 x 10^{-3} to −0.021 x 10^{-3} 1/m (−0.024 x 10^{-3} to −0.006 x 10^{-3} 1/ft) and −0.465 x 10^{-3} to −0.253 x 10^{-3} 1/m (−0.142 x 10^{-3} to −0.077 x 10^{-3} 1/ft). At the first profile date, the CI values encompassed a very narrow range with all sections having a negative curvature except for one section that showed a very slight positive curvature. However, at the last profile date all sections in this data set showed a negative (downward) curvature with the magnitude of downward curvature having increased significantly over time for a majority of the sections.

Figure 43 shows the relationship between the change in IRI between the first and last profile dates and the CI at the last profile date. GPS section 273003 had the largest negative curvature with a CI of −1.64 x 10^{-3} 1/m (−0.5 x 10^{-3} 1/ft), and this data point is not shown in figure 43. This plot shows that 63 percent of the sections have a negative curvature. There was no clear relationship between CI at last profile date and change in IRI.



1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 43. Change in IRI and CI at last profile date for doweled pavements.

Because a majority of doweled pavements at the last profile date had a negative curvature, an investigation was performed to identify parameters having a correlation with negative CI values. This analysis was performed using scatterplots and by computing correlation coefficients between CI and concrete material parameters and environmental parameters. Table 10 shows the parameters that had the strongest relationship with negative CI values.

Higher values of the following parameters resulted in more downward curvature: freezing index, number of days in a year below 0 °C (32 °F), ratio between PCC elastic modulus and split tensile strength, and weight of cement in mix. Higher values of the following parameters resulted in lower downward curvature: mean annual temperature, annual precipitation, days in year with temperature greater than 32 °C (90 °F), and PCC slab thickness.

Table 10. Factors with highest correlation to negative CI for doweled pavements.

Factor	Correlation Coefficient
Freezing index	−0.51
Mean annual temperature	0.49
Number of days below 0 °C (32 °F) per year	−0.44
Ratio between PCC elastic modulus and tensile strength	−0.39
Weight of cement in mix	−0.39
Annual precipitation	0.37
Days in year with temperature greater than 32 °C (90 °F)	0.35
PCC slab thickness	0.27

Comparison Between Doweled and Nondoweled Sections

At the last profile date, 73 percent of nondoweled pavements had a positive curvature; 63 percent of the doweled sections had a negative curvature. The dowels appear to be restraining the slabs in PCC pavements and limiting the magnitude of the upward curvature. Overall, the nondoweled pavements had higher curvature than doweled pavements. The pavements in data set 3 for nondoweled pavements showed a significant increase in upward curvature over the monitored period. A few pavements in data set 3 for doweled pavements showed an increase in downward curvature over the monitored period. For nondoweled pavements, a strong relationship was seen between faulting and slab curvature. It appears that high slab curvature in these pavements increases the potential for faulting and results in a high magnitude of faulting.

FACTORS AFFECTING ROUGHNESS PROGRESSION IN CONCRETE PAVEMENTS

The effect of faulting and slab curvature on roughness was analyzed previously. In this section, the effect of pavement design factors, traffic, environmental conditions, and concrete properties on roughness progression are evaluated. The data available in the LTPP database were reviewed to select parameters that could be studied under each of these factors. The parameters considered for analysis are shown in table 11.

Table 11. Parameters considered for analysis.

Factor	Parameter
Pavement design	Slab thickness
	Base type
	Joint spacing
	Drainage
	Shoulder
Traffic	Cumulative axle loads
Environmental conditions	Annual precipitation
	Annual wet days
	Mean annual temperature
	Number of days with temperature < 0 °C (32 °F), per year
	Number of days with temperature > 32 °C, (90 °F), per year
	Annual freezing index
	Freeze thaw cycles per year
Concrete properties	Split tensile strength
	Elastic modulus
	Compressive strength
	Coefficient of thermal expansion
	Weight of cement in mix
	Weight of fine aggregate in mix
	Weight of coarse aggregate in mix
	Water-to-cement ratio
	Unit weight of concrete

The correlation coefficient between the change in roughness and each parameter shown in table 11 were computed separately for nondoweled and doweled pavements. Scatterplots between the change in roughness and each parameter were also plotted to examine the relationship between the two factors. This procedure was used because high correlation coefficients can sometimes be obtained due to outliers. The factors that had the highest correlation to change in roughness for nondoweled pavements that were reasonable based on the relationship seen in the scatterplots are shown in table 12. No strong correlations or trends were noted for the doweled pavements.

A further evaluation of the effect of parameters listed in table 11 on roughness progression was performed separately for doweled and nondoweled pavements. For nondoweled pavements, the three data sets established previously were used to compare differences in parameters between the three data sets. A meaningful analysis using a similar approach cannot be performed for doweled pavements because there were only a few sections in data sets 2 and 3 (eight and six sections, respectively). Therefore, doweled pavements were analyzed by dividing the pavement sections into two groups, with the first group containing data set 1 and the second group containing data sets 2 and 3.

Table 12. Parameters with highest correlation to change in roughness for nondoweled pavements.

Factor	Correlation Coefficient
Freezing index	0.51
Number of days below 0 °C (32 °F) per year	0.48
Mean annual temperature	−0.48
Freeze thaw cycles per year	0.35
Joint spacing	0.31
Number of days above 32 °C (90 °F) per year	−0.30

The results of the analysis for pavement design factors, traffic, environmental conditions, and concrete properties are shown separately.

Effect of Pavement Design Factors

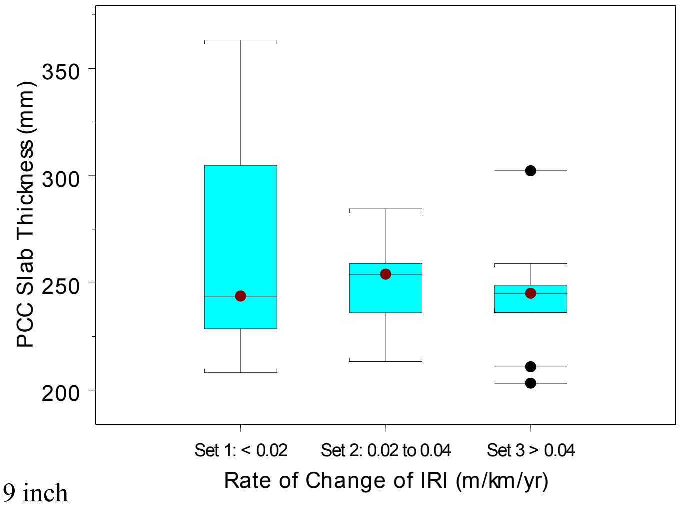
The effect of the following pavement design factors on roughness progression was investigated: concrete slab thickness, base type, joint spacing, drainage, and shoulder type.

Concrete Slab Thickness

The distribution of the PCC slab thickness for the nondoweled data sets is shown in figure 44. The median PCC thickness for pavements in data sets 1, 2, and 3 were 244, 254, and 245 mm (9.6, 10, and 9.7 inches), respectively. Above the median value, the PCC slab thickness for pavements in data set 1 had a much wider range than for data sets 2 and 3.

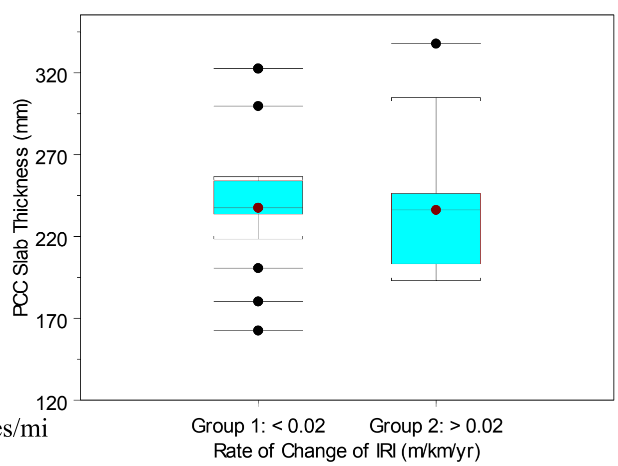
The distribution of the PCC slab thickness for the doweled data sets is shown in figure 45. The median PCC thickness for pavements in groups 1 and 2 were 237 and 236 mm (9.33 and 9.29 inches), respectively. However, the range of thickness below the median for group 2 pavements was much more than that for group 1 pavements.

The PCC slab thickness for both nondoweled and doweled pavements would have been selected based on the expected traffic volumes. If the pavements in data sets 2 and 3 for nondoweled pavements and group 2 for doweled pavements were subjected to a much higher traffic level than the design traffic level, it would be a factor contributing to higher roughness. The relationship between traffic levels and pavement thickness will be discussed in a later section.



1 mm = 0.039 inch
1 m/km = 63.4 inches/mi

Figure 44. Distribution of PCC slab thickness for nondoweled pavements.



1 mm = 0.039 inch
1 m/km = 63.4 inches/mi

Figure 45. Distribution of PCC slab thickness for doweled pavements.

Base Type

The base type for nondoweled pavements was available for eight sections in data set 1, for all sections in data set 2, and for nine sections in data set 3. Table 13 shows the distribution of base types for each data set.

The base type for doweled pavements was available for 17 sections in group 1 and 13 sections in group 2. Table 14 shows the distribution of base type for the two groups. The base types for the six doweled sections that had a rate of increase of IRI greater than 0.04 m/km/yr (2.54 inches/mi/yr) were as follows: two sections, gravel base; three sections, cement-treated base; and one section, asphalt-treated base.

Table 13. Base types for nondoweled pavements.

Data Set	Number of Sections			
	Gravel Base	Cement-Treated Base	Asphalt-Treated Base	Total
1	3	3	2	8
2	5	5	1	11
3	3	5	1	9

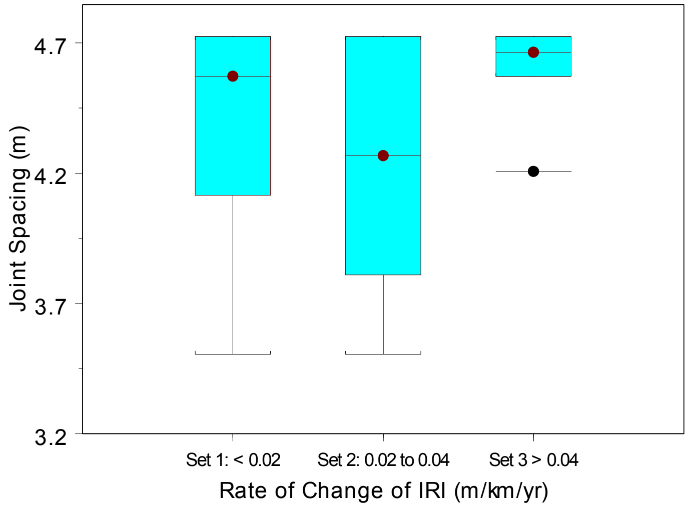
Table 14. Base types for doweled pavements.

Data Group	Number of Sections			
	Gravel Base	Cement-Treated Base	Asphalt-Treated Base	Total
1	7	8	2	17
2	5	6	2	13

No conclusion on the effect of base type on roughness progression can be made for either doweled or nondoweled pavements.

Joint Spacing

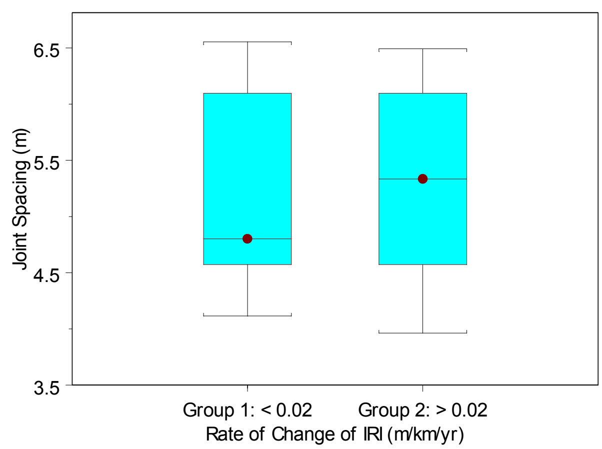
The distribution of the joint spacing for nondoweled data sets is shown in figure 46. Five pavement sections each in data sets 1 and 2 had a joint spacing between 4.6 and 4.7 m (15.1 and 15.4 ft). All sections in data set 3 had a joint spacing of either 4.6 or 4.7 m (15.1 and 15.4 ft) except for one section that had a joint spacing of 4.2 m (13.8 ft). These data suggest that higher joint spacing in nondoweled pavements may be a factor contributing to higher roughness.



1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 46. Distribution of joint spacing for nondoweled pavements.

The distribution of joint spacing for the doweled data groups is shown in figure 47. The median joint spacing for the pavements in data groups 1 and 2 were 4.8 and 5.3 m (15.7 and 17.4 ft), respectively. The overall range of joint spacing for the two data groups was similar, and no conclusion could be made on the effect of joint spacing on roughness for doweled pavements.



1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 47. Distribution of joint spacing for doweled pavements.

Drainage

The LTPP database indicates the type of drainage provided at each test section. The following drainage information on nondoweled sections was determined from the data available in the database:

- Data set 1: Edge drains were provided at two sections; the other sections had no drainage.

- Data set 2: Edge drains were provided at three sections. One section had transverse drains, and no drainage was provided at the other sections.
- Data set 3: No drainage for all sections.

The drainage information for the doweled sections was as follows:

- Group 1: Nine sections had no drainage, seven sections had longitudinal edge drains, and two sections had transverse drainage.
- Group 2: Eleven sections had no drainage, two sections had longitudinal edge drains, and one section had unspecified drainage.

The subgrade type at the sections may have been a consideration in deciding whether drainage was provided at a section. In cases where coarse-grained subgrade was present at a site, a decision may have been made not to provide drainage. An evaluation of the subgrade information for nondoweled sections indicated the following:

- Data sets 1 and 2: Subgrade information was available for 10 sections in each data set. In each data set, seven sections had coarse-grained subgrade and three sections had fine-grained subgrade.
- Data set 3: Subgrade information was available for all sections; eight sections had coarse-grained subgrade and three sections had fine-grained subgrade.

An evaluation of the subgrade information for doweled sections indicated the following:

- Group 1: Subgrade information was available for 15 sections; 7 sections had fine-grained use subgrade and the rest had coarse-grained subgrade.
- Group 2: Subgrade information was available for 14 sections; 6 sections had fine-grained subgrade and the rest had coarse-grained subgrade.

No conclusions on the effect of provision of drainage on roughness progression could be made based on this information.

Shoulder Type

The shoulder type for the nondoweled sections is shown in table 15. The shoulder type was not specified for one section in data set 1. The median width of the paved shoulder for all three data sets was 3 m (9.8 ft).

Table 15. Shoulder types for nondoweled pavements.

Data Set	Number of Sections		
	Gravel Shoulder	Concrete Shoulder	Asphalt Shoulder
Set 1	0	3	7
Set 2	0	5	6
Set 3	1	2	8

The shoulder type for the doweled test sections is shown in table 16. The average widths of the paved shoulder for pavements in groups 1 and 2 were 2.3 and 2.6 m (7.5 and 8.5 ft), respectively. The shoulder information was not specified for two sections each in data groups 1 and 2. For the doweled pavements that had the rate of increase of IRI greater than 0.04 m/km (2.54 inches/mi), the shoulder types were as follows: one section, turf; two sections, granular; one section, asphalt; and two sections, concrete.

No effect of the shoulder type on roughness progression could be concluded based on these data.

Table 16. Shoulder types for doweled pavements.

Data Group	Number of Sections		
	Gravel Shoulder	Concrete Shoulder	Asphalt Shoulder
1	2	8	6
2	4	5	3

Effect of Traffic

The traffic information available in the LTPP database was analyzed to obtain an estimate of the ESALs applied on the pavement the first year after the pavement was opened to traffic and to obtain a traffic growth rate. The method presented by Byrum and Kohn was used to obtain these values.[35] Thereafter, these values were used to compute the cumulative ESALs each test section has received at the last profile date.

Tables 17 and 18 show the distribution of cumulative traffic for the nondoweled and doweled sections, respectively. These tables indicate that some sections showing little increase in roughness have carried a significant amount of traffic. Conversely, some sections that have carried lower traffic volumes are showing a high rate of increase of IRI.

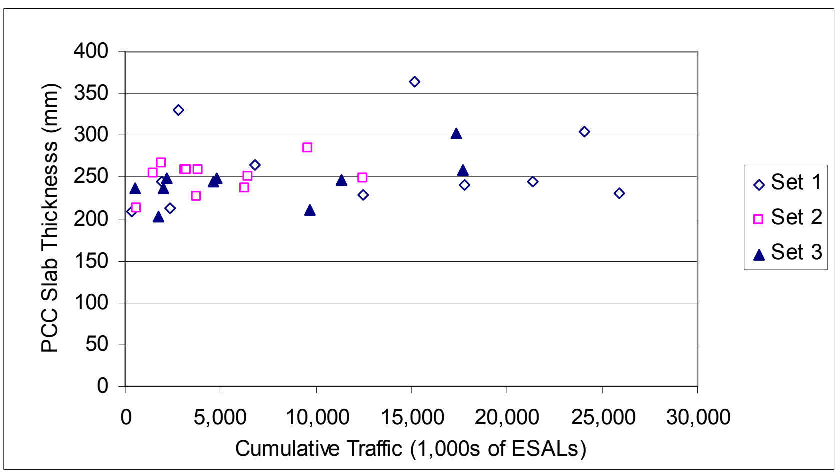
Table 17. Cumulative traffic distribution for nondoweled pavements.

Cumulative ESALs	Number of Sections		
	Data Set 1	Data Set 2	Data Set 3
< 2.5 million	3	3	3
2.5 to 5 million	1	4	4
5 to 7.5 million	1	2	0
7.5 to 10 million	0	1	1
> 10 million	6	1	4

Table 18. Cumulative traffic distribution for doweled pavements.

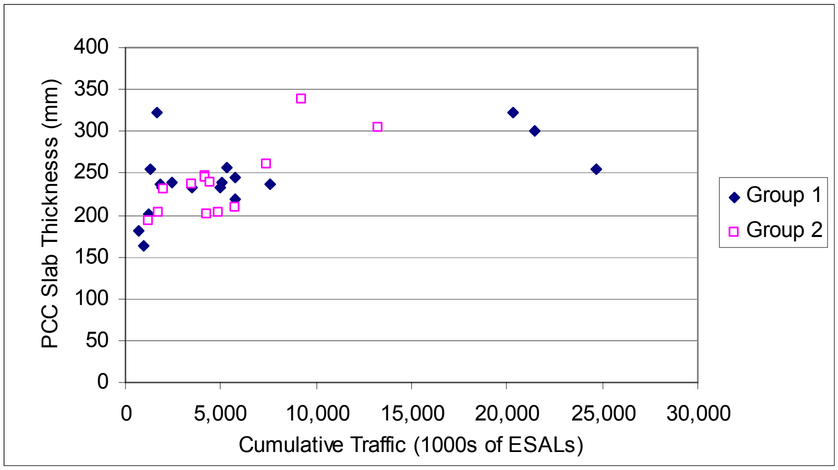
Cumulative ESALs	Number of Sections	
	Group 1	Group 2
< 2.5 million	8	3
2.5 to 5 million	2	6
5 to 7.5 million	4	3
7.5 to 10 million	1	1
> 10 million	3	2

The relationship between cumulative traffic and PCC slab thickness for nondoweled and doweled sections is shown in figures 48 and 49, respectively. These figures show that the sections showing high rates of increase of roughness have not necessarily been subjected to very high traffic volumes.



1 mm = 0.039 inch

Figure 48. Slab thickness versus cumulative traffic for nondoweled pavements.



1 mm = 0.039 inch

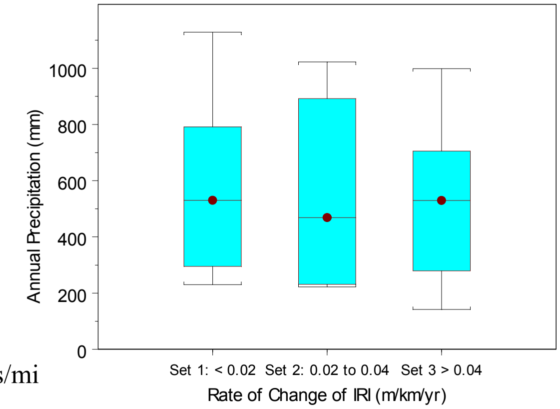
Figure 49. Slab thickness versus cumulative traffic for doweled pavements.

Effect of Environmental Conditions

The effect of annual precipitation, mean annual air temperature, and freezing index on roughness progression was investigated.

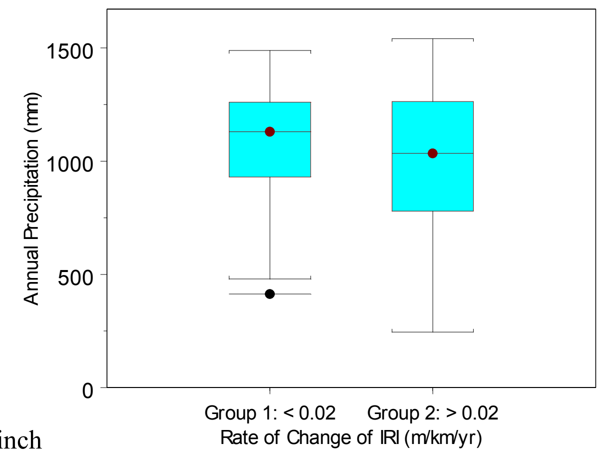
Annual Precipitation

The distribution of annual precipitation at nondoweled and doweled sections is shown in figures 50 and 51, respectively. The median annual precipitation for nondoweled data sets 1, 2 and 3 was 530, 469, and 530 mm (20.9, 18.5, and 20.9 inches), respectively. The median annual precipitation for doweled groups 1 and 2 were 1,130 and 1,034 mm (44.5 and 40.7 inches), respectively. No specific effect of precipitation on roughness progression was noted for either doweled or nondoweled sections.



1 mm = 0.039 inch
1 m/km = 63.4 inches/mi

Figure 50. Distribution of annual precipitation for nondoweled pavements.



1 mm = 0.039 inch
1 m/km = 63.4 inches/mi

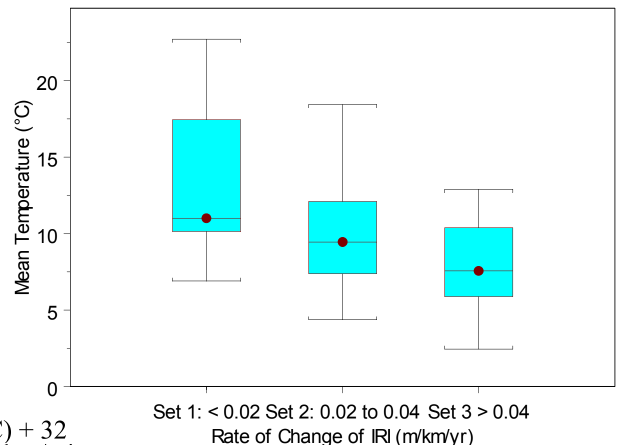
Figure 51. Distribution of annual precipitation for doweled pavements.

Mean Temperature

The distribution of mean annual air temperature for the nondoweled data sets is shown in figure 52. The median temperature for the nondoweled data sets 1, 2, and 3 was 11, 9.5, and 7.6 °C (52, 49, and 46 °F), respectively. The data show that nondoweled pavements in climates having lower mean temperatures have performed poorly compared to those in areas having a higher mean temperature. In nondoweled pavements, load transfer is provided by aggregate interlock. Higher temperatures result in better load transfer, thus reducing faulting and the chances of other distresses occurring at the joints. This situation in turn results in a lower roughness.

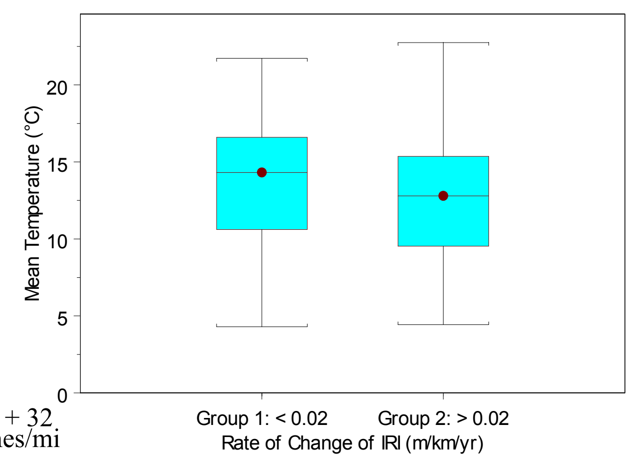
Figure 53 shows the distribution of mean annual air temperatures for the doweled data sets. The median temperature for doweled groups 1 and 2 was 14.3 and 12.8 °C (58 and 55 °F), respectively. No strong effect of mean temperature on roughness progression was observed for the doweled data set, although the range of temperatures between the 25th and 75th percentile values was higher for group 1 compared to group 2.

Trends similar to that observed for mean temperature were seen for the nondoweled and doweled pavements when the effect of number of days above 32 °C (90 °F) on roughness progression was evaluated.



°F = 1.8(°C) + 32
1 m/km = 63.4 inches/mi

Figure 52. Distribution of mean annual temperature for nondoweled pavements.



°F = 1.8(°C) + 32
1 m/km = 63.4 inches/mi

Figure 53. Distribution of mean annual temperature for doweled pavements.

Freezing Index

The distribution of freezing index for the nondoweled data sets is shown in figure 54. The median freezing index for data sets 1, 2, and 3 was 278, 271, and 581 °C days (532.4, 519.8, and 1,077.8 °F days), respectively. Figure 54 shows that pavements in areas having lower freezing indices perform better than those in higher freezing index areas. Areas having higher freezing indices will also have lower temperatures that affect the load transfer capability in nondoweled pavements, resulting in poorer pavement performance. In addition, greater roughness can occur in high freezing index areas due to frost effects, if a frost susceptible base or subgrade is present below the PCC slab.

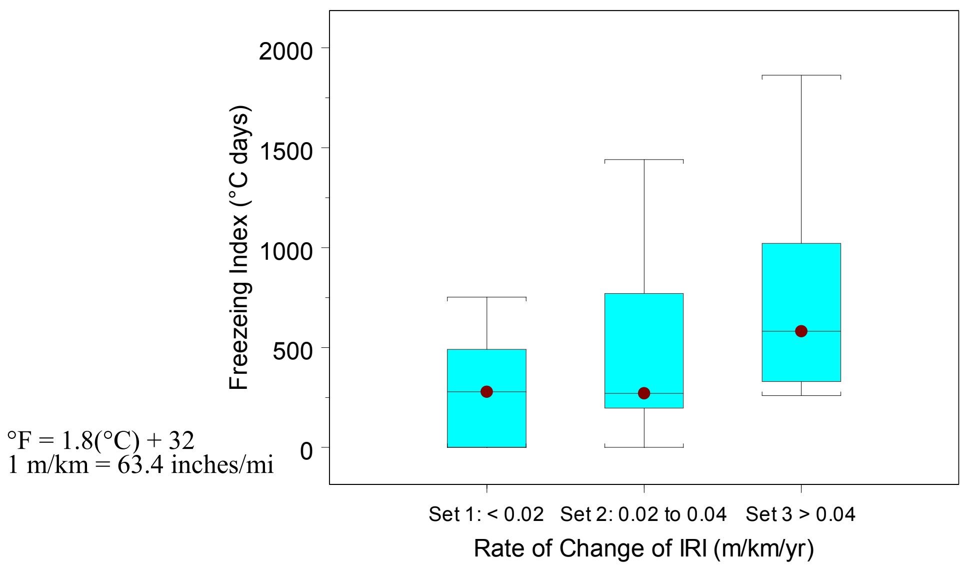
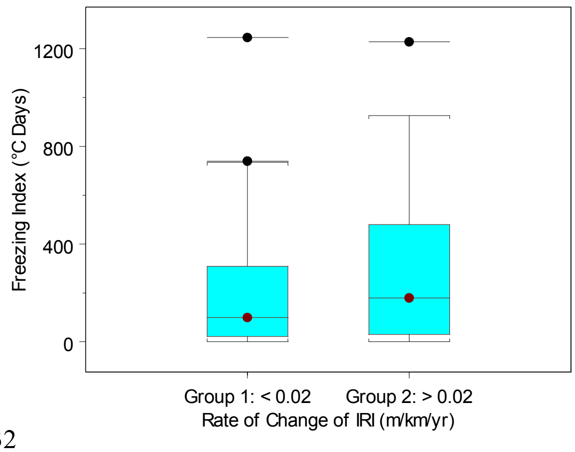


Figure 54. Distribution of freezing index for nondoweled pavements.

The distribution of freezing index for the doweled pavements is shown in figure 55. The median freezing index for pavements in groups 1 and 2 was 99 and 179 °C days (210 and 354 °F days), respectively. As shown in this figure, group 1 pavements overall have a lower freezing index than group 2 pavements, which indicates that pavements located in warmer areas performed better.



°F = 1.8(°C) + 32
1 m/km = 63.4 inches/mi

Figure 55. Distribution of freezing index for doweled pavements.

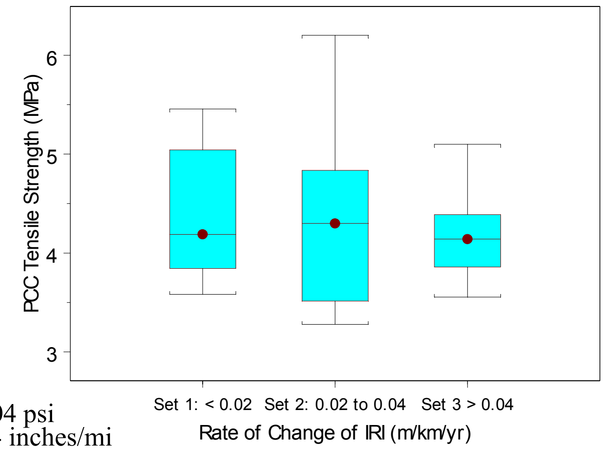
Trends similar to that observed for freezing index were seen for nondoweled and doweled pavements when the effect of number of days below 0 °C (32 °F) on roughness progression was evaluated.

Effect of Concrete Properties

The effect of the following concrete properties on roughness progression was investigated: split tensile strength, elastic modulus, compressive strength, and concrete mix design parameters. Sufficient data were not available for CTE to investigate differences between data sets.

Split Tensile Strength

Figure 56 shows the distribution of split tensile strength of PCC for the nondoweled data sets. The median PCC split tensile strength for data sets 1, 2, and 3 were 4.19, 4.30, and 4.14 megapascals (MPa) (608, 624, and 600 poundforce per square inch (psi)), respectively.

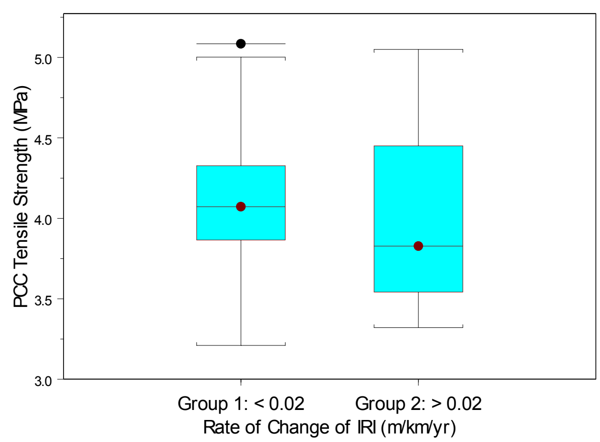


1 MPa = 145.04 psi
1 m/km = 63.4 inches/mi

Figure 56. Distribution of PCC split tensile strength for nondoweled pavements.

Figure 57 shows the distribution of split tensile strength for doweled sections. The median split tensile strength for data groups 1 and 2 were 4.07 and 3.83 MPa (590 and 555 psi), respectively. The values for group 1 had a tighter distribution around the median compared to group 2. In group 1, 22 percent of the sections had a tensile strength of less than 3.8 MPa (551 psi), while this number was 50 percent for group 2.

The split tensile strength of concrete is directly related to the flexural strength of concrete; higher split tensile strength values are associated with higher flexural strengths. Overall, for both nondoweled and doweled pavements, better pavement performance from a roughness point of view was seen for the sections having a higher split tensile strength.

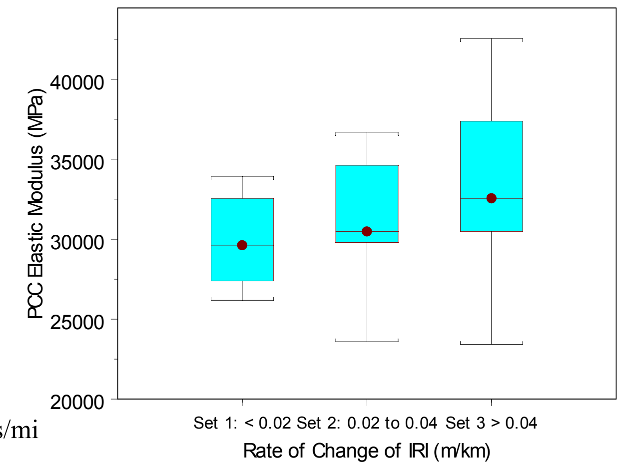


1 m/km = 63.4 inches/mi
1 MPa = 145.04 psi

Figure 57. Distribution of PCC split tensile strength for doweled pavements.

Elastic Modulus

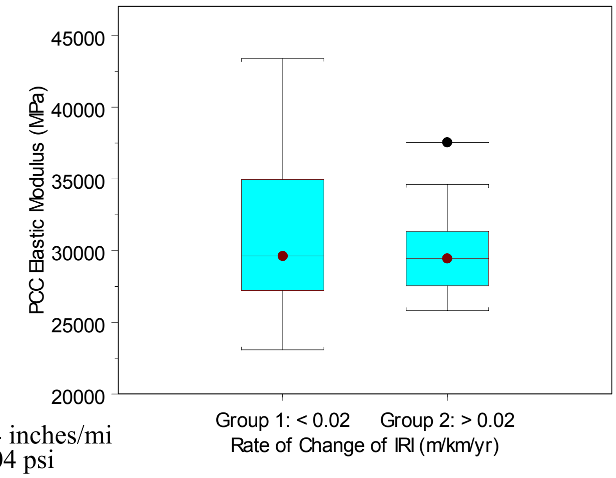
Figure 58 shows the distribution of PCC elastic modulus for the nondoweled data sets. The median PCC elastic modulus for data sets 1, 2, and 3 were 29,627, 30,488, and 32,555 MPa (4.29, 4.42, and 4.72 million psi), respectively. The box plot shows that high PCC moduli values are associated with higher rates of change of IRI.



1 m/km = 63.4 inches/mi
1 MPa = 145.04 psi

Figure 58. Distribution of PCC elastic modulus for nondoweled pavements.

Figure 59 shows the distribution of PCC elastic modulus for the doweled pavements. The median PCC modulus for groups 1 and 2 were 29,627 and 29,455 MPa (4.29 and 4.27 million psi), respectively. The relationship between the roughness progression rate and elastic modulus for the doweled pavements seems to be opposite from the relationship for nondoweled pavements. The pavements with lower rates of change of IRI had higher PCC moduli values.

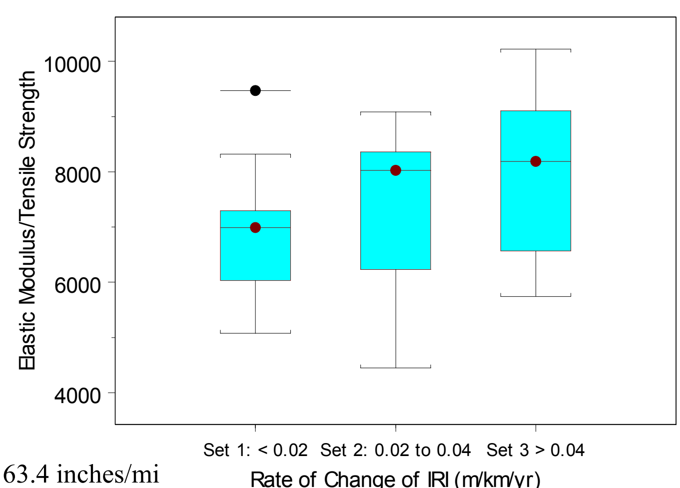


1 m/km = 63.4 inches/mi
1 MPa = 145.04 psi

Figure 59. Distribution of PCC elastic modulus for doweled pavements.

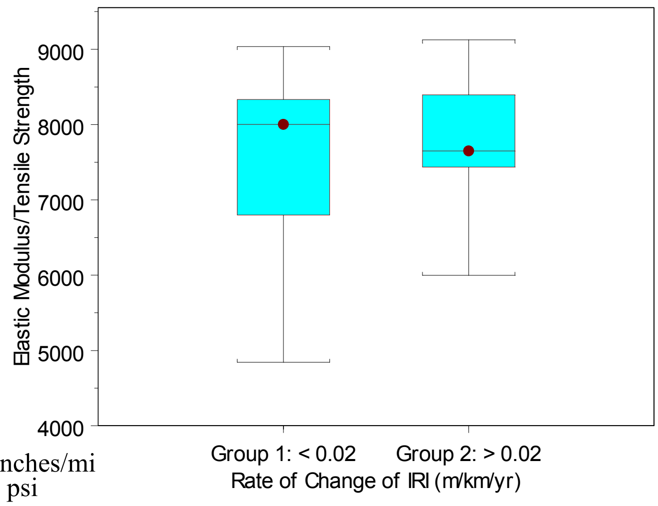
The ratio between the elastic modulus and the split tensile strength of concrete is an indicator of the brittleness of concrete. High values of this ratio indicate a more brittle mix. Figures 60 and 61 show the distribution of this ratio for the nondoweled and the doweled data sets. The median of this ratio for the nondoweled data sets 1, 2, and 3 was 6,989, 8,024, and 8,172, respectively. The median of this ratio for the doweled sections in groups 1 and 2 was 8,002 and 7,649, respectively.

When the range of the ratios above the median was evaluated, it was observed that the range for the nondoweled data set 3 was much higher than that for all other data sets in both the nondoweled and doweled data sets. The numbers indicate that sections with very high values for the ratio between elastic modulus and split tensile strength are associated with poor pavement performance.



1 m/km = 63.4 inches/mi
1 MPa = 145.04 psi

Figure 60. Distribution of ratio between PCC elastic modulus and split tensile strength for nondoweled pavements.

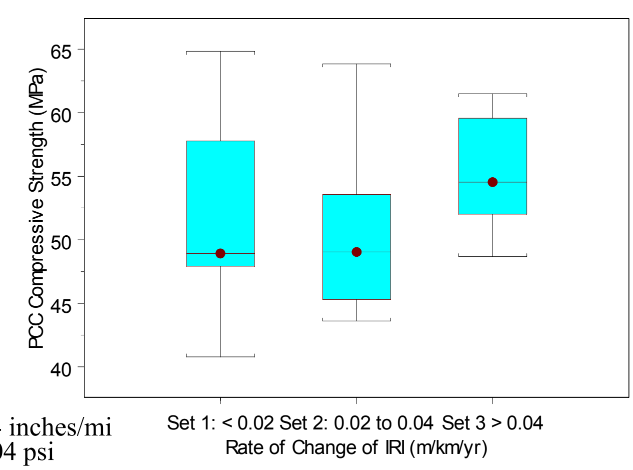


1 m/km = 63.4 inches/mi
1 MPa = 145.04 psi

Figure 61. Distribution of the ratio between PCC elastic modulus and split tensile strength for doweled pavements.

Compressive Strength

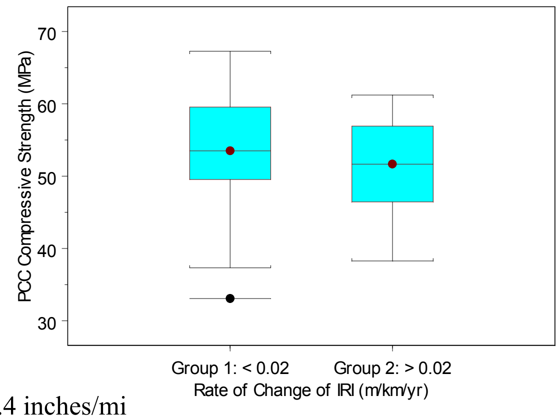
The distribution of PCC compressive strength for the nondoweled data sets is shown in figure 62. The median PCC compressive strength for data sets 1, 2, and 3 were 49, 49, and 55 MPa (7,105, 7,105, and 7,975 psi), respectively. When evaluating both the median and the range of values, it was noted that data set 3 had higher compressive strengths than the other two data sets. Higher cement contents are used for mixes in the freezing regions. It was seen earlier in this report that the pavement sections in data set 3 usually had high freezing indices. Therefore, the higher compressive strength seen for data set 3 is related to these sections being in freezing regions, and thus had higher cement contents that resulted in higher compressive strengths.



1 m/km = 63.4 inches/mi
1 MPa = 145.04 psi

Figure 62. Distribution of compressive strength for nondoweled pavements.

Figure 63 shows the distribution of PCC compressive strength for doweled sections. The median PCC compressive strength for groups 1 and 2 was 54 and 52 MPa (7,830 and 7,540 psi), respectively. No relationship between the compressive strength and rate of increase of roughness can be seen.



1 m/km = 63.4 inches/mi
1 MPa = 145.04 psi

Figure 63. Distribution of compressive strength for doweled pavements.

Concrete Mix Design Parameters

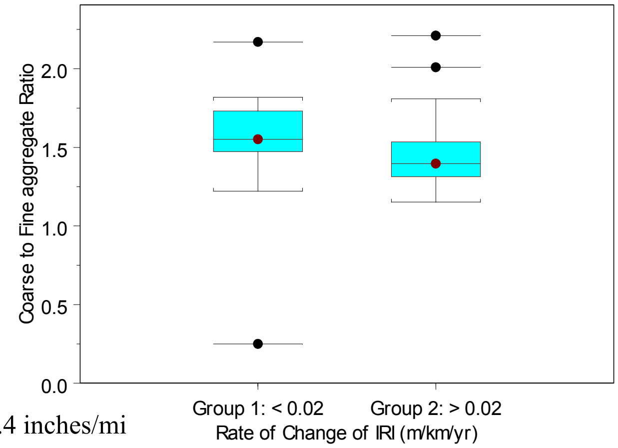
Concrete mix design parameters such as the weight of cement, fine aggregate, coarse aggregate, and the ratio between coarse and fine aggregate in the mix were examined separately for nondoweled and doweled sections to investigate whether differences between the data sets in each pavement type could be observed.

Generally, the amount of cement in the concrete mix in the freezing regions was higher than that in the nonfreezing regions. No differences between the data sets in either nondoweled or doweled sections related to the weight of any mix component could be found.

For nondoweled sections, the ratio between coarse and fine aggregate (by weight) had a very wide range: from 0.43 to 2.01. No distinct differences in this ratio between the three data sets in the nondoweled pavements could be observed. The distribution of the coarse-to-fine aggregate ratio for doweled sections is shown in figure 64. The median of this ratio for groups 1 and 2 was 1.55 and 1.40, respectively. These data indicate that generally the coarse-to-fine aggregate ratio for group 1 pavements is higher than that for group 2 pavements.

GPS-3 SECTIONS SHOWING A HIGH RATE OF INCREASE OF ROUGHNESS

An evaluation of pavement sections that fell into data set 3 was performed to identify the factors that contributed to the high rate of increase of roughness.



1 m/km = 63.4 inches/mi

Figure 64. Distribution of coarse-to-fine aggregate ratio for doweled pavements.

Nondoweled Sections

Eleven nondoweled sections had a rate of change of IRI greater than 0.04 m/km/yr (2.54 inches/mi/yr) (i.e., data set 3). These sections are shown in table 19. The IRI at the first and last profile dates, the change in IRI between these two dates, CI at first and last profile dates, and the total faulting at the section from the distress survey performed closest to the last profile date are also shown in this table. If a fault measurement is not available within 3 years of the last profile date, the faulting at the site is indicated as being not available in this table.

All nondoweled data set 3 sections had a total faulting greater than 60 mm (2.4 inches). Most of these sections also had large changes in CI between the first and the last profile dates. A brief description of each section is set out below.

Section 203060

This section is located in Kansas, was constructed in 1984, and has a 4.6-m (15-ft) joint spacing. The first and last profile years were 1990 and 2002, respectively. During this period, the IRI increased from 1.11 to 1.88 m/km (70 to 119 inches/mi). The pavement structure consists of a 250-mm (9.8-inch) PCC slab placed on an 86-mm (3.4-inch) cement-treated base. The compressive strength and the tensile strength of the PCC were 49 and 3.56 MPa (7,105 and 516 psi), respectively. The elastic modulus of the PCC was 29,110 MPa (4.22 million psi). The total faulting increased from 34 to 83 mm (1.3 to 3.3 inches) from 1994 to 2001. The average faulting at each joint increased from 1 to 2.5 mm (0.04 to 0.10 inches). There was a slight downward curvature in the PCC slabs at the first profile date, and the magnitude of the curvature had increased at the last profile date. This is the only nondoweled section in data set 3 showing a negative curvature at the first and last profile dates.

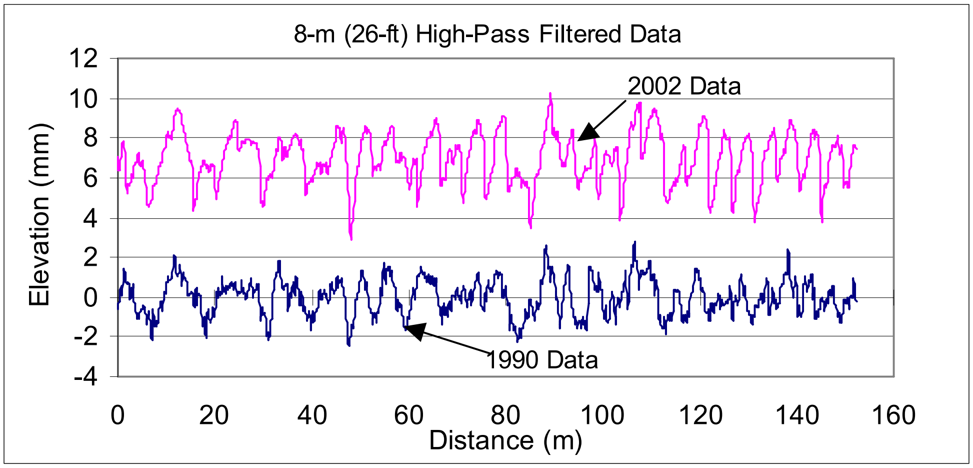
Table 19. Nondoweled data set 3 sections.

GPS Section Number	State or Province	IRI (m/km) First Profile Date	IRI (m/km) Last Profile Date	IRI (m/km) Change	CI x 1,000 (1/m) First Profile Date	CI x 1,000 (1/m) Last Profile Date	Total Faulting (mm (inches))
203060	Kansas	1.11	1.88	0.77	−0.083	−0.358	83 (3.3)
313018	Nebraska	1.29	2.27	0.98	0.005	0.825	158 (6.2)
313033	Nebraska	0.83	1.95	1.12	0.036	0.767	122 (4.8)
323010	Nevada	2.23	3.02	0.79	0.304	0.188	153 (6.0)
323013	Nevada	1.76	2.17	0.42	0.178	0.249	N/A
383005	North Dakota	1.35	1.85	0.50	0.358	1.187	61 (2.4)
463010	South Dakota	2.06	2.52	0.46	0.572	1.035	N/A
493011	Utah	1.32	2.66	1.34	0.094	1.383	105 (4.1)
553015	Wisconsin	1.96	2.89	0.93	0.526	0.574	N/A
563027	Wyoming	2.24	4.20	1.95	0.508	0.889	312 (12.3)
833802	Manitoba	1.83	4.22	2.39	0.287	0.822	102 (4.0)

N/A: Not available

1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Figure 65 shows the high-pass filtered left wheel path profile plots for this section in 1990 and 2002. An increase in downward curvature is seen for the 2002 data. The last distress survey conducted in 2001 indicated that apart from faulting the only other distress present was 0.7 m (2.3 ft) of longitudinal spalling. The increase in IRI at this section is attributed to the increase in downward curvature and faulting.



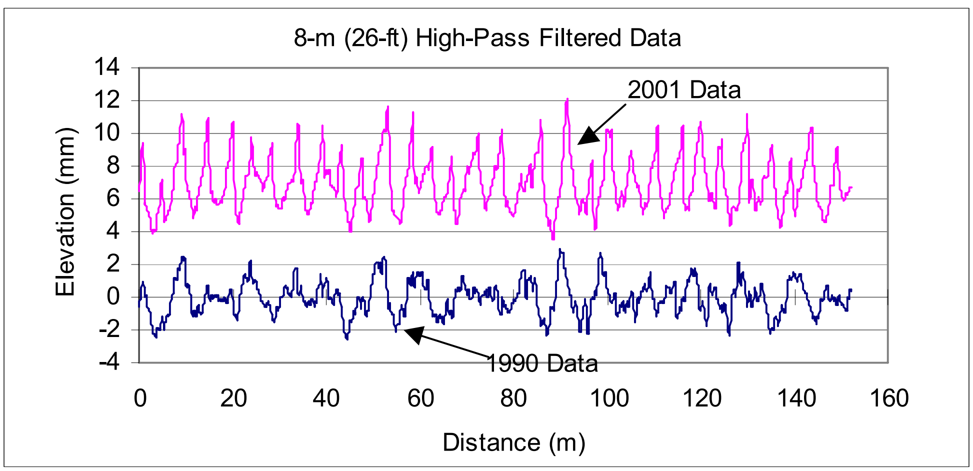
1 m = 3.28 ft
1 mm = 0.039 inch

Figure 65. Profile data from 1990 and 2002—section 203060.

Section 313018

This section is located in Nebraska, was constructed in 1985, and has a 4.7-m (15-ft) joint spacing. The first and last profile years for this section were 1990 and 2001, respectively. During this period, the IRI increased from 1.29 to 2.27 m/km (82 to 144 inches/mi). The pavement structure consists of a 302-mm (11.9-inch) PCC slab placed on a 142-mm (5.6-inch) cement-treated base. Values for elastic modulus, compressive strength, and tensile strength of the PCC are not available for this section. The total faulting obtained from the 2002 distress survey was 158 mm (6.2 inches), with an average faulting of 4.9 mm (0.2 inch) at each joint. The distress survey conducted in 2002 showed no other distresses.

Figure 66 shows the high-pass filtered left wheel path profile plots for this section in 1990 and 2001. The PCC slabs showed very little curvature at the first profile date. However, a significant amount of upward curvature was noted in the PCC slabs at the last profile date. An evaluation of the time-sequence data indicated a trend of increasing curvature over time. The increase in roughness at this section is attributed to the increase in upward curvature and faulting.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 66. Profile data from 1990 and 2001—section 313018.

Section 313033

This section is located in Nebraska, was constructed in 1986, and has a 4.7-m (15-ft) joint spacing. The first and last profile years were 1989 and 2002, respectively. During this period, the IRI increased from 0.83 to 1.95 m/km (53 to 124 inches/mi). The pavement structure consists of a 236-mm (9.3-inch) PCC slab placed on a 117-mm (4.6-inch) asphalt-treated base. Values for elastic modulus, compressive strength, and tensile strength of the PCC are not available for this section. The total faulting at this section from the 1999 distress survey was 122 mm (4.8 inches), with an average faulting of 3.8 mm (0.15 inch) at each joint. The distress survey performed in 1999 indicated that the other distresses present at this section were two transverse cracks, a 1-m (0.3-ft) length of longitudinal cracking, and 1.1-m (3.6-ft) length of pumping.

The PCC slabs had very little curvature at the first profile date and a significant amount of curvature at the last profile date. The increase in curvature noted at this section was similar to that observed at section 313018, also located in Nebraska. An evaluation of the time-sequence data indicated a trend of increasing curvature over time. The increase in roughness at this section is attributed to the increase in curvature and faulting.

Section 323010

The section is located in Nevada, was constructed in 1982, and has a 4.7-m (15-ft) joint spacing. The first and last profile years were 1989 and 1999, respectively. During this period, IRI increased from 2.23 to 3.02 m/km (141 to 191 inches/mi). The pavement structure consists of a 246-mm (9.7-inch) PCC slab placed on a 142-mm (5.6-inch) cement-treated base. The elastic modulus of the PCC was 23,426 MPa (3.39 million psi), while the tensile strength of the PCC was 4.08 MPa (592 psi). The PCC compressive strength is not available for this section. The distress survey conducted in 2000 indicated that the total faulting at this site was 153 mm (6 inches), with an average faulting of 4.8 mm (0.2 inch) at each joint. The distress survey conducted in 2000 showed numerous distresses at this section. The slabs had a slight upward curvature at both the first and last profile dates; the amount of curvature at the last profile date was slightly less than that obtained at the first profile date. The high roughness at this section is attributed to the high faulting and other distress present at this section.

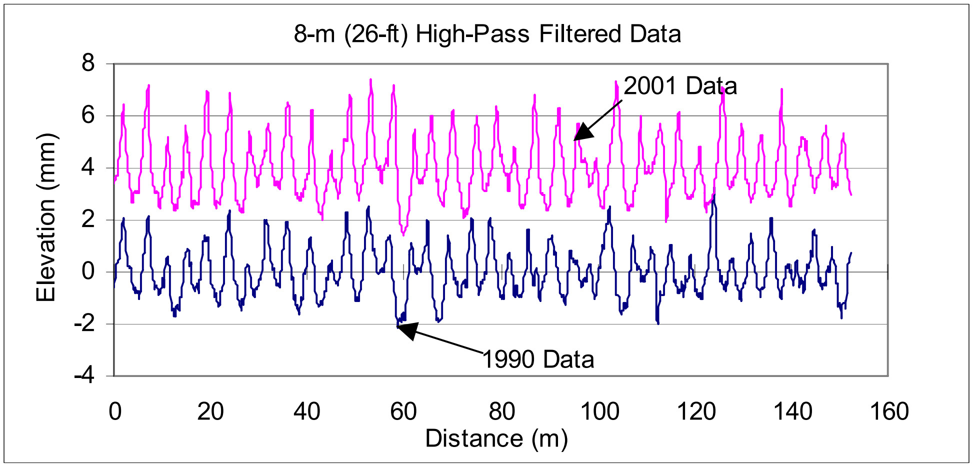
Section 323013

The section is located in Nevada, was constructed in 1981, and has a 4.7-m (15-ft) joint spacing. The first and last profile years were 1989 and 1998, respectively. During this period, the IRI increased from 1.76 to 2.17 m/km (112 to 138 inches/mi). The pavement structure consists of a 211-mm (8.3-inch) PCC slab placed on a 91-mm (3.6-inch) cement-treated base. The compressive strength and the tensile strength of the PCC were 60 and 3.84 MPa (8,700 and 557 psi), respectively. The elastic modulus of the PCC was 37,378 MPa (5.42 million psi). The fault measurements available for this section were obtained in 1992, which was 6 years before the last profile date, and the total faulting at that time was 26 mm (1.02 inches). The distress survey performed in 1997 indicated that there was some longitudinal cracking and transverse cracking at this section. A slight increase in PCC slab curvature was noted at the last profile date. The cause for the increase in IRI at this section is not clear from the available data.

Section 383005

The section is located in North Dakota, was constructed in 1985, and has a 4.2-m (13.8-ft) joint spacing. The first and last profile years were 1989 and 1998, respectively. During this period, IRI increased from 1.35 to 1.85 m/km (86 to 117 inches/mi). The pavement structure consists of a 203-mm (8-inch) PCC slab placed on a 51-mm (2-inch) granular base. The compressive strength and the tensile strength of the PCC were 54 and 3.86 MPa (7,830 and 560 psi), respectively. The elastic modulus of the PCC was 32,728 MPa (4.75 million psi). The total faulting at this site obtained from the 1999 distress survey was is 61 mm (2.4 inches), with an average faulting of 1.7 mm (0.07 inch) at each joint. No other distresses were present at this section.

The PCC slabs had an increase in upward curvature between the first and the last profile dates. An evaluation of time-sequence profile data indicated that the curvature was increasing over time. Figure 67 shows the high-pass filtered left wheel path profile plot for the 1989 and 2001 profiles. As seen in the plot, the PCC slabs were curled upwards on both profile dates, with the 2001 data showing a greater magnitude of curvature. The increase in IRI is attributed to the increase in slab curvature and faulting.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 67. Profile data from 1990 and 2001—section 383005.

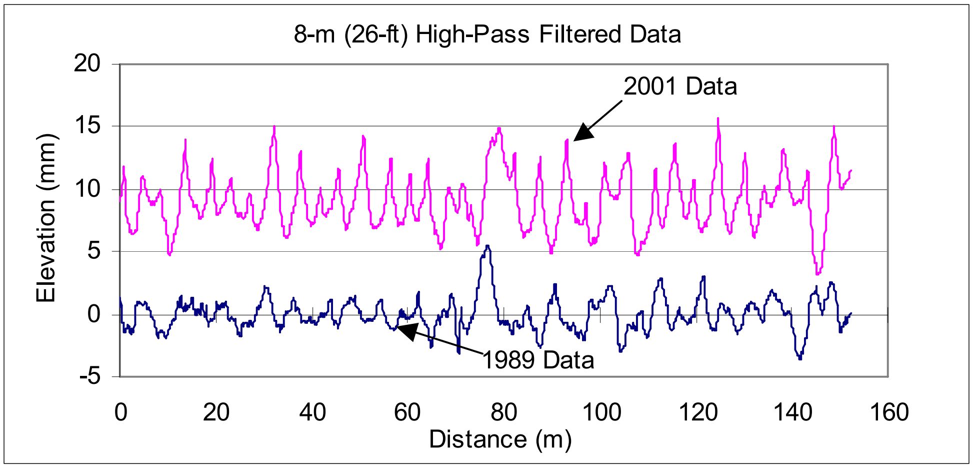
Section 463010

The section is located in South Dakota, was constructed in 1983, and has a 4.6-m (15-ft) joint spacing. The first and last profile years were 1989 and 1999. During this period, IRI increased from 2.05 to 2.52 m/km (130 to 160 inches/mi). The pavement structure consists of a 236-mm (9.3-inch) PCC slab placed on a 41-mm (1.6-inch) granular base. The compressive strength and the tensile strength of the PCC were 55 and 4.39 MPa (7,975 and 637 psi), respectively. The elastic modulus of the PCC was 39,962 MPa (5.79 million psi). Faulting data were not available for this section in the database. Distress data at a date close to the last profile date were not available in the database. The PCC slabs showed an increase in upward curvature between the first and the last profile dates. This increase in curvature has contributed to the increase in roughness.

Section 493011

This section is located in Utah, was constructed in 1980, and has a 4.6-m (15-ft) joint spacing. The first and last profile years were 1989 and 2001. During this period, IRI increased from 1.32 to 2.66 m/km (84 to 167 inches/mi). The pavement structure consists of a 259-mm (10-inch) PCC slab placed on a 102-mm (4-inch) cement-treated base. The elastic modulus of the PCC was 30,488 MPa (4.42 million psi); the tensile strength of the PCC was 4.64 MPa (673 psi). The total faulting at this section in 2002 was 105 mm (4.1 inches), with an average faulting of 3.2 mm (0.13 inch) at each joint. The distress survey performed in 2002 indicated that the other distresses at this section were one transverse crack and a transverse spall.

The PCC slabs had a slight upward curvature at the first profile date and a significant upward curvature at the last profile date. Figure 68 shows the high-pass filtered left wheel path profile plots for the 1989 and 2001 data. The significant increase in slab curvature is evident in the plot. An evaluation of the time-sequence profile data indicated that the curvature of the PCC slabs was increasing over time. The increase in IRI is attributed to the increase in slab curvature and faulting.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 68. Profile data from 1989 and 2001—section 493001.

Section 553015

This section is located in Wisconsin, was constructed in 1984, and has a 4.7-m (15-ft) joint spacing. The first and last profile years were 1990 and 1999, respectively. During this period, the IRI increased from 1.96 to 2.89 m/km (124 to 183 inches/mi). The pavement structure consists of a 244-mm (9.6-inch) PCC slab placed on a 203-mm (8-inch) granular base. The compressive strength and the tensile strength of the PCC were 61 and 4.16 MPa (8,845 and 603 psi), respectively. The elastic modulus of the PCC was 42,546 MPa (6.17 million psi). This section had the highest elastic modulus of the analyzed GPS-3 sections.

The latest faulting data available in the database is from 1992, and the total faulting then was 70 mm (2.8 inches). An evaluation of the profile data collected in 1999 indicated significant faulting. Hence, it appears that the amount of faulting had increased over the years. The distress survey conducted in 1996 indicated that a significant amount of longitudinal spalling was present. The PCC slabs at both profile dates had an upward curvature; however, the magnitude of curvature was virtually the same for the first and the last profile dates. The increase in roughness at this section is attributed to the high faulting.

Section 563027

This section is located in Wyoming, was constructed in 1981, and has a 4.7-m (15-ft) joint spacing. The first and last profile years were 1989 and 2001. During this period, IRI increased from 2.24 to 4.20 m/km (142 to 266 inches/mi). The pavement structure is not available in the database. The compressive strength and the tensile strength of the PCC were 52 and 5.10 MPa (7,540 and 740 psi), respectively. The elastic modulus of the PCC was 31,177 MPa (4.52 million psi). The total faulting at this section in 2001 was 312 mm (12.3 inches), with an average faulting of 9.8 mm (0.4 inch) at each joint. The other distress at this site that was noted during the 2001 distress survey were three transverse cracks, 1.8 m (5.9 ft) of longitudinal spalls, and six transverse spalls having a total length of 1.3 m (4.3 ft). The amount of slab curvature in this section has increased from the first to the last profile dates. The cause for most of the increase in roughness is attributed to faulting, and the increase in slab curvature would have some effect on the increase in IRI.

Section 833802

This section is located in Manitoba, was constructed in 1985, and has a 4.6-m (15-ft) joint spacing. The first and last profile years were 1989 and 2000. During this period, IRI increased from 1.83 to 4.22 m/km (116 to 268 inches/mi). The pavement structure for this section is not available in the database. The compressive strength and the tensile strength of the PCC were 56 and 4.14 MPa (8,120 and 600 psi), respectively. The elastic modulus of the PCC was 32,555 MPa (4.72 million psi). The total faulting at this section observed during the 2000 distress survey was 102 mm (4 inches), with an average faulting of 3.1 mm (0.12 inch) at each joint. A significant amount of transverse spalling is also present at this section. The PCC slabs at this section were curled up during the first and the last profile dates, with the magnitude of the curvature being higher at the last profile date. The increase in roughness is attributed to the high faulting, spalling, and the increase in upward curvature of PCC slabs.

Doweled Sections

Six doweled sections had a rate of increase of IRI greater than 0.04 m/km/yr (2.54 inches/mi/yr) (i.e., data set 3). These sections are shown in table 20. This table also shows IRI at the first and last profile dates, the change in IRI between these two dates, CI at first and last profile dates, and the total faulting at the section from a distress survey performed closest to the last profile date. Only two doweled sections had a total faulting greater than 25 mm (1 inch). A brief description of each of these sections is provided below.

Table 20. Doweled data set 3 sections.

GPS Section Number	State or Province	IRI (m/km)			CI x 1,000 (1/m)		Total Faulting (mm (inches))
		First Profile Date	Last Profile Date	Change	First Profile Date	Last Profile Date	
123804	Florida	1.52	2.12	0.60	−0.005	−0.206	109 (4.3)
163017	Idaho	1.58	2.04	0.45	−0.077	−0.483	25 (1.0)
273003	Minnesota	1.94	2.64	0.69	−0.876	−1.642	10 (0.4)
283019	Mississippi	1.58	2.13	0.55	0.083	−0.043	65 (2.6)
373008	North Carolina	1.79	2.17	0.38	−0.081	−0.392	5 (0.2)
893015	Quebec	1.16	3.02	1.86	−0.069	−0.411	7 (0.3)

1/m = 1/3.28 ft
1 m/km = 63.4 inches/mi

Section 123804

The section is located in Florida, was constructed in 1985, and has a 5.9-m (19-ft) joint spacing. The first and last profile years were 1990 and 2000, respectively. During this period, IRI increased from 1.52 to 2.12 m/km (96 to 134 inches/mi). The pavement structure consists of a 305-mm (12-inch) PCC slab placed on a 170-mm (6.7-inch) cement-treated base. The compressive strength and the tensile strength of the PCC were 42 and 3.54 MPa (6,090 and 513 psi), respectively. The elastic modulus of the PCC was 27,216 MPa (3.94 million psi).

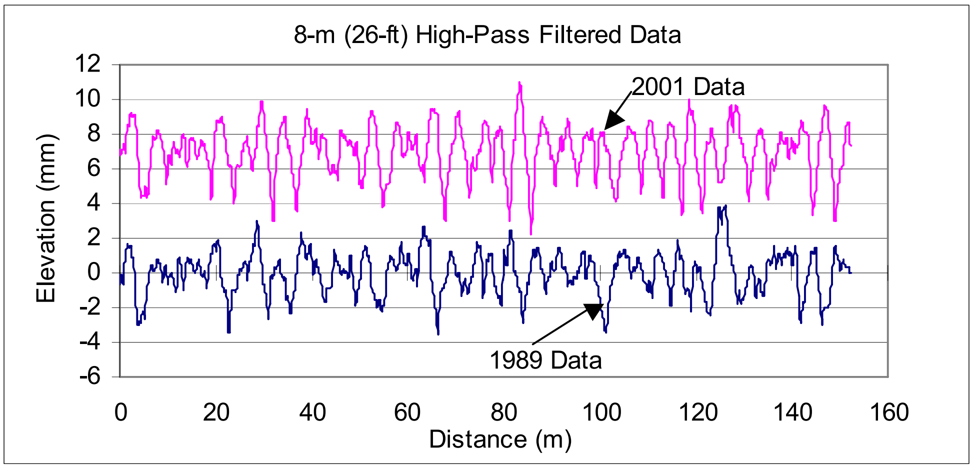
This pavement had the highest faulting of the doweled sections analyzed in this study. This was the only doweled section in which faulting was recorded at the cracks. The total faulting obtained from the 2002 distress survey was 109 mm (4.3 inches), with a total faulting at joints and cracks of 65 and 44 mm (2.6 and 1.7 inches), respectively. A slight increase in downward curvature is seen between the first and the last profile dates. The high increase in roughness is attributed to the faulting. This section had the fourth highest cumulative traffic of the analyzed doweled section.

Section 163017

The section is located in Idaho, was constructed in 1986, and has a 4.4-m (14-ft) joint spacing. The first and last profile years were 1989 and 2001, respectively. During this period, IRI increased from 1.58 to 2.04 m/km (100 to 129 inches/mi). The pavement structure consists of a 262-mm (10.3-inch) PCC slab placed on a 137-mm (5.4-inch) cement-treated base. The elastic modulus of the PCC was 31,350 MPa (4.54 million psi) and the tensile strength of the PCC was 3.73 MPa (544 psi). The compressive strength of the PCC is not available. The total faulting at this site from the 1999 distress survey was 25 mm (1 inch), with an average faulting of 0.7 mm

(0.03 inches) at each joint. The only other distress recorded during the 2002 survey was longitudinal cracking.

The downward curvature of the concrete slabs increased significantly between the first and last profile dates. Figure 69 shows the high-pass filtered left wheel path profile plots for 1989 and 2001. The plots show a clear increase in downward curvature of the PCC slabs between the years. An evaluation of the monitored data indicated that the downward curvature had increased over time. The increase in downward curvature of the slabs contributed to the increase in roughness.



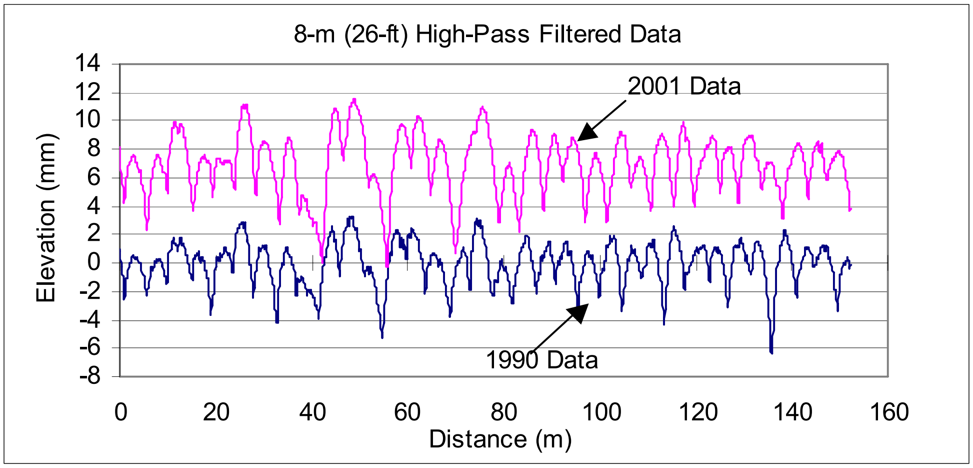
1 m = 3.28 ft
1 mm = 0.039 inch

Figure 69. Profile data from 1989 and 2001—section 163017

Section 273003

The section is located in Minnesota, was constructed in 1986, and has a 4.6-m (15-ft) joint spacing. The first and last profile years were 1990 and 2001, respectively. During this period, IRI increased from 1.94 to 2.64 m/km (123 to 167 inches/mi). The pavement structure consists of a 193-mm (7.6-inch) PCC slab placed on a 127-mm (5-inch) granular base. The compressive strength and the tensile strength of the PCC were 55 and 3.78 MPa (798 and 548 psi), respectively. The elastic modulus of the PCC was 34,450 MPa (5 million psi). The total faulting obtained from the 1999 distress survey was 10 mm (0.4 inch), which is a very low value. The only other distress noted during the 2002 survey was four transverse spalls.

Figure 70 shows the high-pass filtered left wheel path profile plots for 1990 and 2001. The section had a significant downward curvature at the first profile date, and the plot shows the downward curvature has increased from 1990 to 2001. The CI at the last profile date was the highest for all doweled sections. An evaluation of the time-sequence profile data indicated that the downward curvature of the slabs had increased over time. The increase in roughness is attributed to the increase in downward curvature of the PCC slabs.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 70. Profile data from 1990 and 2001—section 273003.

Section 283019

The section is located in Mississippi, was constructed in 1984, and has a 6.1-m (20-ft) joint spacing. The first and last profile years were 1990 and 2001, respectively. During this period, IRI increased from 1.58 to 2.13 m/km (100 to 135 inches/mi). The pavement structure consists of a 239-mm (9.4-inch) PCC slab placed on a 150-mm (6-inch) cement-treated base.

The compressive strength and the tensile strength of the PCC were 52 and 4.80 MPa (7,540 and 696 psi), respectively. The elastic modulus of the PCC was 28,776 MPa (4.17 million psi). The total faulting at this site during the 2000 distress survey was 65 mm (2.6 inches), with an average faulting of 2.7 mm (0.10 inch) at each joint. This section had the second highest faulting of all doweled sections. Numerous other distresses were noted at this section, including longitudinal cracking and spalling. This section had a slight upward curvature at the first profile date and a slight downward curvature at the last profile date. The faulting and other distresses are attributed to the increase in roughness.

Section 373008

The section is located in North Carolina, was constructed in 1984, and has a 6.5-m (21-ft) joint spacing. The first and last profile years were 1990 and 2001, respectively. During this period, IRI increased from 1.79 to 2.17 m/km (113 to 138 inches/mi). The pavement structure consists of a 239-mm (9.4-inch) PCC slab placed on a 150-mm (6-inch) cement-treated base. The compressive strength and the tensile strength of the PCC were 52 and 4.80 MPa (7,540 and 696 psi), respectively. The elastic modulus of PCC was 28,776 MPa (4.17 million psi). The 2002 distress survey indicated the total faulting at this site to be a negligible 5 mm (0.2 inch). The 2002 distress survey indicated a significant amount of longitudinal cracking. There was a slight downward curvature in the PCC slabs at the first profile date, and the magnitude of the curvature had increased at the last profile date. The high amount of cracking is the most likely cause for the increase in roughness.

Section 893015

The section is located in Quebec, was constructed in 1984, and has a 6-m (20-ft) joint spacing. The first and last profile years were 1989 and 1999, respectively. During this period, IRI increased from 1.16 to 3.02 m/km (74 to 191 inches/mi). The pavement structure consists of a 208-mm (8.2-inch) PCC slab placed on a 338-mm (13.3-inch) gravel base. The compressive strength and the tensile strength of the PCC were 42 and 3.32 MPa (6,090 and 481 psi), respectively. The elastic modulus of the PCC was 29,627 MPa (4.29 million psi). The total faulting at this site during the 2000 distress survey was 7 mm (0.28 inch).

There was a slight downward curvature in the PCC slabs at the first profile date, and the magnitude of the curvature had increased at the last profile date. The PCC tensile strength is the third lowest of all doweled sections. A significant amount of transverse spalling was observed, along with numerous patches and other distresses during the last distress survey performed at this section. The downward curvature of the slabs would have increased the potential for transverse cracking. The various distresses on this site appear to be the cause for the high roughness.

Discussion of Observations

Most of the nondoweled sections that were showing a high rate of increase of roughness had high faulting and high CI values. The PCC slabs were curled upwards in all sections except for one. An evaluation of the time-sequence profile data of these sections showed that the amount of slab curvature increased over time at most of these sections. The excessive slab curling at these sections would have been a contributing factor that caused high faulting to occur at these sections. When the slabs are curled upward, there is a potential for water to accumulate at the joints, and because the slabs will deflect as traffic passes, the potential for faulting is high.

For the doweled pavements that showed a high rate of increase of roughness, there were a few cases where the slabs showed an excessive amount of downward curvature. The potential for midslab cracking is high when a PCC slab takes such a shape. This slab shape would have been a contributing factor in causing pavement distress at these sections.

CHAPTER 6. SHORT-TERM CHANGES IN INITIAL SMOOTHNESS OF CONCRETE PAVEMENTS

INTRODUCTION

All SHAs that construct PCC pavements have a smoothness specification. Some smoothness specifications will give a timeframe after construction within which smoothness measurements must be performed while others do not. An issue with PCC pavements is the effect of curling and warping of PCC slabs on smoothness. Some contractors perform smoothness measurements as soon as possible after construction because they perceive that curling and warping effects on the pavement will be negligible immediately after construction. However, if curling and warping occurs in the pavement during its early life, causing a significant reduction in smoothness, the validity of bonuses paid based on smoothness measurements obtained immediately after construction becomes questionable.

A study was performed to investigate how the smoothness of a PCC pavement changes during the first 3 months of its life. Test sections were established at the following paving projects for this study:

- State Route (S.R.) 6220, Centre County, PA.
- U.S. Route 20 (U.S. 20), Hardin County, IA.
- Interstate 80 (I-80), Pottawattamie County, IA.
- U.S. Route 23 (U.S. 23), Monroe County, MI.
- Interstate 69 (I-69), Calhoun County, MI.

All of these pavements were doweled JPC pavements. The smoothness measurements were generally performed at the following time periods using an inertial profiler:

- 1 day after paving.
- 3 days after paving.
- 1 week after paving.
- 3 months after paving.

Smoothness measurements were performed in each wheel path, and three repeat measurements were performed for each data set. At several test sections, measurements were obtained in the morning and the afternoon during the 1-week and 3-month testing. At one test section, measurements were also obtained approximately 1 year after construction.

The following analyses were performed on the collected profile data:

1. Obtain average IRI and RN: Compute IRI and RN for all profile runs. Then compute an average IRI and RN for each data set using the values obtained for the three repeat runs.

2. Investigate the repeatability of the profiler: The IRI values obtained for the entire test section from the repeat runs for each data set were evaluated to determine the repeatability of the profiler.

3. Investigate the short-interval IRI repeatability: This investigation was performed by comparing IRI values at 15-m (49-ft) intervals for the repeat runs in each data set. This investigation indicates whether the distribution of the roughness within the section for the three repeat runs was repeatable. In some cases, IRI from the repeat runs for a pavement section can be very close to each other; however, the distribution of roughness within the section for the repeat runs can be variable. When the overall roughness for the section is computed, the roughness variability within the section can cancel out, and give close IRI values for repeat runs even though they may have very different roughness distributions within a section.

4. Evaluate changes in IRI and RN: Compare IRI and RN obtained the first time the section was profiled with values obtained from subsequent profiling. This analysis provides information on how the smoothness of PCC pavements changes during the first few months of their life relative to the smoothness obtained immediately after paving.

5. Evaluate the effect of joints on profile data and smoothness indices: In the two projects in Iowa, a sawcut that is 6 mm (0.25 inches) wide and 25 mm (1 inch) deep is made on the pavement. Thereafter, the joint is sealed. Profile data collected for these two conditions of the joint were available for the two Iowa projects. In the Michigan and the Pennsylvania projects, a 3-mm (0.12-inch)-wide initial sawcut is made on the pavement. Thereafter, the joint is widened to form a reservoir and then sealed. Profile data collected for all three conditions of the joint were available. These profile data were analyzed to see how the joint appeared in the profile data and to investigate the impact of these different joint conditions on IRI and RN.

6. Use roughness profiles to investigate the distribution of roughness: For this analysis, IRI roughness profiles based on an 8-m (26-ft) base length were used. Roughness profiles for the left wheel path, right wheel path, and an overlaid plot that showed profiles for both wheel paths were used in this analysis.

7. Evaluate profile characteristics: Profile data collected at different time periods were analyzed using profile plots and PSD plots to investigate changes in profile, and to see whether a dominant wavelength or a dominant waveband that is affecting IRI is present in the profile. The CI values of the profiles also were computed to investigate slab curvature.

8. CTE values and microscopical examination: Cores obtained from the pavement or cylinders cast from concrete used in the project were used to determine the CTE of the concrete and to perform a microscopical examination to identify the coarse and the fine aggregate in the concrete. The CTE test was carried out using the test procedure CRD-C 39-81 that is described in the U.S. Army Corps of Engineers Materials Testing Handbook.[36] The microscopical examination of the concrete cylinder or core was performed in accordance

with portions of ASTM C 856-02, Standard Practice for Petrographic Examination of Hardened Concrete.[37]

The results obtained at each project are explained separately below.

STATE ROUTE 6220 PROJECT—PENNSYLVANIA

Project Description

This project involved new construction and was located in Centre County, PA. This roadway is a four-lane divided highway with two lanes in each direction. A 168-m (550-ft)-long test section was established for testing on the two southbound lanes. The test section was established between stations 588+00 and 582+50. (Stations are in U.S. customary units).

Pavement Details

Table 21 shows pavement details. The subgrade consisted of blasted rock mixed with fines. Table 22 provides information about the joints in the pavement.

Table 21. Pavement details—S.R. 6220.

Item	Description	Value
Pavement thickness	Concrete thickness	280 mm (10.9 inches)
	Base thickness	100 mm (3.9 inches) asphalt-treated permeable base over 150 mm (5.9 inches) of aggregate base
Pavement width	Total pavement width	7.30 m (24 ft)
	Width of inside lane	3.65 m (12 ft)
	Width of outside lane	3.65 m (12 ft)
Shoulder	Shoulder type	Concrete, 280 mm (10.9 inches) thick
	Width of shoulder	Inside 1.2 m (3.9 ft), outside 3 m (9.8 ft)
Joint spacing	Joint spacing	4.6 m (15.1 ft)
	Joints skewed?	No
Dowels	Dowel type	Epoxy coated
	Dowel diameter	38 mm (1.5 inches)
	Dowel length	457 mm (17.8 inches)
Tining	Tining type	Transverse tining
	Tining spacing	Random spacing
	Tining depth	3.2 to 4.8 mm (0.1 to 0.2 inches)

Table 22. Joint details—S.R. 6220.

Description	Value
Joint formation	Initial sawcut then reservoir widened
Initial sawcut	3 mm (0.12 inch) wide and 93 mm (3.63 inches) deep
Joint reservoir width	9.5 mm (0.37 inch)
Joint reservoir depth	38 mm (1.5 inches)
Sealant type	Hot-pour asphalt
Depth to top of sealant	5 mm (0.2 inches)

Concrete Mix Design

Table 23 presents the mix proportions used for the concrete mix. An entrained air admixture and a water-reducing admixture were added to the concrete mix. Table 24 shows the gradation of the aggregates used in the concrete mix.

Table 23. Mix proportions—S.R. 6220.

Component	Weight Kilograms per cubic meters (kg/m^3 (lb/yd^3))
Cement Type 1	297 (500)
Fly ash	52 (88)
Coarse aggregate	1,097 (1,849)
Sand	653 (1,100)

Table 24. Gradation of aggregates—S.R. 6220.

Sieve	Percentage Passing	
	Coarse Aggregate	Sand
37.5 mm (1.5 inches)	100	–
25.0 mm (1 inch)	100	–
12.5 mm (0.5 inch)	45	–
9.5 mm (0.4 inch)	–	100
No. 4	8	99
No. 8	3	80
No. 16	–	68
No. 30	–	48
No. 50	–	21
No. 100	–	6

Paving Details

Table 25 shows the date and time of test section paving and other details related to the paving process. The concrete was placed using a slipform paver.

Table 25. Paving Information—S.R. 6220.

Item	Description	Comment
Date and time	Date of paving	9/17/03
	Time of paving	9 a.m.
Paving process	Haul route	Adjacent to inside lane
	Stringline	7.6-m (25-ft) spacing, both sides
	Dowels	Fixed to base
	Tie bars	Inserted by paver
	Concrete deposit method	Belt placers
	Spreader used?	Yes, one
	Paver type	GOMACO™ GHP–2800
Concrete	Temperature	21 °C (69.8 °F)
Curing method	Curing compound	Water-based curing compound sprayed on pavement

Profiling of Section

Eight sets of profile data were collected over a 3.5-month period. The pavement was first profiled 1 day after paving; the second set of profiles were collected 3 days after paving. The third and fourth sets of data were collected 1 week after paving in the morning and afternoon, respectively. The fifth and sixth sets of data were collected 1 month after paving in the morning and afternoon, respectively. The seventh and eighth sets of data were collected about 3.5 months after paving in the morning and afternoon, respectively. The profile data were collected using an ICC lightweight laser profiler. This profiler recorded data at 31-mm (1.2-inch) intervals.

Table 26 shows the dates and times when profile data collection was performed, the approximate age of the pavement at each instance, and the low, high, and mean air temperatures for each profiling day.

The 3-mm (0.12-inch)-wide initial sawcut had been made on the pavement when the profile data were collected 1 day after paving. The joint reservoirs had been formed, but the joints were not sealed when the profile data were collected in the morning of the 1-month data collection. The joints were sealed when the 1-month afternoon runs were performed. During the 3.5-month data collection, it was noted that the first 5.5 m (18 ft) of the inside lane had been diamond ground.

Table 26. Profile data collection—S.R. 6220.

Date of Profiling	Approximate Age of Pavement	Time of Profiling				Air Temperature °C (°F)		
		Inside Lane		Outside Lane				
		Left Wheel Path	Right Wheel Path	Left Wheel Path	Right Wheel Path	Low	High	Mean
9/18/03	1 day	2:32 p.m.	2:34 p.m.	2:37 p.m.	N/A	12 (54)	21(70)	17 (63)
9/20/03	3 days	1:02 p.m.	1:14 p.m.	1:35 p.m.	N/A	11(52)	21(70)	16 (61)
9/24/03	7 days	9:44 a.m.	9:53 a.m.	10:04 a.m.	N/A	7 (45)	22 (72)	15 (55)
		5:09 p.m.	5:21 p.m.	5:33 p.m.	N/A			
10/15/03	1 month	8:38 a.m.	8:48 a.m.	8:59 a.m.	9:05 a.m.	7 (45)	13 (55)	11 (52)
		2:32 p.m.	2:40 p.m.	2:48 p.m.	2:56 p.m.			
12/29/03	3.5 months	9:53 a.m.	10:46 a.m.	11:21 a.m.	11:45 p.m.	1 (34)	16 (61)	8 (46)
		3:51 p.m.	4:02 p.m.	4:23 p.m.	4:41 p.m.			

N/A: Profile data not collected

Roughness Indices

IRI Values

The average IRI values computed from the three repeat runs are presented in table 27 and shown graphically in figure 71. The IRI for the right wheel path in the outside lane is not shown in this figure, because data were collected along this path only during the 1-month and 3.5-month data collection. Table 28 shows the percentage change in IRI values for different test sequences with respect to the IRI obtained at 1 day. Values for right wheel of outside lane are not presented in this table because profiling was performed along this path only during the 1-month and 3.5-month data collection.

Table 27. IRI values for different test sequences—S.R. 6220.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	IRI (m/km)			
			Inside Lane		Outside Lane	
			Left Wheel Path	Right Wheel Path	Left Wheel Path	Right Wheel Path
9/18/03	1 day	2:30 to 2:40 p.m.	1.32	1.21	0.96	N/A
9/20/03	3 days	1 to 1:40 p.m.	1.27	1.24	1.01	N/A
9/24/03	7 days	9:45 to 10 a.m.	1.27	1.18	0.96	N/A
		5:05 to 5:35 p.m.	1.27	1.20	0.98	N/A
10/15/03	1 month	8:40 to 9 a.m.	1.30	1.22	0.97	0.90
		2:30 to 3 p.m.	1.31	1.18	0.94	0.93
12/29/03	3.5 months	10 to 11:30 p.m.	1.21	1.13	0.94	0.91
		3:50 to 4:40 p.m.	1.21	1.10	0.94	0.86

N/A: Profile data not collected

1 m/km = 63.4 inches/mi

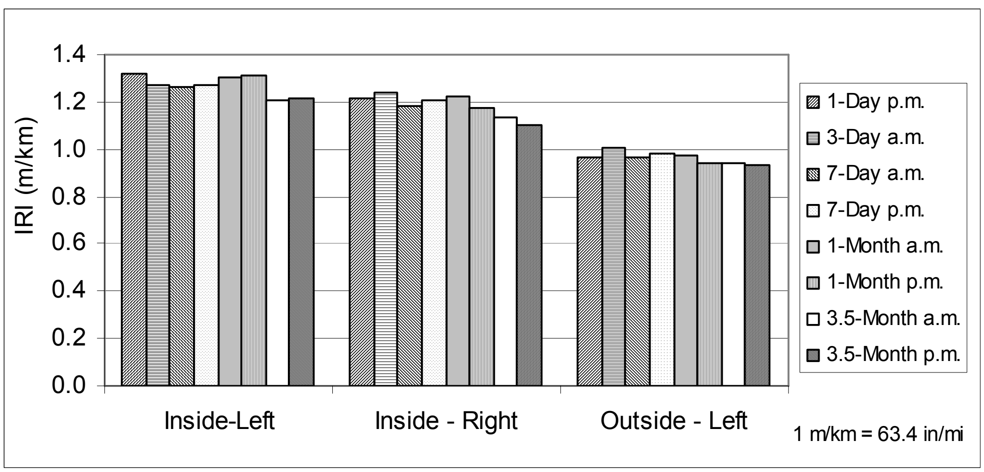


Figure 71. IRI values for different test sequences—S.R. 6220.

Table 28. Percentage change in IRI with respect to 1-day IRI—S.R. 6220.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Percentage Change in IRI with Respect to 1-Day IRI		
			Inside Lane		Outside Lane
			Left Wheel Path	Right Wheel Path	Left Wheel Path
9/20/03	3 days	1 to 1:40 p.m.	−3	2	5
9/24/03	7 days	9:45 to 10 a.m.	−4	−2	0
		5:05 to 5:35 p.m.	−3	−1	2
10/15/03	1 month	8:40 to 9 a.m.	−1	1	1
		2:30 to 3 p.m.	−1	−3	−2
12/29/03	3.5 months	10 to 11:30 p.m.	−8	−7	−2
		3:50 to 4:40 p.m.	−8	−9	−3

Note: 1-day profile data not collected along right wheel path of outside lane

The following observations were noted when evaluating the IRI values:

- The IRI is showing a decrease transversely across the pavement from the left wheel path of the inside lane to the right wheel path of the outside lane. The 1-month afternoon IRI (obtained after the joints were sealed) indicated the IRI of the left wheel path of inside lane, right wheel path of inside lane, left wheel path of outside lane, and right wheel path of outside lane to be 1.31, 1.18, 0.94, and 0.93 m/km (83, 75, 60, and 59 inches/mi), respectively. The difference in IRI between the two wheel paths of the outside lane was small.

- The IRI values obtained during the 3.5-month testing for the inside lane for both the left and the right wheel paths were lower than the IRI obtained during 1-day testing by 8 and 9 percent, respectively. This reduction in IRI is attributed to the diamond grinding that was performed on this lane.

- When the data for the case where measurements were obtained after diamond grinding were omitted, the data show the IRI along each test path has remained relatively constant for all data sets. The changes in IRI with respect to 1-day IRI were usually within ±3 percent. The age of pavement and the time of testing do not appear to have affected the IRI.

- The joints in the pavement were at three different conditions during testing: (1) initial sawcut performed on the pavement (testing at 1 day, 3 days, and 7 days), (2) joint reservoir sawed but not sealed (1-month morning testing), and (3) joints sealed (1-month afternoon and 3.5-month testing). The condition of the joint during testing had negligible impact on the IRI.

RN Values

The average RN values obtained from the three runs are presented in table 29 and shown in figure 72. Table 30 shows the percentage change in RN values for different test dates with respect to the RN obtained at 1 day. The RN for the right wheel path in the outside lane is not shown in figure 72 and table 30 because data were collected along this path only during the 1-month and 3.5-month data collection.

Table 29. RN values for different test sequences—S.R. 6220.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Ride Number			
			Inside Lane		Outside Lane	
			Left Wheel Path	Right Wheel Path	Left Wheel Path	Right Wheel Path
9/18/03	1 day	2:30 to 2:40 p.m.	3.59	3.71	3.84	N/A
9/20/03	3 days	1 to 1:40 p.m.	3.59	3.66	3.80	N/A
9/24/03	7 days	9:45 to 10 a.m.	3.45	3.51	3.62	N/A
		5:05 to 5:35 p.m.	3.44	3.55	3.62	N/A
10/15/03	1 month	8:40 to 9 a.m.	3.37	3.26	3.40	3.40
		2:30 to 3 p.m.	3.68	3.81	3.95	3.88
12/29/03	3.5 months	10 to 11:30 p.m.	3.69	3.78	3.95	3.87
		3:50 to 4:40 p.m.	3.67	3.80	3.97	3.93

N/A: Profile data not collected

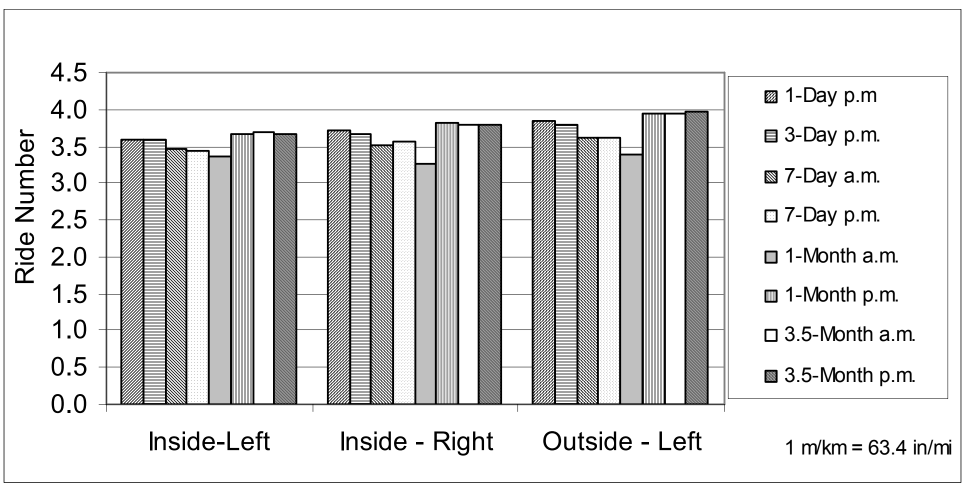


Figure 72. RN values for different test sequences—S.R. 6220.

Table 30. Percentage change in RN with respect to 1-day RN—S.R. 6220.

Date of Profiling	Approximate Age of Pavement	Time Of Profiling	Percentage Change in RN With Respect to 1-Day RN		
			Inside Lane		Outside Lane
			Left Wheel Path	Right Wheel Path	Left Wheel Path
9/20/03	3 days	1 to 1:40 p.m.	0	−1	−1
9/24/03	7 days	9:45 to 10 a.m.	−4	−5	−6
		5:05 to 5:35 p.m.	−4	−4	−6
10/15/03	1 month	8:40 to 9 a.m.	−6	−12	−11
		2:30 to 3 p.m.	2	3	3
12/29/03	3.5 months	10 to 11:30 p.m.	3	2	3
		3:50 to 4:40 p.m.	2	2	3

N/A: Profile data not collected

The following observations were noted when the RN values were evaluated:

- The RN values increased transversely across the pavement from the left wheel path of the inside lane to the left wheel path of the outside lane. The right wheel path of the outside lane had a slightly lower RN than the left wheel path of the outside lane.

- The 7-day RN values were less than the 1-day RN values; the values were lower by 4 to 6 percent for the different wheel paths. An evaluation of the profiles indicated that the backer rods put into the joints appear to have settled when 7-day profiling was performed, and this feature was appearing in the profile as a slight depression and causing lower RN values.

- The 1-month morning profiles had the lowest RN value; the RN was lower than the 1-day value by 6 to 12 percent for the different wheel paths. The joint reservoirs had been sawed when the data were collected, which caused depressions to be recorded in the profile data at the joint locations. This phenomenon caused a reduction in the RN values.

- The RN of profiles collected during the 1-month afternoon profiling when the joints were sealed had higher values than those obtained from profiles collected in the morning when the joint reservoir had been sawed and the joint was unsealed. The afternoon RN values were higher than the morning RN values by 9 to 16 percent for the different wheel paths.

- The RN values obtained after the joints were sealed (1-month afternoon runs) were slightly higher than the RN values obtained from the 1-day profiling by values ranging from 0.09 to 0.11 (2 to 3 percent) for the different wheel paths. An evaluation of the profiles did not indicate the cause for this slight increase in RN. There may have been slurry resulting from joint sawing operations adjacent to the joint during the 1-day profiling, which may have caused lower RN values.

Repeatability of IRI Values

An evaluation of the IRI values obtained from repeat runs for each data set indicated good repeatability. Usually when the IRI from three runs were compared, the difference between the maximum and the minimum IRI was between 0.02 to 0.03 m/km (1.27 to 1.90 inches/mi).

The three repeat runs obtained along the left wheel path of the outside lane for the 1-month afternoon runs were used to evaluate the short interval repeatability of IRI within the section. The IRI of each run was computed at 15-m (49-ft) intervals to perform this evaluation. The 165-m (541-ft)-long section has 11 segments that are 15 m (49 ft) long (the last 3 m (10 ft) of the section was ignored). Figure 73 shows the IRI values obtained at 15-m (49-ft) intervals for each run. Overall, reasonable repeatability in IRI values was obtained for the majority of segments. The difference between the maximum and minimum IRI for a segment obtained from the repeat runs ranged from a low of 0.03 m/km (1.9 inches/mi) at segment 2 to a high of 0.21 m/km (13.3 inches/mi) at segment 8. The average of the difference between maximum and minimum IRI for a segment was 0.09 m/km (5.7 inches/mi).

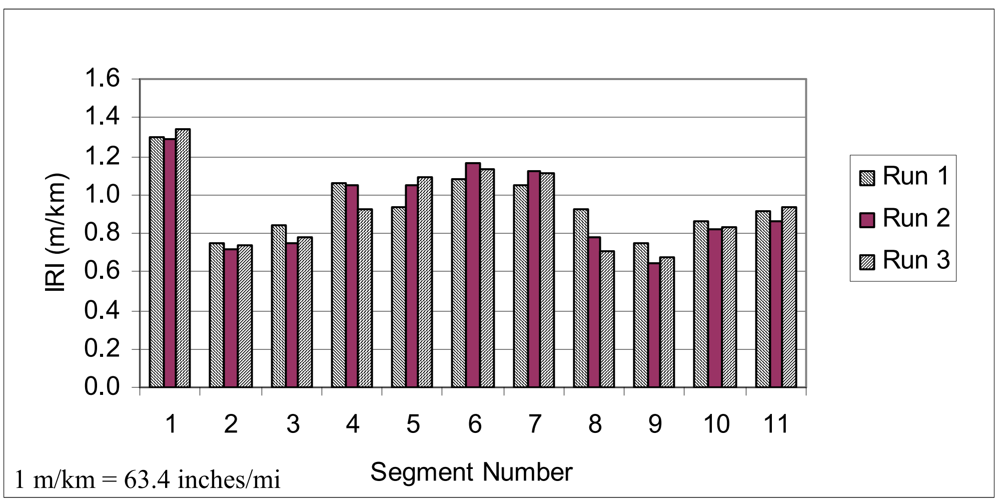


Figure 73. IRI values for repeat runs—S.R. 6220.

Effect of Condition of Joint on Profile Data

During this study, profile data were collected when the joints were in the following conditions:

- The initial sawcut that was 3 mm (0.12 inch) in width and 93 mm (3.7 inches) deep had been made on the pavement. A backer rod had been placed inside the joint. Profile data under these conditions were collected during 1-day, 3-day, and 1-week profiling.
- The joint reservoir that was 9.5 mm (0.4 inch) wide and 38 mm (1.5 inches) deep had been made on the pavement. Profile data under this condition were collected during the 1-month morning data collection.

- The joints had been sealed. Profile data under this condition were collected during the 1-month afternoon and 3.5 month data collection. According to the specification, when the joint is sealed, the depth from the pavement surface to the top of the sealant should be 5 mm (0.2 inch).

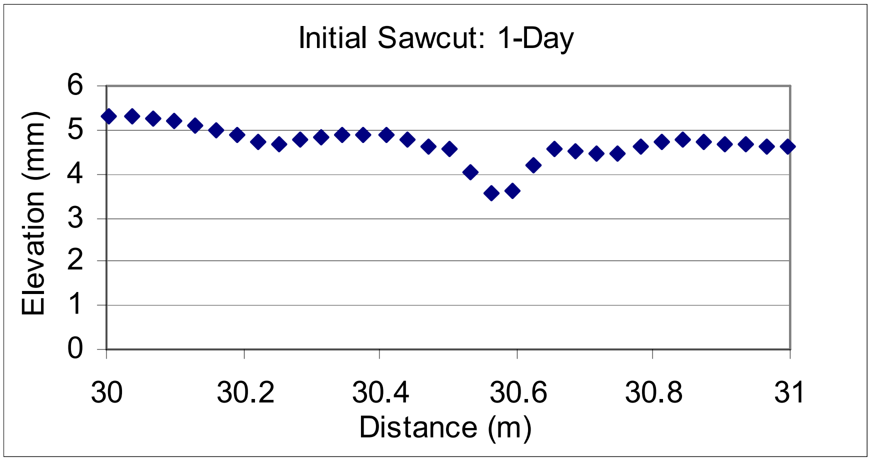
Profile data collected when the joint was at each of the three conditions described previously were evaluated to investigate how the joints showed up on the profile. Data collected along the left wheel path of the outside lane at the following profiling times were evaluated: 1 day (initial sawcut), 1-month morning (joint reservoir sawed), and 1-month afternoon (joint sealed). Figures 74–76 show how a typical joint appeared on the profile for each of these conditions. The joint appears between 30.4 and 30.6 m (99.7 and 100.3 ft) in the plots. The following observations are noted for each case shown in figures 74–76.

- Initial Sawcut: The joint appears in the profile as a small depression spread over a distance of about 220 mm (8.7 inches), with a maximum depth about of 1.5 mm (0.06 inch).

- Joint Reservoir Sawed: The joint appears in the profile as a small depression spread over a distance of about 220 mm (8.7 inches), with a maximum depth of about 3.5 mm (0.14 inch).

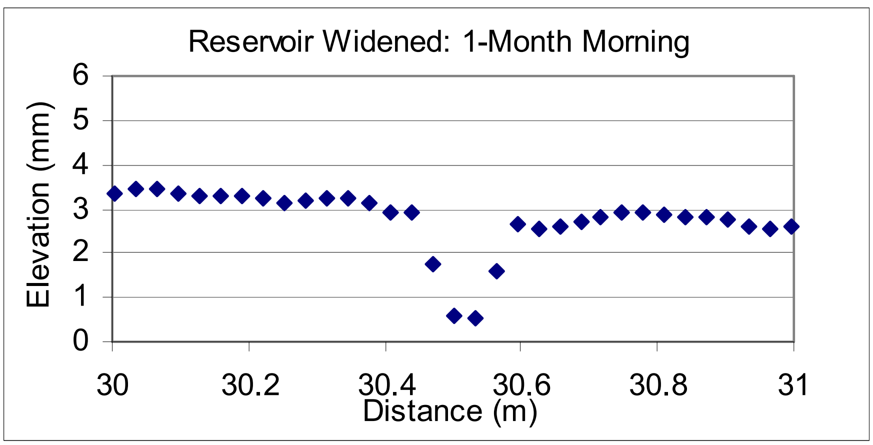
- Joint Sealed: The joint appears in the profile as a small depression spread over a distance of about 220 mm (8.7 inches), with a maximum depth of about 1.5 mm (0.06 inch).

For all three cases, the joint appears in the profile as a small depression spread over a distance of about 220 mm (8.7 inches), when the actual width of the joint is 9.5 mm (0.4 inch). Also, the depth of the joint that appears in the profile for each case is much less than the actual depth. This phenomenon is caused by the averaging performed on the height sensor data and the low-pass filtering that is applied on the profile data to prevent aliasing.



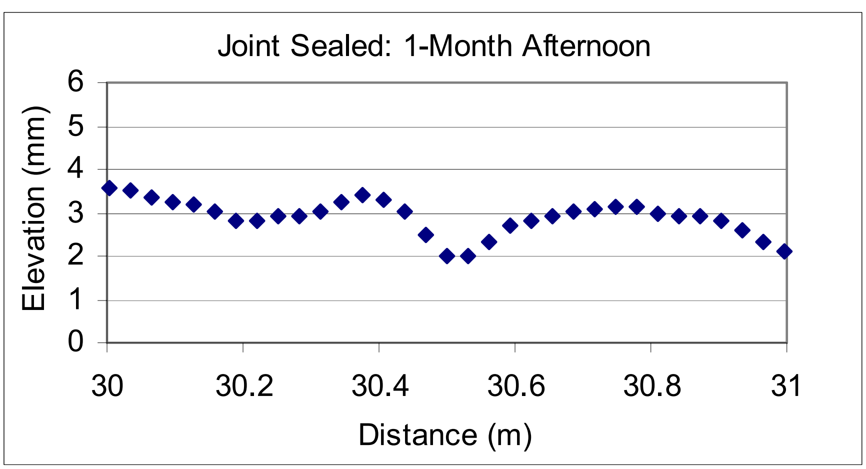
1 m = 3.28 ft
1 mm = 0.039 inch

Figure 74. Measurements at a joint, initial sawcut, 1-day—S.R. 6220.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 75. Measurements at a joint, reservoir widened, 1-month morning—S.R. 6220.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 76. Measurements at a joint, joint sealed, 1-month afternoon—S.R. 6220.

The lightweight profiler used for data collection recorded profile data at 31-mm (1.2-inch) intervals. However, the height sensor in the profiler obtains data at much closer intervals, and an averaged height sensor value is used to compute profile data points at 31-mm (1.2-inch) intervals. In addition, a low-pass filter appears to have been applied on the data further attenuating the depth of the joint, and causing the joint to appear as a much wider feature. Although the profiles obtained when the joint reservoirs had been sawed do show a downward feature at the joint that has a higher depth compared to the other two cases, the depth of the features is not large enough to have an effect on the IRI. Hence, the IRI values computed from profiles collected under the three different scenarios were very close to each other. However, the downward spikes of the magnitude seen for 1-month morning profiling where the joint reservoirs had been sawn do have a significant impact on RN. The 1-month afternoon RN values obtained after the joints were sealed were higher than the morning values by 9 to 16 percent for the different wheel paths.

Roughness Profiles

Figures 77–79 and figures 80–82 show the roughness profiles for a 6-m (20-ft) base length for the outside lane and inside lane, respectively. The roughness profile for the outside lane was computed from data obtained for the 3-month afternoon testing, whereas the roughness profile for the inside lane was computed using data obtained from 3-day testing. The joint spacing of the pavement is 4.6 m (15 ft) and each vertical line in the plots corresponds to a joint. The peaks in the plots indicate areas of high roughness. Many peaks are coinciding with the joint locations or are very close to joint locations. This situation indicates that the dowel baskets may be affecting roughness.

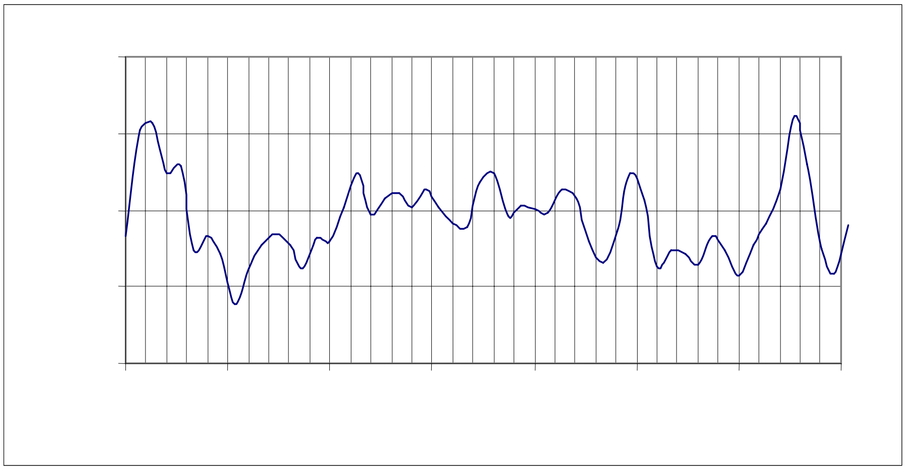
Figure 83 shows a high roughness area along the left wheel path of the inside lane at the start of the section. This area was subsequently diamond ground. Several localized areas of high roughness are noted along the left wheel path of the inside lane, and the roughness of these areas contributed to making this wheel path have the highest roughness of all tested wheel paths.

Evaluation of Profile Data

Apart from the differences in joint conditions, no significant differences between the profiles obtained during the different test sequences could be observed. The profiles obtained along the inside lane during 3.5 months of testing showed the effect of diamond grinding at the start of the section. Also, no distinct profile feature or a dominant waveband could be identified in the profile data.

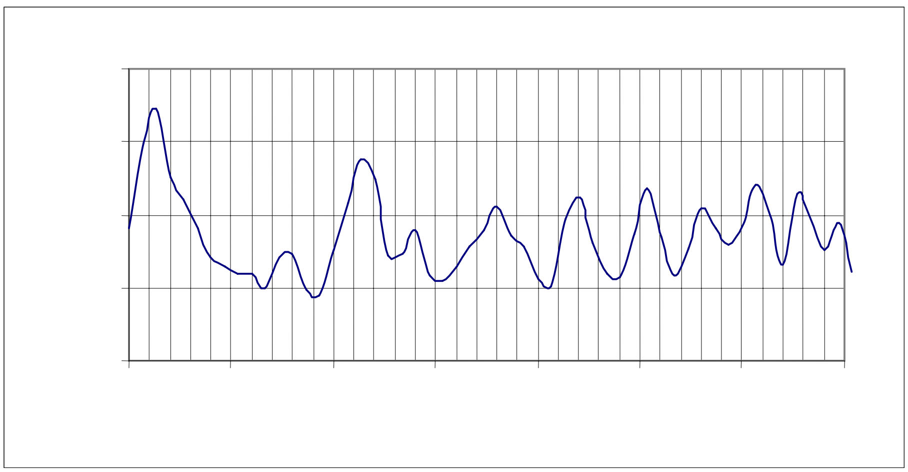
As described previously, the IRI varied transversely across the section. The difference in IRI between the left wheel path of the inside lane and the right wheel path of the outside lane based on 1-month afternoon testing was 0.38 m/km (24 inches/mi). Evaluation of the profile data did not show a distinct profile feature that was responsible for this difference in IRI.

The CI value was computed for one profile run from each data set, and the computed values are shown in table 31. This table does not show values for the right wheel path of the outside lane because profile data were only collected along this path during the 1-month and 3.5-month data collection. The CI values for all data sets were extremely small, which indicates the slabs are more or less flat with virtually no curvature. No noticeable changes in curvature were observed between the data sets.



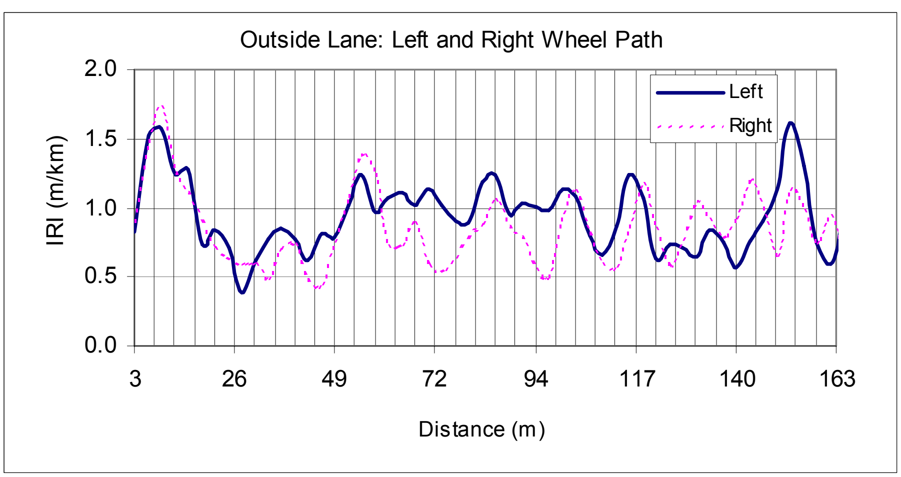
1 m = 3.28 ft
1 m/km =63.4 inches/mi

Figure 77. Roughness profiles for outside lane, left wheel path—S.R. 6220.



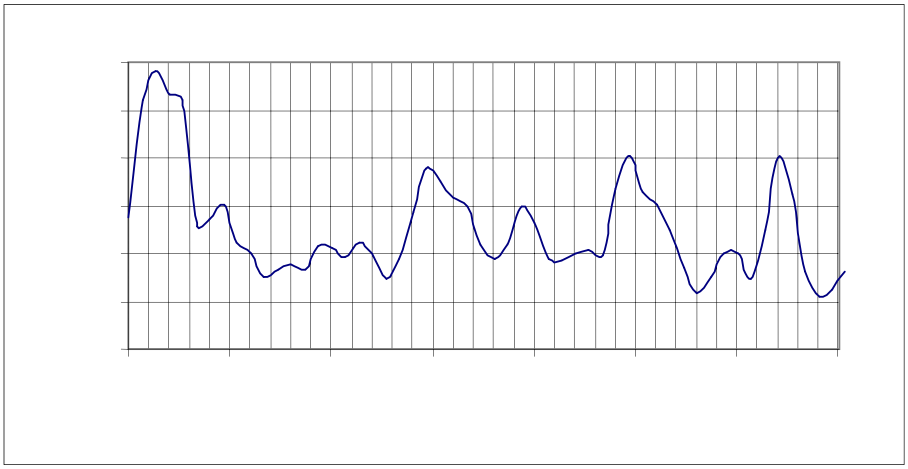
1 m = 3.28 ft
1 m/km =63.4 inches/mi

Figure 78. Roughness profiles for outside lane, right wheel path—S.R. 6220



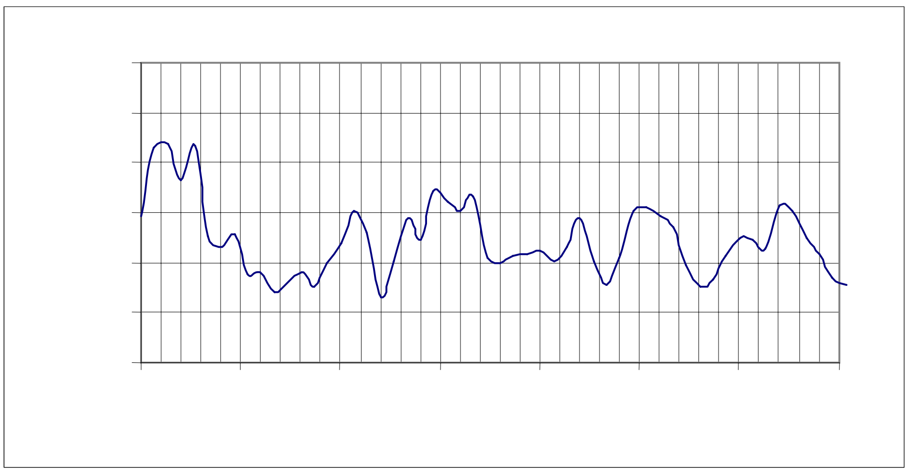
1 m = 3.28 ft
1 m/km =63.4 inches/mi

Figure 79. Roughness profiles for outside lane, left and right wheel path—S.R. 6220.



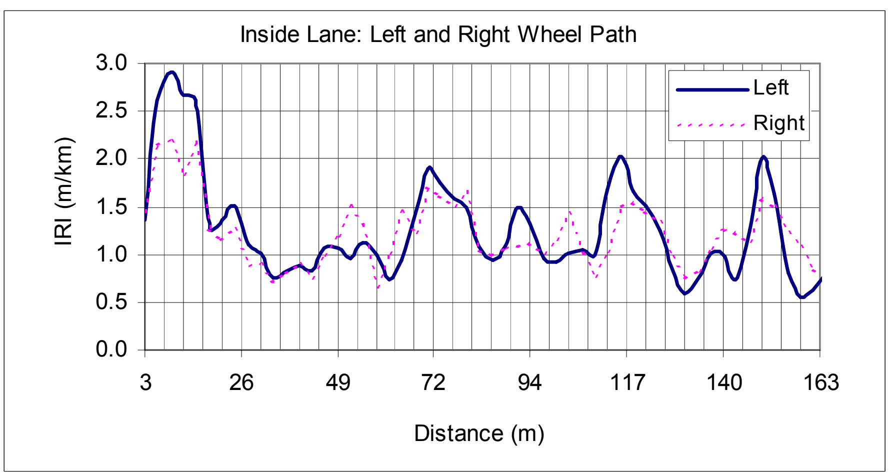
1 m = 3.28 ft
1 m/km =63.4 inches/mi

Figure 80. Roughness profiles for inside lane, left wheel path—S.R. 6220.



1 m = 3.28 ft
1 m/km =63.4 inches/mi

Figure 81. Roughness profiles for inside lane, right wheel path—S.R. 6220.



1 m = 3.28 ft
1 m/km =63.4 inches/mi

Figure 82. Roughness profiles for inside lane, left and right wheel path—S.R. 6220.

Table 31. CI values—S.R. 6220.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Curvature Index x 1,000 (1/m)		
			Inside Lane		Outside Lane
			Left Wheel Path	Right Wheel Path	Left Wheel Path
9/18/03	1 day	2:30 to 2:40 p.m.	0.10	−0.10	0.08
9/20/03	3 days	1 to 1:40 p.m.	−0.13	−0.09	−0.05
9/24/03	7 days	9:45 to 10 a.m.	0.07	0.05	0.08
		5:05 to 5:35 p.m.	0.03	0.08	−0.02
10/15/03	1 month	8:40 to 9 a.m.	0.18	0.05	0.13
		2:30 to 3 p.m.	0.16	−0.02	0.02
12/29/03	3.5 months	10 to 11:30 a.m.	0.05	−0.09	−0.03
		3:50 to 4:40 p.m.	0.07	−0.10	−0.02

1/m = 1/3.28 ft

Coefficient of Thermal Expansion and Microscopical Examination

A CTE test and a microscopical examination were performed on a 150-by-300 mm (5.9-by-11.8 inch) cylinder that was cast from the concrete used in the project. The CTE value for the specimen was 9.60 x 10^{-6} per °C (5.33 x 10^{-6} per °F). The microscopical examination identified the coarse aggregate to be primarily dolomite/dolomitic limestone (carbonate) with traces of granite (silicious). The fine aggregate was classified as silicious.

Summary

- The IRI of the pavement was decreasing transversely from the left wheel path of the inside lane to the left wheel path of the outside lane. Along the outside lane, the left and right wheel path IRI values were close to each other. The 1-month afternoon testing indicated that the IRI of the left wheel path of inside lane, right wheel path of inside lane, left wheel path of outside lane, and right wheel path of outside lane were 1.31, 1.18, 0.94, and 0.93 m/km (83, 75, 60, and 59 inches/mi), respectively. No distinct profile feature that had this effect on the IRI could be identified.

- Researchers noted that the data collected by the lightweight profiler did not accurately record the shape of the joints in the pavement. A joint appeared in the profile as a small depression spread over a distance of 220 mm (8.7 inches), when the actual width of the joint was 9.5 mm (0.37 inch). The depth of this depression varied depending on the condition of the joint. When the initial sawcut that was 3 mm (0.12 inch) wide was present, a depth of 1.5 mm (0.06 inch) was recorded. When the joint reservoir that was 9.5 mm (0.37 inch) wide and 38 mm (1.50 inches) deep was present with the joint being unsealed, a depth of 3.5 mm (0.14 inch) was recorded. When the joint was sealed, a depth of 1.5 mm (0.06 inch) was recorded.

- The profiler showed good repeatability in obtaining IRI values. When the IRI obtained from the three repeat runs for the entire section were evaluated for all data sets, the difference between the maximum and minimum IRI was between 0.02 m/km (2.5 inches/mi) and 0.03 m/km (1.9 inches/mi).

- An evaluation of short interval IRI repeatability using 15-m (49-ft) segment lengths indicated that the profiler was providing reasonable repeatability. The average difference between the maximum and minimum IRI obtained for a 15-m (49-ft)-long segment was 0.09 m/km (5.7 inches/mi).

- Little change in IRI was noted for the different data sets. The IRI can be considered to have remained at the same value over the 3.5-month period. The IRI was not affected by the condition of the joint or the time of day when profiling was performed. The IRI was not affected by the condition of the joint because the data collected by the profiler was attenuating the depth of the joint, and the recorded depths were of such a magnitude that they did not affect the IRI.

- As for the IRI, little change in RN was noted for the different data sets, except for the values obtained when the joint reservoirs had been sawed and the joint was in an unsealed condition. The RN obtained after the joint was sealed was higher than the RN obtained when the joint reservoir was formed but unsealed by 9 to 16 percent for the different wheel paths.

- There was negligible curvature in the PCC slabs for the different data sets. The slabs in the pavement can be considered to be flat. No appreciable changes in curvature were noted between the different data sets.

U.S. 20 PROJECT IN IOWA

Project Description

This was a reconstruction project located in Hardin County, IA. This roadway is a four-lane divided highway with two lanes in each direction. A 183-m (600-ft)-long test section was established for testing on the westbound inside lane. The test section was established between stations 1269+00 and 1275+00. (Stations are in U.S. customary units.)

Pavement Details

Table 32 presents pavement details. Table 33 presents information about the joints in the pavement.

Table 32. Pavement details—U.S. 20.

Item	Description	Value
Pavement thickness	Concrete thickness	260 mm (10.1 inches)
	Base thickness (mm)	278-mm (10.8-inch) semidrainable Unbound granular base
Pavement width	Total pavement width	7.8 m (25.6 ft)
	Width of inside lane	3.6 m (11.8 ft)
	Width of outside lane	4.2 m (13.8 ft)
Shoulder	Shoulder type	150-mm (5.9-inch)-thick granular shoulder
	Width of shoulder	Inside 1.8 m (5.9 ft), outside 2.4 m (7.9 ft)
Joint spacing	Joint spacing	6 m (19.7 ft)
	Joints skewed?	Yes, 6:1
Dowels	Dowel type	Epoxy coated
	Dowel diameter	35 mm (1.4 inch)
	Dowel length	457 mm (18 inches)
Tining information	Tining type	Longitudinal tining
	Tining spacing	20 mm (0.8 inch)
	Tining width	3 mm (0.12 inch)
	Tining depth	3 mm (0.12 inch)

Table 33. Joint details—U.S. 20.

Description	Value
Joint formation	Single sawcut using soft cut saw
Width of cut	6 mm (0.24 inch)
Depth of cut	25 ±6 mm (1 ±0.24 inch)
Sealant type	Hot-pour bituminous
Depth to top of sealant	6 ±3 mm (0.24 ±0.12 inch) from top of pavement

Concrete Mix Design

Table 34 presents the mix proportions used in the concrete mix. An entrained air admixture and a water-reducing admixture were added to the concrete mix. Table 35 presents the gradation of the aggregates used in the concrete mix.

Paving Details

Table 36 presents the date and time when the test section was paved as well as other details about the paving process. The concrete was placed using a slipform paver. Figure 83 is a photograph of the paving operation. The paver had a super-smoother finishing attachment at the back of the paver for finishing the concrete. More than 25 mm (1 inch) of rain occurred after 8 p.m. on the day paving took place.

Table 34. Mix proportions—U.S. 20.

Component	Weight kg/m^3 (lb/yd^3)
Cement type IS	273 (460)
Fly ash—North Omaha Class C	48 (81)
Coarse aggregate	844 (1,423)
Intermediate aggregate	281 (474)
Sand	716 (1,207)

Table 35. Gradation of aggregates—U.S. 20.

Sieve	Percentage Passing		
	Coarse Alden Limestone	Intermediate Alden Limestone	Fine Becker-Brandt Sand
37.5 mm (1.5 inches)	100	100	100
25.0 mm (1 inch)	83	100	100
19.0 mm (0.75 inch)	60	100	100
12.5 mm (0.5 inch)	28	99	100
9.5 mm (0.4 inch)	16	84	100
No. 4	2	15	99
No. 8	1	2	86
No. 16	0.9	1.8	58
No. 30	–	1.5	28
No. 50	–	1.3	7.5
No. 100	–	1	0.9

Profiling of Section

Six sets of profile data were collected over a 3-month period. The pavement was first profiled 1 day after paving, and then the second set of profiles was collected 3 days after paving. The third and fourth sets of data were collected approximately 1 week after paving in the morning and afternoon, respectively. The fifth and sixth sets of data were collected 3 months after paving in the morning and afternoon, respectively, of the same day. The profile data collection was performed using an Ames lightweight profiler that recorded data at 30-mm (1.2-inch) intervals. Table 37 shows the dates and times when profile data collection was performed, the approximate age of the pavement at each of these instances, and the low, high, and mean air temperatures for each profiling day.

A 6-mm (0.25-inch)-wide and 25-mm (1-inch)-deep sawcut had been made on the pavement when the profile data were collected for the first time. The joints were sealed after the 3-day data collection was performed.

Table 36. Paving information—U.S. 20.

Item	Description	Value
Date and time	Date of paving	5/13/03
	Time of paving	9 a.m. to noon
Paving process	Haul route	Adjacent to inside lane
	Stringline	10-m (33-ft) spacing, both sides
	Dowels	Fixed to base
	Tie bars	Inserted by paver
	Concrete deposit method	Belt placers
	Spreader used?	Yes, one
	Paver	CMI 450B
Concrete	Temperature	14 °C (57.2 °F)
Curing method	Curing compound	White pigmented Conspec white wax—IA

Figure 83. Paver in operation.

Table 37. Profile data collection—U.S. 20.

<table>
<tr><th rowspan="2">Date of Profiling</th><th rowspan="2">Approximate Age of Pavement</th><th rowspan="2">Time of Profiling</th><th colspan="3">Air Temperature °C (°F)</th></tr>
<tr><th>Low</th><th>High</th><th>Mean</th></tr>
<tr><td>5/14/03</td><td>1 day</td><td>3 to 4 p.m.</td><td>11 (52)</td><td>21 (70)</td><td>19 (66)</td></tr>
<tr><td>5/16/03</td><td>3 days</td><td>10 to 10:30 a.m.</td><td>6 (43)</td><td>20 (68)</td><td>13 (55)</td></tr>
<tr><td>5/21/03</td><td>8 days</td><td>2 to 3 p.m.</td><td>6 (43)</td><td>18 (64)</td><td>12 (54)</td></tr>
<tr><td>5/22/03</td><td>9 days</td><td>7 to 8 a.m.</td><td>5 (41)</td><td>19 (66)</td><td>12 (54)</td></tr>
<tr><td>8/13/03</td><td>3 months</td><td>8:30 a.m.</td><td rowspan="2">14 (57)</td><td rowspan="2">28 (82)</td><td rowspan="2">22 (72)</td></tr>
<tr><td>8/13/03</td><td>3 months</td><td>2 p.m.</td></tr>
</table>

Roughness Indices

IRI Values

The average IRI values computed from the three repeat runs are presented in table 38 and shown in figure 84. Table 39 shows the percentage change in IRI values for different test sequences with respect to the IRI obtained at 1 day.

Table 38. IRI values for different test sequences—U.S. 20.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	IRI (m/km)	
			Left Wheel Path	Right Wheel Path
5/14/03	1 day	3 to 4 p.m.	1.18	1.65
5/16/03	3 days	10 to 10:30 a.m.	1.24	1.64
5/21/03	8 days	2 to 3 p.m.	1.20	1.60
5/22/03	9 days	7 to 8 a.m.	1.17	1.62
8/13/03	3 months	8:30 a.m.	1.20	1.48
8/13/03	3 months	2 p.m.	1.20	1.50

Note: Joints were sealed after the 3-day data collection

1 m/km = 63.4 inches/mi

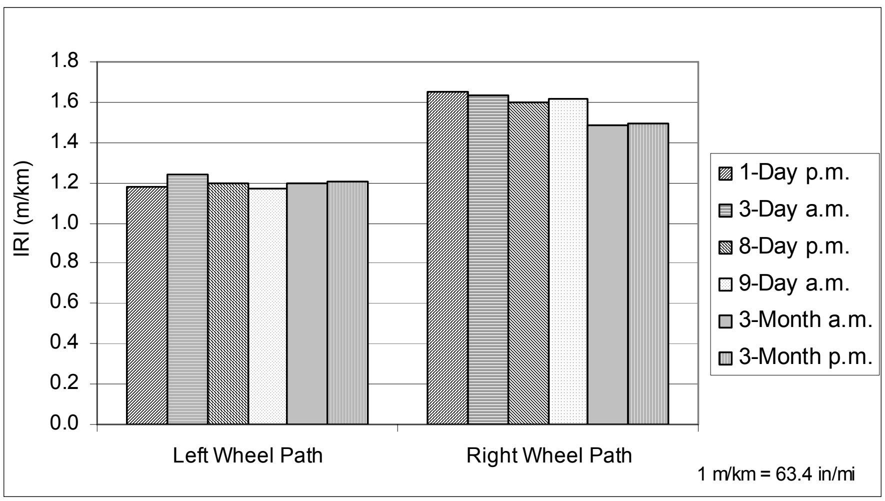


Figure 84. IRI values for different test sequences—U.S. 20.

Table 39. Percentage change in IRI with respect to 1-day IRI—U.S. 20.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Percentage Change in IRI	
			Left Wheel Path	Right Wheel Path
5/16/03	3 days	10 to 10:30 a.m.	5	−1
5/21/03	8 days	2 to 3 p.m.	2	−3
5/22/03	9 days	7 to 8 a.m.	−1	−2
8/13/03	3 months	8:30 a.m.	1	−10
8/13/03	3 months	2 p.m.	2	−9

The following observations were noted when evaluating the IRI values:

- The IRI of the right wheel path was higher than the left wheel path for all test sequences. For the different test sequences, IRI of the right wheel path was higher than that for the left wheel path by amounts ranging from 24 to 40 percent. The highest difference was observed for the testing that was performed 1 day after paving.

- Overall, the IRI remained relatively constant along the left wheel path for all test dates and times. The percentage difference in IRI with respect to the 1-day IRI for the different test sequences ranged from −1 to 5 percent. Along the right wheel path, the percentage difference in IRI with respect to the 1-day IRI for testing performed within the first 9 days ranged from −1 to −3 percent. However, the IRI obtained at 3-month testing was approximately 10 percent lower than that obtained for 1-day testing. The cause for this reduction could not be identified by evaluating the profile data.

- The joints were sealed after the 3-day profiles were obtained. The 1-week IRI values did show a reduction in IRI of 0.04 m/km (2.5 inches/mi) compared to the 3-day IRI value. However, this reduction is extremely small and indicates that similar IRI values were obtained when the joints were unsealed and sealed.

RN Values

The average RN obtained from the three repeat runs are presented in table 40 and shown in figure 85. Table 41 shows the percentage change in RN values for different test sequences with respect to RN obtained at 1 day. The following observations were noted when evaluating the RN values:

- The RN of the left wheel path was higher than that for the right wheel path for all test sequences; the difference in RN ranged from 7 to 10 percent.

- Overall, RN remained relatively constant along both wheel paths for all test sequences. Along the left wheel path, the difference in RN for different test sequences compared to

1-day RN ranged from −1 to 3 percent. This difference for the right wheel ranged from 0 to 5 percent; the highest difference observed was for testing performed at 3 months.

- The joints were sealed after the 3-day profiles were obtained. The 1-week RN values showed an increase in RN of about 0.10 from the 3-day values. This level of increase shows that sealing the joints had a very small effect on RN.

Table 40. RN values for different test sequences—U.S. 20.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	RN	
			Left Wheel Path	Right Wheel Path
5/14/03	1 day	3 to 4 p.m.	3.63	3.28
5/16/03	3 days	10 to 10:30 a.m.	3.59	3.27
5/21/03	8 days	2 to 3 p.m.	3.69	3.36
5/22/03	9 days	7 to 8 a.m.	3.72	3.34
8/13/03	3 months	8:30 a.m.	3.71	3.44
8/13/03	3 months	2 p.m.	3.74	3.45

Note: Joints were sealed after the 3-day data collection

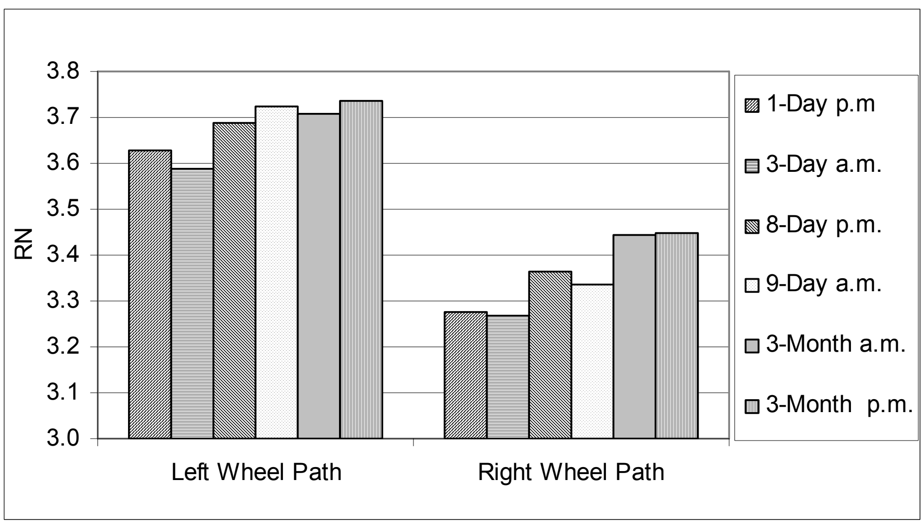


Figure 85. RN values for different test sequences to U.S. 20.

Table 41. Percentage change in RN with respect to 1-day RN—U.S. 20.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Percentage Change in RN	
			Left Wheel Path	Right Wheel Path
5/16/03	3 days	10 to 10:30 a.m.	−1	0
5/21/03	8 days	2 to 3 p.m.	2	3
5/22/03	9 days	7 to 8 a.m.	3	2
8/13/03	3 months	8:30 a.m.	2	5
8/13/03	3 months	2 p.m.	3	5

Repeatability of IRI Values

On average, when all data sets were considered, the difference between the maximum and minimum IRI values obtained from the three repeat runs was 0.06 m/km (3.8 inches/mi).

The three repeat runs collected along the left wheel path for day-1 testing were used to evaluate the short interval IRI repeatability by comparing IRI values obtained at 15-m (49-ft) intervals. For the 183-m (600-ft)-long section, there are twelve 15-m (49-ft) long segments (the last 3 m (10 ft) of the section was ignored). Figure 86 shows the IRI values obtained at 15-m (49-ft) intervals for the three runs.

An evaluation of the IRI values shown in figure 86 indicates that the IRI values from the different runs were variable for many segments. The difference between the maximum and minimum IRI for each segment obtained from the three repeat runs ranged from a low of 0.02 m/km (2.5 inches/mi) that occurred at segment 6, to a high of 0.60 m/km (38 inches/mi) that occurred at segment 8. The average of the difference between maximum and minimum IRI from the three runs when all segments were considered was 0.25 m/km (16 inches/mi).

The IRI for the entire section obtained from the three repeat runs were very close to each other; the IRI values were 1.66, 1.64, and 1.66 m/km (105, 104, and 105 inches/mi). Although the IRI values for the whole section from the three repeat runs were very close to each other, the distribution of the IRI within the section was very different for the three runs. When the overall IRI for the section is computed, the variations of the IRI within the section for a run are averaged out.

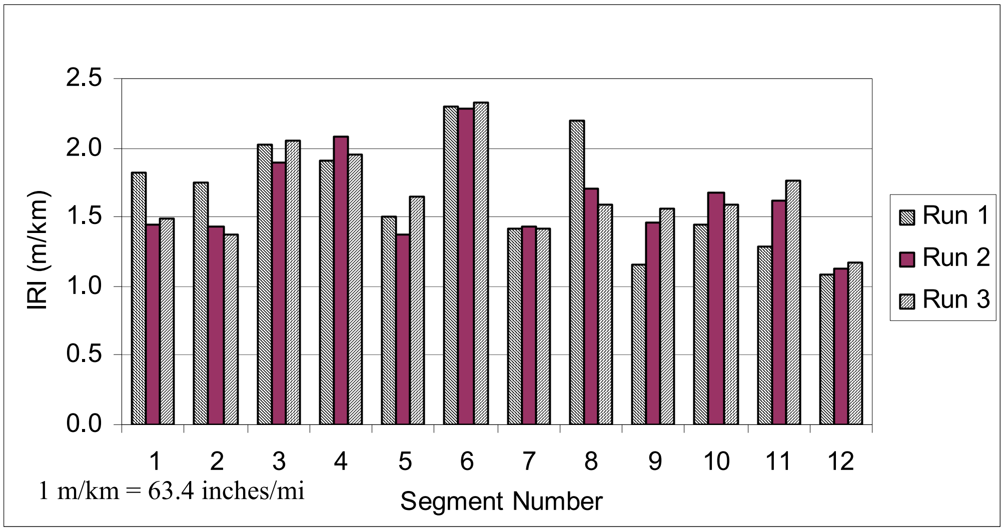


Figure 86. IRI values for repeat runs—U.S. 20.

This pavement section has longitudinal tining. When profile data are collected at this type of section, because of lateral variations in the profile path, the laser dot of the height sensor in the profiler can sometimes traverse along the top of the tining and sometimes dip into the tining and collect data at the bottom of the tining. This phenomenon can result in very different data being collected for repeat runs. The high variability in IRI values between the runs when IRI of 15-m (49-ft) segments were considered is attributed to this phenomenon.

Effect of Joint Condition on Profile Data

The joints were not sealed when the 1-day and 3-day profile data collection was performed. The joints were sealed immediately after the 3-day data collection. Profiles obtained before and after joint sealing were evaluated to investigate how the joints appear on the profile. The left wheel path data collected at 1-day and 1-month afternoon were used in this evaluation.

Figures 87–88 show how a typical unsealed and a sealed joint appeared in the profile data. The joint is approximately at a distance of 48 m (157 ft) in the plots. The joint reservoir is 6 mm (0.25 inch) wide. In the unsealed condition, the joint depth was 25 mm (1 inch). When the joint was sealed, the depth to the top of the sealant from the pavement surface was approximately 6 mm (0.25 inch). Although the joint is 6 mm (0.25 inch) wide, the joint appears in the profile as a feature spread over a distance of approximately 300 mm (11.8 inches). The profile plots indicated a depth at a joint of about 2 mm (0.08 inches) and 1 mm (0.04 inches) for the unsealed and sealed condition, respectively.

1 m = 3.28 ft
1 mm = 0.039 inch

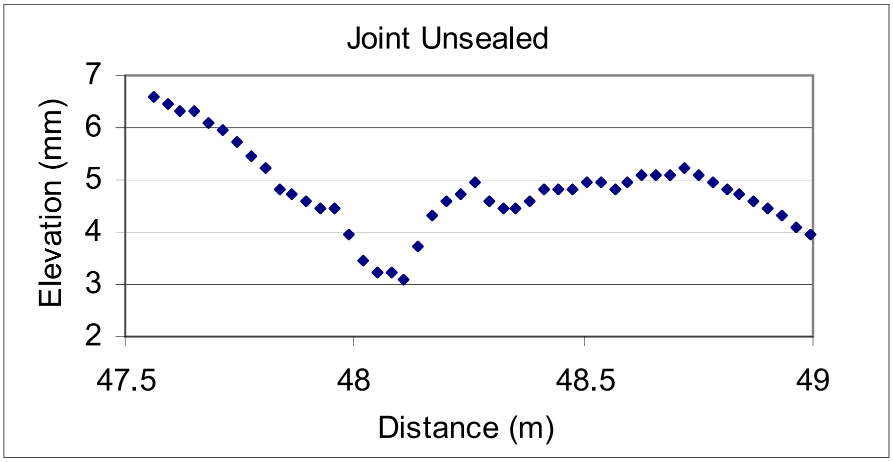


Figure 87. Measurements at a joint, unsealed—U.S. 20.

1 m = 3.28 ft
1 mm = 0.039 inch

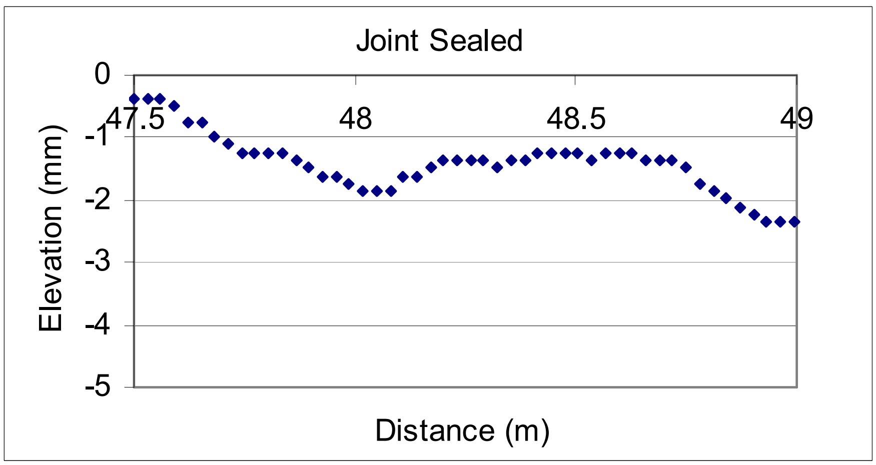


Figure 88. Measurements at a joint, sealed—U.S. 20.

As described previously when discussing joint conditions for the S.R. 6220 project, averaging effects of the height sensor and the anti-alias filter applied on the profile data causes the joint depth to be attenuated as well as the joint to appear as a feature spread over a distance much wider than the actual width of the joint. There is a small difference (approximately 1 mm (0.04 inch)) in the magnitude of the depth of the dip between the unsealed and sealed conditions. However, this difference in magnitude has no effect on IRI; hence, no difference in IRI between the sealed and the unsealed conditions was observed. However, this feature does appear to have a small impact on RN; the RN value increased by 0.1 after the joints were sealed.

Roughness Profiles

The IRI roughness profiles for a 6-m (20-ft) base length for the left wheel path, right wheel path, and an overlaid plot of the two wheel paths obtained from data collected during day-1 testing is shown in figures 89–91. The vertical lines in the plots correspond to joint locations.

The roughness profiles show that the roughness of the right wheel path is much higher than that for the left wheel path up to a distance of 128 m (420 ft), but after that the roughness levels of the two wheel paths are close to each other. Some peaks in the roughness profiles, particularly after a distance of 128 m (420 ft) correspond to or are very close to a joint. This phenomenon may indicate a possible effect of dowel baskets on roughness level.

1 m = 3.28 ft
1 m/km = 63.4 inches/mi

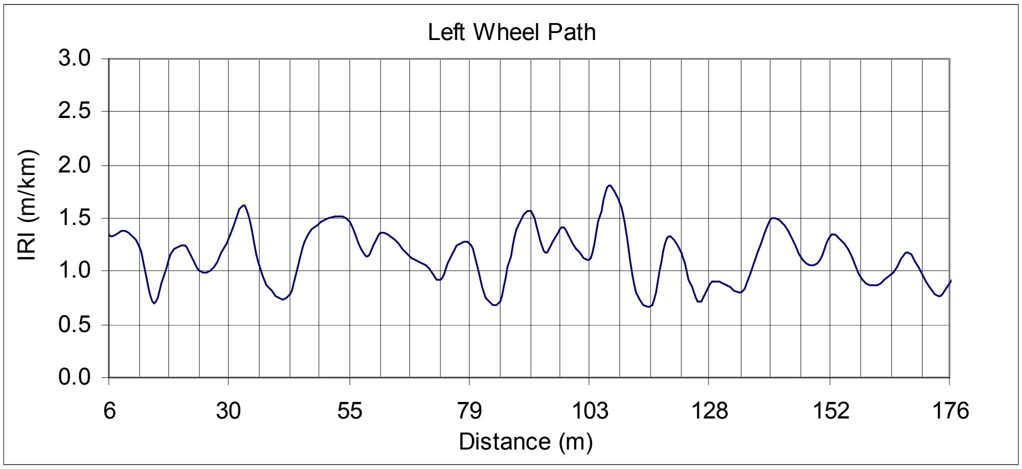


Figure 89. Roughness profiles for U.S. 20, left wheel path.

1 m = 3.28 ft
1 m/km = 63.4 inches/mi

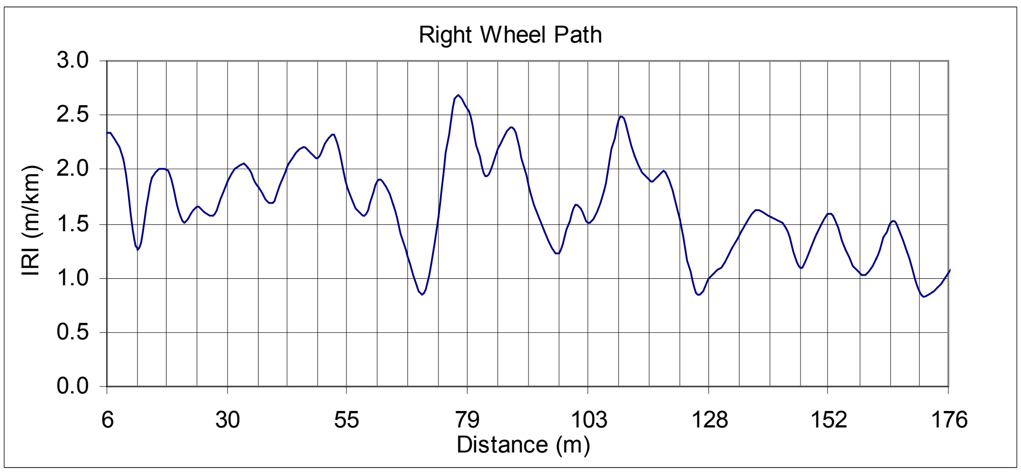


Figure 90. Roughness profiles for U.S. 20, right wheel path.

1 m = 3.28 ft
1 m/km = 63.4 inches/mi

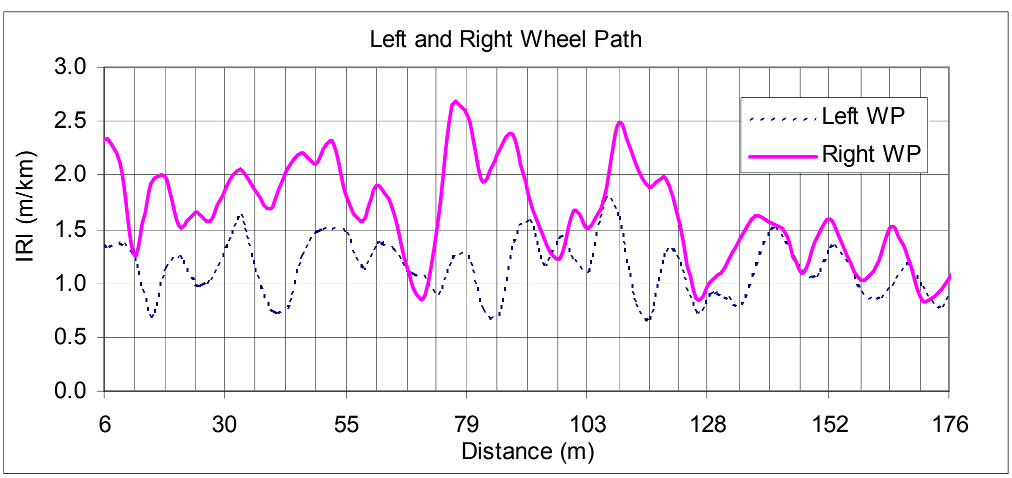


Figure 91. Roughness profiles for U.S. 20, left and right wheel path.

Evaluation of Profile Data

An evaluation of the profile data indicated the presence of a repetitive wave in the right wheel path that had an approximate wavelength of 1.6 m (5.2 ft). This wave was not observed in the left wheel path profile. Figure 92 shows a band-pass filtered profile plot of the data collected along the left and the right wheel paths, with the profiles offset for clarity. The right wheel path shows more waviness, which is caused by the repetitive wave.

1 m = 3.28 ft
1 mm = 0.039 inch

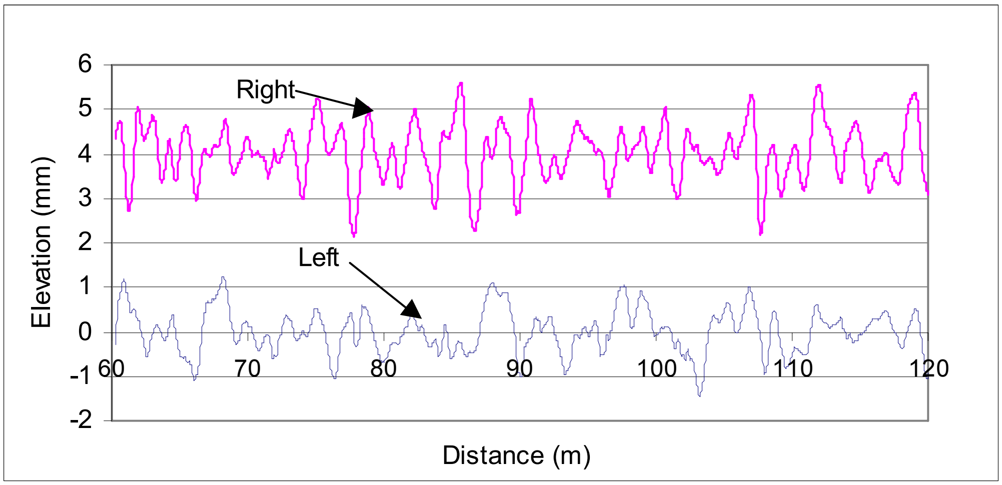
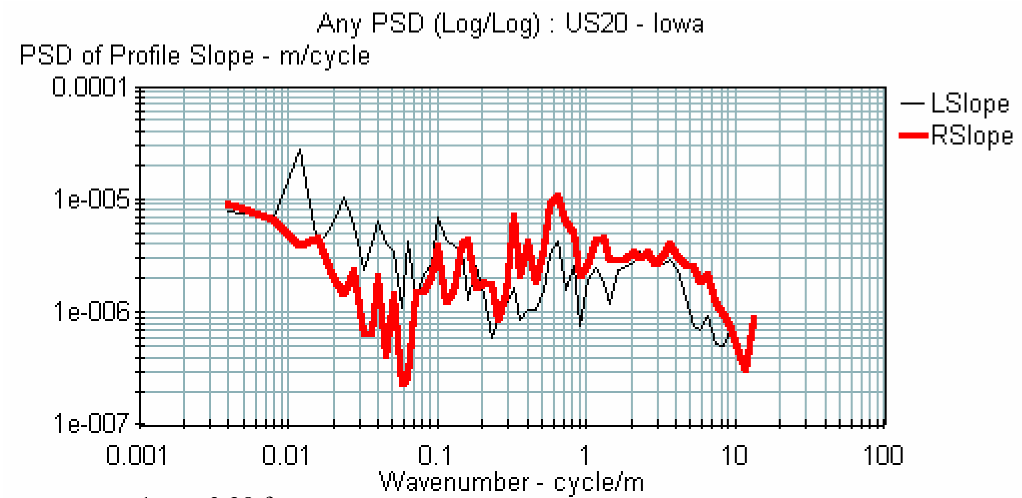


Figure 92. Band-pass filtered elevation profile.

Figure 93 shows the PSD plot of the left and right wheel path data. The right wheel path profile shows higher roughness between wavenumbers of 0.29 and 2.05 m/cycle (0.95 and 6.7 ft/cycle) compared to the left wheel path. These wavenumbers correspond to wavelengths between 0.5 and 3.5 m (11.5 ft). The cause for the high IRI of the right wheel path compared to the left wheel path is attributed to the higher roughness contribution between these wavelengths. The right wheel path PSD plots shows a peak at wavenumber of 0.65 cycles/m (2.1 cycle/ft), which corresponds to a wavelength of 1.6 m (5.2 ft). This is the dominant repetitive wavelength seen in the right wheel path profile.

The CI value was computed for one profile run from each data set. The computed values are shown in table 42. The CI values obtained for this project were somewhat higher than CI values obtained for the other projects. Some changes in CI were also noted between the data sets. The CI is influenced not only by slab curling that affects the slab over the 6-m (20-ft) slab length, but also other curvature within the slab. For example, for this pavement, the repetitive wave that has a wavelength of 1.6 m (5.2 ft) has a significant effect on CI. Evaluation of the profile data did not indicate much movement in the slabs at the joints for the different data sets. However, changes in slab shapes occurring in other areas between the different data sets contributed to changes in CI between the data sets. Generally, CI for data collected in the afternoon had higher CI values; these values were negative, which indicate downward curvature.



1 m = 3.28 ft

Figure 93. PSD plots of profiles.

Table 42. CI values—U.S. 20.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Curvature Index x 1,000 (1/m)	
			Left Wheel Path	Right Wheel Path
5/14/03	1 day	3 to 4 p.m.	−0.33	−0.20
5/16/03	3 days	10 to 10:30 a.m.	0.02	−0.39
5/21/03	8 days	2 to 3 p.m.	−0.04	−0.02
5/22/03	9 days	7 to 8 a.m.	0.06	−0.40
8/13/03	3 months	8:30 a.m.	−0.25	0.11
8/13/03	3 months	2 p.m.	−0.34	−0.41

1/m = 1/3.28 ft

Coefficient of Thermal Expansion and Microscopical Examination

A CTE test conducted on a 100-by-275-mm (4-by-10.8-inch) core indicated a value of 9.83 x 10^{-6} per °C (5.46 x 10^{-6} per °F.) A microscopical examination indicated that the coarse aggregate was limestone (carbonate) and the fine aggregate was silicious.

Summary

- The IRI of the right wheel path was significantly higher than the left wheel path. The right wheel path IRI at 1 day was 1.65 m/km (105 inches/mi), which was 40 percent higher than that of the left wheel path.

- The profile data indicated that the right wheel path had higher roughness between wavelengths of 0.5 and 3.5 m (11.5 ft) compared to the left wheel path. An evaluation of the profile data indicated that a repetitive wave having a wavelength of approximately 1.6 m (5.2 ft) was present on the right wheel path; this wave was a primary contributor to the high IRI observed along the right wheel path. This phenomenon was not observed in the left wheel path data. The factor in the paving process that caused this feature to appear in the right wheel path profile could not be identified. The paver had a super smoother float attached to the back. It is possible that the finishing procedure of this attachment may have caused this feature to appear in the profile.

- Researchers noted that the data collected by the lightweight profiler did not accurately record the shape of the joints in the pavement. In the profile, a joint appeared as a small dip that was spread over a distance of 300 mm (11.8 inches) when the actual width of the joint was 6 mm (0.25 inch). The depth of this dip was about 2 mm (0.08 inch) and 1 mm (0.04 inch) when the joint was unsealed and sealed, respectively. The actual depth of the joint was 25 mm (1 inch) in the unsealed condition. When the joint was sealed, the specification showed that the distance to the top of the sealant from the pavement surface was 6 mm (0.25 inch).

- When the IRI obtained from the three repeat runs for the entire section were evaluated for all data sets, the average of the difference between the maximum and minimum IRI was 0.06 m/km (3.8 inches/mi).

- An evaluation of short-interval IRI repeatability using 15-m (49-ft) segment lengths indicated that the IRI repeatability of the profiler was poor. The average difference between the maximum and minimum IRI obtained from the repeat runs for a 15-m (49-ft)-long segment was 0.25 m/km (16 inches/mi). The PCC pavement had longitudinal tining, and the high difference in IRI obtained between runs is attributed to the longitudinal tining. The profiler is equipped with a laser height sensor that has a diameter of about 1.5 mm (0.06 inches). Due to lateral variability, the laser can take measurements on top of the tining as well as at the bottom when traversing the section. This phenomenon can have a significant effect on the IRI. However, in comparing the IRI values obtained from repeat runs for the entire section that was 183 m (600 ft) long, they showed reasonable repeatability, because variations in IRI within the section tend to compensate when IRI is computed over longer lengths.

- Little change in IRI was noted for the different data sets along the left wheel path, and the IRI can be considered to have remained at the same value over the 3-month monitoring period. Along the right wheel path, little change in IRI was noted for data obtained up to 9 days. However, the data collected at 3 months showed a 10-percent reduction in IRI compared to that obtained 1 day after paving. The cause for this reduction could not be determined from the profile data.

- IRI obtained when the joints were unsealed and sealed were similar. The averaging and low-pass filtering performed on the profile data attenuates the depth of the joint recorded in the profile data. The depth of the joints recorded by the profiler when the joint was not sealed and sealed was 2 mm (0.08 inches) and 1 mm (0.04 inches), respectively. This difference in depth is not significant enough to have an effect on the IRI.

- A slight increase in RN was noted when RN obtained at different times were compared with the 1-day values. However, the increase in RN was less than 5 percent for all cases. The RN obtained when the joints were sealed was 0.1 higher than that obtained when the joint was not sealed.

- The CI values for this pavement were higher than that obtained for the other four projects evaluated in this study. Some changes in CI were observed between data sets that were collected at different times. It appears that the changes in CI are not being influenced by slab curling that occurs over the entire 6-m (20-ft) slab length, but by changes in other wavelengths in PCC slabs that were introduced during the construction.

I-80 PROJECT IN IOWA

Project Description

This project was a reconstruction project and is located in Pottawattamie County, IA. This roadway is a four-lane divided highway with two lanes in each direction. A 305-m (1,000-ft)-long test section was established on the eastbound outside lane.

Pavement Details

Table 43 presents details of the pavement section. The joint details of this section are similar to those for the U.S. 20 project (see table 33).

Concrete Mix Design

Table 44 presents the mix proportions used in the concrete mix. An entrained air admixture and a water-reducing admixture were added to the concrete mix. Table 45 presents the gradation of the aggregates used in the concrete mix.

Paving Details

Table 46 presents the date and time the test section was paved and other details regarding the paving process. The concrete was placed using a slipform paver. Two photographs of the paving process are shown in figures 94 and 95.

Profiling of Section

Five sets of profile data were collected over a 1-month period. The pavement was first profiled 2 days after paving, and the second set of profiles was collected 3 days after paving. The third and fourth sets of data were collected 1 week after paving in the morning and afternoon, respectively. The fifth set of data was collected 1 month after paving in the afternoon. The profile data collection was performed using an Ames lightweight profiler.

Table 43. Pavement details—I-80.

Item	Description	Value
Pavement thickness	Concrete thickness	300 mm (14.8 inches)
	Base thickness (mm)	260 mm (10.2 inches), semidrainable unbound granular base (crushed concrete)
Pavement width	Total pavement width	7.8 m (25.6 ft)
	Width of inside lane	3.6 m (11.8 ft)
	Width of outside lane	4.2 m (13.8 ft)
Shoulder information	Shoulder type	200 mm (7.8 inches) asphalt
	Width of shoulder	Inside 1.8 m (5.9 ft), outside 2.4 m (7.9 ft)
Joint spacing	Joint spacing	6 m (19.7 ft)
	Joints skewed?	Yes, 6:1
Dowel information	Dowel type	Epoxy coated
	Dowel diameter	38 mm (1.5 inches)
	Dowel length	457 mm (17.8 inches)
Tining information	Tining type	Longitudinal
	Tining spacing	20 mm (0.8 inch)
	Tining width	3 mm (0.1 inch)
	Tining depth	3 mm (0.1 inch)

Table 44. Mix proportions—I-80.

Component	Weight (kg/m^3 (lb/yd^3))
Cement type IP	272 (458)
Fly ash—Council Bluffs Class C	48 (81)
Coarse aggregate	870 (1,466)
Intermediate/fine	938 (1,581)

Table 45. Gradation of aggregates—I-80.

Sieve	Percentage Passing	
	Coarse Aggregate	Sand
37.5 mm (1.5 inches)	100	100
25 mm (1 inch)	100	100
19 mm (0.75 inch)	94	–
12.5 mm (0.5 inch)	58	98
9.5 mm (0.4 inch)	26	97
No. 4	4.3	82
No. 8	1.4	67
No. 16	–	53
No. 30	–	35
No. 50	–	12
No. 100	–	0.8
No. 200	0.3	0

Table 46. Paving information—I-80.

Item	Description	Comment
Date and time	Date of paving	10/7/03
	Time of paving	Most of the day
Paving process	Haul route	Adjacent to inside lane
	Stringline	8-m (26-ft) spacing, both sides
	Dowels	Fixed to base
	Tie bars	Inserted by paver
	Concrete deposit method	Belt placer
	Spreader used?	Yes, one
	Paver	Guntert & Zimmerman S850
Concrete	Temperature	24 °C (75 °F)
Curing method	Curing compound	White pigmented Curing compound

Figure 94. Overall view of the paving train.

Figure 95. Slipform paver used for paving.

Table 47 shows the dates and times when profile data collection was performed, the approximate age of the pavement at each of these instances, and the low, high, and mean air temperatures for each profiling day.

Table 47. Profile data collection—I-80.

Date of Profiling	Approximate Age of Pavement	Time at Start of Profiling	Air Temperature °C (°F)		
			Low	High	Mean
10/9/03	2 days	Morning	12 (53.6)	21 (69.8)	17 (62.6)
10/10/03	3 days	Morning	12 (53.6)	26 (78.8)	19 (66.2)
10/14/03	7 days	Morning	4 (29.2)	18 (64.4)	11 (51.8)
	7 days	Afternoon			
11/5/03	1 month	Afternoon	−6 (21.2)	6 (42.8)	1 (33.8)

The joints had been sealed except for the first 76 m (249 ft) of the test section when the 2-day testing was performed.

Roughness Indices

IRI Values

The IRI was computed for the entire 304.8-m (1,000-ft)-long test section as well as for each of the two 152.4-m (500-ft)-long sections contained within the section. The average IRI values (computed using values obtained from three runs) are shown in table 48. The IRI values for the 304.8-m (1,000-ft)-long section for the different test sequences are shown in figure 96. Table 49 shows the percentage change in IRI values for different test sequences with respect to IRI obtained from 2-day testing.

Table 48. IRI values for different test sequences—I-80.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	IRI (m/km)					
			Left Wheel Path			Right Wheel Path		
			Entire Section	First 152.4 m (500 ft)	Second 152.4 m (500 ft)	Entire Section	First 152.4 m (500 ft)	Second 152.4 m (500 ft)
9/18/03	2 days	Morning	1.04	1.09	0.98	0.88	0.97	0.79
9/20/03	3 days	Morning	1.03	1.07	0.97	0.88	0.95	0.82
9/24/03	7 days	Morning	1.14	1.17	1.11	0.94	1.03	0.86
	7 days	Afternoon	1.12	1.11	1.12	0.94	1.00	0.88
10/15/03	1 month	Afternoon	1.11	1.23	1.01	0.93	0.99	0.87

Note: Joints were not sealed in the first 76 m (249 ft) of the section during 2-day profiling. Joints were sealed for all other profiling days.

1 m/km = 63.4 inches/mi

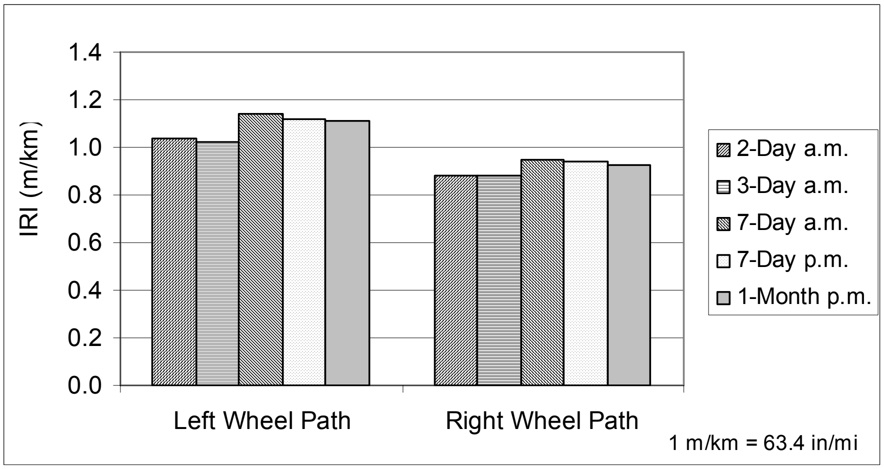


Figure 96. IRI values for different test sequences for entire test section—I-80.

Table 49. Percentage change in IRI with respect to 2-day IRI—I-80.

Approximate Age of Pavement	Time of Profiling	Percentage Change in IRI					
		Left Wheel Path			Right Wheel Path		
		Entire Section	First 152.4 m (500 ft)	Second 152.4 m (500 ft)	Entire Section	First 152.4 m (500 ft)	Second 152.4 m (500 ft)
3 days	Morning	−1	−1	−1	0	−2	3
7 days	Morning	10	7	14	7	6	9
7 days	Afternoon	8	2	14	7	3	12
1 month	Afternoon	7	13	3	6	2	10

The following observations were noted when evaluating the IRI values:

- When the entire 304.8-m (1,000-ft) section was considered, the IRI of the left wheel path was higher than the right wheel path by 16 to 21 percent for the different test sequences.

- When the individual 152.4-m (500-ft)-long sections were considered, very little difference in the IRI was noted between the 2-day and 3-day testing. However, when the 7-day and 1-month IRI values were evaluated, these values were higher than the values obtained from 2-day testing by amounts ranging from 2 to 14 percent for the different test dates and wheel paths. For the first 152.4 m (500-ft) section, the IRI at 1 month was 13 percent and 2 percent higher than the 2-day testing for left and right wheel paths, respectively. For the second 152.4 m (500-ft) section, the IRI at 1 month was 3 percent and 10 percent higher than the 2-day testing for left and right wheel paths, respectively.

RN Values

RN was computed for the entire 304.8-m (1,000-ft)-long section, as well as for the two 152.4-m (500-ft)-long sections contained within it. The average RN values are tabulated in table 50. The RN values for the 304.8-m (1,000-ft)-long section for the different test dates and times are shown in figure 97. Table 51 shows the percentage change in RN for different test sequences with respect to RN obtained from 2-day testing.

Table 50. RN values for different test sequences—I-80.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	RN					
			Left Wheel Path			Right Wheel Path		
			Entire Section	First 152.4 m (500 ft)	Second 152.4 m (500 ft)	Entire Section	First 152.4 m (500 ft)	Second 152.4 m (500 ft)
9/18/03	2 days	Morning	3.79	3.70	3.90	3.94	3.85	4.04
9/20/03	3 days	Morning	3.88	3.81	3.95	4.02	3.97	4.08
9/24/03	7 days	Morning	3.74	3.69	3.79	3.96	3.92	4.01
	7 days	Afternoon	3.78	3.75	3.82	3.95	3.94	3.97
10/15/03	1 month	Afternoon	3.78	3.65	3.90	3.96	3.94	4.00

Note: Joints were not sealed in the first 76 m (249 ft) of the section during 2-day profiling. Joints were sealed for all other profiling days.

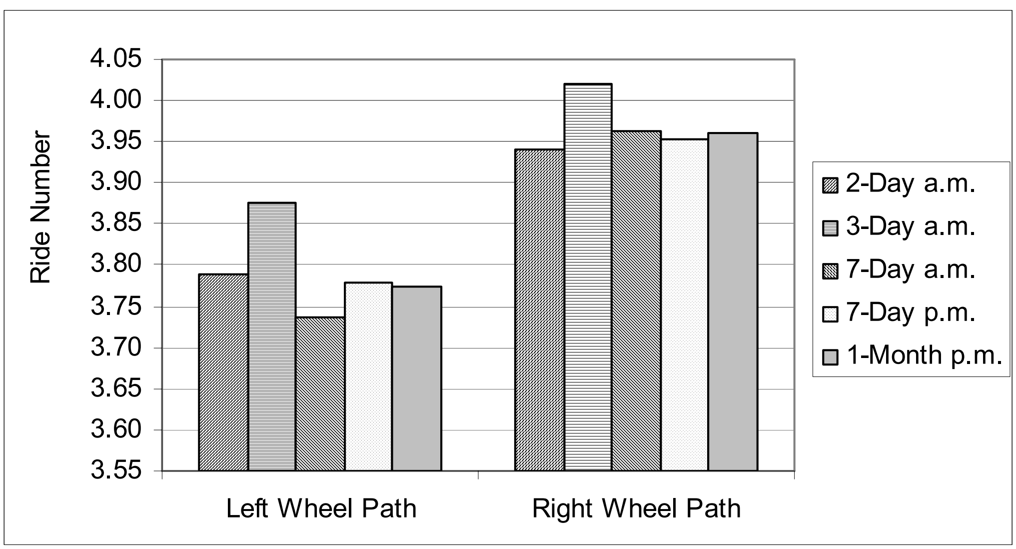


Figure 97. RN values for different test sequences for entire test section—I-80.

Table 51. Percentage change in RN with respect to 2-day RN—I-80.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Percentage Change in RN					
			Left Wheel Path			Right Wheel Path		
			Entire Section	First 152.4 m (500 ft)	Second 152.4 m (500 ft)	Entire Section	First 152.4 m (500 ft)	Second 152.4 m (500 ft)
9/20/03	3 days	Morning	2	3	1	2	3	1
9/24/03	7 days	Morning	−1	0	−3	1	2	−1
	7 days	Afternoon	0	1	−2	0	2	−2
10/15/03	1 month	Afternoon	0	−1	0	1	2	−1

The following observations were noted when evaluating the RN values:

- When the entire 304.8-m (1,000-ft) section was considered, the RN of the right wheel path was higher than that for the left wheel path by amounts ranging by 4 to 6 percent for the different test sequences.

- When the first 152.4-m (500-ft) section was considered, the RN of the two wheel paths changed by amounts varying from −1 to 3 percent from the 2-day RN for the different test sequences. When the second 152.4-m (500-ft) section was considered, the RN of the two wheel paths changed by amounts varying by −3 to 1 percent from the 2-day RN for the different test sequences. These changes are negligible, and for all practical purposes, the RN can be considered to have remained constant for all test sequences.

Repeatability of IRI Values

When all profile data sets were considered, the average difference between the maximum and minimum IRI values from the three repeat runs was 0.09 m/km (5.7 inches/mi) when the entire section was considered. When the two 152.4-m (500-ft) sections within the test section were considered individually, this value was 0.11 and 0.12 m/km (7.0 and 7.6 inches/mi) for the first and the second sections, respectively.

The short-interval IRI repeatability was evaluated using the data collected for 2-day testing along the right wheel path. For each run, the IRI values were computed at 15-m (49-ft) intervals to perform this evaluation. For the 304.8-m (1,000-ft)-long section, there are 20 15-m (49-ft)-long segments (the last 4.8 m (16 ft) was omitted). Figure 98 shows the IRI values that were obtained at each 15-m (49-ft)-long segment for the three runs. The IRI values for the segments obtained from the different runs showed high variability for several segments. The difference between the maximum and minimum IRI of the segments obtained from the three repeat runs ranged from a low of 0.07 m/km (4.4 inches/mi) that occurred at segment 16, to a high of 0.27 m/km (17 inches/mi) that occurred at segment 10, with an average value of 0.14 m/km (8.8 inches/mi).

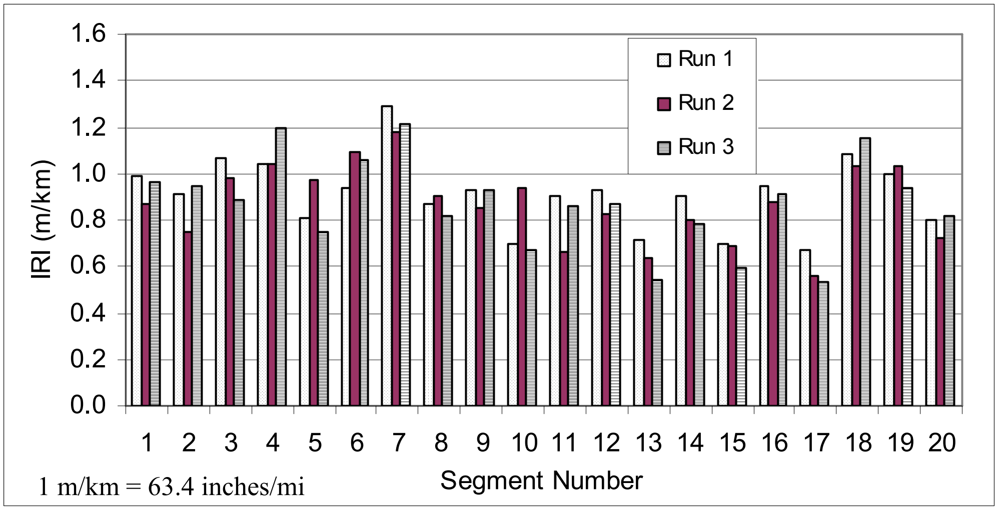


Figure 98. IRI values from repeat runs—I-80.

As in the U.S. 20 project, the IRI for the entire section for the three repeat runs used in this analysis were very close to each other, with the IRI values being 0.91, 0.87, and 0.87 m/km (58, 55, and 55 inches/mi). However, the distribution of the IRI within the section was different for the three runs. The IRI variations tend to compensate for each other when data are averaged over the entire section, and this is the reason why the overall IRI for the three repeat runs were very close to each other, even though the IRI distribution within the section for the three runs showed variability.

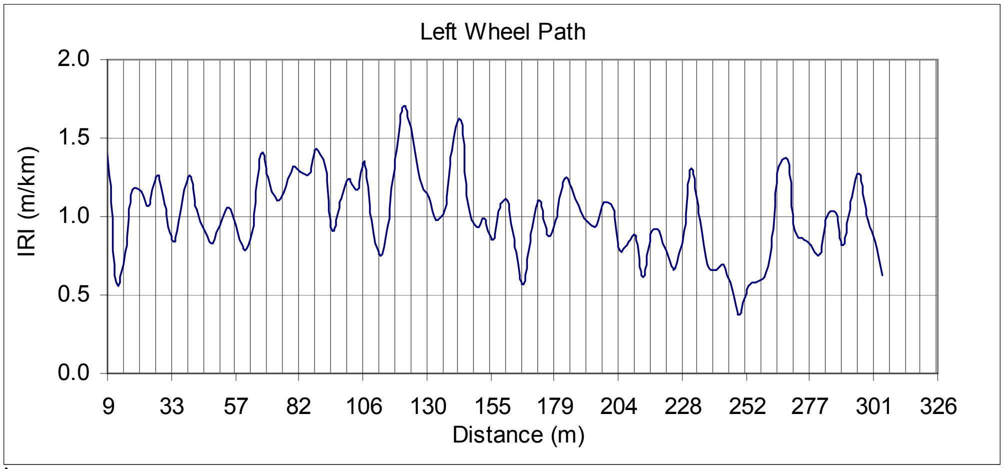
This pavement has longitudinal tining similar to the U.S. 20 project. As described for the U.S. 20 project, the variability between runs for short interval IRI in this project is also attributed to the longitudinal tining.

Effect of Condition of Joint on Profile Data

The method used to form joints in this section was similar to that used for the test section in U.S. 20, where joints were formed using a soft cut saw that sawed a joint reservoir 6 mm (0.25 inch) wide and 25.4 mm (1 inch) deep. Profile data collection for this section was performed using the same profiler that collected data at the U.S. 20 test section. The effect of the condition of the joint (sealed versus unsealed) that was seen in the profile data at this test section was similar to the observations noted at the test section located on U.S. 20.

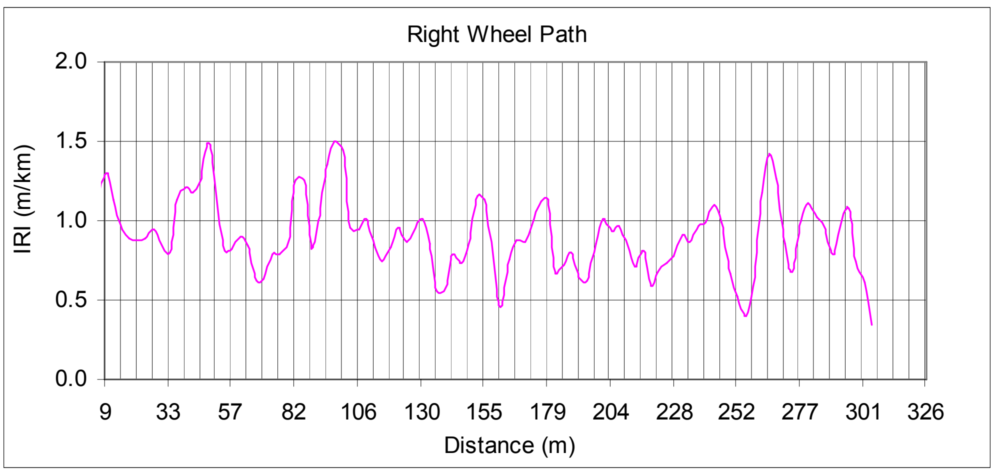
Roughness Profiles

Figures 99–101 show IRI roughness profiles based on a 6-m (20-ft) base length for day-3 testing for the left wheel path, right wheel path, and an overlaid plot of the two wheel paths.



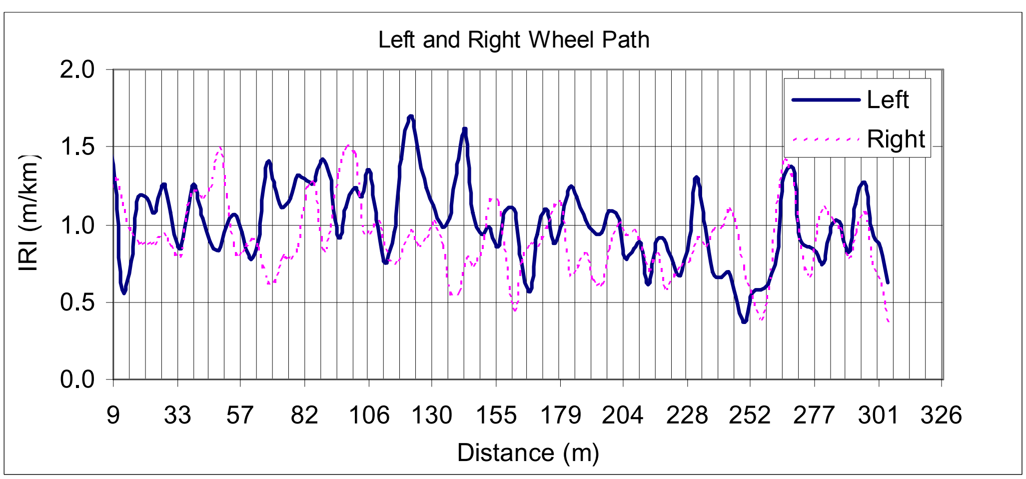
1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 99. Roughness profiles for I-80, left wheel path.



1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 100. Roughness profiles for I-80, right wheel path.



1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 101. Roughness profiles for I-80, left and right wheel path.

The vertical lines in the plots correspond to joint locations. The left wheel path shows a higher roughness level than the right wheel path, particularly for the first 155 m (508 ft) of the section. Except for some localized areas, the 6-m (20-ft) base length IRI along the right wheel path was less than 1 m/km (63 inches/mi). Some peaks in the roughness profiles are coinciding with the joint or are very close to the joint, which may indicate a possible effect of dowel baskets on the roughness.

Evaluation of Profile Data

No differences could be observed in profile data for the different test sequences. In addition, no distinct profile feature or dominant waveband that had a significant influence on the roughness could be identified in the profile data.

The CI value was computed for one profile run from each data set, and the computed values are presented in table 52. These CI values are very small, which indicates that the pavement is essentially in a flat condition. No noticeable changes in curvature have occurred over the monitored period.

Table 52. CI values—I-80.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Curvature Index x 1,000 (1/m)	
			First 152.4 m (500 ft)	Second 152.4 m (500 ft)
9/18/03	2 days	Morning	−0.10	0.11
9/20/03	3 days	Morning	−0.07	0.00
9/24/03	7 days	Morning	−0.10	−0.05
	7 days	Afternoon	−0.17	−0.04
10/15/03	1 month	Afternoon	0.02	−0.11

1/m = 1/3.28 ft

Coefficient of Thermal Expansion and Microscopical Examination

A CTE test was conducted on a 100-by-200-mm (4-by-8-inch) cylinder made from the concrete used in this project. The CTE value for the specimen was 13.2 x 10^{-6} per °C (7.33 x 10^{-6} per °F). A microscopical examination indicated that the coarse aggregate was quartzite (silicious) and the fine aggregate was silicious.

Summary

- The IRI of the left wheel path was higher than that of the right wheel path by 16 to 21 percent for the different test sequences.

- The joint dimensions in the pavement for this project were similar to that in the U.S. 20 project. Also, the same lightweight profiler was used to collect profile data. Similar observations noted for the U.S. 20 project regarding measurement of joints and impact of joints on roughness indices were noted for this project.

- When the IRI values obtained from the three repeat runs for each individual 152.4-m (500-ft)-long sections were evaluated for all data sets; the average of the difference between the maximum and minimum IRI was 0.12 m/km (7.6 inches/mi). The longitudinal tining in the sections caused this high difference to occur.

- An evaluation of short-interval IRI repeatability using 15-m (49-ft) segment lengths indicated that the IRI repeatability of the profiler was generally low. The average difference between the maximum and minimum IRI obtained from the runs for a 15-m (49-ft)-long segment was 0.14 m/km (8.9 inches/mi). The PCC pavement had longitudinal tining, and the high difference in IRI obtained between runs is attributed to the effect of longitudinal tining.

- Changes in left as well as right wheel path IRI compared to the 2-day IRI were noted for testing performed at 7 days and 1 month. The lowest changes in IRI were noted along the right wheel path of the first 152.4-m (500-ft)-long section, where the change in IRI ranged from −2 to 6 percent. The highest change in IRI was noted along the left wheel path of the second 152.4-m (500-ft)-long section, where the change in IRI ranged from −1 to 14 percent.

- The RN values for both wheel paths of the two 152.4-m (500-ft)-long sections showed little change for the different profiling dates over the 1-month period. The change in RN compared to the 2-day RN was within ±3 percent for the different profiling times over the 1-month period.

- There was negligible curvature in the PCC slabs for the different data sets. Hence, the PCC slabs can be considered to be flat. No noticeable changes in curvature were noted between the different test sequences.

U.S. 23 PROJECT IN MICHIGAN

Project Description

This project was a reconstruction project located in Monroe County, MI. This roadway is a four-lane divided highway with two lanes in each direction. A 157-m (515-ft)-long test section was established for testing on the two southbound lanes. The test section was established between stations 739+85 and 745+00. (Stations are in U.S. customary units.)

Pavement Details

Table 53 provides pavement details. Table 54 presents information about the joints in the pavement.

Table 53. Pavement details—U.S. 23.

Item	Description	Value
Pavement thickness	Concrete thickness	280 mm (10.9 inches)
	Base thickness (mm)	100 mm (4 inches) open-graded aggregate drainage course on 100 mm (4 inches) of dense-graded aggregate base
Pavement width	Total pavement width	7.9 m (25.9 ft)
	Width of inside lane	3.6 m (11.8 ft)
	Width of outside lane	4.3 m (14.1 ft)
Shoulder information	Shoulder type	Asphalt
	Width of shoulder	Inside 1.2 m (3.9 ft), outside 3 m (9.8 ft)
Joint spacing	Joint spacing	4.6 m (15.1 ft)
	Joints skewed?	No
Dowel information	Dowel type	Epoxy coated
	Dowel diameter	32 mm (1.25 inches)
	Dowel length	457 mm (17.8 inches)
Tining information	Tining type	Transverse tining
	Tining spacing	12.5 mm (0.5 inch)
	Tining width	3 mm (0.12 inch)
	Tining depth	3 to 6 mm (0.12 to 0.23 inch)

Table 54. Joint details—U.S. 23.

Description	Value
Joint formation	Initial sawcut then reservoir widened
Initial sawcut	3 mm (0.12 inch) wide and 89 mm (3.5 inches) deep
Width of cut	9.5 mm (0.4 inch)
Depth of cut	38 mm (1.5 inches)
Sealant type	Neoprene
Depth to top of sealant	6 mm ±1.5 mm (0.25 ±0.06 inch)

Concrete Mix Design

Table 55 presents the mix proportions used in the concrete mix. An air-entraining admixture and a water-reducing admixture were added to the concrete mix. Table 56 presents the gradation of the aggregates used in the concrete mix. The coarse aggregate consisted of a mix of Michigan DOT (MDOT) 4AA and 6AAA limestone aggregate.

Table 55. Mix proportions—U.S. 23.

Component	Weight kg/m^3 (lb/yd^3)
Cement Type 1	279 (470)
Coarse Aggregate—4AA Limestone	377 (635)
Coarse Aggregate—6AAA Limestone	688 (1,160)
Sand	866 (1,460)

Table 56. Gradation of aggregates—U.S. 23.

Sieve	Percentage Passing	
	Combined Coarse Aggregate	Sand
75 mm (3 inches)	100	–
37.5 mm (1.5 inches)	100	–
50 mm (2 inches)	94	–
37.5 mm (1.5 inches)	80	–
25 mm (1 inch)	67	–
19 mm (0.75 inch)	49	–
12.5 mm (0.5 inch)	17	–
9.5 mm (0.4 inch)	8	100
No. 4	3	100
No. 8	3	85
No. 16	3	66
No. 30	3	44
No. 50	3	18
No. 100	3	4
No. 200	–	1.2

Paving Details

Table 57 presents the date and time of paving of the test section as well as other details about the paving process. The paving commenced at 6:30 a.m. and stopped at 11:30 a.m. because rain was predicted later for that day. It started to rain in the afternoon.

Figure 102 shows the view of the base course ahead of the paver and shows the tie bars and dowel baskets placed on the base. Figure 103 shows a view of the paving train used in the project, which consisted of a spreader, slipform paver, and the curing/texturing unit.

Table 57. Paving information—U.S. 23.

Item	Description	Comment
Date and time	Date of paving	8/3/03
	Time of paving	6:30 to 11:30 a.m.
Paving process	Haul route	Adjacent to inside shoulder
	Stringline	7.6-m (25-ft) spacing, both sides
	Dowels	Fixed to base
	Tie bars	Fixed to base
	Concrete deposit method	Belt placers
	Spreader used?	Yes, one
Curing method	Curing compound	ChemMasters' Safe-Cure 1000™

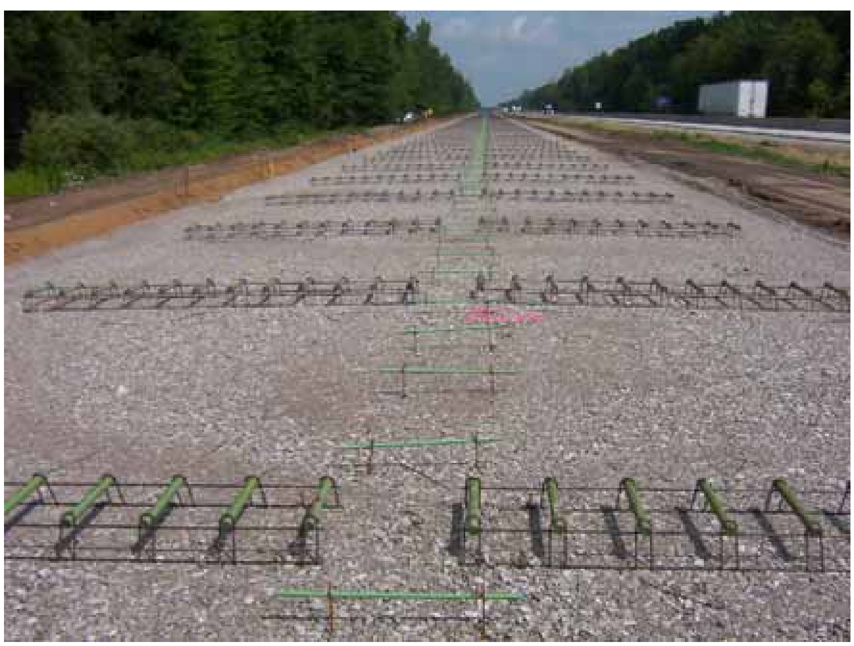


Figure 102. Tie bars and dowel basket placed on base.

Profiling of Section

Five sets of profile data were collected at the test section in 2003 within a 10-day period. The profile data were collected using a lightweight profiler that was owned by MDOT, which was using software developed by Transology Association. Figure 104 shows a photograph of the lightweight profiler. This profiler is equipped with a laser height sensor and recorded profile data at 76-mm (3-inch) intervals.

The pavement was first profiled 1 day after paving. Initial sawcuts over joints had been completed a short time before profiling, and dry slurry from the saw-cutting operation was present adjacent to the joints. The second set of data was collected 5 days after paving. The third set of data was collected 9 days after paving in the morning. The joint reservoirs were sawed and unsealed when the profile data were collected. The contractor sealed the joints in the afternoon. The fourth and fifth sets of data were collected on the 10^{th} day after paving in the morning and the afternoon, respectively.

Figure 103. View of the paving train.

Figure 104. Profile data collection using a lightweight inertial profiler.

Approximately 1 year after paving, profile data were collected at the test section using a Dynatest® high-speed profiler. This profiler is equipped with laser height sensors and recorded profile data at 25-mm (1-inch) intervals. Figure 105 shows a photograph of the Dynatest high-speed profiler.

Figure 105. High-speed inertial profiler.

Table 58 presents the dates and times when profile data collection was performed, the approximate age of the pavement at each instance, and the low, high, and mean air temperatures for each data collection date.

Table 58. Profile data collection—U.S. 23.

<table>
<tr><th rowspan="4">Date of Profiling</th><th rowspan="4">Approximate Age of Pavement</th><th colspan="4">Time of Profiling</th><th colspan="3" rowspan="3">Air Temperature °C (°F)</th></tr>
<tr><th colspan="2">Inside Lane</th><th colspan="2">Outside Lane</th></tr>
<tr><th>Left</th><th>Right</th><th>Left</th><th>Right</th></tr>
<tr><th>Wheel Path</th><th>Wheel Path</th><th>Wheel Path</th><th>Wheel Path</th><th>Low</th><th>High</th><th>Mean</th></tr>
<tr><td>8/4/03</td><td>2 days</td><td>10:33 a.m.</td><td>10:57 a.m.</td><td>10:38 a.m.</td><td>10:48 a.m.</td><td>16 (60)</td><td>27 (80)</td><td>22 (71)</td></tr>
<tr><td>8/8/03</td><td>5 days</td><td>2:39 p.m.</td><td>12:25 p.m.</td><td>1:03 p.m.</td><td>11:49 a.m.</td><td>19 (66)</td><td>28 (82)</td><td>23 (73)</td></tr>
<tr><td>8/12/03</td><td>9 days</td><td>10:34 a.m.</td><td>11:11 a.m.</td><td>10:44 a.m.</td><td>10:58 a.m.</td><td>19 (66)</td><td>28 (82)</td><td>24 (75)</td></tr>
<tr><td rowspan="2">8/13/03</td><td>10 days</td><td>10:47 a.m.</td><td>10:21 a.m.</td><td>10:42 a.m.</td><td>10:05 a.m.</td><td rowspan="2">18 (64)</td><td rowspan="2">29 (84)</td><td rowspan="2">24 (75)</td></tr>
<tr><td>10 days</td><td>3:04 a.m.</td><td>3:34 p.m.</td><td>2:55 p.m.</td><td>3:54 p.m.</td></tr>
<tr><td>8/5/04</td><td>1 year</td><td>6:30 p.m.</td><td>6:30 p.m.</td><td>6:02 p.m.</td><td>6:02 p.m.</td><td>13 (55)</td><td>23 (73)</td><td>18 (64)</td></tr>
</table>

Roughness Indices

IRI Values

The average IRI values (computed from the three repeat runs) are presented in table 59 and shown in figure 106. Table 60 shows the percentage change in IRI values for the different test sequences with respect to the IRI obtained at 1 day.

Table 59. IRI values for different test sequences—U.S. 23.

<table>
<tr><th rowspan="3">Date of Profiling</th><th rowspan="3">Approximate Age of Pavement</th><th rowspan="3">Time of Profiling</th><th colspan="4">IRI (m/km)</th></tr>
<tr><th colspan="2">Inside Lane</th><th colspan="2">Outside Lane</th></tr>
<tr><th>Left Wheel Path</th><th>Right Wheel Path</th><th>Left Wheel Path</th><th>Right Wheel Path</th></tr>
<tr><td>8/4/03</td><td>1 day</td><td>10:30 a.m. to 10:50 a.m.</td><td>1.11</td><td>0.99</td><td>1.07</td><td>0.85</td></tr>
<tr><td>8/8/03</td><td>5 days</td><td>11:50 a.m. to 2:40 p.m.</td><td>1.01</td><td>0.83</td><td>0.81</td><td>0.79</td></tr>
<tr><td>8/12/03</td><td>9 days</td><td>10:35 to 11:15 a.m.</td><td>1.03</td><td>0.99</td><td>1.10</td><td>0.84</td></tr>
<tr><td rowspan="2">8/13/03</td><td>10 days</td><td>10:05 to 10:50 a.m.</td><td>0.89</td><td>0.79</td><td>0.84</td><td>0.70</td></tr>
<tr><td>10 days</td><td>2:55 to 3:55 p.m.</td><td>0.90</td><td>0.81</td><td>0.85</td><td>0.72</td></tr>
<tr><td>8/4/04</td><td>1 year</td><td>6 to 6:30 p.m.</td><td>0.74</td><td>0.74</td><td>0.70</td><td>0.73</td></tr>
</table>

Notes: 1. Initial sawcut 3-mm (0.12-inch)-wide present when data were collected at 1 and 5 days.

2. Joint reservoir had been sawed but unsealed when data were collected at 9 days.

3. Joints were sealed when data were collected at 10 days.

1 m/km = 63.4 inches/mi

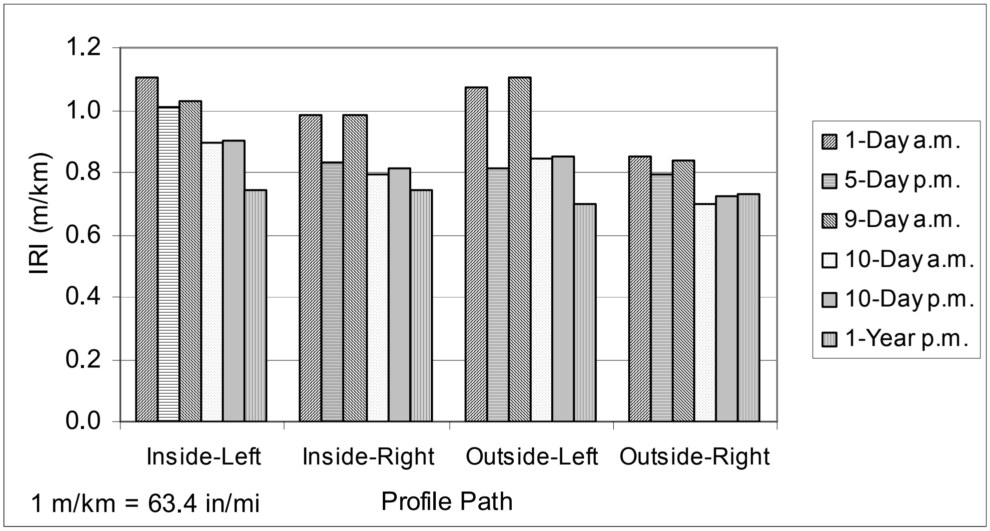


Figure 106. IRI values for different test sequences—U.S. 23.

Table 60. Percentage change in IRI with respect to 1-day IRI—U.S. 23.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Percentage Change in IRI			
			Inside Lane		Outside Lane	
			Left Wheel Path	Right Wheel Path	Left Wheel Path	Right Wheel Path
8/8/03	5 days	11:50 a.m. to 2:40 p.m.	−9	−16	−24	−7
8/12/03	9 days	10:35a.m. to 11:15 a.m.	−7	0	3	−2
8/13/03	10 days	10:05 a.m. to 10:50 a.m.	−19	−20	−21	−18
	10 days	2:55 p.m. to 3:55 p.m.	−18	−17	−21	−15
8/4/03	1 year	6 to 6:30 p.m.	−33	−25	−35	−14

Notes: 1. Joint reservoir had been sawed but unsealed when data were collected at 9 days.

2. Joints were sealed when data were collected at 10 days.

The following observations were noted when evaluating the IRI values:

- The mean IRI of the inside lane was slightly higher than the IRI of the outside lane. Based on the average obtained for all test sequences, the inside lane had a mean IRI that was 9 percent higher than the IRI of the outside lane.

- For all wheel paths, the IRI obtained at 5 days was lower than the IRI obtained at 1 day, with the reduction in IRI ranging from 7 to 24 percent for the different wheel paths.

- For all wheel paths, the IRI obtained at 9 days was higher than the IRI obtained at 5 days. When the 5-day data collection was performed, the initial sawcut that was 3 mm (0.12 inch) wide had been made on the pavement. When the 9-day data collection was performed, the joint reservoirs had been sawed on the pavement, but the joints had not been sealed. The increase in IRI for the 9-day testing is attributed to these open joint reservoirs present on the pavement. The increase in IRI at 9-day data collection with respect to the 5-day data collection was 2, 18, 35, and 5 percent for the inside lane left wheel path, inside lane right wheel path, outside lane left wheel path, and outside lane right wheel path, respectively.

- For all wheel paths, the IRI obtained at 10 days was lower than the IRI obtained at 9 days. When the 9-day data collection was performed, the joint reservoirs had been sawed on the pavement, but the joints had not been sealed. The joints had been sealed when the 10-day data collection was performed. The reduction in IRI for the 10-day testing compared to the 9-day testing is attributed to the sealing of the joints. The reduction in IRI due to sealing of the joints was 13, 19, 23, and 16 percent for the inside lane left wheel path, inside lane right wheel path, outside lane left wheel path, and outside lane right wheel path, respectively.

- No appreciable changes in IRI values were noted for the morning and afternoon IRI values obtained from data collected during the 10-day profiling.

- The IRI values obtained from day-10 profiling was less than the IRI obtained from day-1 profiling by values ranging from 18 to 21 percent for the different wheel paths.

- The IRI obtained from data collected 1 year after paving was the lowest IRI obtained for all test sequences, except for the right wheel path of the outside lane. The IRI at 1 year was lower than the IRI at 1 day by 33, 25, 35, and 14 percent for the inside lane left wheel path, inside lane right wheel path, outside lane left wheel path, and outside lane right wheel path, respectively. The IRI at 1 year was lower than the IRI at 10-day by 17, 9, and 18 percent for the inside lane left wheel path, inside lane right wheel path, and outside lane left wheel path, respectively. For the outside lane right wheel path, the 1-year IRI was 1 percent higher than the 10-day IRI.

- Overall, a general reduction in IRI over time was noted.

RN Values

The average RN values are presented in table 61 and shown in figure 107. Table 62 shows the percentage change in RN values for the different test sequences with respect to the RN obtained at 1 day.

The following observations were noted when evaluating the RN values:

- A slight increase in RN was obtained for 5-day data compared to those from 1-day data. For the different wheel paths, the 5-day RN values were higher by amounts ranging from 1 to 6 percent.

- For all wheel paths, the RN obtained at 9 days was lower than the RN obtained at 1 day. When the 1-day data collection was performed, the initial sawcut that was 3-mm (0.12 inch) wide had been made on the pavement. When the 9-day data collection was performed, the joint reservoirs had been sawed on the pavement, but the joints had not been sealed. The decrease in RN for the 9-day testing is attributed to these open joint reservoirs. The RN for 9-day testing was lower than the RN from 1-day testing by amounts ranging from 17 to 25 percent for the different wheel paths.

- The 10-day RN values were very close to 1-day RN values. The difference between 10-day RN and 1-day RN ranged from −1 to 4 percent for the different wheel paths.

- The 1-year RN values were very close to 1-day RN values, but slightly higher for all cases. The difference between 1-year RN and 1-day RN ranged from 2 to 3 percent for three wheel paths; in contrast, for the right wheel path of the outside lane this difference was 7 percent.

- Overall, little difference in RN occurred over a 1-year period at this section.

Table 61. RN values for different test sequences.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Ride Number			
			Inside Lane		Outside Lane	
			Left Wheel Path	Right Wheel Path	Left Wheel Path	Right Wheel Path
8/4/03	1 day	10:30 a.m. to 10:50 a.m.	3.81	3.84	3.83	3.72
8/8/03	5 days	11:50 a.m. to 2:40 p.m.	3.86	3.93	4.06	3.87
8/12/03	9 days	10:35 a.m. to 11:15 a.m.	3.00	2.86	2.88	3.10
8/13/03	10 days	10:05 a.m. to 10:50 a.m.	3.75	3.70	3.79	3.87
	10 days	2:55 p.m. to 3:55 p.m.	3.81	3.69	3.77	3.84
8/4/04	1 year	6 p.m. to 6:30 p.m.	3.90	3.91	3.94	3.99

Notes: 1. Initial sawcut 3 mm (0.12 inch) wide present when data were collected at 1 and 5 days.

2. The joint reservoir had been sawed when data were collected at 9 days.

3. Joints were sealed when data were collected at 10 days.

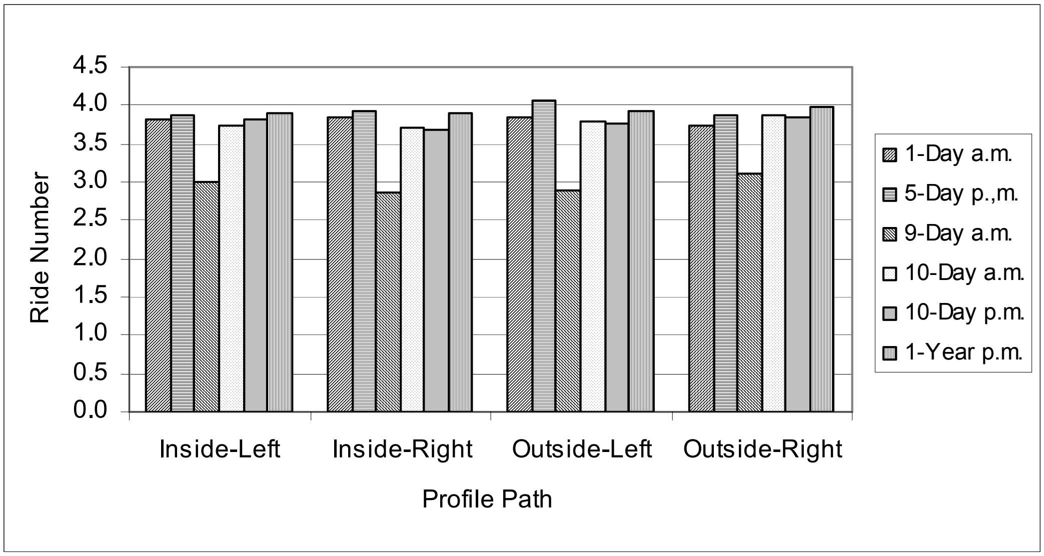


Figure 107. RN values for different test sequences—U.S. 23.

Table 62. Percentage change in RN with respect to 1-day RN—U.S. 23.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Percentage Change in RN with Respect to 1-Day RN (%)			
			Inside Lane		Outside Lane	
			Left Wheel Path	Right Wheel Path	Left Wheel Path	Right Wheel Path
8/8/03	5 days	11:50 a.m. to 2:40 p.m.	1	2	6	4
8/12/03	9 days	10:35 a.m. to 11:15 a.m.	−21	−25	−25	−17
8/13/03	10 days	10:05 a.m. to 10:50 a.m.	−2	−3	−1	4
	10 days	2:55 p.m. to 3:55 p.m.	0	−4	−2	3
8/4/04	1 year	6 p.m. to 6:30 p.m.	2	2	3	7

Notes: 1. Initial sawcut 3 mm (0.12 inch) wide present when data were collected at 1 and 5 days.
2. The joint reservoir had been sawed when data were collected at 9 days.
3. Joint was sealed when data were collected at 10 days.

Repeatability of IRI Values

When the IRI from three runs for all test sequences were evaluated, the difference between the maximum and the minimum IRI was between 0.01 to 0.11 m/km (0.6 to 7.0 inch/mi), with the average difference being 0.06 m/km (3.8 inches/mi).

An evaluation of the repeatability of short-interval IRI was performed by computing IRI at 15-m (49-ft) intervals for the three repeat runs collected along the left wheel path of the inside lane for the day-10 morning runs. For the 156-m (512-ft)-long section, there are 10 15-m (49-ft) long segments (the last 6 m (20 ft) of the section were not considered). Figure 108 shows the IRI values that were obtained at 15-m (49-ft) intervals for each run for each segment.

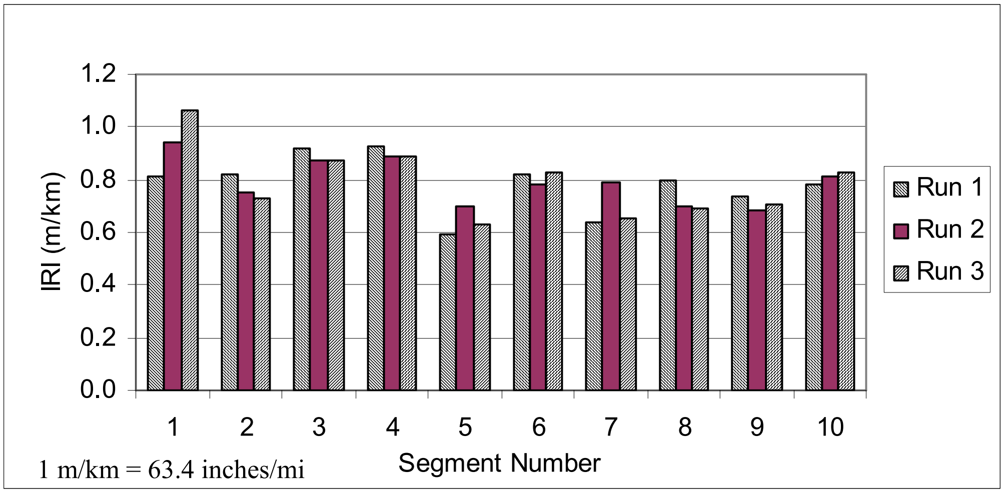


Figure 108. Repeatability of IRI values—U.S. 23.

A few sections had significant differences in IRI values between runs. The difference between the maximum and minimum IRI for each segment obtained from the three repeat runs ranged from a low of 0.04 m/km (2.5 inches/mi) that occurred at segment 4, to a high of 0.25 m/km (15.6 inches/mi) that occurred at segment 1, with an average 0.10 m/km (6.3 inches/mi). The IRI values obtained from the three runs for the entire section were 0.79, 0.80, and 0.79 m/km (50, 51, and 50 inches/mi). Significant differences in IRI were shown for repeat runs at a few 15-m (49-ft) segments. However, when IRI of the entire section from the three runs was compared, the three values were almost identical. As described previously for the other projects, this result is caused by compensating effects.

Effect of Joint Condition on Profile Data

During this study, the lightweight profiler collected profile data when the joints were in the following conditions:

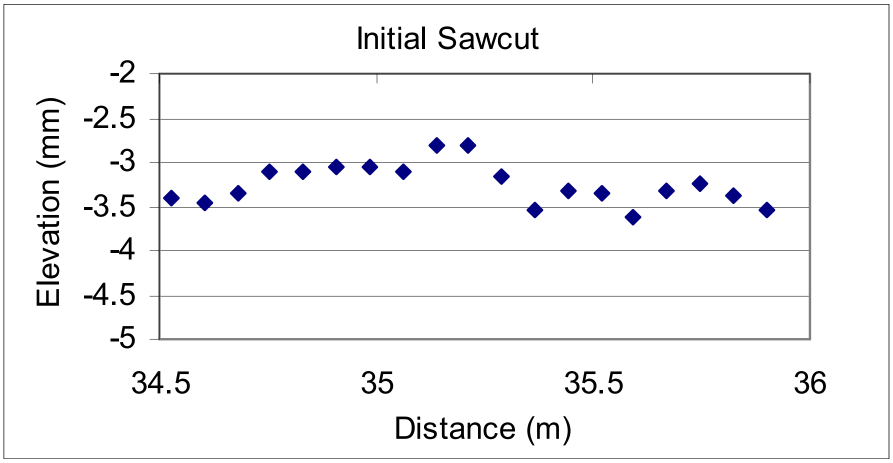
- The initial sawcut that was 3 mm (0.12 inch) wide and 93 mm (3.7 inches) deep had been made on the pavement (1-day profiling).
- The joint reservoir that was 9.5 mm (0.4 inch) wide and 38 mm (1.5 inches) deep had been made on the pavement, but the joint was not sealed (9-day profiling).
- The joints had been sealed (10-day profiling).

The data collected under these three conditions were evaluated to investigate how the joints showed up on the profile. Data collected along the left wheel path of the outside lane were used in this investigation. Figures 109–111 show how a typical joint appeared on the profile for each of these conditions. The joint is located between 35 and 35.5 m (114.8 and 116.4 ft). Note the following observations for each case shown in figures 109–111:

- Initial sawcut: The joint location cannot be detected in the profile data plot.

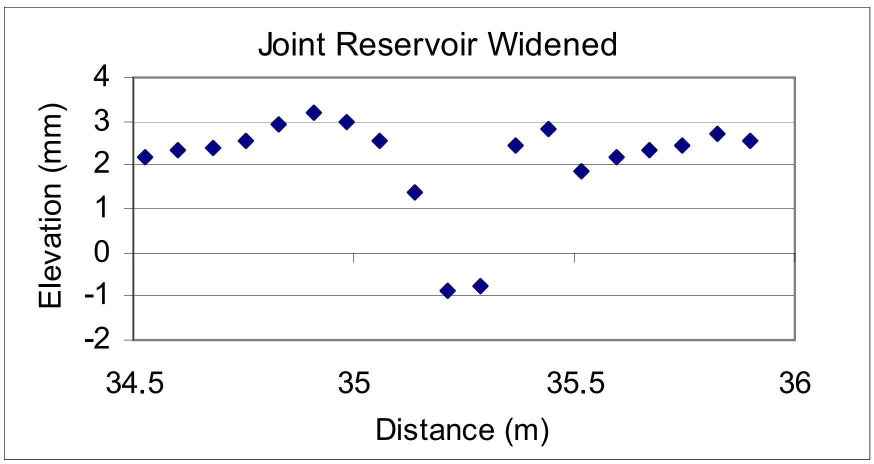
- Joint reservoir sawed: In the profile, the joint appears as a feature spread over a distance of about 450 mm (17.7 inches), with a depth of about 4 mm (0.16 inches). Sharp differences in elevation between adjacent data points were noted at data collected over the joints. For the entire profile, there were 21 joint locations where the difference between adjacent data points was greater than 3 mm (0.12 inches).

- Joint sealed: In this profile, the joint appears as a feature spread over about 450 mm (17.7 inches), with a depth of 1.25 mm (0.05 inches). Unlike for the previous case, sharp differences in elevation did not occur in the data collected over a joint. In this profile, there were only three locations where the elevation difference between adjacent points exceeded 2 mm (0.08 inches).



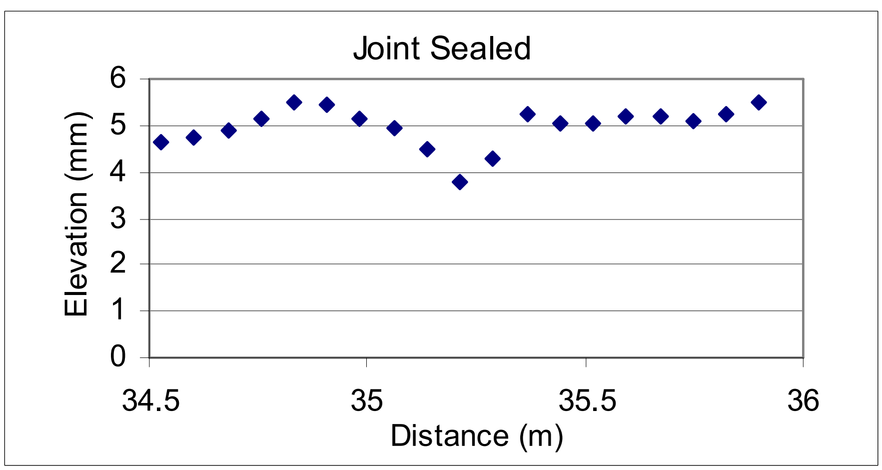
1 m = 3.28 ft
1 mm = 0.039 inch

Figure 109. Measurement at a joint, initial sawcut—U.S. 23.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 110. Measurement at a joint, joint reservoir widened—U.S. 23.



1 m = 3.28 ft
1 mm = 0.039 inch

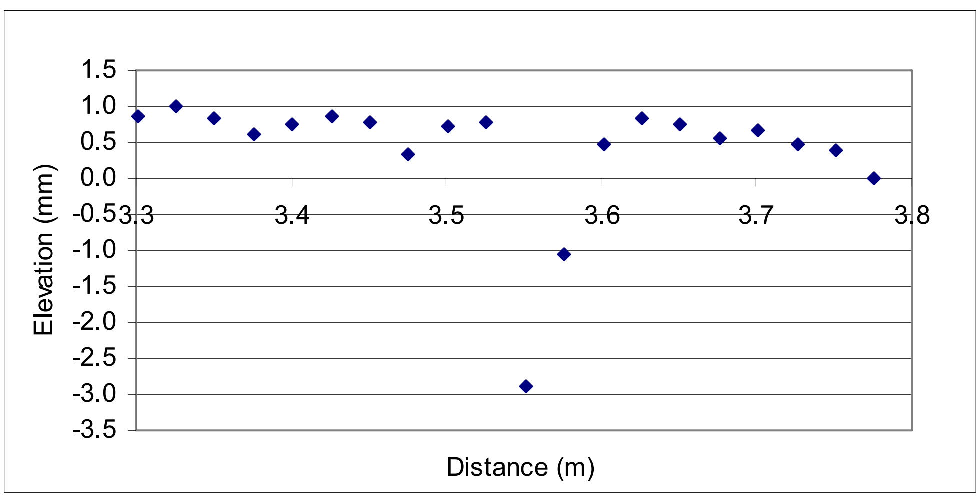
Figure 111. Measurement at a joint, joint sealed—U.S. 23.

An evaluation of IRI values shown in table 59 shows that IRI obtained after the joints were sealed was lower than that obtained when the joint reservoir was sawed but the joint not sealed. IRI decreased after the joint was sealed because the lower depth of the joint is recorded in the profile.

As described previously for the other projects, the averaging performed on the height sensor data and the anti-alias filter applied on the profile data cause the depth of the joint to be attenuated. In addition, these factors cause the joint to appear in the profile data as a feature spread over a much wider distance than the actual width of the joint. The actual width of the joint in this case is 9.5 mm (0.4 inch), but the joint appears in the profile as a dip that is spread over a distance of 450 mm (17.7 inches). The depth of the joint reservoir is 38 mm (1.5 inches) and when sealed the depth to the top of the sealant from the pavement surface according to the specification should be 6 mm (0.25 inch). The depths noted for these two cases in the profile were approximately 4 and 1.5 mm (0.16 and 0.06 inch), respectively.

The 1-year data at this section were collected using a Dynatest high-speed profiler. The locations of the joints were clearly visible in the data collected by this profiler. Figure 112 shows the data that were typically collected by this profiler at a joint. The joint is located between 3.5 and 3.6 m (11.5 and 11.8 ft).

This profiler recorded the joint as a feature that was 75 mm (3 inches) wide and about 3.75 mm (0.15 inch) deep. The width of the feature recorded by this profiler was much less than that recorded by the lightweight profiler. However, the depth of the feature recorded by this profiler was higher than that recorded by the lightweight profiler. These observations show the two profilers treated a joint very differently. This difference is attributed to differences in height sensor averaging methods and anti-alias filters that are employed in the two profilers.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 112. Data collected over a joint by the high-speed profiler.

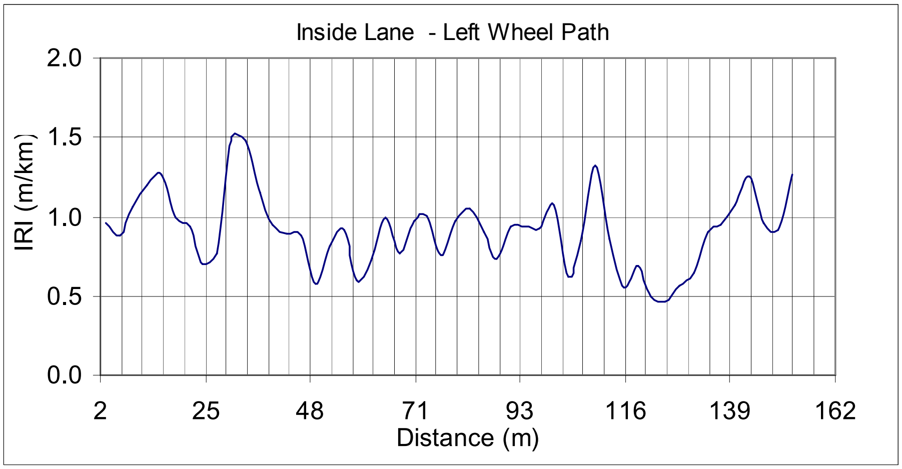
Roughness Profiles

The 6-m (20-ft) base length IRI roughness profiles for the day-10 morning data for the inside and outside lanes are shown in figures 113–118, respectively. In each roughness profile, the vertical lines correspond to a joint location.

As seen in figures 113–115, several localized rough spots were along both wheel paths of the inside lane. The roughness at these localized locations was not excessive, except in the left wheel path of the inside lane, where there was a noticeable rough spot at about 35 m (115 ft). Figures 116–118 shows that in the outside lane, the first 16 m (49 ft) of the left wheel path has a higher roughness compared to the rest of the left wheel path and the right wheel path. In the outside lane apart from the first 16 m (49 ft) of the left wheel path, the left and right wheel paths show a close roughness distribution along the section.

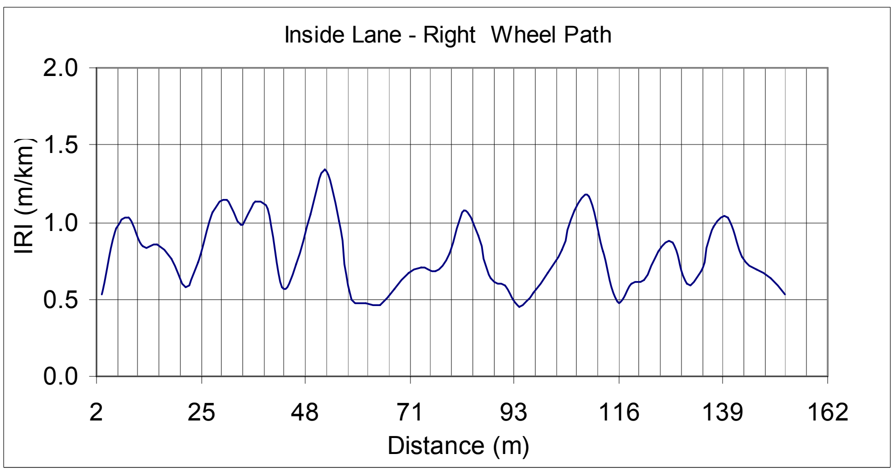
Evaluation of Profile Data

An evaluation of profile data collected at different time sequences indicated that the profiles obtained at day 1 were different from the other profiles because they showed a small hump around each joint. Figure 119 shows how the humps appeared in the profile. The profile data collection at day 1 was performed immediately after the joints were sawed, and residue from the sawing operation was present adjacent to the joint. Figure 120 shows a photograph of the pavement that was obtained at the time profiling was performed at day 1. The humps seen in the profile appear to have been caused by the residue from the sawing operation. These humps were not noticeable for data collected after day-1 profiling because the residue would have been washed from the rain experienced after profiling. The decrease in IRI after day-1 profiling is attributed to elimination of these humps from the profile.



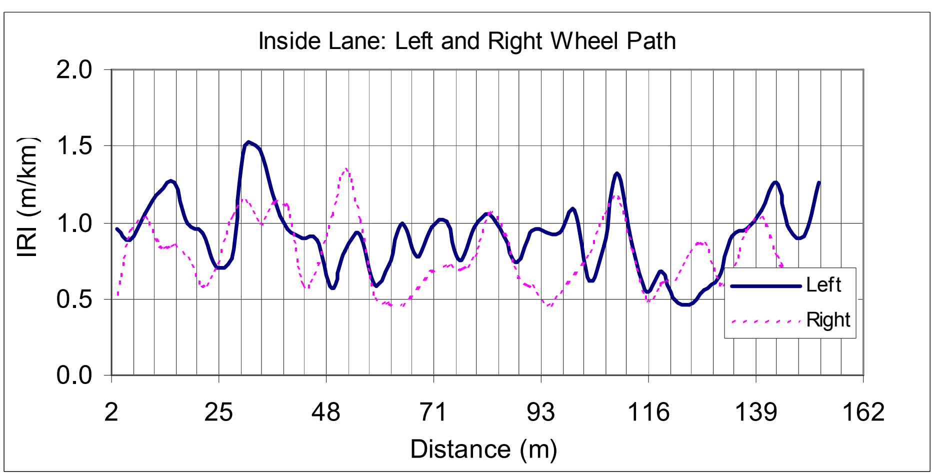
1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 113. Roughness profiles for inside lane, left wheel path—U.S. 23.



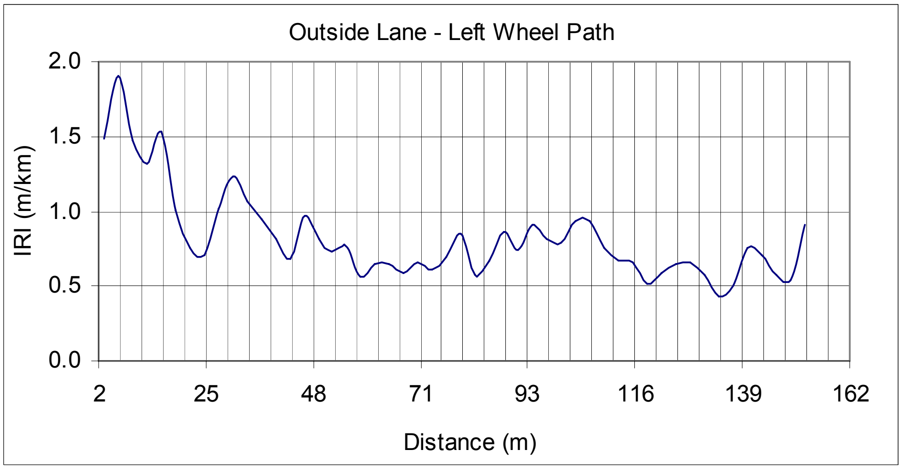
1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 114. Roughness profiles for inside lane, right wheel path—U.S. 23.



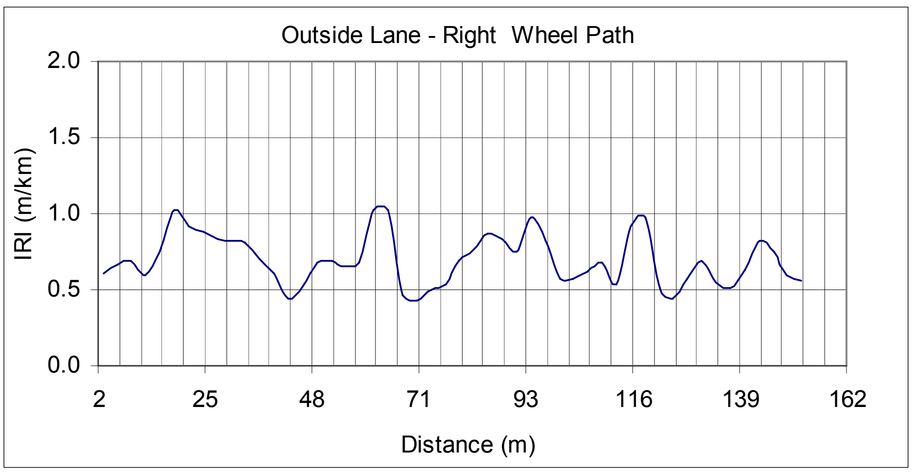
1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 115. Roughness profiles for inside lane, left and right wheel path—U.S. 23.



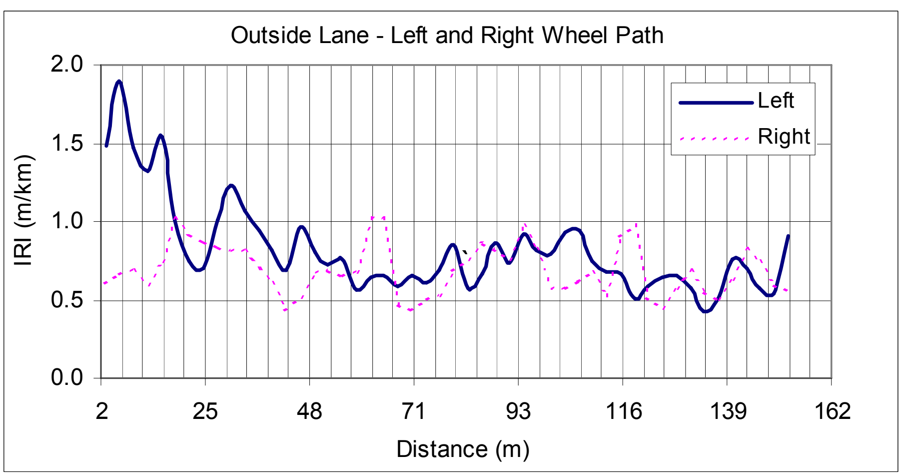
1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 116. Roughness profiles for outside lane, left wheel path—U.S. 23.



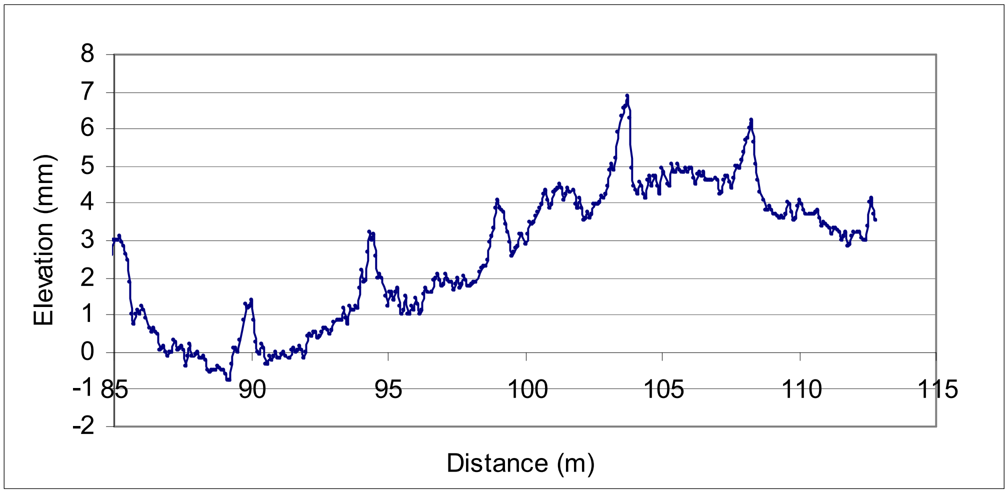
1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 117. Roughness profiles for outside lane, right wheel path—U.S. 23.



1 m = 3.28 ft
1 m/km = 63.4 inches/mi

Figure 118. Roughness profiles for outside lane, left and right wheel path—U.S. 23.



1 m = 3.28 ft
1 mm = 0.039 inch

Figure 119. Humps in profile for 1-day profile data.

Figure 120. Residue from joint sawing operation adjacent to a joint.

IRI obtained from 1-year profiling (data collected by high-speed profiler) was lower than IRI obtained from day-10 profiling (data collected by lightweight profiler). There were some differences in these two profile data sets, but no distinct profile feature that caused the differences in IRI to occur could be identified. Analysis of the data collected over the joints indicated these two profilers have different averaging and/or low-pass filtering methods. It is not clear whether the reduction in IRI was related to changes in the pavement profile or to differences between the profiling equipment.

CI was computed for one profile run from each data set, and the computed values are shown in table 63. These CI values are very small indicating the pavement is essentially in a flat condition. No noticeable changes in curvature have occurred over the monitored period.

Table 63. CI values—U.S. 23.

<table>
<tr><th rowspan="3">Date of Profiling</th><th rowspan="3">Approximate Age of Pavement</th><th rowspan="3">Time of Profiling</th><th colspan="3">Curvature Index x 1,000 (1/m)</th></tr>
<tr><th>Inside Lane</th><th colspan="2">Outside Lane</th></tr>
<tr><th>Right Wheel Path</th><th>Left Wheel Path</th><th>Right Wheel Path</th></tr>
<tr><td>8/4/03</td><td>1 day</td><td>10:30 .to 10:50 a.m.</td><td>0.07</td><td>0.13</td><td>0.07</td></tr>
<tr><td>8/8/03</td><td>5 days</td><td>11:50 a.m. to 2:40 p.m.</td><td>0.00</td><td>−0..07</td><td>0.01</td></tr>
<tr><td>8/12/03</td><td>9 days</td><td>10:35 to11:15 a.m.</td><td>0.12</td><td>−0.16</td><td>−0.17</td></tr>
<tr><td rowspan="2">8/13/03</td><td>10 days</td><td>10:05 to 10:50 a.m.</td><td>−0.02</td><td>−0.16</td><td>−0.10</td></tr>
<tr><td>10 days</td><td>2:55 to 3:55 p.m.</td><td>−0.08</td><td>−0.16</td><td>−0.11</td></tr>
<tr><td>8/4/04</td><td>1 year</td><td>6 to 6:30 p.m.</td><td>−0.01</td><td>−0.26</td><td>0.18</td></tr>
</table>

1/m = 1/3.28 ft

Coefficient of Thermal Expansion and Microscopical Examination

The core obtained from this pavement was lost in the mail when it was shipped to the laboratory to perform the CTE test. Hence, a CTE value is not available for the concrete used in this project.

Summary

- The mean IRI obtained from day-10 testing indicated that the inside lane had an IRI of 0.86 m/km (0.10 yd/mi), which was 9 percent higher than that of the outside lane. However, testing performed 1 year after paving indicated that the mean IRI of the inside and outside lanes were very close to each other, with the values being 0.74 and 0.72 m/km (47 and 46 inches/mi), respectively.

- When IRI obtained from the three repeat runs for the entire section were evaluated for all data sets collected by the lightweight profiler, the average of the difference between the maximum and minimum IRI was 0.06 m/km (3.8 inches/mi).

- An evaluation of short-interval IRI repeatability of the lightweight profiler showed the average difference between the maximum and minimum IRI obtained from the repeat runs for a 15-m (49-ft)-long segment was 0.10 m/km (6.3 inches/mi).

- An evaluation of profile data collected 1 day after paving showed humps appearing at joint locations. Profile data collection on this day was performed immediately after the joints were sawed. It appears that the residue from the joint sawing operation was responsible for the humps. Profile data collected at other times did not show these humps. The residue present on the first day is likely to have been washed away by the rain that occurred after the day-1 profiling was performed. These humps caused the day-1 IRI to be higher than IRI values obtained from all other data collection sequences.

- The data collected by the lightweight profiler did not accurately record the shape of the joint reservoir in the pavement. The joint could not be observed in the profile data when data collection was performed when the 3-mm (0.12-inch)-wide initial sawcut was present on the pavement. However, in the data collected after the 9.5-mm (0.4-inch)-wide joint reservoir was formed, the joint appeared as a small dip that was spread over a distance of 450 mm (17.7 inches), with a maximum depth of about 4 mm (0.16 inch). Data collected after the joint was sealed showed the joint appeared as a feature that was spread over a distance of 450 mm (17.7 inches), with a maximum depth of about 1.25 mm (0.05 inch). The actual depth of the joint was 38 mm (1.5 inches) in the unsealed condition, and when the joint was sealed, the distance to the top of the sealant from the pavement surface was specified to be 6 mm (0.25 inch). The distortion of the joint reservoir shape noted in the profile data was due to averaging of the height sensor data and the probable application of the anti-alias filter to the profile data.

- A reduction in IRI from that obtained at 1 day was seen for data collected at 5 days, 10 days, and 1 year after paving. For the 10-day data, the reduction in IRI varied from 15 to 21 percent for the different wheel paths. This reduction is attributed to the high IRI that was obtained when 1-day profiles were obtained, which was caused by humps that occurred at the joints because of residue from the joint sawing operation.

- IRI obtained after the joints were sealed was lower than that obtained when the joints were in an unsealed condition with the joint reservoir sawed. On average, the reduction in IRI achieved after the joints were sealed was 18 percent. A similar observation was made for RN, where an increase in RN of 27 percent occurred because of joint sealing.

- IRI obtained after 1 year of paving was lower than that obtained at 10 days after paving for all wheel paths except for the right wheel path of the outside lane. The reduction in IRI between the 1-year and the 10-day data averaged 15 percent for the three wheel paths where a reduction in IRI occurred. In the right wheel path of the outside lane, the 1-year IRI was 1 percent higher than the 10 day IRI. It is unclear whether this reduction in IRI was due to changes in pavement profile or due to differences in profile data collection capabilities of the lightweight profiler that collected the 10-day data and the high-speed profiler that collected the 1-year data.

- Apart from the RN values that were obtained when the joints were in an unsealed condition, little change in RN occurred for all other profiling times that varied from 1 day after paving to 1 year after paving. The change in RN for these profiling sequences differed from the RN obtained during 1-day profiling by amounts ranging from −4 to 7 percent for the different wheel paths.

- There was negligible curvature in the PCC slabs in the pavement for the different data sets. The PCC slabs can be considered to be flat. No changes in curvature were noted between the different test dates and times.

I-69 PROJECT IN MICHIGAN

Project Description

This project was a reconstruction project located in Calhoun County, MI. This roadway is a four-lane divided highway with two lanes in each direction. A 148-m (485-ft)-long test section was established for testing on the inside lane in the southbound direction. The test section was established between stations 1450+84 and 1445+97. (Stations are in U.S. customary units.)

Pavement Details

Table 64 presents details regarding the pavement. The joint details in this project were similar to those used in the U.S. 23 project (see table 54).

Table 64. Pavement details—I-69.

Item	Description	Value
Pavement thickness	Concrete thickness	280 mm (10.9 inches)
Pavement width	Total pavement width	7.9 m (25.9 ft)
	Width of inside lane	3.6 m (11.8 ft)
	Width of outside lane	4.3 m (14.1 ft)
Shoulder information	Shoulder type	Asphalt
Joint spacing	Joint spacing	4.6 m (15.1 ft)
	Joints skewed?	No
Dowel information	Dowel type	Epoxy coated
	Dowel diameter	32 mm (1.25 inches)
	Dowel length	457 mm (17.8 inches)
Tining information	Tining type	Transverse tining
	Tining spacing	12.5 mm (0.5 inch)
	Tining width	3 mm (0.12 inch)
	Tining depth	3 mm (0.12 inch) to 6 mm (0.25 inch)

Concrete Mix Design

Table 65 presents the mix proportions that were used in the concrete mix. Table 66 presents the gradation of the aggregates used in the concrete mix.

Table 65. Mix proportions—I-69.

Component	Weight (kg/m^3 (lb/yd^3))
Cement	237 (400)
Fly ash	42 (71)
Coarse aggregate	1071 (1,805)
Sand	838 (1,412)

Table 66. Aggregate gradation—I-69.

Sieve	Percentage Passing	
	Coarse Aggregate	Sand
62.5 mm (2.4 inches)	100	100
50 mm (2 inches)	97.7	100
37.5 mm (1.5 inches)	77.5	100
25 mm (1 inch)	62.3	100
19 mm (0.75 inches)	52.6	100
12.5 mm (0.5 inches)	28.3	100
9.5 mm (0.4 inches)	16.4	100
No. 4	2.8	99.4
No. 8	1.3	89.2
No. 16	1	67.8
No. 30	1	39.5

Paving Details

Table 67 presents the date and time the test sections were paved and other details about the paving process. The paving for that day began at 6 a.m. and stopped at noon because rain was predicted later for that day. It started to rain in the afternoon. The test section was observed to have had less than ideal (probably less than specified) tining depth, which was probably the result of the rain that occurred in the afternoon. Figure 121 shows a photograph of the front view of the paver and figure 122 shows a photograph of the finishing process behind the paver.

Table 67. Paving information.

Item	Description	Comment
Date and time	Date of paving	7/8/03
	Time of paving	6 a.m. to noon
Paving process	Haul route	Adjacent to inside shoulder
	Stringline	7.6-m (25-ft) spacing, both sides
	Dowels	Inserted by paver
	Tie bars	Inserted by paver
	Concrete deposit method Spreader used?	Deposited by trucks in front of paver No
Curing method	Curing compound	White curing compound

Figure 121. View from the front of the paver.

Profiling of Section

Four sets of profile data were collected within a 10-day period with a lightweight profiler that is owned by MDOT. This profiler uses software provided by Transology Association. The data recording interval in the profiler is 76 mm (3 inches). The pavement was first profiled 1 day after paving. Initial sawcuts over joints had just been completed before the runs were made. Wet slurry from the saw-cutting operation was present on the pavement adjacent to the joints when the pavement was profiled. The second set of profiles was collected 6 days after paving. The third and fourth sets of data were collected on the 10th day; separate data sets were collected in the morning and in the afternoon. After the morning runs were made, and before performing the afternoon runs, the contractor widened the initial sawcut to create the reservoir for placing the joint sealant. Day-10 afternoon profiles were obtained when the joints were in this condition.

Figure 122. Finishing process behind the paver.

About 4.5 months after paving, this section was profiled again with the high-speed profiler owned by MDOT. Table 68 presents the dates and times when data collection was performed, the approximate age of the pavement at each of these instances, and the low, high, and mean air temperature for each data collection date.

Table 68. Profile data collection—I-69.

Date of Profiling	Approximate Age of Pavement	Time of Profiling		Air Temperature On Day of Profiling ° C (° F)		
		Left Wheel Path	Right Wheel Path	Low	High	Mean
7/9/03	1 day	11:41 a.m.	11:53 a.m.	17 (62)	26 (78)	16 (60)
7/14/03	6 days	11:27 a.m.	11:53 a.m.	12 (53)	28 (82)	21 (69)
7/18/03	10 days	9:18 a.m.	9:34 a.m.	13 (55)	26 (78)	19 (66)
	10 days	3:54 p.m.	3:47 p.m.			
11/26/03	4.5 months	12:10 p.m.	12:10 p.m.	−2 (28)	8 (46)	3 (37)

Roughness Indices

IRI Values

The average IRI values (computed by averaging data obtained from repeat runs) are tabulated in table 69 and shown in figure 123. Table 70 shows the percentage change in IRI values for the different test sequences with respect to the IRI obtained at 1 day.

Table 69. IRI values for different test sequences—I-69

Date of Profiling	Approximate Age of Pavement	Time of Profiling	IRI (m/km)	
			Left Wheel Path	Right Wheel Path
7/9/03	1 day	11:40 to 11:55 a.m.	1.15	1.17
7/14/03	6 days	11:25 to 11:55 a.m.	1.05	1.12
7/18/03	10 days	9:15 to 9:35 a.m.	1.09	1.04
	10 days	3:45 to 3:55 p.m.	1.29	1.48
7/9/03	4.5 months	12:10 p.m.	1.25	1.43

Notes:

1. Initial sawcut 3 mm (0.12 inch) wide present when data were collected at 1-day, 6-day, and 10-day morning.
2. Joint reservoir was formed and joint not sealed when data were collected on the afternoon of day 10.

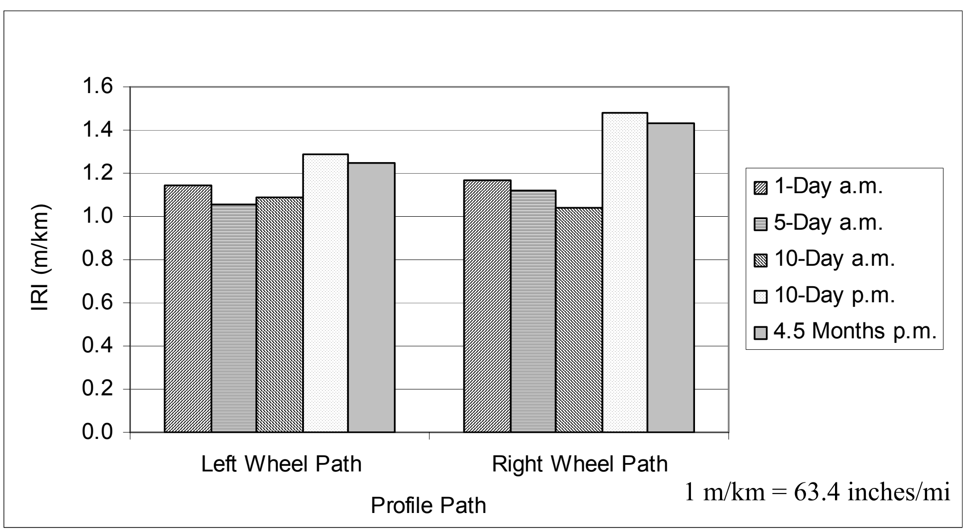


Figure 123. IRI values for different test sequences—I-69.

Table 70. Percentage change in IRI with respect to 1-day IRI—I-69.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Percentage Change in IRI	
			Left Wheel Path	Right Wheel Path
7/14/2003	6 days	11:25 to 11:55 a.m.	−8	−4
7/18/2003	10 days	9:15 to 9:35 a.m.	−5	−11
	10 days	3:45 to 3:55 p.m.	13	27
7/9/2003	4.5 months	12:10 p.m.	9	22

Notes:
1. Initial sawcut 3 mm (0.12 inch) wide present when data were collected at 6-day and 10-day morning.
2. Joint reservoir was formed and joint not sealed when data were collected on the afternoon of day 10.
3. Joints sealed during 4.5-month data collection.

The following observations were noted when evaluating the IRI values:

- The left and right wheel path IRI values were very close to each during 1-day testing, with the values for left and right wheel paths being 1.15 and 1.17 m/km (72.9 and 74.2 inches/mi), respectively. However, during the 4.5-month testing, IRI of the right wheel path was 14 percent higher than the left wheel path, which had an IRI of 1.25 m/km (79 inches/mi).

- The IRI values obtained from the 10-day morning testing (performed before joint reservoirs were sawed) for both the left and the right wheel paths were lower than IRI obtained from 1-day testing by 5 and 11 percent, respectively. It is possible that the sawcut slurry residue present on the pavement when 1-day profiles were obtained may have influenced IRI and caused the 1-day IRI to have an upward bias.

- The IRI values obtained for the day-10 afternoon profiling were higher than those obtained from the morning data collection. The higher values resulted because the joint reservoirs were sawed on the pavement after the morning data collection, the joints were not sealed when the afternoon runs were performed, and the joints appeared in the collected data. The IRI obtained from the afternoon data collection was higher than that obtained from morning values by 19 and 42 percent along the left and right wheel paths, respectively. An evaluation of the profile data indicated that the depth of the joint reservoir was higher along the right wheel path than the left wheel path. This situation caused the magnitude of the IRI increase to be greater along the right wheel path.

- The IRI values obtained from data collected 4.5 months after paving showed higher values than those obtained 1 day after paving. IRI from 4.5-month testing was higher than 1-day IRI values by 9 percent and 22 percent along the left and right wheel paths, respectively. The 1-day profiles were obtained with a lightweight profiler, whereas the 4.5-month profiles were obtained with a high-speed profiler. It is unclear whether the differences in data collection capabilities of the two devices contributed to the difference in IRI. When the 4.5-month data collection was performed, the northbound lanes were under construction, and the northbound traffic was using the inside lane of the southbound lanes. Hence, this section was profiled while traveling in the northbound direction. There was a median barrier between the two southbound lanes, and the profiled path that was followed during the 4.5-month profiling may have been different from the path that was followed during the 1-day profiling. This could be another factor that contributed to differences in IRI. Hence, it is unclear whether the increase in IRI observed at 4.5 months was occurring because of changes in pavement profile or whether equipment differences and variations in the wheel paths contributed to this difference.

RN Values

The average RN values (computed by averaging values of the three repeat runs) are presented in table 71 and shown in figure 124. Table 72 shows the percentage change in RN values for the different test sequences with respect to the RN obtained at 1 day.

Table 71. RN values for different test sequences.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Ride Number	
			Left Wheel Path	Right Wheel Path
7/9/03	1 day	11:40 to 11:55 a.m.	3.66	3.66
7/14/03	6 days	11:25 to 11:55 a.m.	3.73	3.74
7/18/03	10 days	9:15 to 9:35 a.m.	3.81	3.91
	10 days	3:45 to 3:55 p.m.	2.87	2.58
11/26/03	4.5 months	12:10 p.m.	3.50	3.45

Notes:

1. Initial sawcut 3 mm (0.12 inch) wide was present when data were collected at 1-day, 6-day, and 10-day morning.
2. Joint reservoir formed when data were collected on afternoon of day 10.
3. Joints sealed during 4.5-month data collection.

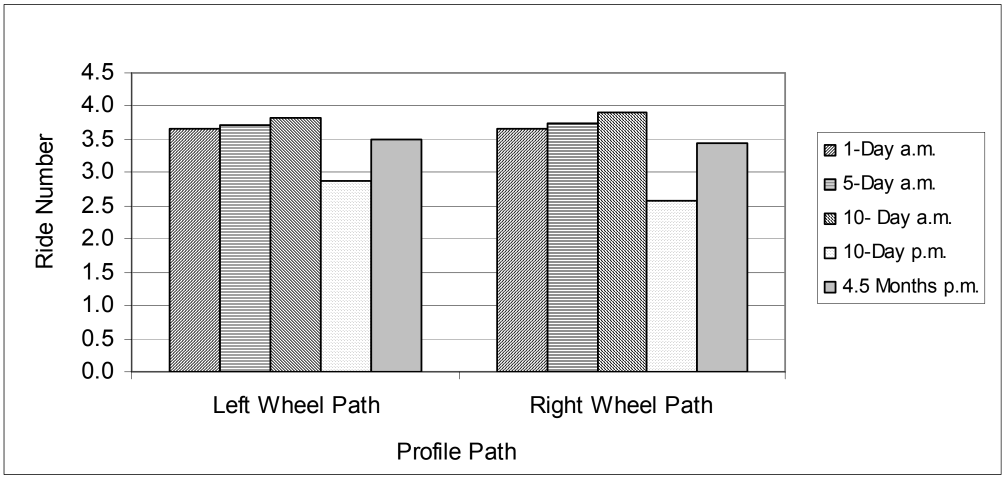


Figure 124. RN values for different test sequences—I-69.

Table 72. Percentage change in RN with respect to 1-day RN—I-69.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Percentage Change in RN	
			Left Wheel Path	Right Wheel Path
7/14/2003	6 days	11:25 to 11:55 a.m.	2	2
7/18/2003	10 days	9:15 to 9:35 a.m.	4	7
	10 days	3:45 to 3:55 p.m.	−21	−30
11/26/2003	4.5 months	12:10 p.m.	−4	−6

Notes:

1. Initial sawcut 3 mm (0.12 inch) wide present when data were collected at 1-day, 6-day, and 10-day morning.
2. Joint reservoir formed when data were collected on the afternoon of day 10.
3. Joints sealed during 4.5-month data collection.

The following observations were noted when evaluating the RN values:

- The left and right wheel path RN values were identical at 1-day testing; RN was 3.66 for both. At 4.5 months, the RN values of the wheel paths were very close to each other; the RN of right wheel path was less than that of the left wheel path by 0.05.

- The RN values obtained along the left and right wheel paths from the morning testing performed on day-10 were higher than those obtained from day-1 testing by 4 and 7 percent, respectively. As described previously for IRI, the lower RN at day-1 testing may have been caused by the presence of slurry from the saw-cutting operation that subsequently was washed away by rain and resulted in a higher RN for the 10-day morning profiles.

- The RN values obtained during the afternoon profiling of day 10 (when the joint reservoirs had been sawed, but the joint was not sealed) were much lower than the RN values obtained during the morning profiling when only the 3-mm (0.12-inch)-wide initial sawcut was present on the pavement. For the left and the right wheel paths, the RN from the afternoon runs were lower than the morning runs by 25 and 34 percent, respectively. As described previously, the depth of the reservoir along the right wheel path was higher than that along the left wheel path, and this difference resulted in a much higher decrease in RN along the right wheel path.

- RN for the left and right wheel paths at 4.5 months were lower than the RN obtained from 1-day testing by 4 percent and 6 percent, respectively.

Repeatability of IRI Values

When the IRI values obtained for the entire section from all profile sequences for the lightweight profiler were compared, the difference between the maximum and minimum IRI for the three runs ranged between 0.01 to 0.10 m/km (6.3 inches/mi), with the average difference of 0.04 m/km (2.5 inches/mi).

An evaluation of the short-interval IRI repeatability of the lightweight profiler was performed by computing IRI values at 15-m (49-ft) intervals for day-9 morning right wheel path data. For the 148-m (485-ft)-long test section, there are nine 15-m (49-ft)-long segments. Figure 125 shows the IRI values that were obtained at 15-m (49-ft) intervals for each run for each segment. Overall, the repeatability appeared to be satisfactory for most segments, but a few segments had significant differences in IRI values between runs. The difference between the maximum and minimum IRI for each segment obtained form the three repeat runs ranged from a low of 0.06 m/km (3.8 inches/mi) at segment 3, to a high of 0.24 m/km (15.2 inches/mi) at segment 1. The average of the difference between maximum and minimum IRI from the three runs for the segments was 0.11 m/km (7 inches/mi). The IRI values obtained from the three runs for the entire section were 1.04, 1.03, and 1.06 m/km (66, 65, and 67 inches/mi). Although there were differences in individual 15-m (49-ft) segments between runs, when the overall IRI of the entire section from the three runs were compared, the three values were almost identical. As described previously for the other projects, this closeness occurs because of compensating effects of IRI over the section.

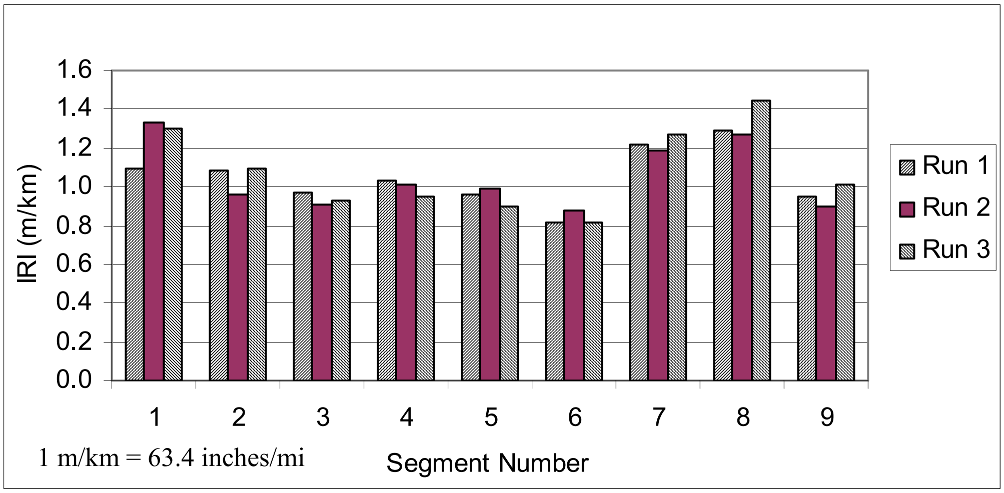


Figure 125. Repeatability of IRI values—I-69.

Effect of Joint Condition on Profile

The method used to form joints in this section was similar to that used for the test section on U.S. 23. Profile data collection for this section was performed using the same profiler that collected data at the U.S. 23 test section. The effect of joints seen in the profile data at this test section was similar to the observations noted at the test section located on U.S. 23.

Roughness Profiles

Figures 126–128 show the 6-m (20-ft) base length roughness profile for the section. The vertical lines in the plots correspond to joint locations. These roughness profiles are for data that were collected at 9 days after paving in the morning. The IRI values of the entire section for the two wheel paths for that test date were extremely close to each other: IRI of the left and the right wheel paths were 1.09 and 1.04 m/km (67 and 66 inches/mi), respectively. Although the overall roughness values for the two wheel paths are close to each other, the roughness profiles shown in figures 126–128 show that the variability of roughness within the section is greater for the left wheel path. Along the left wheel path, the roughness profile exceeded 1.5 m/km (95 inches/mi) at five locations compared to one location for the right wheel path.

Evaluation of Profile Data

Five profile data sets were obtained at this test section. The first four data sets were obtained with a lightweight profiler; the last data set was obtained with a high-speed profiler. There were significant differences in the spatial distribution of roughness within the section obtained by the two profilers. Hence, it is unclear whether the differences in IRI noted between the 4.5-month profiling and 1-day profiling were caused by a difference in the pavement profile, or whether they were related to differences between the two profilers' data collection capabilities. As was indicated previously, when profile data collection was performed with the high-speed profiler, a median barrier was present between the lanes. Hence, the high-speed profiler may not have been able to follow the exact path that was profiled by the lightweight profiler. This is another factor that could have contributed to differences in IRI obtained by the high-speed profiler and the lightweight profiler.

An evaluation of the profile data did not indicate a distinct profile feature or a waveband that had a dominant contribution to IRI.

CI was computed for one profile run from each data set, and the computed values are presented in table 73. All CI values are small, which indicates that the PCC slabs are essentially in a flat condition with negligible curvature. No noticeable changes in curvature were noted between the different data sets.

1 m = 3.28 ft
1 m/km = 63.4 inches/mi

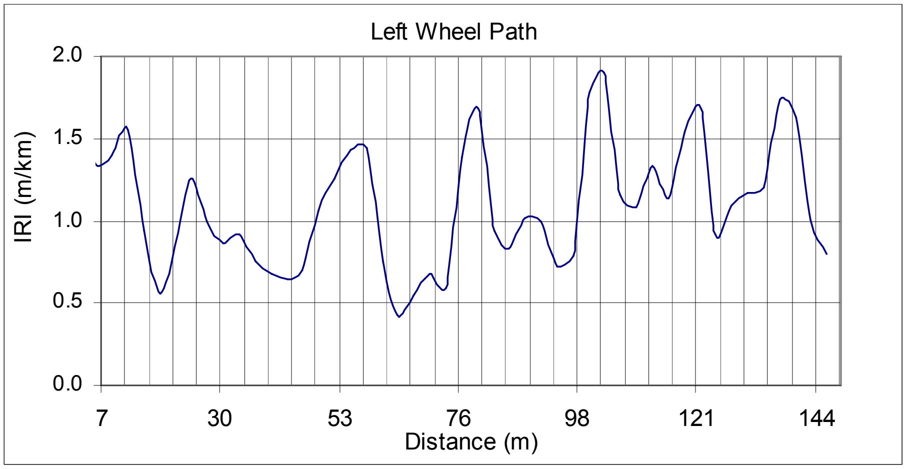


Figure 126. Roughness profiles for I-69, left wheel path.

1 m = 3.28 ft
1 m/km = 63.4 inches/mi

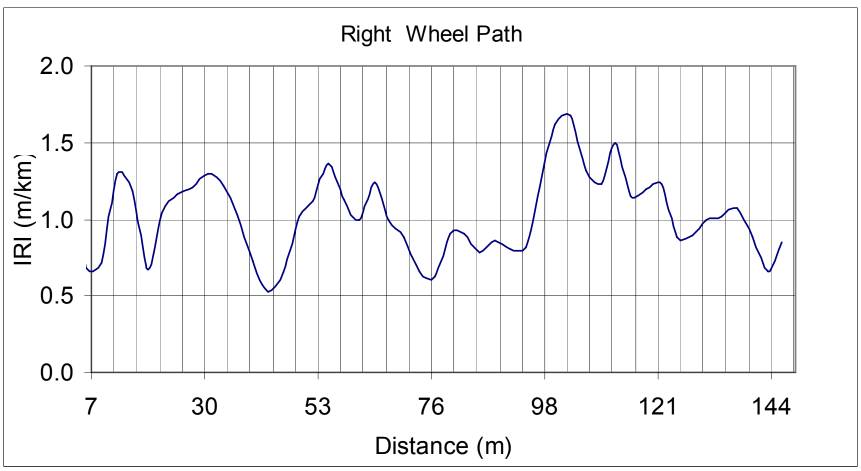


Figure 127. Roughness profiles for I-69, right wheel path.

1 m = 3.28 ft
1 m/km = 63.4 inches/mi

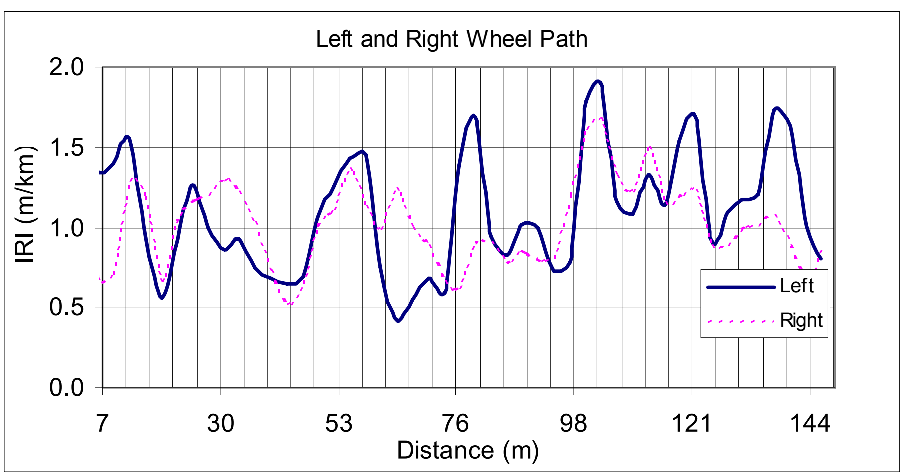


Figure 128. Roughness profiles for I-69, left and right wheel path.

Table 73. CI values—I-69.

Date of Profiling	Approximate Age of Pavement	Time of Profiling	Curvature Index x 1,000 (1/m)	
			Left Wheel Path	Right Wheel Path
7/9/03	1 day	11:40 to 11:55 a.m.	0.049	0.184
7/14/03	6 days	11:25 to 11:55 a.m.	0.187	0.194
7/18/03	10 days	9:15 to 9:35 a.m.	0.082	0.062
	10 days	3:45 to 3:55 p.m.	0.095	0.125
7/9/03	4.5 months	12:10 p.m.	0.000	0.127

Coefficient of Thermal Expansion and Microscopical Examination

A CTE test was conducted on a 150-by-300-mm (5.9-by-11.8-inch) core from the pavement. The CTE value for the specimen was 8.25 x 10^{-6} per °C (4.58 x 10^{-6} per °F). A microscopical examination of the core indicated that the coarse aggregate was dolomite/dolomitic limestone (carbonate) and the fine aggregate was silicious.

Summary

- Testing performed immediately after paving indicated that the left and right wheel path IRI were very close to each other; the values from day-1 testing were 1.15 and 1.17 m/km (73 and 74 inches/mi), respectively.

- When IRI obtained from the three repeat runs for the entire section were evaluated for all data sets collected by the lightweight profiler, the average of the difference between the maximum and minimum IRI was 0.04 m/km (2.5 inches/mi).

- An evaluation of short-interval IRI repeatability of the lightweight profiler using 15-m (49-ft) segment lengths indicated that the average difference between the maximum and minimum IRI obtained from the repeat runs was 0.11 m/km (7 inches/mi).

- The method used to form joints in this project was similar to that used at the test section on U.S. 23. The lightweight profiler used to collect profile data at this test section was the same device that collected data at the test section established on U.S. 23. The observations regarding the condition of the joint on profile data and roughness indices noted at this section were similar to those at the test section on U.S. 23.

- A reduction in IRI from that obtained at 1 day was seen for data collected on day-6 and day-10 morning (joint reservoir was not sawed at this time). The reduction in IRI ranged from 4 to 11 percent for the different wheel paths and test dates. The joints in the pavement had been sawn just before profile data collection on the first day the pavement was profiled, and residue from the joint sawing operation was present adjacent to the joints at the time of profiling. Hence, the higher IRI obtained for day 1 may have been due to this residue.

- IRI obtained after the joint reservoirs were formed—but the joints not sealed—was much higher than that obtained from data collected when only the initial sawcut was present on the pavement. The IRI values obtained when the joint reservoir was sawed but joint not sealed were higher than the IRI obtained before forming the reservoir by 18 and 42 percent for the left and the right wheel paths, respectively. The joint reservoir was noted to have had a higher depth along the right wheel path than the left, which was the cause for the higher increase in IRI along the right wheel path. Profile data were not collected immediately after the joints were sealed; hence, a comparison of IRI values obtained after the joints were sealed could not be performed.

- IRI obtained 4.5 months after paving was higher than the 1-day IRI by 9 percent and 22 percent along the left and the right wheel paths, respectively. The 1-day profiling was performed with a lightweight profiler, whereas the 4.5-month profiling was performed with a high-speed profiler. It is unclear whether the difference in IRI was caused by a change in the pavement profile or whether it was related to differences between the two profilers' data collection capabilities. Also, a median barrier was present when the 4.5-month data were collected, and the path followed by the high-speed profiler during profiling may have been different from that followed by the lightweight profiler. This situation might also be a contributing factor to differences in IRI.

- Apart from the RN values obtained when the joints were in an unsealed condition with the joint reservoirs sawed, little change in RN occurred for the other profiling times that varied from 1 day after paving to 4.5 months after paving. When all profiling paths and test sequences were considered, the changes in RN that occurred compared to the 1-day RN ranged from −6 to 7 percent.

- There was negligible curvature in the PCC slabs. No noticeable changes in curvature were noted over the monitored period.

CHAPTER 7. CONCLUSIONS

PERFORMANCE OF DOWELED AND NONDOWELED PAVEMENTS

Roughness Progression

Analysis of LTPP data indicated major differences in roughness progression between doweled and nondoweled pavements. The doweled and nondoweled pavements were divided into three data sets based on the rate of increase of roughness:

- Data set 1—less than 0.02 m/km/yr (1.27 inches/mi/yr).
- Data set 2—between 0.02 and 0.04 m/km/yr (1.27 to 2.54 inches/mi/yr).
- Data set 3—greater than 0.04 m/km/yr (2.54 inches/mi/yr).

The percentage of nondoweled pavements that fell into data sets 1, 2, and 3 was equal (each set consisted of 33 percent of nondoweled pavements). The percentage of doweled sections that fell into data sets 1, 2, and 3 was 56, 25, and 19 percent, respectively. These results clearly show that doweled pavements are providing a superior performance from a roughness point of view.

The roughness progression plots for PCC pavements showed a parallel pattern. This finding indicates that pavements that are built smoother will provide a longer service life before reaching a terminal roughness value, compared to pavements having a lower initial smoothness level.

Faulting

There was a significant difference in faulting between doweled and nondoweled pavements. Ninety-four percent of the doweled pavements had a total faulting within a 152.4-m (500-ft)-long section that was less than 38 mm (1.5 inches), compared to 39 percent for the nondoweled pavements. A total faulting of 38 mm (1.5 inches) corresponds to an average faulting of less than 1.2 mm (0.05 inch) at a joint. The average total faulting for data sets 1, 2, and 3 for doweled pavements were 15, 13, and 38 mm (0.59, 0.51, and 1.5 inches), respectively, whereas the values for nondoweled pavements were 32, 47, and 119 mm (1.26, 1.85, and 4.69 inches), respectively. A strong relationship existed between the roughness level and total faulting for nondoweled pavements (correlation coefficient of 0.70), whereas a similar relationship was not seen for doweled pavements (correlation coefficient of 0.07). The dowels in the doweled pavements are serving their intended function by providing load transfer and thus preventing faulting.

Slab Curvature

Nondoweled Pavements

The majority of nondoweled pavements had an upward curvature; 73 percent of the sections showed an upward curvature at the last profile date. Generally, the pavements that fell into data set 1 have not shown a major change in curvature over the monitored period, which on average was about 10 years. However, pavements that fell into data sets 2 and 3 have shown an increase in curvature over time. Sections in data set 3 showed significant upward curvature, and the amount of curvature was increasing over time. The pavements with a high degree of curvature had high IRI values, showed a high rate of increase of roughness, and had high faulting. The pavements that had very little curvature initially and whose curvature did not change over time showed very good performance from a roughness point of view. Increasing values of the following factors were associated with higher curvature: CTE, freezing index, and elastic modulus of concrete, while increasing values of the following factors were associated with lower curvature: mean annual temperature, annual precipitation, number of wet days per year, and PCC slab thickness.

Doweled Pavements

Overall, the magnitude of slab curvature seen in doweled pavements was much less than that seen for nondoweled pavements. The majority of the sections had downward curvature; 63 percent of the sections showed a downward curvature at the last profile date. Generally, the pavements that fell into data set 1 had very little slab curvature, and they were essentially flat, and showed little change in curvature over time. However, the range of curvature seen for pavements in data set 2 increased over time. The slabs in all pavements that fell into data set 3 showed downward curvature. Sections that showed excessive amounts of downward curvature had high roughness values and high rates of increase of roughness. For doweled pavements, higher values of the following factors resulted in more downward curvature: freezing index, number of days in a year below 0 °C (32 °F), ratio between PCC elastic modulus and split tensile strength, and weight of cement in mix. Higher values of the following parameters resulted in lower downward curvature: mean annual temperature, annual precipitation, days in a year with temperature greater than 32 °C (90 °F), and PCC slab thickness.

FACTORS AFFECTING ROUGHNESS PROGRESSION

Nondoweled Pavements

The strongest factor affecting roughness progression was faulting. Many sections that had high faulting also had high CI values. It appears that excessive PCC slab curvature increases the potential for faulting. The worst performing nondoweled pavements from a roughness point of view were mostly located either in areas where the mean annual temperature was low or the freezing index was high. (In freezing regions, the freezing index is correlated to mean annual temperature.) In nondoweled pavements, load transfer is provided by aggregate interlock. When the temperatures are low, the amount of load transfer between the slabs will be low. This situation increases the potential for faulting.

Generally, pavements having higher split tensile strength values appear to be performing better from a roughness point of view. Higher split tensile values indicate higher flexural strengths. Generally, pavements having high elastic modulus values (greater than 35,000 MPa (5.08 million psi)), or pavements having a high value for the ratio between elastic modulus of concrete and split tensile strength (greater than 8,000) appear to be showing high rates of increase of roughness. Several pavement sections showed a high rate of increase of IRI, where a primary factor causing the increase in IRI was the increase in upward slab curvature over time.

Doweled Pavements

No factor that had a strong relationship to roughness progression could be identified. There were some weak trends in the data that slab length, split tensile strength, and coarse-to-fine aggregate ratio may be affecting roughness progression. The median slab length of the set of pavements that showed a rate of change of IRI of less than 0.02 m/km/yr (1.27 inches/mi/yr) and greater than 0.02 m/km/yr (1.27 inches/mi/year) were 4.8 and 5.3 m (15.7 and 17.4 ft), respectively. Overall, a higher split tensile strength, which is related to higher flexural strength of concrete, appears to be beneficial to providing a good performance from a roughness viewpoint. It appears that higher values of coarse-to-fine aggregate ratio in the concrete provided pavements that maintained their smoothness over a long period. The trends seen for nondoweled pavements for elastic modulus, or the ratio between elastic modulus and tensile strength, were not seen for the doweled pavements. Several doweled sections had high elastic modulus values or high values for the ratio between elastic modulus and split tensile strength that did not exhibit poor performance.

EFFECT OF JOINTS ON PROFILE AND SMOOTHNESS INDICES

In the two Iowa projects, the joints were formed by performing a sawcut that had a width of 6 mm (0.25 inch) and a depth of 25 mm (1 inch), and then the joints were sealed. Profile data were collected on these projects when the joint was in an unsealed as well as in the sealed condition. For both cases, the lightweight profiler recorded the joint as a feature spread over a distance of 300 mm (11.8 inches). The recorded depths of the joint in the unsealed and sealed condition were 2 mm (0.08 inch) and 1 mm (0.04 inch), respectively.

In the test section located in Pennsylvania, the joints were formed by performing an initial sawcut that was 3 mm (0.12 inch) wide, then a joint reservoir having a width of 9.5 mm (0.37 inch) and a depth of 38 mm (1.50 inches) was sawed, and then the joint was sealed. Profile data were collected when the joints were in these three conditions. For all three cases, the lightweight profiler recorded the joint as a feature spread over a distance of 220 mm (8.7 inches). The recorded depth of the joint when the initial sawcut was present, when the reservoir was sawed with the joint unsealed, and when the joint was sealed were 1.5, 3.5, and 1.5 mm (0.06, 0.14, and 0.06 inch), respectively.

In the test sections located in Michigan, the joint forming procedure was similar to that used in the Pennsylvania project. The joint dimensions were also similar to those in the Pennsylvania project. The joint could not be detected in the data collected when the 3-mm (0.12-inch)-wide initial sawcut was present on the pavement. For the other two cases, the profiler recorded the joint as a feature spread over a distance of 450 mm (17.7 inches). The recorded depth of the joint when the reservoir was sawed with the joint unsealed, and when the joint was sealed, was 4 and 1.4 mm (0.16 and 0.05 inch), respectively.

The lightweight profilers used to measure profiles on the five paving projects had data recording intervals ranging from 30 to 76 mm (1.2 to 3.0 inches). However, these profilers obtain height sensor data at much closer intervals, and average the values when computing the profile at each data recording interval. In addition, the profilers appear to be applying a low-pass anti-alias filter on the data. The averaging procedure used on the height sensor data and the anti-alias filter will flatten out the depth of the joint that appears in the profile and cause the joint to be spread over a much wider distance than the actual width. There are differences in these procedures between profilers manufactured by different manufacturers. Hence, the differences observed in these projects on how a joint was recorded in the profile data are attributed to different procedures used by the different profilers in manipulating the data before recording. Depending on the procedure used to manipulate the data, the condition of the joint during data collection may or may not have an influence on the smoothness index computed from that data.

The profilers used in the Iowa and Pennsylvania projects significantly attenuated the depth that was recorded at a joint when the joint was unsealed. In these projects, the IRI after the joint was sealed was almost identical to the IRI obtained when the joint was unsealed. This is because the depth of the joint that was recorded when the joint was unsealed was not large enough to affect the IRI. For the projects in Michigan, IRI obtained when the joint reservoir was formed, but with the joint unsealed, was 18 percent lower that that obtained when the reservoir was sealed. In the Michigan projects, the depth of the joint recorded by the profilers when the joint was unsealed was large enough to affect the IRI.

For the Iowa project, the RN values obtained after the joints were sealed were about 2 to 3 percent higher than those obtained before sealing the joints. For the Michigan projects and the Pennsylvania project, RN obtained from data collected when the joint reservoir was formed but the joint was unsealed was on average 14 and 28 percent lower, respectively, than that obtained after the joint was sealed.

These results indicate that profiling should not be performed when the joint reservoir is made on the pavement with the joint unsealed if RN is to be computed from the data. It is best to follow a similar approach when collecting data to compute IRI. For some profilers, this may not be an issue, but it could be for some profilers. There was no evidence to suggest IRI or RN computed from data collected under the following two conditions would be different: (1) 3-mm (0.12-inch)-wide initial sawcut was present on the pavement and (2) joint reservoir was sawed and sealed.

SHORT-TERM CHANGES IN PROFILE AND SMOOTHNESS

Slab Curvature

For the project in Pennsylvania, the two projects in Michigan, and the I-80 project in Iowa, negligible PCC slab curvature was observed for all data collection sequences. The slabs were essentially flat. In all of these projects, virtually no changes in slab curvature were observed between the different profiling sequences.

The U.S. 20 project had the highest CI at the first profile date of all projects. Some changes in CI between the profiling sequences were noted at this section. An evaluation of the profile data indicated that CI was being influenced by variations in the profile within the slab, and not by changes that occurred because of movements at the joints. The right wheel path in this section had a repetitive wave having a wavelength of approximately 1.6 m (5.2 ft). The amplitude of this wavelength in the profile appeared to be different during the different profiling sequences, and this phenomenon was having an influence on the CI.

Changes in IRI

For the Pennsylvania project, little change in IRI was noted for the different test sequences. IRI can be considered to have remained at the same value over the 3.5-month monitored period.

At the U.S. 20 project in Iowa, little change in IRI was noted for the different test sequences along the left wheel path over the 3-month monitoring period. Along this wheel path, changes in IRI compared to IRI obtained immediately after paving ranged from −1 to 5 percent for the different profiling sequences. Along the right wheel path, little change in IRI was noted for data obtained up to 9 days (change of −1 to −3 percent). However, the data collected at 3 months showed a reduction in IRI of 10 percent compared to IRI obtained 1 day after paving. The cause for this reduction could not be determined from the profile data.

In the I-80 project in Iowa, two 152.4-m (500-ft)-long sections were considered. The lowest changes in IRI were noted along the right wheel path of the first 152.4-m (500-ft)-long section, whereas the changes in IRI with respect to IRI immediately after paving for the different profiling sequences ranged from −2 to 6 percent. The highest changes in IRI were noted along the left wheel path of the second 152.4-m (500-ft)-long section, where changes in IRI with respect to IRI immediately after paving ranged from −1 to 14 percent.

For the U.S. 23 project in Michigan, profile data collected 1 day after paving showed humps appearing at joint locations. These humps appear to have been caused by the joint saw residue and caused IRI to increase. A reduction in IRI from that obtained immediately after paving was seen for data collected at 5 days, 10 days, and 1 year after paving. For the 10-day data, the reduction in IRI compared to IRI obtained immediately after paving varied from 15 to 21 percent for the different wheel paths. This reduction is attributed to the residue from the joint sawing operation being washed away by rain. IRI obtained 1 year after paving was lower than that obtained at 10 days after paving for all wheel paths, except for the right wheel path of the outside lane. The reduction in IRI between 1-year and 10-day data averaged 15 percent for the three wheel paths where a reduction in IRI occurred, while for the other wheel path IRI increased by 1 percent. It is unclear whether this reduction in IRI was due to changes in pavement shape or due to differences in profile data collection capabilities between the lightweight profiler used to obtain the 10-day data and the high-speed profiler used to obtain the 1-year data.

For the I-69 project in Michigan, a reduction in IRI from that obtained at 1 day after paving was seen for data collected at 6 days and 10 days after paving. The reduction in IRI ranged from 4 to 11 percent for the different wheel paths and test dates. The joints in the pavement had been sawn just before profile data collection on the first day the pavement was profiled, and residue from the joint sawing operation was present adjacent to the joints at the time of profiling. The higher IRI obtained for day 1 may have been caused by this residue. IRI obtained 4.5 months after paving was higher than that obtained immediately after paving by 9 and 22 percent for the left and the right wheel paths, respectively. The 1-day profiling was performed with a lightweight profiler, while the 4.5-month profiling was performed with a high-speed profiler. It is unclear whether the difference in IRI was caused by a change in the pavement profile or is related to differences in the two profilers' data collection capabilities. Also, a median barrier was present when the 4.5-month data were collected, and the path followed by the high-speed profiler during profiling may have been different from that followed by the lightweight profiler. The fact that different paths were followed might also have been a contributing factor to differences in IRI.

The CTE values of the concrete used in these projects ranged from 8.25 x 10^{-6} to 13.2 x 10^{-6} per °C (4.58 x 10^{-6} to 7.33 x 10^{-6} per °F). No effect of CTE on short-term changes in IRI could be detected.

Changes in RN

The effect of joint condition (sawed, not sealed versus sealed) was discussed previously. When addressing changes in RN for the different projects, RN obtained when the joint reservoir is sawed but unsealed will not be considered.

In the Pennsylvania project, little change in RN was noted for the different data sets. The changes in RN of the different wheel paths over the 3.5-month monitoring period, compared to RN obtained 1 day after paving, ranged from −6 to 3 percent.

For the U.S. 20 project in Iowa, a slight increase in RN was noted over the 3-month monitoring period when RN obtained at different test sequences was compared with the 1-day values. However, the increase in RN was less than 5 percent.

For the I-80 project in Iowa, the RN values for both wheel paths of the two 152.4-m (500-ft)-long sections showed little change for the different profiling sequences over a 1-month period. The change in RN compared to RN obtained immediately after paving was within ±3 percent.

For the U.S. 23 project in Michigan, the RN obtained at profiling times that varied from 5 days after paving to 1 year after paving showed little change when compared to RN obtained immediately after paving. The variations in RN that occurred, compared to the 1-day RN, ranged from −4 to 7 percent with no consistent trend occurring over time.

For the I-69 project in Michigan, little change in RN occurred for the different profiling sequences that varied from 1 day after paving to 4.5-months after paving. When all profiling paths and test sequences were considered, the variations in RN that occurred, compared to the 1-day RN, ranged from −6 to 7 percent.

MEASUREMENT OF SMOOTHNESS FOR CONSTRUCTION ACCEPTANCE

Surface Condition

The surface of the pavement must be clean when performing profile measurements. Residue from the sawcutting operation present adjacent to the transverse joint can appear as small humps in the measured profile and can affect the smoothness indices computed from the profile data. On one paving project where such residue was observed, an increase in IRI ranging from 17 to 25 percent for the four evaluated wheel paths could be attributed to the humps created by the residue.

Repeatability of IRI

For each data set, the lightweight profilers obtained three repeat runs. When IRI obtained from the three repeat runs for the entire section were evaluated for all data sets, the average difference between the maximum and minimum IRI for the S.R. 6220 project in Pennsylvania, U.S. 23 project in Michigan, and I-69 project in Michigan were 0.03, 0.06, and 0.04 m/km (1.9, 3.8, and 2.5 inches/mi), respectively. These three projects had transverse tining. When a similar analysis was performed for the two projects in Iowa that had longitudinal tining, the average difference between the maximum and minimum IRI was 0.06 and 0.09 m/km (3.8 and 5.7 inches/mi) for the U.S. 20 and I-80 project, respectively. The lateral wander during profiling would cause the laser dot of the height sensor to obtain measurements on top of the tine as well as the bottom of the tine. This is attributed as the cause for the lower repeatability of IRI observed on the longitudinally tined pavements.

Repeatability of Short-Interval IRI

An evaluation of short-interval IRI repeatability using 15-m (49-ft) segment lengths showed the average difference between the maximum and minimum IRI obtained from the repeat runs for the 15-m (49-ft)-long segments in the S.R. 6220 project in Pennsylvania, U.S. 23 project in Michigan, and I-69 project in Michigan were 0.09, 0.10, and 0.11 m/km (5.7, 6.3, and 7.0 inches/mi), respectively. A similar analysis for the two projects in Iowa indicated values of 0.25 and 0.12 m/km (15.9 and 7.6 inches/mi) for the U.S. 20 and I-80 projects, respectively. The short-interval IRI repeatability for the U.S. 20 project was very poor. These results indicate that implementing an IRI-based specification that relies on short-interval IRI (e.g., 15 m (49 ft)) is not suitable for pavements that have longitudinal tining. There were several cases where significant differences in IRI between the runs were noted for individual 15-m (49-ft)-long segments in all projects. Although there can be differences in IRI between 15-m (49-ft)-long segments, when IRI is computed over longer distances such as 152.4 m (500 ft), these differences can cancel out, and IRI from repeat runs can give excellent agreement.

Certification of Profilers

If a profiler is used to measure smoothness of concrete pavements, it is advisable to certify the profiler on PCC sections. There could be differences in the way profilers treat joints, which can affect smoothness indices obtained from the profile data. In addition, treatment of tining may be different between devices. Hence, certifying profilers on asphalt surfaces and using the profiler to measure smoothness on concrete surfaces may not necessarily mean comparable smoothness indices will be obtained between devices.

Time for Profiling

Based on the five projects used in this study, it appears that smoothness measurements can be performed at any time within the first few months after paving.

USE OF PROFILE DATA FOR CONSTRUCTING SMOOTH PAVEMENTS

Usually smoothness indices like IRI are computed for each lane over a 161-m (528-ft) length for construction acceptance. The IRI for the overall section does not provide any information about how IRI varies within the section, or where rough spots within the section are located. A roughness profile of the section can be used to investigate how roughness varies within the section and identify where rough spots within the section are located. If rough spots are detected within the section, the location of these events could be correlated to the construction process or a pavement feature to obtain information about a specific construction event or a pavement feature at that location that resulted in a high roughness value. Overlaid roughness profiles of the left and right wheel paths can be used to see how roughness varies between the wheel paths. In addition, overlaid roughness profiles of both wheel paths of the inside and outside lane can be used to see how roughness varies across the entire pavement width. These procedures help detect problems in the construction process that results in rough spots.

PSD plots can be used to identify whether a roughness associated with a specific wavelength is predominant in the pavement. A wavelength that has a significant contribution to the roughness will appear as a spike in the PSD plot. If spikes are detected, the cause for the prominent wavelength to occur in the profile can be investigated, whether it stems from the equipment used for paving or the finishing process. For example, data from the U.S. 20 project showed a repetitive wave with a wavelength of 1.6 m (5.2 ft) appearing along the right wheel path. Any repetitive feature occurring in a profile can be easily detected by this technique.

Analyzing roughness profiles, as well as using PSD plots to evaluate data collected at the start of a paving project, will indicate whether any features in the profile are contributing to a high roughness. If such features are detected, these methods will provide an opportunity to troubleshoot, identify the problem, and then correct it at the start of the project.

CONSTRUCTION CONSIDERATIONS

Overall, when all tested wheel paths and lanes were considered, the average IRI values for the S.R. 6220 project in Pennsylvania, U.S. 20 project in Iowa, I-80 project in Iowa, U.S. 23 project in Michigan, and I-69 project on Michigan were 1.11, 1.44, 0.95, 0.80, and 1.07 m/km (70, 91, 60, 51, and 68 inches/mi), respectively.

The best smoothness was obtained at the U.S. 23 project. This is the only project where tie bars were not inserted by the paver, but fixed to the base on chairs. Fixing the tie bars to the base will be more costly than inserting them during paving. It appears that the contractor in this project believed that better smoothness could be achieved by using this procedure. The contractor did indeed construct a very smooth pavement in this project.

The only project where a spreader was not used, and also where dowel bars were inserted, was the I-69 project. Out of the five projects studied, this project had the third best smoothness.

EFFECT OF MIX DESIGNS ON SMOOTHNESS

A survey of State DOT personnel and concrete industry personnel was performed to get their opinion on whether contractors have been adjusting their mix designs to achieve higher smoothness. The general consensus was that no modifications have been required in the concrete mix design to achieve higher smoothness. It was indicated that when the mix design for a project is approved by the DOT personnel, it usually cannot be changed without the approval from the DOT engineers. It was also indicated that no pavement performance problems have been encountered because of the implementation of a smoothness specification. The general consensus was that the smoothness is mostly affected by the paving equipment and the construction process.

CHAPTER 8. RECOMMENDATIONS AND GUIDELINES

This chapter presents recommendations and guidelines for design features, PCC material properties, and construction procedures for constructing pavements having high initial smoothness while ensuring good long-term performance. Recommendations and guidelines for smoothness testing and use of profile data to troubleshoot problems are also presented in this chapter.

VALUE OF BUILDING SMOOTH PAVEMENTS

The roughness progression plots for JPC pavements in the LTPP program showed a parallel pattern in roughness progression. This finding indicates that pavements that are built smoother will provide a longer service life. These pavements will also provide road users with a better ride quality.

DESIGN FEATURES

- Construction of nondoweled pavements in freezing regions is not recommended. Nondoweled pavements in freezing regions have shown poor performance from a roughness point of view.

- Some nondoweled pavements have shown an excessive amount of upward slab curvature. The upward slab curvature in these pavements has been increasing over time. These pavements have a high amount of faulting and are thus showing poor performance from a roughness point of view. In addition, many of these pavements are showing other distress. Factors associated with higher amounts of slab curvature over time in nondoweled pavements are high values of freezing index, CTE of PCC, and PCC elastic modulus. Higher values of the following factors were associated with lower curvature: mean annual temperature, annual precipitation, number of wet days per year, and slab thickness. To prevent an increase in upward slab curvature over time, it is recommended that dowels are used for all pavements constructed in freezing areas, as well as for projects that utilize PCC having a high CTE or a high modulus (greater than 35,000 MPa (5.08 million psi)).

- The provision of dowels in pavements has served its intended function by preventing faulting. If there is any reason to believe there is even the slightest possibility for faulting to occur, dowels are recommended. Providing dowels will ensure a long lasting smooth pavement.

- Doweled pavements with a joint spacing of 4.8 m (15.7 ft) or less seem to perform better than those having a higher joint spacing. It is recommended that SHAs utilizing a joint spacing much greater than 4.8 m (15.7 ft) investigate whether using 4.8 m (15.7 ft) or lesser spaced joints will give better performance.

PCC MATERIAL PROPERTIES

- For both doweled and nondoweled pavements, using PCC with higher split tensile strength (which results in a higher flexural strength) appears to be beneficial for long-term performance from a roughness point of view.

- In nondoweled pavements, PCC having high elastic modulus values (greater than 35,000 MPa (5.08 million psi)) or PCC having a high ratio (greater than 8,000) between elastic modulus of concrete and split tensile strength appear to be showing high rates of increase of roughness. These trends were not seen for doweled pavements.

- Evidence suggests that higher values of coarse-to-fine aggregate ratio in concrete results in pavements that maintain their smoothness over longer periods.

- A survey of State DOT personnel and concrete industry personnel showed no evidence suggesting that contractors have been adjusting their mix designs to achieve higher smoothness. The general consensus was that no modifications have been required in the concrete mix design to achieve higher smoothness. In addition, no pavement performance problems were thought to have been encountered because of the implementation of a smoothness specification.

CONSTRUCTION CONSIDERATIONS

- An overview of the construction procedures needed to construct a smooth pavement were outlined in this report. More detailed descriptions are presented in ACPA publication *Constructing Smooth Concrete Pavements*(17) and FHWA publication *PCC Pavement Smoothness Characteristics and Best Practices for Construction*.(25) Adherence to these procedures is necessary to construct a smooth pavement.

- Analysis of data from five projects indicated that the smoothest pavement was constructed in the project where the tie bars were attached to chairs fixed to the base before paving, whereas in all other projects the tie bars were inserted by the paver. The IRI of this project was 0.80 m/km (51 inches/mi). The IRI of the other four projects where the tie bars were inserted by paver were 1.11, 1.44, 0.95, and 1.07 m/km (70, 91, 60, and 68 inches/mi). Although fixing the tie bars to the base before paving may be more costly, these results indicate that doing so may achieve a smoother pavement.

MEASUREMENT OF SMOOTHNESS

- The surface of the pavement must be clean when performing profile measurements. Residue from saw-cutting operation that is present adjacent to the transverse joints can appear as small humps on the measured profile, and these can affect the smoothness indices computed from the profile data.

- The data collected by a profiler do not measure the shape of the joint accurately. Lightweight profilers from different manufacturers are using different methods to filter data. As a result, joints are being measured differently. Some profilers eliminate most of the effect of a joint by filtering, which flattens out the profile at the joint. There is a possibility that smoothness indices obtained by devices manufactured by different manufacturers are different because of differences in the way the joints are measured.

- In many States, an initial sawcut is made on the pavement, a joint reservoir is formed, and then the joint is sealed. Profile measurements should not be obtained when the joint reservoir is formed and the joint has not yet been sealed. At that point, data can yield high roughness values if the joint reservoir appears in the profile data.

- On transverse tined surfaces, when three repeat runs were obtained with lightweight profilers, the difference between the maximum and minimum IRI of the three runs over an approximate distance of 161 m (528 ft) typically ranged from 0.03 to 0.06 m/km (1.9 to 3.8 inches/mi). However, in longitudinally tined surfaces, this value was much higher and ranged from 0.06 to 0.09 m/km (3.8 to 5.7 inches/mi). Because of the interaction between the laser sensor and the longitudinal tining, the IRI obtained on longitudinally tined surfaces is less repeatable than that obtained on transverse tined surfaces.

- The short-interval IRI repeatability over 15-m (9-ft) segment lengths for transverse tined surfaces showed that the average difference in IRI between the maximum and minimum IRI obtained from three repeat runs ranged from 0.09 to 0.11 m/km (5.7 to 7 inches/mi). These results for a longitudinal tined surface ranged from 0.12 to 0.25 m/km (7.6 to 15.6 inches/mi). However, when the IRI over the entire section was computed, the repeat runs showed very close IRI values, due to compensation effects that cause the IRI differences occurring within short intervals to cancel out. These results indicate that implementing an IRI-based specification that relies on short-interval IRI (e.g., 15 m (49 ft)) is not practical for pavements that have longitudinal tining.

- Certifying profilers will ensure that consistent measurements are obtained between devices. Profilers used to measure smoothness of PCC pavements should be certified on PCC sections. There are differences in the way profilers treat joints as well as differences in the way tining is treated. Hence, certifying profilers on asphalt surfaces and using the profilers on a PCC section may not necessarily result in comparable data being collected on the PCC surface by the different profilers.

- Testing at five projects over a 3-month period indicated that negligible built-in slab curling was present on the pavement. Curvature of the slabs changed little over the 3-month period, and no noticeable effect of slab curvature affecting the IRI was noted.

- A study performed by collecting data at various periods over a 3-month interval at five projects showed somewhat variable results in changes that occur in IRI over the first 3 months after paving. In general, it appears that smoothness measurements can be performed at any time within the first few months after a project is paved. In some projects, very little change in IRI was noted (within ±5percent for a wheel path). In some projects, changes up to

±10 percent along a wheel path were noted. It is unclear whether these changes were occurring because of changes in pavement profile or whether they were related to either equipment effects or lateral wander during profiling.

- That study also found that little change in RN occurred over a 3-month period from that obtained immediately after paving. For most cases, the RN measured at different times within a 3-month period varied from that obtained immediately after paving by about ±5 percent.

- A study involving two profilers from two different manufacturers showed no evidence to suggest IRI or RN computed from data collected under the following two conditions would be different: (1) 3-mm (0.12-inch)-wide initial sawcut was present on the pavement and (2) joint reservoir sawed and sealed.

USE OF PROFILE DATA FOR CONSTRUCTING SMOOTH PAVEMENTS

- Usually smoothness indices like IRI are computed for each lane over a 161 m (528 ft) length for construction acceptance. The IRI for the overall section does not provide any information on how IRI varies within the section or where rough spots within the section are located. A roughness profile of the section can be used to investigate how roughness varies within the section and where rough spots within the section are located. If rough spots are detected within the section, their location could be correlated to the construction process to obtain information on a specific construction event that occurred at the location that resulted in a high roughness value. Overlaid roughness profiles of the wheel paths of the inside and outside lane can be used to see how roughness varies across the pavement. Using this approach may help detect problems in the construction process that result in rough spots.

- Advanced profile analysis techniques, such as PSD plots, can be used to identify whether a roughness associated with a specific wavelength is present in the new pavements. A wavelength that significantly contributes to the roughness will appear as a spike in the PSD plots. If such spikes are detected, the cause for the prominent wavelength to occur in the profile can be investigated. Repetitive wavelengths in the profile can be caused by equipment that is used for paving, finishing process, or stringline effects.

- Using roughness profiles as well as PSD plots on profile data collected at the start of a paving project will indicate whether there are any features in the profile that are contributing to a high roughness. If such features are identified, they can then be corrected at the start of the project.

APPENDIX A. CURVATURE INDEX

REVIEW OF SLAB FUNDAMENTALS AND CURVATURE CONCEPTS

Curvature is a key concept in structural analysis and applied mechanics, especially in the area of soil-structure interaction problems. The mathematical expression for curvature for function, $z = f(x)$, which in this case is an elevation = *f*(distance) profile along a jointed concrete pavement, is as shown in figure 129:

$$\kappa = \frac{\dfrac{d^2 z}{dx^2}}{\left[1 + (\dfrac{dz}{dx})^2\right]^{3/2}}$$

Figure 129. Equation. Kappa.

A key assumption in applied mechanics is the assumption of small strains or small changes in curvature from the initial state to the stressed state. When the slope, dz/dx, (or change in slope for structural analysis) is small, the bottom component of the above equation for curvature is very close to a value of 1. This allows the following simplified small strains approximation for curvature when there is a small slope, as shown in figure 130:

$$\text{Curvature} = \mathrm{K} \approx \frac{d^2x}{dx^2}$$

Figure 130. Equation. Curvature.

In the case of a road profile, the unit for curvature is $1/L$, where L is a length unit. Whenever a stress in a concrete pavement is calculated from a wheel load or curling/warping phenomena, the soil-structure interaction problem is first solved using equations of equilibrium for curvature. The pavement materials and thickness are then used to estimate stress for the solved values of curvature, which are based on the relative stiffness of the slab and foundation support. Slabs, including pavements, can be modeled in three dimensions with partial differential equations of form similar to that shown in figure 131:(38)

Bending in the X direction in a thin slab:

$$M_x = \frac{Eh^3}{12(1-\mu^2)} \frac{\partial^2 z}{\partial x^2}$$

Figure 131. Equation. M_x.

where,

M_x = Bending moment.
E = Slab elastic modulus.
$\frac{h^3}{12}$ = Moment of inertia of a unit wide slab element of height, h.
μ = Poisson's ratio for the slab material.

Flexural stress in a slab or beam is then related to the moment by figure 132:

$$\sigma = M\frac{y}{I}$$

Figure 132. Equation. Sigma.

where,

y = Distance from the neutral axis of the slab (mid-depth).
σ = Stress at a distance y from the neutral axis.

The analysis of curvatures and stress in rigid pavements is primarily based on Westergaard's research from the 1920s for predicting PCC slab deflections and stresses for wheel loads(39) and for temperature warping.(38) Based on that research and some recent trends, the equations in figures 133 and 134 are key moment-curvature equilibrium equations representing pavement slabs:

$$-\frac{\partial^2 z}{\partial x^2} = \frac{12}{Eh^3}(M_x - \mu M_y) + A + B + C + D$$

Figure 133. Equation. Negative second derivative of *z* over *x*.

$$-\frac{\partial^2 z}{\partial y^2} = \frac{12}{Eh^3}(M_y - \mu M_x) + A + B + C + D$$

Figure 134. Equation. Negative second derivative of *z* over *y*.

where,

A = Temperature-related curvature, curling = $f(t)$.

B = Moisture-related curvature, warping = $f(t)$.

C = Construction-related curvature = $f(A,x,y,t)$.

D = Long-term curvature creep = $f(A,B,C,x,y,t)$.

M_x, M_y = Bending moments in the slab = $f(A,B,C,D,x,y,t)$.

These two moment-curvature equilibrium equations can be more intuitively thought of as having the form in figure 135:

$$\kappa_{total} = \kappa_{stress} + \kappa_{temperature} + \kappa_{moisture} + \kappa_{construction} + \kappa_{creep}$$

Figure 135. Equation. κ_{total}.

Although A, B, C, and D curvature components vary over the area of the slab, the A and B components (in situ temperature- and moisture gradient-related curvatures) can generally be assumed to be close to constant over the slab area. However, the construction-related curvature and the creep of curvature over time, which is caused by stress relaxation in the slab or subgrade, are generally not constant over the slab area but are a function of the soil-structure interaction's sensitivity to joint spacing, foundation type, materials, and the like. The soil structure interaction is a function of all the various effects and slab geometry.

BREAKING DOWN THE PAVEMENT PROFILE

Modern profiling devices measure a profile of the total curvature in each slab from all of the curvature causing factors. They measure a slice through the solution (κ_{total}) for those partial differential equations shown previously. It is not easy to determine how much of the total curvature is occurring from each of the different types of curvature and thus it is also very difficult to quantify how much stress is actually present. The pavement profile can be divided into two separate matrices: one with only continuous slab segment data and one with only data for zones immediately around probable faults and cracks or questionable data. The following two categories of slab shape features are quantified:

- Imperfections: Usually discontinuities associated with moment hinges at cracks and joints, and these are obvious breaks in the continuity of the road profile, $z = f(x)$. The size and location of the imperfections are calculated.

- Average slab curvature: Between each imperfection, the average curvature present in each of the apparent continuous slabs is calculated.

Inertial profiling devices actually obtain a discontinuous profile consisting of evenly spaced point elevations, which can be conveniently analyzed using finite difference methods. The slope and curvature anywhere along the profile, for any span length, can readily be estimated using the finite difference forms of the first and second derivatives as follow in figures 136 and 137:

$$\frac{dz}{dx} = slope_n = \frac{z_{n+i} - z_n}{x_{n+i} - x_n}$$

Figure 136. Equation. First derivative of *z* over *x*.

$$\frac{d^2z}{dx^2} = curvature_n = \frac{slope_{n+i} - slope_n}{x_{n+i} - x_n}$$

Figure 137. Equation. Second derivative of *z* over *x*.

where, for the 152.4-mm (6-inch) spacing LTPP profiles:

$i = 1$ for 152.4-mm (6-inch) interval data points.
$i = 2$ for 304.8-mm (12-inch) interval data points.
$i = 3$ for 457.2-mm (18-inch) interval data points.
$i = 4$ for 604.6-mm (24-inch) interval data points.
$i = 5$ for 762-mm (30-inch) interval data points.
$i = 6$ for 914.4-mm (36-inch) interval data points.
$i = 7$ for 1,066.8-mm (42-inch) interval data points.
$i = 8$ for 1,219.2-mm (48-inch) interval data points.

CURVATURE INDEX

The computation of CI is illustrated using data obtained at a 152.4-m (500-ft) length GPS-3 section. The LTPP inertial profiler collected data at 25-mm (1-inch) intervals, but profile data elevations are moving-averaged to 152.4-mm (6-inch) intervals. The elevation value reported at a given point on the profile is the average of the surrounding 12, 25-mm (1-inch) interval elevation samples, or ±152.4 mm (6 inches), which is a moving average filter. These filtering techniques significantly reduce long wavelength vertical curves and micro/macro texture features having short wavelengths from the final profile data.

Slab curvature estimates were obtained by calculating curvature using the moving-arc finite difference second derivatives, using several different base lengths across a slab. Profile data within about 0.75 to 1 m (2 to 3 ft) of imperfections are separated from the apparent continuous portions of the profile data. Within each of these isolated slab zones, moving 0.152-, 0.305-, 0.457-, 0.610-, 0.762-, 1.067-, and 1.219-m (6-, 12-, 18-, 24-, 30-, 36-, 42-, and 48-inch) interval curvatures are calculated at 152.4-mm (6-inch) intervals over the full length of the individual slab segments between the imperfections using the three-point moving arc type data filter. This procedure is performed for all isolated slab zones within the section. For each wheel path, all previously described curvature values are averaged to obtain the CI of the wheel path.

Thereafter, the CI values for the two wheel paths are averaged to obtain the overall CI of the section.

APPENDIX B. GPS-3 SECTIONS USED FOR ANALYSIS

Tables 74 and 75 present information about nondoweled and doweled GPS-3 sections used for analysis.

Table 74. Nondoweled GPS sections.

Data Set	GPS Section	State	Environmental Region	Construction Date
Set 1	47613	AZ	Dry no-freeze	10/1/81
Set 1	63013	CA	Dry no-freeze	7/1/82
Set 1	67493	CA	Dry no-freeze	6/1/83
Set 1	163023	ID	Dry freeze	12/1/83
Set 1	313023	NE	Wet freeze	6/1/84
Set 1	313024	NE	Wet freeze	12/1/84
Set 1	313028	NE	Wet freeze	4/1/81
Set 1	404157	OK	Wet no-freeze	3/1/86
Set 1	463053	SD	Wet freeze	10/1/85
Set 1	533014	WA	Dry freeze	4/1/87
Set 1	556354	WI	Wet freeze	8/1/89
Set 2	63024	CA	Dry no-freeze	11/1/80
Set 2	203013	KS	Wet freeze	1/1/84
Set 2	383006	ND	Dry freeze	10/1/87
Set 2	493015	UT	Dry freeze	9/1/85
Set 2	497082	UT	Dry freeze	11/1/90
Set 2	497083	UT	Dry freeze	7/1/89
Set 2	533019	WA	Dry freeze	8/1/86
Set 2	537409	WA	Dry freeze	5/1/81
Set 2	553016	WI	Wet freeze	9/1/86
Set 2	556351	WI	Wet freeze	8/1/89
Set 2	556353	WI	Wet freeze	8/1/89
Set 3	203060	KS	Wet freeze	12/1/84
Set 3	313018	NE	Wet freeze	6/1/85
Set 3	313033	NE	Wet freeze	9/1/86
Set 3	323010	NV	Dry freeze	8/1/82
Set 3	323013	NV	Dry freeze	12/1/81
Set 3	383005	ND	Wet freeze	7/1/85
Set 3	463010	SD	Wet freeze	9/1/83
Set 3	493011	UT	Dry freeze	3/1/80
Set 3	553015	WI	Wet freeze	10/1/84
Set 3	563027	WY	Dry freeze	6/1/81
Set 3	833802	MB	Wet freeze	10/1/85

Table 75. Doweled GPS sections.

Data Set	GPS Section	State	Environmental Region	Construction Date
Set 1	53011	AR	Wet no-freeze	5/1/83
Set 1	124059	FL	Wet no-freeze	6/1/89
Set 1	124109	FL	Wet no-freeze	3/1/89
Set 1	133007	GA	Wet no-freeze	12/1/81
Set 1	133020	GA	Wet no-freeze	9/1/85
Set 1	193033	IA	Wet freeze	1/1/84
Set 1	203015	KS	Dry freeze	1/1/85
Set 1	213016	KY	Wet freeze	11/1/85
Set 1	353010	NM	Dry no-freeze	6/1/84
Set 1	373807	NC	Wet no-freeze	8/1/80
Set 1	393801	OH	Wet freeze	12/1/83
Set 1	421623	PA	Wet freeze	6/1/83
Set 1	423044	PA	Wet freeze	12/1/85
Set 1	453012	SC	Wet no-freeze	4/1/82
Set 1	483010	TX	Wet no-freeze	10/1/84
Set 1	556352	WI	Wet freeze	8/1/89
Set 1	556355	WI	Wet freeze	8/1/89
Set 1	893016	QB	Wet freeze	9/1/84
Set 2	47614	AZ	Dry no-freeze	5/1/84
Set 2	87776	CO	Dry freeze	8/1/88
Set 2	124057	FL	Wet no-freeze	6/1/86
Set 2	133019	GA	Wet no-freeze	12/1/81
Set 2	183030	IN	Wet freeze	1/1/81
Set 2	193028	IA	Wet freeze	6/1/85
Set 2	273013	MN	Wet freeze	10/1/85
Set 2	283018	MS	Wet no-freeze	11/1/84
Set 3	123804	FL	Wet no-freeze	9/1/85
Set 3	163017	ID	Dry freeze	11/1/86
Set 3	283019	MS	Wet no-freeze	11/1/84
Set 3	373008	NC	Wet no-freeze	6/1/84
Set 3	893015	QB	Wet freeze	9/1/84
Set 3	273003	MN	Wet freeze	11/1/86

APPENDIX C. FAULTING AT GPS SECTIONS

Tables 76 and 77 present information about faulting at nondoweled and doweled GPS sections.

Table 76. Faulting at nondoweled sections.

Data Set	Section ID	State	First Profile Date	Last Profile Date	Average IRI (m/km) First Profile Date	Average IRI (m/km) Last Profile Date	Fault Survey Date	Total Faulting Over 152.4-m-Long Section (mm (inch)) Joints	Cracks	Total
Set 1	47613	AZ	3/22/90	2/18/99	1.61	1.56	12/8/98	30 (1.17)	0	30 (1.17)
Set 1	63013	CA	1/29/90	3/19/02	1.83	1.76	3/8/02	41 (1.60)	0	41 (1.60)
Set 1	67493	CA	1/30/90	3/9/00	1.41	1.46	3/20/00	34 (1.33)	0	34 (1.33)
Set 1	163023	ID	10/17/89	8/1/01	1.53	1.59	10/9/02	23 (0.90)	0	23 (0.90)
Set 1	313023	NE	4/16/90	5/15/01	1.13	1.27	4/5/00	6 (0.23)	0	6 (0.23)
Set 1	313028	NE	5/3/90	4/25/02	1.20	1.31	1/23/02	23 (0.90)	0	23 (0.90)
Set 1	404157	OK	1/25/91	5/9/01	1.08	1.21	9/12/00	58 (2.26)	0	58 (2.26)
Set 1	463053	SD	11/18/89	10/3/01	1.16	1.31	8/14/02	34 (1.33)	0	34 (1.33)
Set 1	533014	WA	10/25/89	5/9/01	0.97	1.08	4/27/00	60 (2.34)	0	60 (2.34)
Set 1	556354	WI	6/5/90	7/16/01	1.24	1.24	10/10/02	10 (0.39)	0	10 (0.39)
Set 2	63024	CA	1/29/90	3/21/02	1.52	1.88	3/11/02	54 (2.11)	0	54 (2.11)
Set 2	203013	KS	4/11/90	4/23/02	1.59	1.99	7/19/01	36 (1.40)	0	36 (1.40)
Set 2	383006	ND	10/23/89	10/7/01	0.89	1.38	8/5/99	59 (2.30)	0	59 (2.30)
Set 2	493015	UT	6/29/89	9/9/01	1.97	2.30	6/19/02	76 (2.96)	0	76 (2.96)
Set 2	497082	UT	9/18/91	9/2/01	0.91	1.29	5/14/02	48 (1.87)	0	48 (1.87)
Set 2	497083	UT	10/23/91	9/8/01	1.26	1.31	6/21/02	12 (0.47)	4 (0.16)	17 (0.66)
Set 2	533019	WA	10/25/89	5/10/01	1.16	1.28	5/1/00	66 (2.57)	0	66 (2.57)
Set 2	537409	WA	10/24/89	5/9/01	1.21	1.48	5/3/00	77 (3.00)	1 (0.04)	78 (3.04)
Set 2	553016	WI	6/5/90	4/19/99	1.24	1.54	11/3/94	61 (2.38)	0	61 (2.38)
Set 2	556351	WI	6/4/90	7/16/01	1.99	2.29	10/10/02	17 (0.66)	0	17 (0.66)
Set 2	556353	WI	6/5/90	7/16/01	1.50	1.82	10/10/02	10 (0.39)	0	10 (0.39)
Set 3	203060	KS	4/11/90	4/23/02	1.11	1.88	7/16/01	83 (3.24)	0	83 (3.24)
Set 3	313018	NE	4/16/90	6/13/01	1.29	2.27	5/14/02	158 (6.16)	0	158 (6.16)
Set 3	313033	NE	11/19/89	4/26/02	0.83	1.95	11/16/99	122 (4.76)	0	122 (4.76)
Set 3	323010	NV	9/6/89	10/15/99	2.23	3.02	4/25/00	150 (5.85)	3 (0.18)	153 (5.97)
Set 3	323013	NV	9/30/89	8/26/98	1.76	2.17	2/4/92	26 (1.01)	0	26 (1.01)
Set 3	383005	ND	10/24/89	10/9/01	1.35	1.85	8/4/99	61 (2.38)	0	61 (2.38)
Set 3	463010	SD	11/13/89	5/15/99	2.06	2.52	8/21/02	0	0	0
Set 3	493011	UT	8/2/89	9/9/01	1.32	2.66	6/20/02	105 (4.10)	0	105 (4.10)
Set 3	553015	WI	6/5/90	4/20/99	1.96	2.89	2/25/92	70 (2.73)	0	70 (2.73)
Set 3	563027	WY	6/28/89	10/18/01	2.24	4.20	8/20/01	307 (11.97)	5 (0.20)	312 (12.17)
Set 3	833802	MB	10/18/89	3/12/00	1.83	4.22	5/16/00	102 (3.98)	0	102 (3.98)

1 m/km = 63.4 inches/mi

Table 77. Faulting at doweled sections.

Data Set	Section ID	State	First Profile Date	Last Profile Date	Average IRI (m/km) First Profile Date	Average IRI (m/km) Last Profile Date	Fault Survey Date	Total Faulting Over 152.4-m-Long Section (mm (inch)) Joints	Cracks	Total
Set 1	53011	AR	11/29/90	1/23/01	1.13	1.25	6/27/00	0	0	0
Set 1	124059	FL	7/13/90	9/12/00	0.99	1.03	5/13/02	32 (1.25)	0	32 (1.25)
Set 1	124109	FL	7/13/90	9/12/00	1.90	1.94	2/9/00	11 (0.43)	0	11 (0.43)
Set 1	133007	GA	8/3/90	8/6/01	1.76	1.80	2/12/02	38 (1.48)	0	38 (1.48)
Set 1	133020	GA	6/14/90	8/15/01	1.37	1.37	2/20/02	18 (0.70)	2 (0.08)	20 (0.78)
Set 1	193033	IA	6/13/90	8/8/01	1.69	1.67	10/4/02	8 (0.31)	0	8 (0.31)
Set 1	203015	KS	4/14/90	4/18/02	1.10	1.29	10/16/02	37 (1.44)	0	37 (1.44)
Set 1	213016	KY	12/4/89	2/8/02	1.53	1.55	5/16/02	10 (0.39)	0	10 (0.39)
Set 1	353010	NM	10/18/90	11/14/01	1.38	1.42	5/10/02	26 (1.01)	0	26 (1.01)
Set 1	373807	NC	2/6/90	7/6/00	1.81	1.83	8/29/00	26 (1.01)	0	26 (1.01)
Set 1	393801	OH	9/28/89	11/6/01	1.90	1.97	3/20/01	10 (0.39)	0	10 (0.39)
Set 1	421623	PA	11/5/89	11/13/00	1.32	1.33	7/30/98	15 (0.59)	0	15 (0.59)
Set 1	423044	PA	11/22/89	6/11/00	2.37	2.19	1/29/02	0	0	0
Set 1	453012	SC	7/31/90	8/18/01	1.12	1.20	7/18/01	5 (0.20)	0	5 (0.20)
Set 1	483010	TX	4/12/90	10/23/01	2.17	2.18	8/28/00	10 (0.39)	0	10 (0.39)
Set 1	556352	WI	6/5/90	7/16/01	1.23	1.33	10/10/02	6 (0.23)	0	6 (0.23)
Set 1	556355	WI	6/27/90	7/16/01	1.68	1.61	10/10/02	10 (0.39)	0	10 (0.39)
Set 1	893016	QB	9/23/89	5/12/99	3.00	2.97	10/19/00	10 (0.39)	0	10 (0.39)
Set 2	47614	AZ	3/24/90	11/13/01	1.02	1.33	12/10/01	7 (0.27)	0	7 (0.27)
Set 2	87776	CO	4/27/90	9/4/01	1.39	1.67	7/22/02	7 (0.27)	0	7 (0.27)
Set 2	124057	FL	7/3/90	9/7/00	0.76	1.00	2/7/00	16 (0.62)	0	16 (0.62)
Set 2	133019	GA	8/3/90	3/14/02	1.46	1.82	2/11/02	30 (1.17)	0	30 (1.17)
Set 2	183030	IN	10/4/89	11/9/01	1.55	1.95	9/27/01	6 (0.23)	0	6 (0.23)
Set 2	193028	IA	6/13/90	8/8/01	1.69	1.99	4/5/02	10 (0.39)	0	10 (0.39)
Set 2	273013	MN	6/21/90	8/15/01	1.28	1.66	9/19/02	3 (0.18)	0	3 (0.18)
Set 2	283018	MS	12/5/90	3/26/01	1.69	1.95	1/25/00	20 (0.78)	0	20 (0.78)
Set 3	123804	FL	7/3/90	9/7/00	1.52	2.12	5/16/02	65 (2.54)	44 (1.72)	109 (4.25)
Set 3	163017	ID	9/7/89	8/2/01	1.58	2.04	8/11/99	25 (0.98)	0	25 (0.98)
Set 3	273003	MN	6/20/90	8/22/01	1.94	2.64	9/23/02	7 (0.27)	0	7 (0.27)
Set 3	283019	MS	12/6/90	3/26/01	1.58	2.13	1/26/00	65 (2.54)	0	65 (2.54)
Set 3	373008	NC	3/15/90	4/22/02	1.79	2.17	4/17/02	5 (0.20)	0	5 (0.20)
Set 3	893015	QB	9/23/89	5/12/99	1.16	3.02	9/2/98	18 (0.70)	0	18 (0.70)

1 m/km = 63.4 inches/mi

APPENDIX D. CURVATURE INDEX VALUES FOR GPS-3 SECTIONS

Tables 78 and 79 present curvature index values at nondoweled and doweled GPS-3 sections, respectively.

Table 78. CI at nondoweled sections.

Data Set	Section ID	State	Construction Date	First Profile Date	Last Profile Date	Average IRI (m/km)		CI x 1,000 (1/m)	
						First Profile Date	Last Profile Date	First Profile Date	Last Profile Date
Set 1	47613	AZ	10/1/81	3/22/90	2/18/99	1.61	1.56	0.236	0.108
Set 1	63013	CA	7/1/82	1/29/90	3/19/02	1.83	1.76	1.079	0.560
Set 1	67493	CA	6/1/83	1/30/90	3/9/00	1.41	1.46	0.044	0.187
Set 1	163023	ID	12/1/83	10/17/89	8/1/01	1.53	1.59	0.048	0.234
Set 1	313023	NE	6/1/84	4/16/90	5/15/01	1.13	1.27	0.132	−0.018
Set 1	313024	NE	12/1/84	5/3/90	4/25/02	1.44	1.60	−0.071	−0.055
Set 1	313028	NE	4/1/81	5/3/90	4/25/02	1.20	1.31	−0.005	−0.234
Set 1	404157	OK	3/1/86	1/25/91	5/9/01	1.08	1.21	−0.009	0.066
Set 1	463053	SD	10/1/85	11/18/89	10/3/01	1.16	1.31	0.150	−0.001
Set 1	533014	WA	4/1/87	10/25/89	5/9/01	0.97	1.08	0.117	0.159
Set 1	556354	WI	8/1/89	6/5/90	7/16/01	1.24	1.24	−0.042	−0.150
Set 2	63024	CA	11/1/80	1/29/90	3/21/02	1.52	1.88	0.337	0.279
Set 2	203013	KS	1/1/84	4/11/90	4/23/02	1.59	1.99	0.102	−0.033
Set 2	383006	ND	10/1/87	10/23/89	10/7/01	0.89	1.38	0.030	0.676
Set 2	493015	UT	9/1/85	6/29/89	9/9/01	1.97	2.30	−0.051	−0.432
Set 2	497082	UT	11/1/90	9/18/91	9/2/01	0.91	1.29	−0.021	0.266
Set 2	497083	UT	7/1/89	10/23/91	9/8/01	1.26	1.31	−0.043	0.127
Set 2	533019	WA	8/1/86	10/25/89	5/10/01	1.16	1.28	0.218	0.391
Set 2	537409	WA	5/1/81	10/24/89	5/9/01	1.21	1.48	0.071	0.050
Set 2	553016	WI	9/1/86	6/5/90	4/19/99	1.24	1.54	0.167	0.105
Set 2	556351	WI	8/1/89	6/4/90	7/16/01	1.99	2.29	−0.362	−0.319
Set 2	556353	WI	8/1/89	6/5/90	7/16/01	1.50	1.82	−0.124	0.206
Set 3	203060	KS	12/1/84	4/11/90	4/23/02	1.11	1.88	−0.083	−0.358
Set 3	313018	NE	6/1/85	4/16/90	6/13/01	1.29	2.27	0.005	0.825
Set 3	313033	NE	9/1/86	11/19/89	4/26/02	0.83	1.95	0.036	0.767
Set 3	323010	NV	8/1/82	9/6/89	10/15/99	2.23	3.02	0.304	0.188
Set 3	323013	NV	12/1/81	9/30/89	8/26/98	1.76	2.17	0.178	0.249
Set 3	383005	ND	7/1/85	10/24/89	10/9/01	1.35	1.85	0.358	1.187
Set 3	463010	SD	9/1/83	11/13/89	5/15/99	2.06	2.52	0.572	1.035
Set 3	493011	UT	3/1/80	8/2/89	9/9/01	1.32	2.66	0.094	1.383
Set 3	553015	WI	10/1/84	6/5/90	4/20/99	1.96	2.89	0.526	0.574
Set 3	563027	WY	6/1/81	6/28/89	10/18/01	2.24	4.20	0.508	0.889
Set 3	833802	MB	10/1/85	10/18/89	3/12/00	1.83	4.22	0.287	0.822

1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

Table 79. CI at doweled sections.

Data Set	Section ID	State	Construction Date	First Profile Date	Last Profile Date	Average IRI (m/km) First Profile Date	Average IRI (m/km) Last Profile Date	CI x 1,000 (1/m) First Profile Date	CI x 1,000 (1/m) Last Profile Date
Set 1	53011	AR	5/1/83	11/29/90	1/23/01	1.13	1.25	0.093	0.046
Set 1	124059	FL	6/1/89	7/13/90	9/12/00	0.99	1.03	−0.045	−0.121
Set 1	124109	FL	3/1/89	7/13/90	9/12/00	1.90	1.94	0.127	0.084
Set 1	133007	GA	12/1/81	8/3/90	8/6/01	1.76	1.80	−0.006	0.313
Set 1	133020	GA	9/1/85	6/14/90	8/15/01	1.37	1.37	−0.128	0.044
Set 1	193033	IA	1/1/84	6/13/90	8/8/01	1.69	1.67	−0.171	−0.010
Set 1	203015	KS	1/1/85	4/14/90	4/18/02	1.10	1.29	−0.058	−0.253
Set 1	213016	KY	11/1/85	12/4/89	2/8/02	1.53	1.55	0.066	0.501
Set 1	353010	NM	6/1/84	10/18/90	11/14/01	1.38	1.42	0.085	0.274
Set 1	373807	NC	8/1/80	2/6/90	7/6/00	1.81	1.83	−0.008	0.023
Set 1	393801	OH	12/1/83	9/28/89	11/6/01	1.90	1.97	−0.065	−0.211
Set 1	421623	PA	6/1/83	11/5/89	11/13/00	1.32	1.33	−0.028	0.013
Set 1	423044	PA	12/1/85	11/22/89	6/11/00	2.37	2.19	−0.091	−0.127
Set 1	453012	SC	4/1/82	7/31/90	8/18/01	1.12	1.20	0.005	−0.057
Set 1	483010	TX	10/1/84	4/12/90	10/23/01	2.17	2.18	−0.246	−0.031
Set 1	556352	WI	8/1/89	6/5/90	7/16/01	1.23	1.33	−0.116	−0.053
Set 1	556355	WI	8/1/89	6/27/90	7/16/01	1.68	1.61	−0.044	−0.380
Set 1	893016	QB	9/1/84	9/23/89	5/12/99	3.00	2.97	−0.108	−0.475
Set 2	47614	AZ	5/1/84	3/24/90	11/13/01	1.02	1.33	−0.057	0.462
Set 2	87776	CO	8/1/88	4/27/90	9/4/01	1.39	1.67	−0.022	−0.162
Set 2	124057	FL	6/1/86	7/3/90	9/7/00	0.76	1.00	−0.023	0.028
Set 2	133019	GA	12/1/81	8/3/90	3/14/02	1.46	1.82	0.039	−0.238
Set 2	183030	IN	1/1/81	10/4/89	11/9/01	1.55	1.95	−0.043	0.176
Set 2	193028	IA	6/1/85	6/13/90	8/8/01	1.69	1.99	0.021	−0.427
Set 2	273013	MN	10/1/85	6/21/90	8/15/01	1.28	1.66	−0.142	0.563
Set 2	283018	MS	11/1/84	12/5/90	3/26/01	1.69	1.95	−0.075	−0.115
Set 3	123804	FL	9/1/85	7/3/90	9/7/00	1.52	2.12	−0.005	−0.206
Set 3	163017	ID	11/1/86	9/7/89	8/2/01	1.58	2.04	−0.077	−0.483
Set 3	273003	MN	11/1/86	6/20/90	8/22/01	1.94	2.64	−0.876	−1.642
Set 3	283019	MS	11/1/84	12/6/90	3/26/01	1.58	2.13	0.083	−0.043
Set 3	373008	NC	6/1/84	3/15/90	4/22/02	1.79	2.17	−0.081	−0.392
Set 3	893015	QB	9/1/84	9/23/89	5/12/99	1.16	3.02	−0.069	−0.411

1 m/km = 63.4 inches/mi
1/m = 1/3.28 ft

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