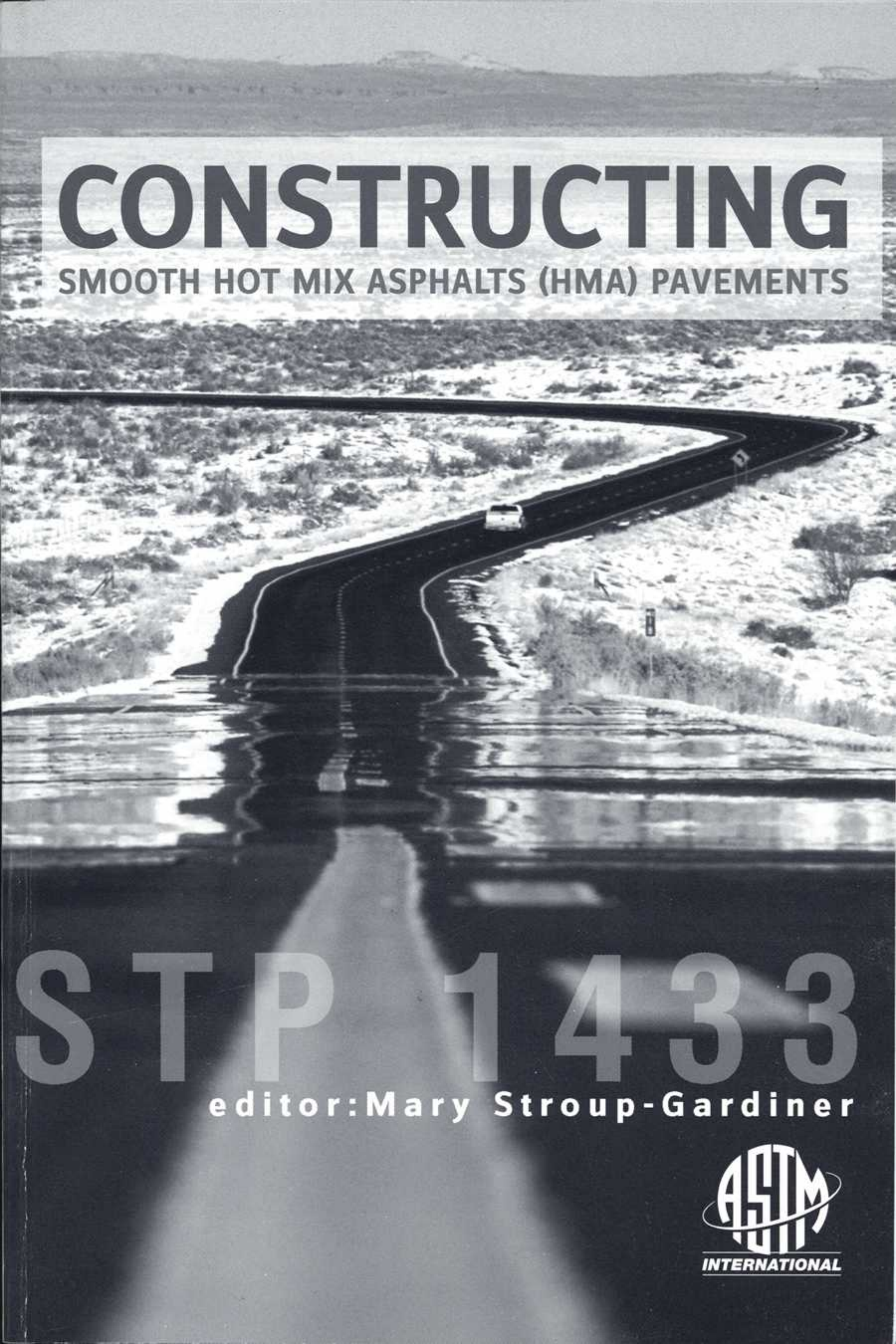

CONSTRUCTING

SMOOTH HOT MIX ASPHALTS (HMA) PAVEMENTS

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Constructing Smooth Hot Mix Asphalt (HMA) Pavements

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Foreword

This publication, *Constructing Smooth Hot Mix Asphalt (HMA) Pavements,* contains papers presented at the symposium of the same name held in Dallas, Texas, on 4 December 2001. The symposium was sponsored by ASTM International Committee D4 on Road and Paving Materials. The symposium chairperson was Mary Stroup-Gardiner, Auburn University.

Contents

Overview

The number of miles in America's highway infrastructure increases each year, however the funds available for the construction, maintenance, and repair of this infrastructure traditionally lag far behind these needs. It is now, more than ever, critically important to maximize the quality and longevity of any highway work. The construction of smooth, or conversely, less rough, pavement surfaces has been identified as a major factor in accomplishing this goal. There is evidence that initially smoother pavements perform longer with fewer needed maintenance activities than initially rougher pavements. While this concept has spurred most agencies to formulate specifications that control the initial roughness of the pavement, there is no consensus among the agencies on what roughness parameter or equipment is best. There is also little understanding of the correlations between the types of equipment and roughness parameters.

This book represents the work of a number of authors prepared for the American Society for Testing and Materials Symposium on Constructing Smooth Hot Mix Asphalt (HMA) Pavements, December 4, 2001, Dallas, Texas. Papers and presentations were selected to highlight the state-of-the-art agency research, equipment comparisons, and innovative methods for processing profile data. This effort represents the commitment of ASTM committee D4 on Road and Paving Materials to provide a timely look at hot mix asphalt (HMA) smoothness measurements, specifications, and equipment.

State Agency Perspectives

Five papers provide the reader with insight into both the history of the development and the implementation of roughness specifications for new hot mix asphalt pavements in Alabama, Arizona, New Jersey, Virginia, and Tennessee. These papers highlight the wide range of differences in equipment and approaches used to quantify HMA smoothness by state agencies across the country. This information will provide the readers with insight into complexities associated with developing and implementing ride quality specifications.

National and International Perspectives

One paper uses an analysis of the Long Term Pavement Performance (LTPP) national pavement data base to determine the affect of various construction alternatives on the smoothness of the final HMA surface. This paper also presents correlation equations that relate measurements with traditional, but slow, hand-operated profilograph to measurements with the state-of-the-art vehicle-mounted equipment. A second paper compares the use of six devies for measuring roughness on recently constructed Taiwan highways. This information will prove especially useful for agencies faced with assessing ride quality in confined urban areas.

Equipment Comparisons, Materials Considerations, and Analyses

One paper provides information as to how various HMA mixtures, friction courses, and construction practices influence smoothness measurements and pavement quality. A second compares the results obtained from an inclinometer profiler and a vehicle mounted profiler when used to test a wide range of HMA mixtures. Correlations between construction practices and their influence on roughness are also presented. The third paper discusses a new method for analyzing the raw profile data obtained by a wide range of profilers. This analysis method can be used to improve data processing for any equipment that collects the raw profile.

In summary, this collection of papers provides the reader with the necessary overview to understand the current state-of-the-art approaches to constructing smooth HMA pavements.

Mary Stroup Gardiner
Auburn University
Auburn University, AL
Symposium chairperson and editor

State Agency Perspectives

Brian Bowman,[1] B. Parker Ellen, III,[2] and M. Stroup Gardiner[1]

Evaluation of Pavement Smoothness and Pay Factor Determination for the Alabama Department of Transportation

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Reference: Bowman, B., Ellen, B.P., III, and Stroup Gardiner, M., **Evaluation of Pavement Smoothness and Pay Factor Determination for the Alabama Department of Transportation,"** *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433,* M. S. Gardiner Ed., American Society for Testing and Materials International, West Conshohocken, PA. 2003.

Abstract: In 1989 the Alabama Department of Transportation (ALDOT) added a policy to their smoothness specification that enables payments made to paving contractors to be based on the level of smoothness. The contractor can receive a 5 % bonus for above average or a 5 % penalty for below average smoothness readings. The measurement of smoothness has been based on the manual extraction of data from profilograph traces based on a 0.2 blanking band and resolution of 0.05. ALDOT has determined that more than three-quarters of all the 0.1 mile segments tested since the implementation of the specification have fallen in the 5 % bonus range without an improvement in pavement ride quality. This observation resulted in the decision to conduct a study to determine; 1) if the ProScan™ hardware and software could be used to provide a reliable method of reducing profilograph traces, and 2) to investigate the feasibility and consequences of different smoothness pay factors. The results of the study support the ProScan™ system as a quick, accurate, and replicable method of reducing the profilographs. In addition, it was concluded that ALDOT should change the blanking band to a width of 0.0 and should adopt a combined step and continuous function method of determining incentive pay factors. With these pay factors in place ALDOT would have paid only 96.8% of the bid price for paving projects that brought 102% pay under the old step-wise function.

Keywords: roughness, smoothness, International Roughness Index, ProScan

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Introduction

Alabama Department of Transportation (ALDOT) measures pavement smoothness based on the California Rolling profilograph. This device is a 25 ft (7.62 m) long, multi-wheeled rolling straightedge that is propelled by hand. It measures the vertical deviations from a moving fixed-length reference plane. The result of this test is usually a graphical record; a profilograph trace. A perfectly plane surface would have no vertical deviations and measure 0 in/mile. Most States allow small deflections recorded by the profilograph nulled out of the measurements to compensate for equipment vibrations and other minor movements. The amount of deflection to be nulled out is determined by the specification of a blanking band. Only deflections occurring outside of the blanking band tolerance are recorded as deviations from a smooth surface.

In 1989, ALDOT added a policy to their smoothness specification that enables payments made to a paving contractor to be based on the level of smoothness. Contractors can receive a 5% bonus for above average smoothness readings or a 5% penalty for below average profile index (PI) ratings. Alabama is among a majority of state highway agencies that currently offers an incentive/disincentive policy, a practice which is encouraged by American Association of State Highway and Transportation officials. However, an analysis by ALDOT indicates that more than three-quarters of all the 0.1 mile segments tested since the implementation of the specification have fallen in the 5 % bonus range (Figure 1) without an improvement in pavement ride quality. ALDOT officials believe that some inferior pavement sections have received bonus payments. These bonus payments were believed to occur due to large incentive payment increments (5%) resulting in a skewed payment distribution.

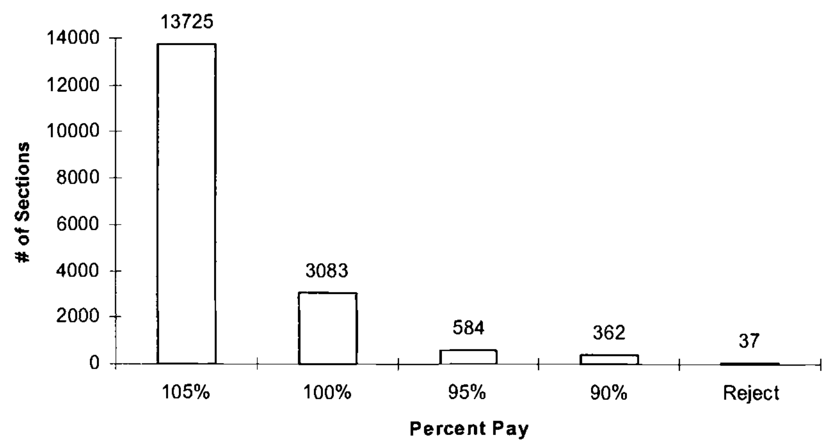

Figure 1 - ALDOT pay adjustment distribution

Other studies indicate that a thick blanking band tolerance zone, in the manual method of trace analysis, allows minor defects in the pavement to go unnoticed [1,2,3]. Alabama as well as most states that use the manual method for trace analysis specifies a blanking band width of 0.2 inches (5mm). In 1990, The Kansas Department of Transportation (KDOT) began studying the affect that the 0.2 inch (5mm) blanking band has on the analysis results [4]. They noticed a series of low amplitude waves in the profile of some pavements that are not being incorporated in the smoothness analysis. These low amplitude waves can dramatically affect ride quality but are not measured because they fall inside the blanking band tolerance zone. KDOT has changed their specifications to use a zero "null" blanking band width that eliminates the tolerance limit.

Objectives

There were two major objectives of this project. The first was to conduct an analysis of an electronic scanning device called ProScan™ [3] as a feasible alternative to the manual method of trace analysis. This required determining the repeatability of ProScan™ to insure that the results are consistent. The second objective was to revise ALDOT's current smoothness pay scale such that it produces a distribution of payments that encourages smooth pavements.

Background

During a 1960 study on the evaluation of ride quality, Carey and Irick introduced the "serviceability-performance concept" as a measure of ride quality [5]. Carey and Irick instituted a system of rating panels that numerically rated different pavement sections based on the perceived quality of ride that each provided. The rating panels consisted of pavement specialists who gave a rating between 0 and 5 for each section, based on their perception of ride quality. The results of each panelist were combined and used to calculate a PI for each section. The sections that were assigned a PI rating between 4 and 5 were considered to have a superior ride quality, sections rating between 2 and 4 were of average ride quality, and the sections falling in the 0 to 2 PI range were considered poor pavements. The categorized test sections were analyzed to determine which pavement factors influenced ride quality. It was determined that 95% of a pavement's ride quality is due to the smoothness of the surface profile. Even though other factors such as vehicle dynamics and human response can influence ride quality, they do not affect the perceived ride quality as much as pavement smoothness.

Pavement smoothness is a measure of the distortions of the pavement profile from a level plane. When evaluating smoothness for newly constructed pavements or overlays the focus lies entirely on the construction process. Any irregularities in construction, such as a lack of uniformity in the thickness of the pavement layers, or poor construction can result in smoothness distortions. When evaluating pavement smoothness on pavements that have been in service the emphasis is not on the quality of their construction, but on other factors as well. Pavement distresses such as cracking and

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[3] The ProScan system was developed and the software programmed by Devore Systems, Inc., Manhattan, KS.

rutting, which are functions of repeated loads, will contribute to the level of smoothness. These distresses can be a reflection of a poorly constructed pavement, but in most cases are due to the quality of material used to construct the roadway. The environment also plays a key role in the performance of a pavement over time. Deterioration of one or more pavement layers due to shrinking and swelling of the subgrade in conjunction with repeated load applications can create pavement distresses that lead to smoothness variations .

Between 1971 and 1982, the World Bank supported several studies in Brazil, Kenya, the Caribbean, and India and developed the International Road Roughness Index (IRI) as a standard that can be used to evaluate smoothness [6]. The IRI is based on mathematically simulating the response of one tire on a car traveling at 50 mph (80 km/h). This quarter-car model (Figure 2) is represented by standardized parameter values of a sprung mass, unsprung mass, suspension spring rate, and suspension linear damping. The IRI is based on the relative displacement of the sprung and unsprung masses at a 50 mph (80 km/m) test speed over the length of the test section and is reported as inches of roughness per mile (mm/km).

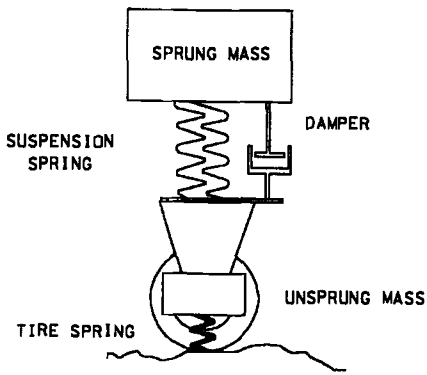

Figure 2 - Quarter-car model

The first profilographs, called longitudinal profilographs, were hand propelled and consisted of a rigid beam or frame mounted on a multiple-wheel support system. The California Department of Transportation developed the first profilograph in the 1940's. Many variations exist, with lengths ranging from 7 to 25 feet and 4 to 12 supporting wheels - in addition to the "profile wheel". These traditional models are walk-behind profilographs and are operated at low speeds (5 mph (8km/h)or less). The profilograph trace, developed by the device, can be analyzed either manually or electronically to evaluate smoothness; some newer models are linked directly with computers . The resulting PI is reported as inches of roughness per mile (mm per km).

Profilogram Reduction Methods and Procedures

Manual Method

The manual method of reducing the trace produced by a mechanical profilograph involves using a special plastic scale that is approximately 1.7 in (43 mm) wide and 21.12 in (535 mm) long. The length of the template represents 528 ft (161 m). At the center of the template is a solid color band that can vary in width from 0 to 0.2 in (0 to 5 mm). The solid color portion of the template covers up a portion of the trace and is the "blanking band". The template is centered on the profilograph trace and the number and magnitude of the vertical deviations "scallops" above and below the blanking band are recorded. When the measurements are summed and divided by the test section length the PI, expressed as inches per mile (millimeters per kilometer), is obtained.

Some state highway agencies also require locating bumps on the profile trace. Bumps are deviations that exceed 0.3 in (7.6 mm) in height and require corrective action such as grinding or milling by the contractor. Bumps are identified on the trace with a clear, plastic template that is approximately 3 in (75 mm) wide and 5 in (125 mm) long. On the front of the bump template is a horizontal line that is 1 in (25 mm) long and terminated by two short vertical lines that are in (3 mm). A 1 in (25 mm) slit in the template is located 0.3 in (7.6 mm) above and parallel to the scribed line and is just wide enough to fit the tip of a pencil. The template is placed on the profilogram so to align the scribed line under the base of a bump. A line is drawn through the slit in the template onto the trace to note the area of the bump that exceeds 0.3 in (7.6 mm) in height. The template is then moved to the next bump and the procedure is repeated .

Proscan Automated Profilogram Reduction System

An alternative method of reducing the trace produced by a mechanical profilograph is with the use of an automated profilogram reduction system. ProScan is a DOS based system developed by Devore Systems, Inc. that consists of a hand scanner mounted on a paper transport unit. The transport unit scrolls the trace paper produced by the mechanical profilograph at a continuous rate while the scanner captures the trace information. The trace is digitized by an image enhancement program and stored on a disk. A two-sided moving-average filter is applied to the recorded profile. The purpose of the filter is to remove the sharp deviations caused from pavement texture or profilograph vibration. The ProScan software performs a least-square error analysis to determine the best fit linear line and measures scallop heights to determine the PI. It can also indicate the location of bumps that occur on the profile trace. The results can be displayed on screen or can be printed in report format. ProScan also offers the ability to change the reduction parameters at which the profile trace is analyzed. The operator has the ability to define certain criteria such as blanking band width, segment length, filter length, scallop resolution, minimum scallop height, minimum scallop width, and minimum bump height as required in the specifications. A big advantage of ProScan is the relative ease and speed that a profilogram trace can be analyzed compared to the manual reduction procedure.

Data Analysis

The data used for this project consisted of profile traces from 20 ALDOT paving projects that were constructed during the period from 1991 to 1995. Table 1 summarizes the profilogram data provided for the project by ALDOT. The profilograms were produced using a California type profilograph. Included with the traces were calculated values of the PI for each 0.1 mile segment of the trace as determined manually by ALDOT personnel. The data consisted of 326 lane miles of data which resulted in 3310 segments of 0.1 mile or less in length. The profilogram for each segment was scanned and analyzed by ProScan five times using the same reduction criteria currently used by ALDOT for manual trace reduction. The traces were also analyzed using a variation of different reduction parameters by changing the blanking band widths to 0.1 and 0.0 in and the scallop resolution to 0.01 in. The data analysis steps included the following.

- *ProScan Consistency* - Each of the 3310 profilograph traces was reduced five times by the scanning reduction system. The purpose of this multiple scan was to determine if the ProScan system was capable of providing consistently reliable readings. Consistency was ascertained by inspection and analysis of the population standard deviation and the ability of the ProScan system to identify bumps and scallops.

- *Comparison of Manual and ProScan Readings* - A comparison of the manual and ProScan methods was performed after the reliability of the ProScan system had been established. This comparison was performed to determine if ProScan could acceptably simulate the manual method of data reduction. The analysis is not as straight forward as may first appear. For example, the original profilograms that were used to obtain the manual readings were also used for obtaining the five ProScan readings. Because the ProScan system was applied five times and consistent results were obtained there is a high degree of confidence in the results. The same is not true for the manual readings. Only one manual reading for each analysis segment was obtained by unknown individuals in uncontrolled environments.

- *Determination of Pay Adjustment Factors* - After the feasibility of using ProScan for smoothness measurement had been established, the data was used as a model for developing new pay adjustment factors. Alabama has been predominantly paying 5% bonuses on pavement segments constructed since the adoption of the incentive/disincentive policy in 1989. The incentive payments are intended to motivate contractors to achieve smoothness levels above the minimum requirement. However, when the incentive payment threshold is at a level that can be reached repeatedly, the motivation for improving quality does not exist. By using the database as a sample of the overall pavement smoothness levels in Alabama, new pay adjustment factor levels can be created to produce a more even distribution of payments.

Table 1 - Summary of ALDOT pavement smoothness data for flexible pavements.

Division	Project Number	Project Designer	Date	Lanes	Type	Length (lane miles)[1]	# of Test Segment s
1	IM-65-3(132)	1A	Aug-93	4		19.07	192
	STPAA-398(39)	1B	Aug-95	4		14.25	144
2	NHF-398(40)	2A	Oct-93	2	Overlay	8.39	85
	D-2113(1)	2B	Nov-93	2		11.77	118
	APD-471(29)	2C	Oct-94	4	New Construction	22.17	223
3	IR-59-1(172)105	3A	Jun-91	8		26.93	271
	IR-459-4(64)24	3B	Jun-91	6		22.78	228
	99-303-644-069-301	3D	Oct-93	2		9.73	99
	STPAA-6407-107	3E	Jul-95	2	New Construction	13.94	142
4	IM-85-1(116)	4A	Apr-94	4	Overlay	15.94	160
	DBAAF-9062(2)/ MAAA-9062(3)	4B	Jul-95	2	New Construction	2.95	30
5	99-305-543-006-401	5A	Jun-94	2	Overlay	10.75	109
6	IM-85-1(111)	6A	Jan-93	4		22.22	238
7	NFH-449(8)	7A	Mar-95	4		21.52	220
	99-307-234-053-502	7B	May-95	4		23.39	237
	1A-003-000-002	7D	Sep-95	2		1.40	14
8	IM-59-1(175)	8A	Aug-92	4	Overlay	16.04	161
	IM-59-1(179)	8C	May-93	4	Overlay	4.62	48
9	IM-65-1(206)	9A	Nov-94	4	New Construction	47.16	480
	DE-0019(802)	9B	Sep-95	4	Overlay	10.92	111

1. kilometer = miles x 1.61

Proscan Consistency

The data set consists of profilograms of 3310 roadway segments 0.1 of a mile or less in length. The 0.1 mile segments are the result of ALDOT standard procedures with the shorter lengths resulting from total project lengths not equaling 0.1 of a mile multiples. The data was obtained from 20 projects performed in nine ALDOT Divisions by various contractors. Each project was completed by different road crews, using different equipment and asphalt mix on different terrain and subbase conditions. The profilograph traces between projects is, therefore, both independent and mutually exclusive. The same consideration can be applied to each 0.1 mile segment within each project. While it is expected that the contractor and possibly the equipment remain the same, variations in the subbase and surface preparations, asphalt mix, and equipment operations and performance can result in different smoothness readings between segments. The smoothness readings between each segment are, therefore independent and mutually exclusive.

The profilograms of each project were accompanied by a manually derived PI. These PIs were developed by ALDOT personnel using a 0.2 in (5 mm) blanking band with the scallops measured to the nearest 0.5 in (1.3 mm) called "resolution". No additional manual readings were obtained because the variability of manual PI reductions was determined from prior studies. These studies indicate that the variability in the manual

PIs for each segment, as measured by the standard deviation, varied from 0.7 to 4.8 in/mi, (18 to '22 mm/km)with an average standard deviation of 2.9 in/mi (74 mm/km)

The five ProScan runs were performed to enable an assessment of the repeatability of the system. The analysis was performed, by project, on the standard deviation of the repeat measurements for each segment. Table 2 summarizes the distribution of the standard deviation for the entire ProScan data base using a 0.2 in (5 mm) blanking band at a resolution of 0.05 in (1.3 mm). ProScan is capable of reducing the data at a number of different blanking bands and resolutions but the blaniking band and resolution/settings used for ProScan match the blanking band and resolution used for manual data extraction in Alabama. The range of the ProScan standard deviation is from a minimum of 0 (2596 observations) to a maximum of 0.38 (6.0 mm/km) (one observation) with 90% of all observations less than a standard deviation of 0.20 (3.2 mm/km).

Table 2 - Summary of ProScan Standard Deviation for Entire Data Base (0.2 in. (3.2 mm/km) Blanking Band with 0.05 in. (1.3 mm) Resolution)

	Percentile			Range	
	50	90	95	Minimum	Maximum
Standard Deviation in/mi [mm/km]	0.0 [0.0]	0.2 [3.2]	0.24 [3.8]	0.0 [0.0]	0.38 [6.0]

Table 3 summarizes the range and average standard deviation of the ProScan readings for each project. The variability exhibited in Table 2 ranges from a minimum of zero to a maximum of 0.38 in/mi (6.0 mm/km). The largest average standard deviation was 0.120 in/mi (1.9 mm/km). These results are considerably lower than the range of 0.7 to 4.8 in/mi (18 to 122 mm/km), and average of 2.9 in/mi (74 mm), determined from manual observations. These comparisons indicate that the ProScan system will provide more consistent results than the manual ratings. Reducing variability has the advantages of:

- Reducing the influence of the ability, experience and subjectivity of the individual performing the profilograph reduction,
- Helping ensure a uniform reduction of profilograph data within and between divisions, and
- Reducing possible contractor complaints pertaining to the perceived experience and inherent accuracy of the individual performing the profilograph reduction.

Five readings of the ProScan system were obtained for this study to enable the evaluation of system consistency. Actual applications of the ProScan system, however, will consist of obtaining only one reading. Determining which of the five readings to use for further analysis in this project yields four alternatives. These alternatives are using a representative reading such as the mean, median, mode or the random selection of one reading.

Table 3 - Analysis of ProScan Consistency by Project (0.2 in. (5 mm)Blanking Band with 0.05 in. (1.3 mm) Resolution)

		Parameters of Five ProScan Readings Parameters Range in/mi [mm/km]		
Project ID	Number of Segments	Minimum	Maximum	Mean
1A	192	0	0.28 [4.4]	0.039 [0.6]
1B	144	0	0.37 [15.8]	0.067 [1.1]
2A	85	0	0.24 [3.8]	0.033 [0.5]
2B	118	0	0.37 [5.8]	0.094 [1.5]
2C	223	0	0.38 [6.0]	0.071 [1.1]
3A	271	0	0.28 [4.4]	0.024 [0.4]
3B	228	0	0.20 [3.2]	0.005 [0.1]
3D	99	0	0.37 [5.8]	0.117 [1.8]
3E	142	0	0.37 [5.8]	0.052 [0.8]
4A	160	0	0.37 [5.8]	0.034 [0.5]
4B	30	0	0.37 [5.8]	0.076 [1.2]
5A	109	0	0.37 [5.8]	0.091 [1.4]
6A	238	0	0.37 [5.8]	0.077 [1.2]
7A	220	0	0.37 [5.8]	0.050 [0.8]
7B	237	0	0.37 [5.8]	0.044 [0.7]
7D	14	0	0.24 [3.8]	0.120 [1.9]
8A	161	0	0.24 [3.8]	0.026 [0.4]
8C	48	0	0.24 [3.8]	0.027 [0.4]
9A	480	0	0.37 [5.8]	0.015 [0.2]
9B	111	0	0.37 [5.8]	0.097 [1.5]

The mean was chosen as the evaluation variable because it provides the best estimate of the expected long term outcome. In addition, comparisons between the mean, median, and mode for the ProScan data base revealed little difference between them. This small difference is evidenced by the small standard deviation that existed in the ProScan readings.

Comparison of Manual and Proscan Readings

Becasue only one measure of ProScan will be obtained it is necessary to determine the type of association between the ProScan and the manual methods. This is necessary because adopting the ProScan system could result in a completely different set of smoothness measurements. If the ProScan measurements are not the same as those obtained in the past then a difference in the incentive payments made to the Contractor will result, necessitating a new pay adjustment scale. The association between ProScan and manual methods was performed by considering the following.

- *Linear Association* - The ProScan and manual methods should exhibit a linear association if the methods are equivalent. For example, consider a hypothetical case where the manual method exhibits uniform fluctuations, and the ProScan method quadratic fluctuations, in their smoothness measures. Such a difference in data distribution would result in drastic differences in manual and PI readings for different data reduction conditions.
- *Similar Trends* - The ProScan and manual methods should exhibit similar trends if the methods are equivalent. If the manual method indicates that a segment has a lower rank than an adjacent segment then the ProScan measures should exhibit the same trend. Measures of smoothness should not be subjective. The measured value between segments may change in magnitude, but the relative ranking between segments should be consistent regardless of the method of measure.
- *Categorical Equality* - The level of approval for pavement smoothness is based on acceptance categories. PI values falling between specified intervals result in different incentive payments. If the manual and ProScan methods result in different categorical equivalents then it will be necessary to determine different incentive thresholds.

Measures of Association Defined

A graphical and statistical comparison of the ProScan and manual PIs was conducted to determine if ProScan provides similar results as the manual method for a wide range of data reduction condition. The graphical analysis consists of scatter plots of the manual versus ProScan smoothness readings for each project. These scatter plots were developed to provide a visual clue of any association between the manual and ProScan readings. They reveal a linear relationship between the manual and ProScan methods since the observations are clustered around a straight line. Constructing a 95% confidence interval around this line indicates the majority of observations are within ± 2.5 % of the average.

Figure 3 is an example of the graph that was constructed for project 1B. Notice that numbers are annotated at the observation points that are outside of the 95% confidence interval. These are the case numbers of the data observation and were investigated to determine their source. In all cases they were due to manual observations and appear to be outliers. With this in mind, the original idea was to remove them from the analysis. Upon further consideration it was determined that the manual readings were the actual readings that were used to determine the contractor incentive payout. Removing the outliers resulted in a smaller confidence band and the migration of other manual readings to the peripheral of the band extent. The outliers of the manual readings were retained in the analyses.

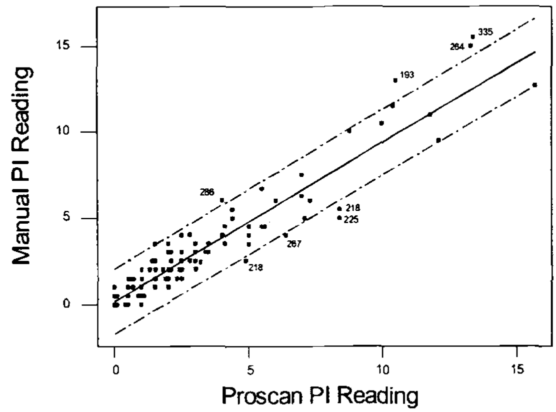

Figure 3 Example of graphical comparison of ProScan and manual PI reading

Regression modeling was performed to determine if the ProScan readings were a dependable predictor of the manual data. The scatter plots indicated that a straight, linear, line provided the best fit. A linear regression was performed, therefore, with the manual readings modeled as the dependent and the mean ProScan readings as the independent variables.

Table 4 presents the regression parameters, and summarizes the statistical measures, of linear association between the manual and average ProScan ratings. The intercept and slope result from a linear regression model between the manual and average ProScan ratings for all of the segments within each of the 20 projects. The intercept is the expected value of the manual rating when the average ProScan rating is equal to zero. The slope is the expected change in the manual rating when the ProScan rating changes by one unit. An exact linear relationship has an intercept of 0 and a slope of 1.

For linear regression the intercept and slope provide the parameters to write the equation of the straight line as the statistical model. For example, the estimated model for project 1B is:

Inches/Mile

$$\text{Manual PI} = 0.133 + 0.940 \text{ ProScan PI} \qquad (1)$$

Millimeters/Kilometer

$$\text{Manual PI} = 1.70 + 0.940 \text{ ProScan PI} \qquad (2)$$

The simple correlation between manual and the mean ProScan readings is provided by the R^2 statistic. The R^2 is often interpreted as the proportion of the total variation in the manual readings accounted for by the mean ProScan readings. If there is no linear relationship between the dependent and independent variable the value of R^2 is 0 or very small. If all of the observations fall on the regression line, R^2 is 1.

The last measure of association between the manual and ProScan readings is the correlation coefficient. This measure is easily interpretable, does not depend upon the units of measurement, and provides an absolute measure of how well the model fits the data. Selecting the correct correlation test however, requires knowledge of how the data is distributed. This was determined by first assuming a normal distribution and then applying the K-S test. The K-S test compares the cumulative distribution of the actual data set with the distribution that would occur if it was normally distributed. The analysis for the manual, and the mean of the ProScan, index profile indexes are summarized in Table 4. Because the data does not exhibit a normal distribution the Spearman rank order correlation test was used to determine if the manual and ProScan readings exhibited similar trends. A negative correlation between the two data sets indicates that an increase in one data set tends to result in a decrease in the other data set. Similarly a positive correlation indicates that an increase in one data set tends to cause an increase in the other.

Summary of Statistical Relationship Tests

The coefficient of determination, R^2, the intercept, and the slope values indicate that the ProScan readings provide good estimates of the manual readings. Not only is a good estimate received but the ProScan method is not subject to the wide variations in measurements between analysis segments exhibited by the manual readings. Table 4 indicates that the (K-S) normality test with the exception of Project 7D, provides no evidence of normality for either the manual or average ProScan ratings. This influences the type of statistical tests and methods that are appropriate for the smoothness data. For example, the absence of normality results in the need to use an ordinal measure of association between the manual and average ProScan ratings. The Spearman correlation coefficient, displayed in the last two columns of Table 4, indicates a significant monotonic relationship between the two variables. This implies that a high (low) ranking with a manual observation tends to occur jointly with a high (low) ranking of the ProScan observation. High and low manual profile readings are, therefore, accompanied by respective high and low mean ProScan readings.

Because neither the manual or ProScan data exhibited normal distribution characteristics, a non-parametric test was used to determine if they were statistically equal. This was accomplished by considering the manual and average ProScan rankings as paired (related) observations for each segment. The results of the non-parametric Wilcoxon paired samples test is summarized in Table 5. The manual and average ProScan ratings were not statistically equal for the majority of projects. Statistical equality was only identified for projects 1B, 3D, 5A, and 8C. In addition, the ProScan readings yield consistently lower PI ratings; as summarized in Table 6. Sufficient information was not available on the characteristics of each project to determine the possible reasons for differences in the tests of statistical difference.

Table 4 - Measures of association between manual and Proscan smoothness readings

LINEAR ASSOCIATION				K-S NORMALITY TEST		TREND TEST	
	Regression	Parameters				Spearman Rank Order	
Proj	Intercept in/mi (mm/km)	Slope in/mi (mm/km)	R^2	Manual	ProScan	Correlation Coeff.	Significant[1]
1A	0.16 (2.52)	0.31 (4.90)	0.91	no	no	0.90	yes
1B	0.11 (1.79)	0.94 (9.94)	0.91	no	no	0.93	yes
2A	0.19 (3.00)	0.81 (12.78)	0.64	no	no	0.78	yes
2B	1.03 (16.25)	1.00 (15.77)	0.84	no	no	0.91	yes
2C	1.48 (23.34)	1.02 (16.09)	0.75	no	no	0.83	yes
3A	0.33 (5.21)	1.11 (17.51)	0.80	no	no	0.79	yes
3B	0.11 (1.79)	1.24 (19.56)	0.67	no	no	0.75	yes
3D	1.06 (16.72)	0.78 (12.30)	0.67	no	no	0.73	yes
3E	1.02 (16.09)	0.88 (13.88)	0.70	no	no	0.86	yes
4A	0.79 (12.46)	0.93 (14.67)	0.53	no	no	0.70	yes
4B	1.92 (30.28)	0.61 (9.62)	0.57	no	no	0.71	yes
5A	0.29 (4.57)	0.91 (14.35)	0.89	no	no	0.93	yes
6A	0.50 (7.89)	1.21 (19.09)	0.76	no	no	0.88	yes
7A	0.67 (10.57)	1.01 (15.93)	0.82	no	no	0.84	yes
7B	0.96 (15.14)	1.17 (18.45)	0.69	no	no	0.75	yes
7D	2.48 (39.12)	0.76 (12.00)	0.67	yes	yes	0.80	yes
8A	0.53 (8.36)	1.09 (17.19)	0.55	no	no	0.71	yes
8C	0.00 (0.00)	1.29 (20.35)	0.89	no	no	0.84	yes
9A	0.47 (7.41)	0.95 (15.00)	0.67	no	no	0.59	yes
9B	0.33 (5.21)	0.76 (12.00)	0.91	no	no	0.96	yes

[1] Level of significance = 0.05

Table 5 - Summary of Wilcoxon Paired Samples Test Between Manual and ProScan

Project	Segments Analyzed	Wilcoxon z value	Statistically Equal[1]
1A	192	6.31	no
1B	144	0.23	yes
2A	85	2.04	no
2B	118	8.47	no
2C	223	12.05	no
3A	271	9.48	no
3B	228	5.99	no
3D	99	0.13	yes
3E	142	7.19	no
4A	160	8.93	no
4B	30	3.79	no
5A	109	1.59	yes
6A	238	9.64	no
7A	220	9.89	no
7B	237	12.21	no
7D	14	3.17	no
8A	161	7.70	no
8C	48	1.81	yes
9A	480	12.77	no
9B	111	5.73	no

[1] Level of significance = 0.05

Table 6 - Summary of differences in manual and ProScan readings by segment

		ProScan less than Manual		ProScan equals Manual		Manual less than ProScan	
Pro ject	Segments Analyzed	Frequency	Percent	Frequency	Percent	Frequency	Percent
1A	192	81	40.5	91	47.4	20	12.1
1B[2]	144	48	33.6	40	28.0	55	38.4[1]
2A	85	24	28.2	46	54.1	15	17.7
2B	118	101	85.6	8	6.8	9	7.6
2C	223	196	87.9	19	8.5	8	3.6
3A	271	132	48.7	127	46.9	12	4.4
3B	228	51	22.4	175	76.8	2	0.8
3D[2]	99	46	46.5	6	6.1	47	47.4[1]
3E	142	95	66.9	26	18.3	21	14.8
4A	160	121	75.6	26	16.3	13	8.1
4B	30	25	83.3	2	6.7	3	10.0
5A[2]	109	37	33.9	14	12.8	58	53.3[1]
6A	238	153	64.3	54	22.7	31	13.0
7A	220	146	66.4	52	23.6	22	10.0
7B	237	194	81.9	37	15.6	6	2.5
7D	14	13	92.9	0	0	1	7.1
8A	161	102	63.4	44	27.3	15	9.3
8C[2]	48	14	29.2	26	54.2	8	16.6
9A	480	238	49.6	223	46.5	19	3.9
9B	111	17	15.3	23	20.7	71	64.0[1]

[1] Manual ranking predominantly less than ProScan ranking
[2] Manual and ProScan statistically equal (See Table 5)

Determination of Pay Adjustment Factors

Two different styles of pay adjustment factors were considered: stepwise payment increments and continuous payment functions. Alabama currently uses a step function with 5% increments. These relatively large increments between payment levels results in the potential for a large payment difference between two borderline segments. For example, consider two segments that do not vary significantly in overall rideability but fall into two different payment ranges. One may be at the low end of the 105% payment range with a PI of 2.5 in/mi (39.4 mm/km) and the other at the high end of the 100% range with a PI of 3.0 in/mi (47.3 mm/km). The PI values in this case are not significantly different or at least not different enough to warrant such a large difference in payment.

- *Smaller Steps* - One solution to this problem is to create adjustment factors with smaller steps (1 or 2%) so that two borderline segments do not receive payments that differ as much as they do with 5% increment steps.

- *Continuous Linear Relationship* - Another solution is to apply a continuous linear relationship between PI values and pay adjustment factors instead of using a step function pay scale. This alternative assigns pay factors that are strictly a function of the PI value instead of creating pay factor ranges that allow for a range of PI values to achieve the same bonus or deduction. Figure 5 shows a graphical representation of the relationship between Pay Factor and PI for this alternative. The problem with this method is that there is only one PI value that will yield a pay factor of 100%. This means that there is no specified acceptance range. Almost all pavement segments on a project will receive either some type of bonus or deduction leaving it impossible for contractors to bid on a project when they know that actual payment will be different. Therefore, this method will not be considered when producing new pay adjustment scales.

- *Combination of Step and Continuous Relationship* - A third alternative is to combine the step function relationship with the continuous function concept by specifying a 100% acceptance range with linear relationships in the bonus and penalty ranges. Figure 6 shows a graphical relationship of this method. This allows for bonuses and penalties to be a function of the PI while still specifying an acceptance range. The advantage of this method is that a pavement section that has a PI value that falls just outside the 100% acceptance range receives only a minor bonus or penalty instead of a large bonus or penalty that it would receive with a step function pay scale. At the same time there is an acceptance region that gives contractors a tolerance range that they can expect to receive full pay for their work.

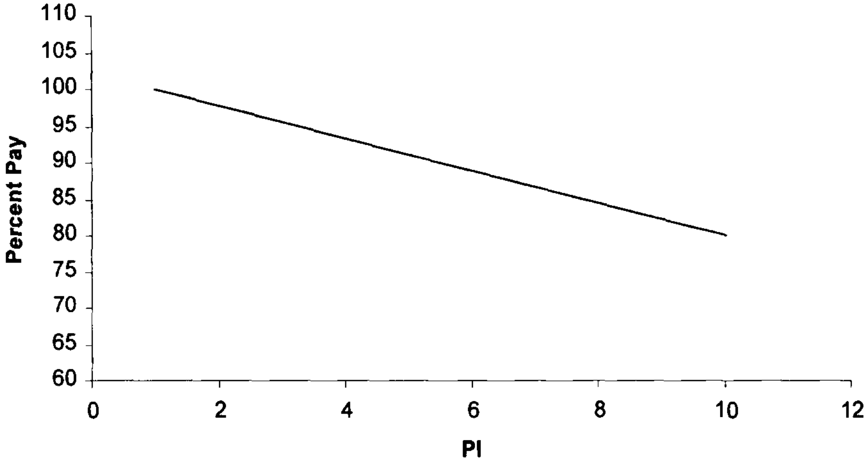

Figure 5 - Continuous Function Relationship Between Pay Factor and PI

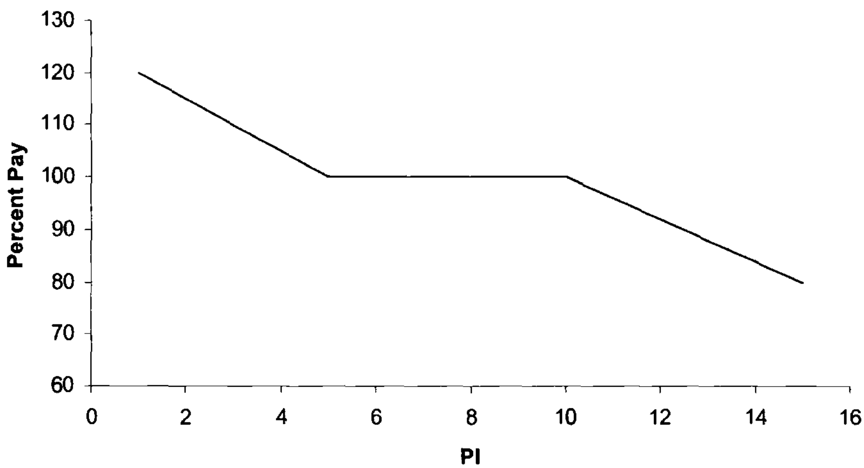

Figure 6 - Combination Continuous Function/Step Function Relationship Between Pay Factor and PI

Current Experience

The experience of ALDOT, and many other States, is that too many pavement segments are currently receiving higher pay factors than warranted. What is required is new pay scales set at levels that reward exceptional pavements. At the same time the pay scale cannot be set so stringently that acceptable pavement segments are penalized. To determine where to set these levels, it is necessary to decide what percentage of pavements should receive bonuses and what percentage of pavements should receive penalties. Ultimately this will be the decision of ALDOT. For the purpose of this research a variety of different pay scale proposals will be presented that will produce different percentages of segments that receive bonuses and penalties.

Percentile Values

Percentile values were used to assist in determining the bonus and penalty ranges. Percentiles separate data sets into 100 equal parts, and represent a number such that it separates the highest percent from the bottom percent. For example, the 85th percentile is the number from the data set that 15% of the observations are greater than and 85% are less than it. The value to set the bonus range, therefore, can be determined by finding the PI value that corresponded to the percentage of segments that are to receive bonus payments. For this research, it was necessary to determine which PI values correspond to a variety of different percentiles, because the percentages to use for determining bonus and penalty ranges were not specified. The identification of natural break points were used to identify percentile levels for use as bonus and penalty range values.

Formulation of New Pay Factors

The new bonuses and penalties for contractor pay were determined through an examination of 330 lane miles of profile indices analyzed in this project. Manual reductions, calculated and provided by ALDOT, were analyzed along with the computerized reductions performed by ProScan based on the ALDOT specifications of a 0.2 in (5mm) blanking band and a scallop resolution of 0.05 in (1.3 mm). PI values were reported using the manual method for each 0.1 lane mile (0.16). The first step in identifying data outliers was to determine the mean and standard deviation for each lane for each project. An allowable range of the mean plus two standard deviations for each lane of each project was calculated, and then values exceeding this upper limit were removed. This process was repeated until no outliers could be identified. Typically a lower limit would also be calculated, however, in this case the mean was only one standard deviation above the 0.0 PI value (for 0.2 in (5mm) blanking band). This same approach was used to identify outliers in the ProScan data base.

Suggestions for the revised smoothness specification are based on several assumptions. First, smoothness values statistically greater than the normal (average) ALDOT hot mix asphalt (HMA) pavement smoothness indicate that an incentive is warranted. This limit is set at one standard deviation (2.20 in/mi (34.7 mm/km) PI) above the grand average manual method smoothness value of 1.90 in/mi (30 mm/km) PI. This sets the risk for the agency at about 15% for paying an incentive for a standard smoothness. Since 1.90 minus 2.20 in/mi (30 - 34.7 mm/km) would be a negative number, the lowest value reported for this test, using the 0.2 inch (5 mm) blanking band, requires a value of 0.0 to indicate the extra quality.

Second, the mean Alabama HMA smoothness values of 1.90 in/mi (30 mm/km) (manual method) and 1.50 in/mi (27.7 mm/km) (ProScan) were averaged to obtain a value of 1.70 in/mi (26.8 mm/km). Because the standard deviation for both methods was the same, 2.20 in/mi (34.7 mm/km) PI was added to this value to obtain a value of 3.90 in/mi (61.5 mm/km) PI. This value represents a seller's risk of about 15% of having a pay adjustment assessed to an acceptable HMA smoothness.

Lastly, subsequent pay factor percentages and increments were kept the same. Considerably more information as to the initial PI and subsequent loss of rideability is needed before these percentages can be adjusted. The pay factors initially invisualized during the research and the current values are summarized in Table 7.

Table 7 - Schedule of initial research and existing PI values and corresponding price adjustment

Contract Price Adjustment Percent of Pavement Unit Bid Price	Research inches/mile/section (millimeters/kilometers/section)	Existing Profile Index inches/mile/section (millimeters/kilometer/section)
105%	0.0	Under 3.0 (47.3)
100%	0.1 - 3.9 (1.6 - 61.5)	3.0 - 6.0 (47.3 - 94.6)
95%	4.0 - 5.9 (63.1 - 93.1)	6.0 - 8.0 (94.6 - 126.2)
90%	6.0 - 7.9 (94.6 - 124.6)	8.0 - 10.0 (126.2 - 157.7)
Unacceptable	Over 7.9 (124.6)	Over 10.0 (157.7)

Effect of New Pay Factor Adjustment

New pay factors were developed and applied to the profile indexes for each section of this study to obtain the resulting pay adjustment. With the adjustment, it was determined that ALDOT would have paid only 96.8% of the bid price for the paving projects that brought 102% pay under the old step-wise pay function. The majority of the adjustment occurred in the bonus range of the pay scale with a limited number of sections migrating into the penalty range, as shown in Figure 7.

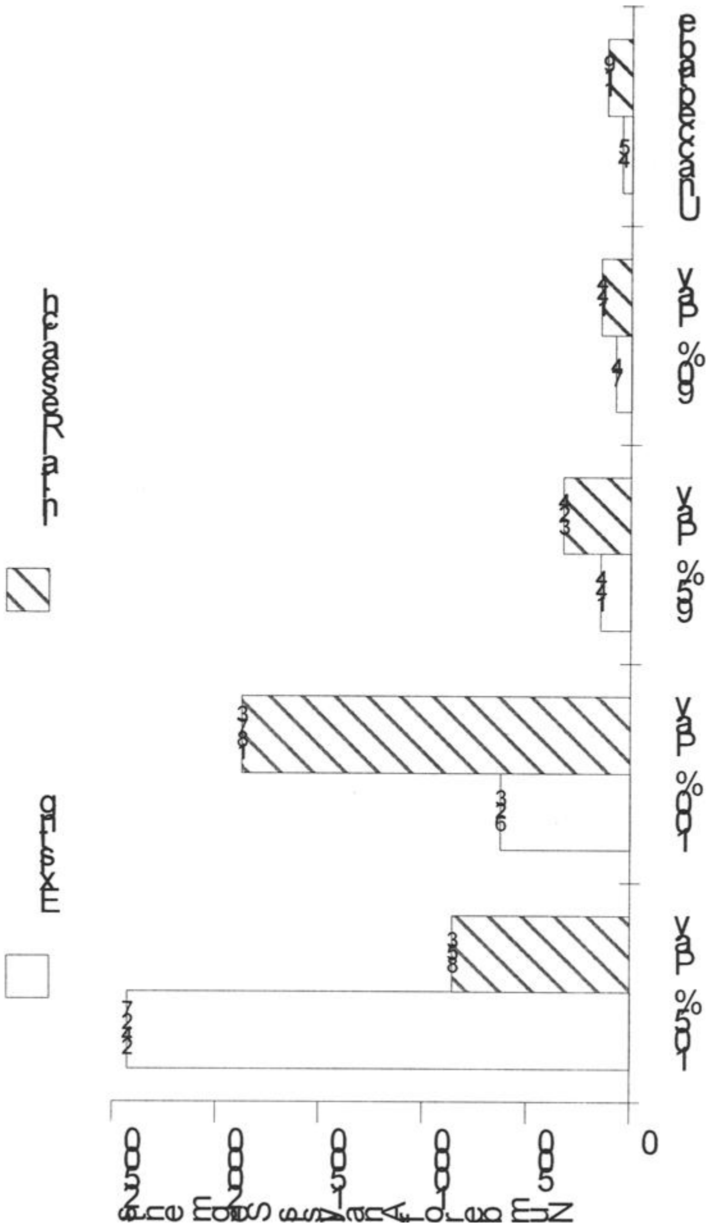

Figure 7 - Comparison of performance incentives between existing and initial pay factors

The lowest PI value that can be attained with the 0.2 in (5 mm) blanking band is 0.0. This eliminates the ability to achieve the desired linear/stepwise combination function for the new pay scale. It is recommended that a null, or 0.0 blanking band be used in order to eliminate this problem. Figure 8 was generated using data from the NCAT test track in Opelika, AL. It shows that the 0.2 in/mi (232 mm/km) blanking band is well correlated with the 0.0 blanking band. Using the correlation equation, a new specification can be developed that will allow for a graduated pay scale in the incentive range, creating the linear/stepwise combination. The y-intercept from the line for the 0.0 blanking band (approximately 14.7) becomes the highest PI value for the bonus range. A PI just greater (14.8in/mi (233.4 mm/km) is the lowest value in the100% pay range. This range is the middle step of the suggested pay function presented as Figure 9. The remaining values of the suggested pay factors for a 0.0 blanking band are found below in Table 8.

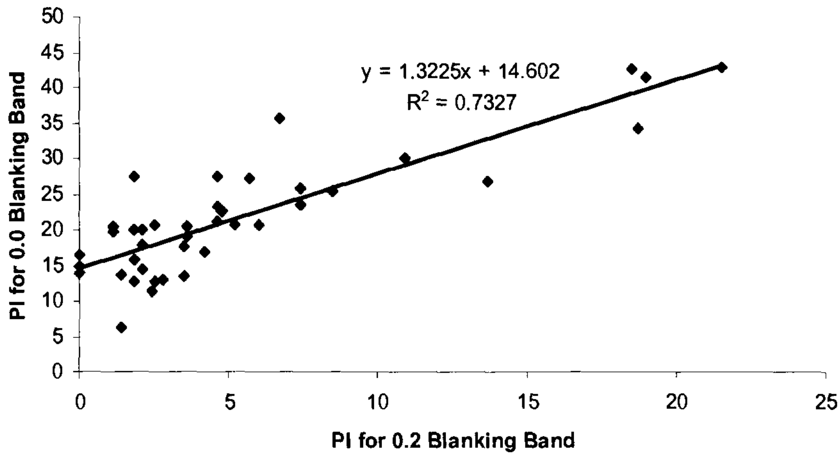

Figure 8 -Formulation of 0.0 blanking band pay function

Table 8 - Schedule of proposed PI values and corresponding price adjustment

Contract Price Adjustment of Pavement Unit Bid Price	Proposed Profile Index inches/mile/section (millimeters/kilometer/section)
Bonus 100% to 105% max	0.0 to 14.7 (0.0 to 232.0)
100%	14.8 to 20.0 (233.0 to 316.0)
Penalty 90% to 100%	20.1 to 26.0 (317.0 to 410.0)
Unacceptable	greater than 26.0 (410.0)

The suggested pay function, presented as Figure 9, developed from Table 8 becomes the following.

Inches/mile

Bonus:

$$\text{Percent Pay} = (-1.667 * \text{PI}) + 124.5 \tag{3}$$

Penalty:

$$\text{Percent Pay} = (-1.667 * \text{PI}) + 133.33 \tag{4}$$

Millimeters/kilometer

Bonus:

$$\text{Percent Pay} = (-0.1057 * \text{PI}) + 124.5 \tag{5}$$

Penalty:

$$\text{Percent Pay} = (-0.1057 * \text{PI}) + 133.33 \tag{6}$$

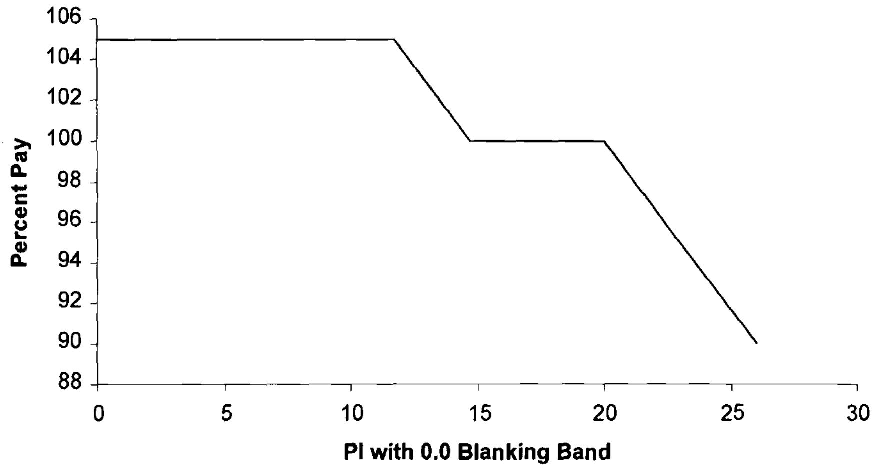

Figure 9 - Suggested pay function with 0.0 blanking band

Examples of Pay Function Application

Example 1: Inches/mile

Bonus
PI = 13.0; Percent Pay = (-1.667*13) + 124.5 = 102%
Penalty
PI = 23.5; Percent Pay = (-1.667*23.5) + 133.33 = 94%

Example 2: Millimeters/kilometer

Bonus
PI = 205; Percent Pay = (-0.1057*205) + 124.5 = 102.8%
Penalty
PI = 371; Percent Pay = (-0.1057*371) + 133.33 = 94.1%

Notice that the slope of the bonus and penalty portions of the incentive diagram are equal. The result is that the same monetary increment is provided as a bonus or penalty for a unit increase or decrease in the PI value.

Summary of Conclusions and Recommendations

1. The ProScan system provides a more reliable and faster method of determining the PI than manual data extraction. The reduced variability of the Proscan readings has the advantages of
   - Reducing the influence of the experience and subjectivity of the individual performing the profilograph reduction,
   - Helping ensure a uniform reduction of profilograph data within, and between ALDOT divisions, and
   - Reducing possible contractor complaints pertaining to the accuracy of the profilograph readings.
2. The ProScan ratings, while exhibiting less variability, yield consistently lower PI ratings than data extracted manually.
3. The pay factors developed during this research effort were applied to the project segments. This analysis revealed that ALDOT would have paid only 96.8% of the pavement bid price with the proposed pay factors. These same analysis segments earned the contractors 102% of the pavement bid price under the old step wise pay function.

4. ALDOT is currently using the California type profilograph to measure pavement smoothness. This project was intended to ascertain the reliability of the ProScan data extraction system and to develop an equitable contract price adjustment factor. These methods are intended to stay in place while ALDOT identifies and makes the transition to more reliable and faster technologies, such as laser-based methods, currently available.

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Asphalt Concrete Smoothness Incentive Results by Highway Type and Design Strategy

Reference: Delton, J., Li, Y., and Johnson E., "**Asphalt Concrete Smoothness Incentive Results by Highway Type and Design Strategy,**" *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433*, M. S. Gardiner, Ed., American Society for Testing and Materials International, West Conshohocken, PA, 2003.

Abstract: The Arizona Department of Transportation (ADOT) implemented an incentive/disincentive asphalt concrete (AC) smoothness specification in 1990. Since then hundreds of projects have been tested for smoothness. These projects have included a wide variety of layer combinations of one or more of the following: overlay, remove, replace, and finishing course. The number of projects and variation in design allows comparisons of the smoothness results for different design strategies as well as trends in smoothness results over time. In addition to the tests on the final surface, many projects were also tested on intermediate lift surfaces.

A statistical analysis of the smoothness data is conducted to study the smoothness distribution and to compare the pavement smoothness of different categories. The smoothness correlation between various layers is also studied. The economic benefits of the implementation of ADOT's smoothness specification are evaluated. The results of this study can be useful in establishing target levels for newly implemented or revised pavement smoothness specifications.

Keywords: pavement smoothness test, smoothness specification, incentive, disincentive, smoothness distribution, cost and benefit

Introduction

The initial smoothness of the pavement immediately after construction is a key component to a smooth-riding roadway during its life cycle. First, initial smoothness of pavement is usually an indicator of the overall quality of construction. If the pavement is constructed with a very smooth surface, there is a greater likelihood that the contractor has provided good quality workmanship in many other aspects of construction. In addition, initial pavement smoothness affects pavement long-term performance. It has also been shown that initial pavement smoothness measurements are highly correlated with smoothness measurement made 10 years after construction [*1*].

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Achieving a higher level of initial smoothness on highways during construction results in longer highway life, smooth-riding pavements during its life cycle, and savings to the taxpayer due to reduced wear and tear on vehicles. Therefore, since the early 1990s many highway agencies have developed and implemented initial smoothness based incentive/disincentive provisions in their pavement construction specifications to motivate the contractor to provide a high level of smoothness quality [2].

To develop and implement an effective smoothness specification is a challenge to many highway agencies. This process involves both the highway agencies and contractors. Highway agencies always desire that contractors produce as smooth a pavement as possible. However, achieving high smoothness levels requires extra effort by the contractor during the construction process. More accurate paving equipment may be required. People in business for a profit are less likely to make that effort without a monetary incentive. Therefore, a smoothness-based specification that would set a goal for smoothness and then pay the contractors extra money - above the contract amount for meeting that goal - would be necessary. One key problem associated with developing such a incentive-based specification is finding the balance for an incentive amount that is large enough to make it appealing to the contractor and yet, not so large that the agency pays more incentive than they gain in benefit. Another challenge is dealing with the perception that a state agency is giving away money. Because of a lack of hard data, many highway agencies are hesitant to move forward unless the benefits to themselves and the public can be demonstrated. To answer these challenges and ultimately develop an effective and reliable pavement smoothness specification, it is critically important to analyze statistically the historical smoothness data from the states that have applied such a specification for many years.

The Arizona Department of Transportation (ADOT) has been using smoothness based incentive/disincentive specifications since 1990. Since that time hundreds of projects have been tested for smoothness. These specifications cover new construction pavement projects and various rehabilitation projects. These projects have included a wide range of layer combinations of one or more of the following: overlay, remove, replace and finishing course. The well-documented smoothness data for the large number of the projects and variation in design provide a sufficient database to evaluate the benefits of the smoothness specification. The result of this analysis provides a solid statistical base for an improved ADOT smoothness specification.

Objectives of Paper

This main objective of this paper is to describe the results of ADOT's 10-year use of an incentive-based specification for pavement smoothness and its effect on pavement construction smoothness. A statistical analysis of the smoothness data (years 1992 through 2000) is conducted to investigate the smoothness distribution and to compare the pavement smoothness of projects in different categories. The smoothness correlation between various layers and factors affecting the pavement construction smoothness are also studied. In addition, the economic benefits of the implementation of ADOT's smoothness specification are evaluated.

ADOT Smoothness Specifications

Based on a study, a K.J. Law profiler was selected as the smoothness measuring equipment for the ADOT smoothness specification [*3*]. The K.J. Law profiler is a high-speed inertial profiler and is an ASTM Class I profile measurement device. The profiler measures and records Class I pavement profiles in each wheel path with two infrared height sensors at the wheel path positions. The smoothness is represented in Mays Ride value, which is a roughness index similar to the International Roughness Index (IRI) [*4*]. The less the Mays Ride value the smoother the pavement. The accuracy and repeatability of the measurement of smoothness by ADOT's K.J. Law^{TM} profiler is high. The standard deviation of the measurements of Mays values on a section is less than 0.003 m/km.

The Mays values for each 0.16 km are used to determine the incentive or disincentive for that length. The incentive /disincentive for the project is the sum of the incentive/disincentive for every 0.16 km within the project.

Although the incentive/disincentive is determined by the measured smoothness on the final layer, the smoothness on the old pavement surface and other intermediate layers has also been measured. Thus, a complete set of smoothness measurements for a project often includes the smoothness data per 0.16 km of each layer for every lane.

The first smoothness specification was implemented in 1992. A revised specification was developed with the involvement of contractors in 1996 and has been used since then. The incentive/disincentive formulas are

$$\textit{Incentive Value} = [(IV - AS)/(IV + 0.032)] * COEF \tag{1}$$

$$\textit{Disincentive Value} = [(DV - AS)/(IV + 0.032)] * 1000 \tag{2}$$

where, *IV* and *DV* are the thresholds (window values) of Mays value for incentive and disincentive, respectively. *AS* is the measured Mays value of the finished layer. *COEF* is a parameter relating the measured Mays value to the amount of incentive or disincentive in dollars. The values of *COEF* for the two specifications are shown in Tables 1 and 2, respectively. In both specifications, incentive/disincentive rates are determined based on the opportunities for leveling and road classes. An opportunity for leveling consists of each instance of the following: milling of existing surface, placement of a lift of AC and placement of a friction course. In the 1992 specification, three categories were classified with different incentive or disincentive rate. In the 1996's revised specification eight categories were classified.

Table 1 – *Parameters in ADOT's 1992 Smoothness Specification*

		IV, m/km	*DV*, m/km	*COEF*, $
Interstate		0.576	0.768	2000
Non-Interstate	Overlay with ACFC	0.832	1.024	2000
	Overlay without ACFC	0.960	1.152	2000

Analysis of Smoothness Data

In this analysis, the total of 194 projects from 1992 to 2000 are grouped into the eight categories. The majority are divided and non-divided highway projects with at least two leveling opportunities. The number of divided projects with at least two opportunities is 77 while that of non-divided projects is 72. All of these projects with at least two leveling opportunities have asphalt concrete friction course (ACFC) or asphalt rubber concrete friction course (ARACFC). The numbers of projects for category 2, 5, 6 and 7 are 3, 18, 20 and 4, respectively. The only opportunity for leveling in category 2 is ACFC/ARACFC. There are no projects for category 4. Since the incentive or disincentive is determined for every one-tenth mile on each lane, one tenth of mile on each lane is treated as one sample in this smoothness analysis.

Table 2 – *Parameters in ADOT's 1996 Smoothness Specification*

Category	IV, m/km	DV, m/km	COEF, $
1, Divided, at least two leveling opportunities	0.528	0.720	2500
2, Divided, one leveling opportunity	0.608	0.800	2500
3, Non-divided, at least two leveling opportunities with ACFC	0.608	0.800	2500
4, Non-divided, at least two leveling opportunities without ACFC	0.688	0.880	2500
5, Non-divided, ACFC	0.848	1.040	2500
6, Non-divided, AC	0.848	1.040	2500
7, Non-divided, at least two leveling opportunities with ACFC, new construction	0.608	0.800	2500
8, Non-divided, at least two leveling opportunities without ACFC, new construction	0.688	0.880	2500

Smoothness Comparison between Standard ACFC and ARACFC

A statistical *t*-test was made for interstate highway projects in category 1 to determine if there is a significant difference in smoothness between the two types of pavement surfaces. To eliminate the potential effect of the contractors' improvement on project smoothness over time on the comparison, only projects built in years 1994 and 1995 were included in this test. For the projects with ACFC, the average smoothness value of every lane prior to ACFC surfacing is greater than 0.88 m/km. To eliminate the potential effect of the existing pavement smoothness on the smoothness of following friction courses, those projects with ARACFC lane for which existing smoothness value was less than 0.88 m/km were excluded from the comparison.

The *t*-test shows that at a 95% significance level there is no significant difference in smoothness between a standard ACFC and ARACFC. Therefore, the effect of the difference between ACFC and ARACFC can be ignored when the specification is developed. Samples of the two types of pavement projects were pooled together for the further analysis.

Relationships between Smoothness of Layers

Two cases were analyzed to investigate the effect of the smoothness of previous layer on that of the following layer. The first case is that the following layer is 12.7 mm ACFC/ARACFC, which is always the final course. The other case is that the following layer is a 50.8 mm to 101.6 mm inlay AC (removed and replaced AC) or AC overlay. To eliminate the potential effect of project categories and rehabilitation strategies, the analysis was conducted for each category.

For each category, a linear regression analysis was conducted relating the smoothness of current layer to the smoothness of old or new AC layer immediately before the current layer in the following general form

$$R_{current} = a + b\, R_{previous} \tag{3}$$

where $R_{current}$ represents the smoothness of the current layer in question; $R_{previous}$ is the smoothness of old pavement or new AC layer immediately before the current layer. $\boldsymbol{a}$ and $\boldsymbol{b}$ are regression coefficients.

The results of the linear regression analysis provide the regression coefficients, $\boldsymbol{a}$ and $\boldsymbol{b}$, and information on the significance of the independent variable, $R_{previous}$ on the dependent variable, $R_{current}$. Coefficient $\boldsymbol{b}$ reflects the magnitude of how the smoothness of the old pavement or previous AC layer affects the smoothness of the current AC layer. If this constant is approximately 1.0, this indicates that there is strong one-to one relation between the smoothness of the old pavement or previous AC layer and that of the current AC layer. This means that if one old pavement is 0.08 m/km smoother than another, then the smoothness of a new AC layer over that pavement will remain 0.08 m/km smoother than that of a new AC layer on the other pavement. If the regression coefficient $\boldsymbol{b}$ is approximately zero, this shows that the smoothness of the current AC layer is not affected at all by the previous layer.

The regression analysis also provides tests for the statistical significance of the regression coefficient $\boldsymbol{b}$. The statistical significance of $\boldsymbol{b}$ is evaluated using $\boldsymbol{p}$-value, which shows the probability that the significance of the effect of the independent variable, the smoothness of the previous layer or old pavement, on the dependent variable, the smoothness of the current AC layer, is due to chance alone. Obviously, the smaller $\boldsymbol{p}$-value, the stronger the indication that the smoothness of the previous layer or old pavement has a truly significant effect on that of the current AC layer. For this evaluation, a significance level of 0.1 was selected. This means that if the $\boldsymbol{p}$-value of the regression coefficient $\boldsymbol{b}$ is less than 0.1, the results are considered significant.

The analysis results of case one (Table 3) shows that $\boldsymbol{b}$ value is around zero, indicating there is no relationship between the smoothness of AC overlay or inlay AC and that of its previous layer. Thus, it can be concluded that the smoothness of AC layers is not affected by the smoothness of its previous layer. For case two (Table 4), the $\boldsymbol{p}$-value is zero for the all analyzed categories, demonstrating that the smoothness of the previous layer or old pavement truly has an effect on that of the finished ACFC/ARACFC course. The magnitude of regression coefficient $\boldsymbol{b}$ varies from 0.226 to 0.298 for categories 1, 2, 3, 5 and 7 with an average of 0.260. This means that on average if one old pavement is 0.08

m/km smoother than another, then the smoothness of a new AC layer over that pavement will be 0.021 m/km smoother than that of a new AC layer on the other pavement. This degree of the effect of the smoothness of the previous layer or old pavement on that of the finished ACFC/ARACFC course deserves a consideration in the development of a smoothness specification.

The R^2 of the linear regression equation is small, varying from 0.200 to 0.432 for categories 1, 2, 3, 5 and 7. This suggests that the smoothness of the finished ACFC/ARACFC course also heavily depends on other factors in addition to the smoothness of the previous layer or old pavement.

Table 3 -- *Linear Regression Analysis Results of Smoothness of New AC or Inlay Layer vs its Previous Layer or Old Pavement*

Category	Layers	No. of Samples	R^2	a, m/km	b	p-Value
1	Old Pavement - Inlay AC	5540	0.0004	0.726	0.009	0.157
1	Inlay AC - New AC	2498	0.0001	0.778	-0.004	0.644
3	Old Pavement - Inlay AC	1103	0.0005	0.873	-0.011	0.443
3	Old Pavement - New AC	4077	0.0119	0.872	-0.048	0.000
6	Old Pavement - New AC	1327	0.0115	0.809	-0.057	0.000

Table 4 -- *Linear Regression Analysis Results of Smoothness of ACFC/ARACFC vs its Previous Layer or Old Pavement*

Category	Layers	No. of Samples	R^2	a, m/km	b	p-Value
1	ACFC - New AC	8999	0.3348	15.003	0.298	0.000
2	ACFC - Old Pavement	410	0.4321	13.114	0.261	0.000
3	Inlay AC - ACFC	1075	0.2006	21.720	0.252	0.000
5	ACFC - Old Pavement	410	0.4321	13.114	0.261	0.000
7	New AC - ACFC	473	0.3198	18.041	0.226	0.000

Smoothness Distribution and Change with Years

The smoothness distribution is evaluated per individual years when the years have large number of samples. For the years that do not have enough projects, the projects in two or three sequential years are combined for the analysis. However, the projects in and before 1996 when the revised specification was implemented are not combined with the projects after 1996.

The histograms are plotted for investigating the distribution model of smoothness. The histograms show that the smoothness in years 1994 and 1996 approximately follow normal distributions. With the increase of years, not only the smoothness range that has the maximum frequency shifts significantly toward smaller value, but also the shape of the histogram gets more unbalanced, showing the smoothness does not follow normal distribution any more. In these cases, the pattern of the histograms clearly shows the features of Lognormal distribution (Figure 1).

The Lognormal P-P plot of smoothness was conducted for each year. P-P plot presents a variable's cumulative proportions against the cumulative proportions of the test distribution model. The P-P plots determine whether the distribution of a variable matches the given distribution. The Lognormal P-P plots show that for most of the categories, the smoothness matches Lognormal distribution extremely well. The estimated parameters for the Lognormal distribution model are presented (Table 5), with smoothness average, standard deviation and coefficient of variance for each category.

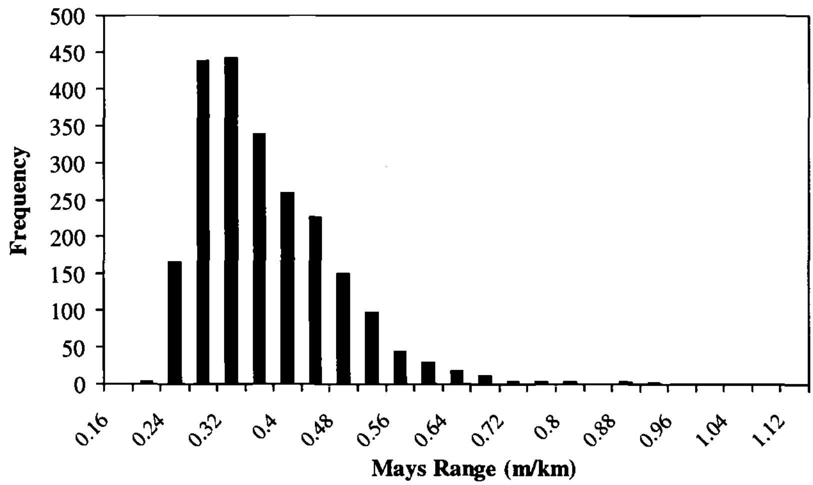

Figure 1 - *Smoothness Histogram of Projects in Category 1, Year 2000*

Figure 2 shows that the average smoothness value of category 1 projects reduces significantly with years except in year 1997 from 0.547 m/km in year 1995 to 0.349 m/km in year 2000. For category 3, the average smoothness value decreases sharply from year 1994 to 1996. Year 1997 has seen a significant increase in smoothness value and after that year the smoothness value continues to decrease. The average smoothness value of category 6 reduces dramatically from year 1998 to 2000. The significant increase in smoothness value from year 1996 to 1997 for categories 1 and 3 may reflect the disturbing effect of the implementation of the revised specification started in 1996. The standard deviation does not vary significantly with years after year 1996 for all three categories even though the standard deviation of categories 3 and 6 is higher than that in category 1.

Different from the results of categories 1, 3 and 6, the average smoothness value of the projects in categories 5 and 7 increases with years. It should be noted that the projects in categories 5 and 7 are very limited.

Table 5 – *Summary of Smoothness Parameters for All Categories*

		Category 1							Category 2	Category 3						
Year		1994	1995	1996	1997	1998	1999	2000	98,99& 2000	1994	1995	1996	1997	1998	1999	2000
No. of 0.16 km		1979	2812	3406	1593	2658	7239	2240	537	291	858	858	791	2012	3430	3900
Mean (m/km)		0.54	0.55	0.49	0.54	0.43	0.37	0.35	0.61	0.85	0.60	0.52	0.65	0.53	0.51	0.50
Stdev. (m/km)		0.11	0.15	0.10	0.10	0.11	0.09	0.10	0.19	0.21	0.11	0.15	0.15	0.16	0.16	0.15
Coef. of Variance		0.21	0.27	0.20	0.19	0.26	0.25	0.28	0.31	0.24	0.19	0.28	0.23	0.30	0.31	0.30
Log-normal Distrib.	Scale (m/km)	0.53	0.53	0.48	0.53	0.41	0.36	0.34	0.58	0.83	0.59	0.51	0.63	0.51	0.49	0.48
	Shape	0.20	0.26	0.18	0.19	0.24	0.24	0.26	0.31	0.24	0.18	0.25	0.22	0.27	0.30	0.29

		Category 5		Category 6				Category 7			Category 8
Year		94& 95	2000	94&95&96	97&98	1999	2000	94&95&96	97&98	2000	94&95&96
No. of 0.16 km		152	410	338	719	516	475	722	802	325	332
Mean (m/km)		0.59	0.72	0.82	0.91	0.73	0.54	0.53	0.59	0.59	0.78
Stdev. (m/km)		0.13	0.22	0.15	0.17	0.17	0.14	0.14	0.20	0.18	0.18
Coef. of Variance		0.22	0.31	0.18	0.19	0.23	0.26	0.27	0.35	0.31	0.23
Lognormal Distrib.	Scale (m/km)	0.57	0.68	0.81	0.90	0.71	0.52	0.52	0.51	0.61	0.76
	Shape	0.24	0.32	0.18	0.19	0.22	0.24	0.29	0.29	0.27	0.21

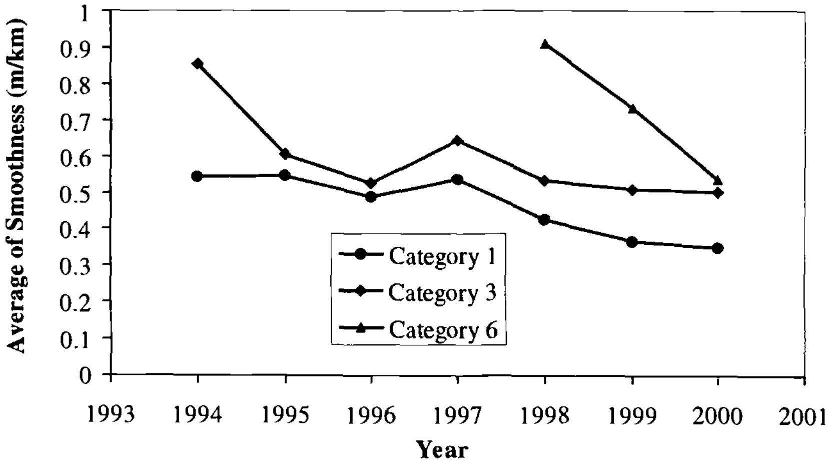

Figure 2 – *Average Smoothness vs Year*

Theoretically, new construction projects should have more opportunities for contractors to achieve smoother pavements. However, the results of finished projects do not support this. Compared to the smoothness of projects completed before 1997, the smoothness value in the recent years even increase 0.05 m/km. The causes are not clear.

Comparison of the Smoothness of Projects among 6 Categories

The *t*-test was used to test the significance of the effect of highway types (divided or non-divided) and the number of leveling opportunities on the pavement smoothness. The following hypotheses were tested

(1) $H_0: S_1 = S_3$, $H_1: S_3 > S_1$;
(2) $H_0: S_3 = S_7$, $H_1: S_7 > S_3$;
(3) $H_0: S_2 = S_5$, $H_1: S_5 > S_2$;
(4) $H_0: S_3 = S_6$, $H_1: S_6 > S_3$;
(5) $H_0: S_5 = S_6$, $H_1: S_5 > S_6$.

where S_1 , S_2 , S_3 , S_5 , S_6 and S_7 are the means of smoothness of categories 1, 2, 3, 5, 6 and 7, respectively. The comparison was conducted for each year or two or three years period. For the comparison where one side includes multi-year periods and the other side includes individual years, the samples in the individual years were combined into the same period of years as the other category. The 95% significance level was used in the *t*-tests. The smoothness variances of the two compared categories were assumed unequal. The *t*-tests results show that all H_0 hypotheses are rejected.

Test (1) is designed to show the effect of road class (interstate or non-interstate) on the contractor's quality of work. At a 95% significant level, in most years, the *t*-statistics are significantly greater than the t-critical, showing the smoothness of the projects on the two classes of roads are significantly different. In the current ADOT specification, the difference in target value for these two categories is assumed as 0.08 m/km. Then, the hypothesis: H_0: $S_3 = S_1 + 0.08$, H_1: $S_3 \neq S_1 + 0.08$, was also tested showing that the difference between S_3 and $S_1 + 0.08$ is still significant especially for the projects in years 1999 and 2000 where the *t*-statistics are significantly greater than the *t*-critical.

Test (2) is designed to compare the non-divided highway rehabilitation projects with two or more leveling opportunities and new construction projects. It is usually assumed that it is easier for the contractors to achieve smoother pavements in new construction projects. In the current ADOT specification, the target smoothness values for the two categories are set the same. However, the results of the finished projects show that at the 95% significance level, the difference between the two is significant. The average smoothness for the new construction projects is even higher than that of rehabilitation projects.

The test (3) is to compare the smoothness of ACFC/ARACFC on old divided and non-divided highways. The smoothness difference between two categories is significant, with *t*-statistics of 7.58. However, the current ADOT specification assumes a difference in target value between two cases as large as 0.24 m/km. The t-test on the hypothesis: H_0: $S_5 = S_2 + 0.24$, H_1: $S_5 > S_2 + 0.24$, results in a *t*-statistic of –9.94 showing that the difference in target value between the two categories adopted in the specification is large.

The test (4) is designed to show the significance of the effect of the number of leveling opportunities on the pavement smoothness for the non-interstate projects. At a 95% significance level, there is a significant difference in smoothness between the projects with only AC overlay and those with two leveling opportunities. However, the results of years 1999 and 2000 projects show that the 0.24 m/km difference in target values for the two categories is too large for the recent years' projects.

In the current ADOT specification, either ACFC/ARACFC only or AC only are considered as one leveling opportunity and given the same target values. However, *t*-test (5) shows that the smoothness difference between the two types of jobs in the year of 2000 is significantly large with *t*-statistics of 14.31. The smoothness value of ACFC/ARACFC is 0.181 m/km larger than that of AC overlay.

Smoothness Variance by Projects and Contractors

Only the projects in categories 1 and 3 were included in this analysis because other categories do not have enough number projects. Standard deviation of project average smoothness reflects the variability in overall smoothness among projects. As Figure 3 shows, the variability of work quality among projects does not decrease with years as expected.

Table 6 shows the averages and standard deviations of smoothness of projects in categories 1 and 3 for several contractors. Other categories are not included because they do not have enough number of projects in individual contractors. The variability of smoothness of different contractors' is significant. The project smoothness of the best contractor, G, is 33% better than that of the worst contractor, C in category 1, which includes the non-divided highway projects with two or more leveling opportunities. In category 3 contractor G is also the best, with project smoothness value 12% less than that of the worst contractor. The smoothness variability of contractors G's projects is also relatively small. Possibly as the result of contractor G's good quality giving a bidding advantages, 42% of category 1 projects and 78% of category 3 project contracts were awarded to contractor G.

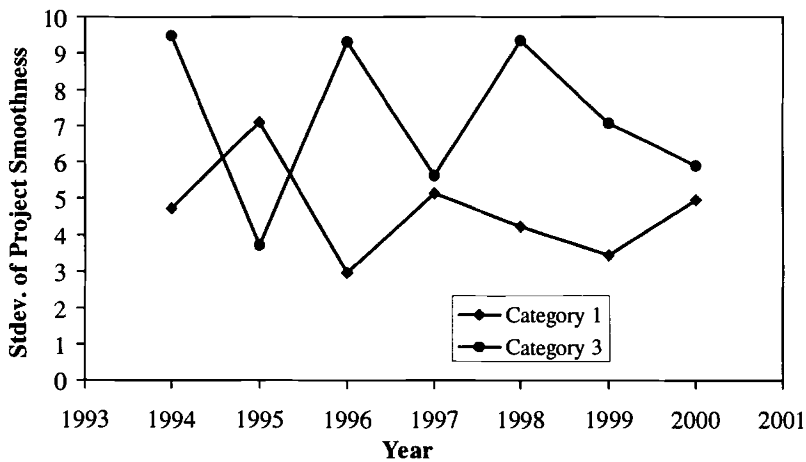

Figure 3 – *Variability of Average Smoothness among Projects*

Table 6 – *Smoothness Comparison by Contractors*

Category	Contractor Code	No. of Projects	Length (0.16 km)	Average (m/km)	Standard Deviation
1. Non-divided-Two or More Opportunities	A	2	530	0.540	0.105
	B	3	456	0.492	0.094
	C	4	532	0.649	0.141
	D	2	503	0.546	0.143
	E	4	694	0.539	0.157
	F	7	1404	0.438	0.153
	G	23	2713	0.432	0.125
	H	8	911	0.537	0.115
	I	2	545	0.530	0.124
3. Divided-Two or More Opportunities	F	6	1574	0.394	0.073
	G	28	6772	0.346	0.094
	J	2	596	0.391	0.095

Cost and Benefit of Pavement Smoothness

As a result of the smoothness specification the typical smoothness on Interstate AC pavements has improved approximately 35%. To achieve that, ADOT pays a bonus (or charges a penalty) in accordance with the formulas (1) and (2).

The essential features of the formulas are that there is about 0.2 m/km window that spans smoothness values between 0.528 m/km and 0.72 m/km. If the contractor works within that window neither bonus nor penalties occur. If the contractor achieves a smoothness better than the low end of the window a bonus is earned. For example, if a smoothness of 0.4 m/km is achieved a bonus of $570 is earned for that 0.16 lane-km. If a 0.64 m/km is achieved there is neither bonus nor penalty. If a 0.88 m/km is achieved a penalty of approximately $290 is incurred. As the formulas show, the bonus side rises faster than the penalty side. The primary reason this formulation was used is that it becomes quickly apparent to contractors that working on the smooth side is where significant bonuses can be achieved. It is also the area, however, where incremental smoothness improvements become progressively more difficult.

Figure 4 shows a comparison of the cost of the smoothness specification on an average per square meter basis. As is shown on the chart, there is a modest difference between the cost per square meter of including the smoothness specification whether it is on a new construction project or on a rehabilitation project.

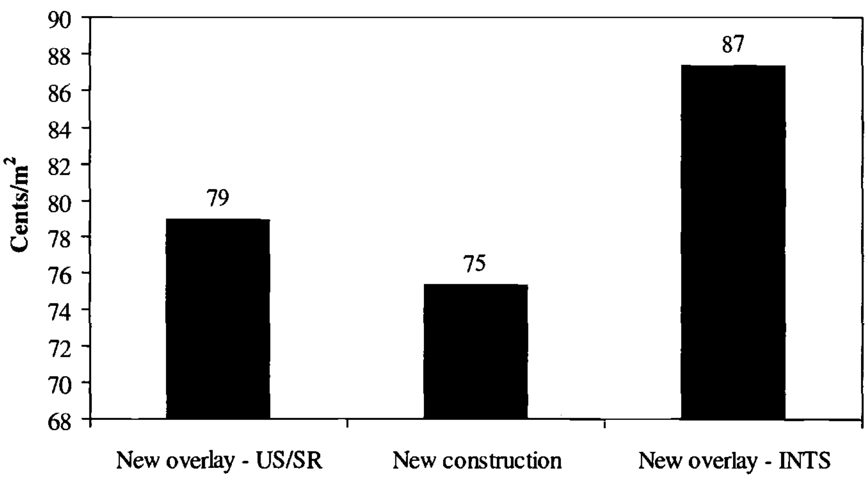

Figure 4 – *Bonus, New Construction vs. New Overlay, 1995-2000*

Figures 5 and 6 show the Incentive/Disincentive amounts for individual contractors for the year 2000. Figure 5 shows details regarding the number of projects of each size for each contractor. Figure 6 includes detailed information on the type of design each contractor was built. The detailed information is in the parenthesis following each contractor's name.

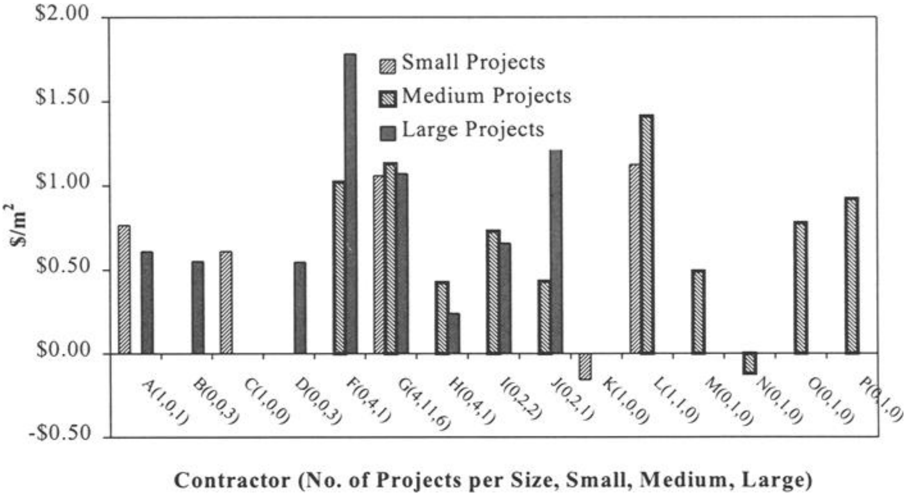

Figure 5 – *Incentive Pay by Contractor, Number of Projects per Size (Small, Medium, Large)*

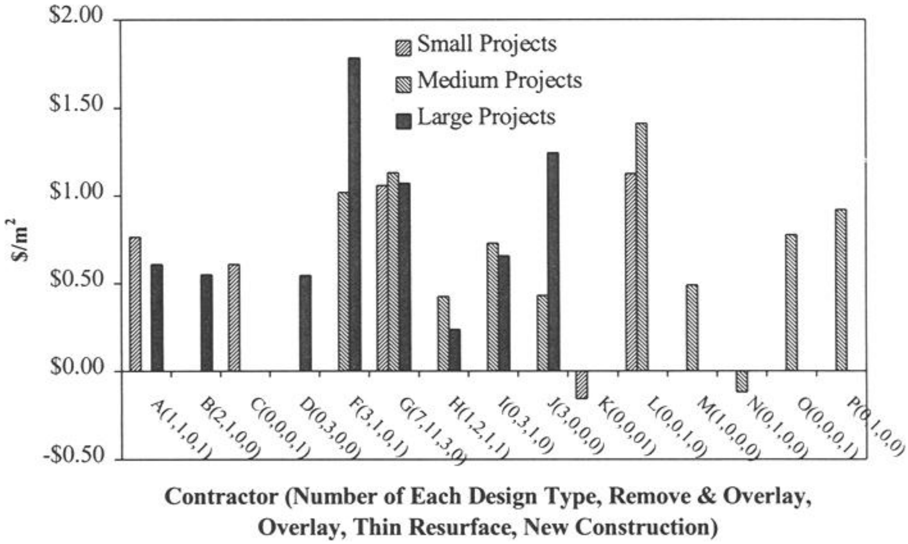

Figure 6 – *Incentive Pay by Contractor, Number of Each Design Type (Remove & Overlay, Overlay, Thin Resurface, New Construction)*

The ultimate concern is the amount of benefit gained from having the smoothness specification versus its cost. Although there is the intangible benefit of being one of the states with the smoothest pavements the best justification is tangible dollar benefits to the Department and to the Citizens of Arizona. To show specific numbers a cost benefit analysis of a typical project was done.

As a general rule of thumb, a pavement is expected to have 0.08 m/km increase in smoothness value per year. The smoothness specification decreases the average initial smoothness value for a project from 0.72 m/km to 0.56 m/km. If the trigger for the next project is the point when the pavement smoothness value reaches 1.52 m/km then a pavement that has an initial smoothness value of 0.72 m/km will have its next project in 10 years. A pavement having an initial smoothness value of 0.56 m/km, as a result of the inclusion of a smoothness specification, will need its next project in 12 years. Inclusion of the smoothness specification has, in this example, extended the pavement life by two years. Accordingly, over a life-cycle period of 60 years this will eliminate the need for one rehabilitation project. This would save ADOT about $68 000 per km in direct costs. It also saves road users $28 000 per km in imputed costs by eliminating the traffic delay associated with the eliminated project. The total savings for ADOT and users are $96 000 per km. Based on the recent history of incentive cost for the smoothness specification, a typical amount ADOT pays a contractor is $24 000 per km, slightly more than $1.2 per square meter. Therefore, for the comparative pavements in this example, the project with an initial smoothness value of 0.56 m/km (as a result of implementation of smoothness specification), has a cost/benefit ratio is 4 ($96 000 benefit / $24 000 cost).

In addition to the savings to users from the cost of delays due to construction work zone activities as stated above, implementation of the smoothness specification results in a smoother ride over the pavement service life. The smoother ride reduces the intangible costs attributable to driver and passenger discomfort as well as the real costs of increased fuel and oil consumption, tire wear and vehicle repair and maintenance.

Discussion

The study shows that the smoothness of old pavements has no significant impact on the smoothness of following layers except when the project is ACFC/ARACFC only. In other words, no matter how rough the old pavement is, the contractors have the abilities and opportunities to achieve a smooth pavement if the job involves AC overlay or removing and replacing AC. Based on this finding, it would be logical to expect the smoothness of the projects on divided and non-divided highways to be close to each other. However, the analysis results of the projects do not completely support this inference. For all the projects completed in years 1999 and 2000, the smoothness value of non-divided highway projects is about 0.16 m/km (46%) less than that of divided highway projects. For the projects that were completed by contractor F in years 1998-2000, the smoothness value of non-divided highway projects is only 0.044 m/km (11%) less than that of divided highway projects. For the projects completed by contractor F, which makes up the majority of all the projects, the smoothness value of non-divided highway projects is 0.086 m/km less than that of divided highway projects. The difference of 0.086 m/km is very close to the 0.08 m/km that is used in the current ADOT specification.

Therefore, it may be concluded that the difference in smoothness between divided and non-divided highway projects is not related to smoothness difference in old pavements where non-divided highway pavements are usually much rougher than that of divided highway pavements. Instead, the difference may be mainly associated with the greater variability in quality within the same contractor and among different contractors. For overall projects, the standard deviation of non-divided highway projects completed in years 1996-2000 is 0.152, which is significantly greater than that of divided highway projects, 0.1. Greater variability of non-divided highway projects may be caused by many factors such as contractors' expectation and effort on their job, construction environments, geometry, and others. Because quality variability is a complex issue and impossible to eliminate, it is reasonable to recognize the smoothness difference caused by job variability in developing smoothness specifications. Since contractor G represents the highest quality and majority of all the projects, the smoothness difference of 0.086 m/km between its projects in two categories justify the difference of 0.08 m/km, which is used in the current ADOT specification.

The same window values are adopted in the current ADOT smoothness specification for category 2, divided highway projects with only one leveling opportunity (ACFC/ARACFC so far), and category 3, non-divided highway projects with two or more leveling opportunities. However, the data shows that there is a significant difference in smoothness between the projects in the two categories. The average smoothness value of category 2 projects is 0.613 m/km and that of category 3 is 0.512 m/km. Thus it seems that the window values in the current specification are high for category 3.

A difference in window values of 0.24 m/km are adopted in the current ADOT smoothness specification for category 2, divided highway projects with only one leveling opportunity (ACFC/ARACFC so far), and category 5, non-divided highway projects with only friction course. The data shows that the difference in project smoothness between the two categories is only 0.104 m/km with the smoothness value being 0.613 m/km and 0.717 m/km for category 2 and 5, respectively. It seems that the window values in the current specification are high for category 5.

A difference of window values of 0.24 m/km are adopted in the current ADOT smoothness specification for category 3, non-divided highway projects with two or more leveling opportunities, and category 6, non-divided highway projects with only AC overlay. For the projects in year 2000, the corresponding difference is 0.04 m/km, which is relatively small. This, combined with the finding that the AC overlay smoothness is not significantly affected by the smoothness previous layers or old pavements, shows that there is great potential to eliminate the difference of 0.24 m/km in the future smooth specification for the two categories.

Conclusion

Based on the analysis of the smoothness data of the pavement rehabilitation and new construction projects completed from 1994 to 2000, the following conclusions can be drawn.

(1) Implementation of an AC smoothness specification at ADOT has greatly improved the new pavement smoothness. From 1994 to 2000, the new pavement smoothness has increased 36% for the projects done on divided highway and 30% for non-divided highway.
(2) The amount of benefit to the Department and to the Citizens of Arizona gained from having the smoothness specification versus its cost is significant. The cost/benefit ratio is about 4.
(3) The smoothness of old pavements does not have a significant effect on the smoothness of projects that have AC layers involved while it has some effect on the smoothness of the ACFC/ARACFC projects completed by some contractors.
(4) The smoothness of 0.16 km follows Lognormal distribution.
(5) There is a significant difference in smoothness variability among projects, contractors, and categories. The smoothness of the projects on divided highways has much less variability than that on non-divided highways.
(6) Some of the target and window values in the current ADOT smoothness specification, which was determined based on the smoothness data from a limited number of projects, have become inconsistent with the smoothness data of the much greater number of projects completed in recent years. These values need to be modified based on the statistical analysis of the recent smoothness.

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Use of Automated Profilers to Replace NJDOT Rolling Straightedges

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Reference: Zaghloul, S. M. and Vitillo, N., **"Use of Automated Profilers to Replace NJDOT Rolling Straightedges,"** *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433,* M. S. Gardiner, Eds., American Society for Testing and Materials International, West Conshohocken, PA, 2003.

Abstract: A study was performed to evaluate the applicability of using automated profilers to replace the Rolling Straightedges (RSEs) currently used by NJDOT to implement the department smoothness specifications. Two categories of profilers were considered in the study, two lightweight and three full size profilers, in addition to two NJDOT RSEs. Several analyses were performed on the collected data, which include preliminary analysis, RSE simulation, statistical analysis, speed effect analysis and correlation analysis. The RSE simulation analysis consisted of simulating a 10-ft (3.048 m) straightedge over the profile and calculating the deviation at the mid-point of the straightedge. The statistical analyses were performed to investigate the equipment repeatability and the differences among devices, including the two RSEs. Three correlation analysis studies were performed to correlate the RSE measurements with the results of the simulation analysis, to correlate IRI measured with different devices and to correlate the IRI and % defective length.

Keywords: Pavement smoothness, pavement rideability, smoothness specifications, International Roughness Index (IRI), Rolling Straightedge simulation, percentage defective length and profilers

Introduction

Improving the performance of asphalt pavements is an on-going goal for highway agencies, pavement designers and contractors. Advances in pavement design, asphalt mix testing and design, and construction equipment may help in achieving this goal. However, no real improvement will be achieved without improving the current Quality Control/Quality Assurance (QC/QA) practices. Smoothness testing is one of the QC/QA tests that requires improvements. The current New Jersey Department of Transportation (NJDOT) QC/QA specifications require inspecting the finished asphalt surface with a 10-ft (3.048m) Rolling Straightedge (RSE).

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Surface tolerances measured with the RSE are then used to evaluate the initial pavement smoothness and the construction quality. Although the RSE inspection is simple and does not require expensive equipment or operators with engineering training, it is time and labor consuming and requires lane closures.

Also, the repeatability of RSE is not always high and in some cases, the results are misleading. In addition, RSE inspection cannot address the roughness associated with wavelengths longer than the straightedge base length and does not provide the information required for long-term monitoring.

Study Objectives

A study was performed to evaluate the applicability of using automated highway profilers to replace RSEs currently used by NJDOT to implement the department smoothness specifications *[1]*. Two categories of profilers were considered in the study, low speed and high speed profilers. The low speed profilers included the KJ Law Lightweight Profiler (T 6400) and the ICC Lightweight Profiler (MDR 4082-PLT). The high speed profilers included: the KJ Law Road Surveyor Profiler (T 6600) and an ARAN with a profiling subsystem. A Stantec RT 3000 equipped with laser sensors was also used in the project to scan several candidate sites.

The scope of the study was limited to asphalt surfaced pavements. A Design of Experiment (DOE) was prepared to allow selecting the test sections. Three levels of initial smoothness were considered in the DOE as follows:

- Very smooth pavements (Level 1) - % Defective Length (%DL) <= 1.5,
- Smooth pavements (Level 2) - 1.5 < %DL <= 3.5, and
- Relatively rough pavements (Level 3) - %DL > 3.5.

Based on the DOE, three sections were required for each smoothness level. Each of the selected sections has to be tested with two NJDOT RSEs, the two lightweight profilers and the two full size profiles.

Scan Tests and Site Selection

Figure 1 outlines the steps followed to achieve the study objects. An initial list of test sections was compiled from the construction projects of 1997, 1998 and 1999 construction seasons. The as-built RSE measurements of these sections were reviewed and used to classify these sections into the appropriate initial smoothness group. Since the RSE and low speed profilers tests require lane closures, site conditions, such as traffic and number of lanes, were considered in the selection process. Test sections with high traffic volumes or with a single lane per direction were excluded from the initial list.

The field testing program of the project consisted of three phases. In Phase I, a laser based profiler was used to scan the test sections of the initial list. Analysis was performed on the collected data to select test sections that satisfy the DOE requirements. In Phase II, the selected sections were then surveyed using NJDOT RSEs (two devices) and the lightweight profilers (ICC and KJ Law). Tests were performed on the left and right wheel paths of each of the test sections. Three runs were performed with the lightweight profilers on each wheel path at the speed recorded by the equipment manufacturer. In Phase III, the selected test sections were surveyed using the high speed profilers. The number of runs varied among the devices. As a minimum, three repeated runs were performed on each of the test sections. Also, multiple runs at different speeds (40, 50 and 60 mph [64.4, 80.5 and 96.6 km/h]) were performed using one of the high-speed profilers.

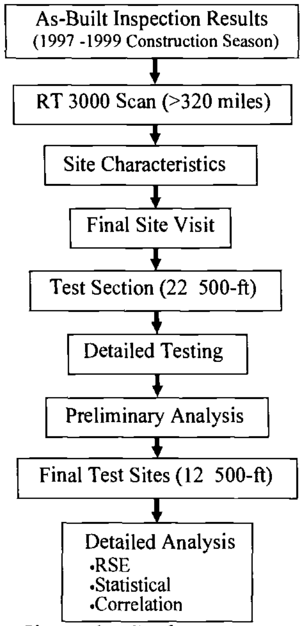

Figure 1 - *Study overview.*

Analysis of the Scanning Tests

More than 320 miles (515.2 km) were scanned to ensure getting the required number of test sites (9 sites, 500-ft [152.4 m] each). The 100-ft (30.48 m) IRI values were calculated for the scanned sections and used to calculate the 500-ft (152.4 m) moving average IRI values. In total, over 16500 500-ft (152.4 m) moving averages were obtained from the scanned sections. These moving averages were analyzed to categorize the sections as very smooth, smooth or relatively rough, using the limits shown in Table 1. A short list of candidate sections was prepared based on the results of the scan data analysis. The candidate sections were then visited to evaluate the site characteristics from the safety viewpoint, such as number of lanes and sight distance.

Table 1 - *Roughness categories.*

	Minimum 500-ft (152.4 m)		Maximum 500-ft (152.4 m)	
Roughness Category	IRI[1]	PSI[2]	IRI[1]	PSI[2]
Very Smooth			< 0.85	< 4.29
Smooth	0.85	4.29	1.00	4.29
Relatively Rough	> 1.00	> 4.18		

[1]IRI in m/km (1 m/km = 63.5 m/km),
[2]0-5 scale.

Detailed Field Tests

Low Speed Devices

The lightweight profilers considered in this study are equipped with a single laser sensor and a single accelerometer. These profilers measure the longitudinal profile of only one wheel path and reccrd it at a sample rate of 1-in (25.4 mm).. The testing speed may vary between 10-20 mph (16.1-32.2 km/h). However, the typical testing speed, as recommended by the equipment manufacturers, is 20 mph (32.2 km/h).

Traffic control was provided to close the slow lanes of the selected sections. Test sections (500-ft [152.4 m] each) were marked and numbered. In total, 22 test sections were tested using the ICC and KJ Law Lightweight Profilers and two NJDOT RSEs. The lightweight profilers testing consisted of testing the right and left wheel paths, three times each, at the speed recommended by the manufacturer. Also, three runs were performed at speeds of 10, 15 and 20 mph (16.1, 24.15 and 32.2 km/h) to investigate the impact of speed on the lightweight profiler measurements.

Two NJ DOT RSEs were considered in this study. Both RSEs were calibrated prior of the testing on the same bench. The RSEs were calibrated following the standard NJ DOT procedure. The cut-off limit for the RSEs was set to 1/8 in. (3.175 mm). Therefore, the RSEs will mark areas where the tolerance exceeds 1/8 in. (3.175 mm).

A single RSE test was performed on each wheel path of the test sections. Also, ten repeated runs were planned for two test sections (one from the very smooth group and one from the relative rough group). However, these repeated runs provided inaccurate results because it was hard to isolate the trace of each run.

High Speed Devices

The high speed profilers included in this study were equipped with laser sensors and accelerometers. These profilers measure the longitudinal profiles of both wheel paths and record them at sampling rates in the range of 1 to 6 inches (25.4 to 152.4 mm).

The marked test sections were tested with the high speed profilers. The number of runs varied among the equipment. As a minimum, three repeated runs were performed on each of the test sections. In some cases, the number of repeated runs was increased to five. Also, three speeds were used in the ARAN tests (40, 50 and 60 mph [64.4, 80.5 and 96.6 km/h]), with at least three runs at each speed.

Preliminary Analysis and RSE Simulation

Several analyses were performed on the collected data. These analyses included: preliminary analysis, RSE simulation, statistical analysis, effect of speed analysis and correlation analysis. Details of these analyses are shown in the following sections.

Preliminary Analysis

Results of the RSE tests were reviewed to identify the test sections that match the DOE requirements. As a result, 12 500-ft [152.4 m] test sections were selected. Nine test sections out of the 12 test sections were selected to satisfy the DOE requirements, while the remaining three sections were selected to verify the analysis results.

RSE Simulation Analysis

RSE computer simulation analysis was performed on the measured profiles. The simulation analysis consisted of driving a 10-ft (3.048 m) straightedge over the measured profiles and calculating the deviation at the mid-point of the straightedge The step used to move the RSE forward was set equal to the sampling rate used in the data collection. This sampling rate varied among the devices and ranged from 1 in to 6 inches (25.4 to 152.4 mm). The results of the computer simulation analysis were verified manually. Tolerances were manually calculated for a few profiles and compared with the computer simulation results. Perfect agreement was obtained for all cases considered in this verification.

Locations where tolerance exceeded the limit (0.125 in. [3.175 mm]) were identified. The total defective length is then calculated as the summation of the lengths where the tolerance exceeded the limit. The %DL was then calculated as the total defective length divided by the total section length. Results of the simulation analysis performed on different devices/test sections, along with the corresponding results of the RSEs are presented in Table 2.

The simulation analysis results from the collected profiles were further analyzed by sectioning each of the 500-ft (152.4 m) sections into a set of 50-ft (15.24 m) segments. The main reason for this is to have enough degrees of freedom for the statistical analysis.

Table 2 - *Results of the simulation analysis.*

	Simulated %DL				Measured %DL	
Section Number	ICC Light*	KJ Law Light*	KJ Law Full*	ARAN*	RSE I	RSE II
1	4.35	1.22	2.38	0.60	4.20	3.60
2	4.99	1.22	1.32	0.54	4.60	3.80
3	4.62	5.94	2.69	0.54	4.20	3.00
4	2.00	0.33	1.28	0.42	1.80	0.80
5	3.89	0.79	1.15	1.58	2.80	0.20
6	3.22	2.53	0.35	1.24	2.40	0.20
7	2.03	3.14	1.18	0.90	1.20	0.60
8	1.44	4.13	1.08	0.18	1.00	0.60
9	0.71	5.24	0.01	1.84	0.80	0.80
10	0.00	0.00	0.10	0.00	0.00	0.00
11	0.00	0.03	0.00	0.00	0.00	0.00
12	0.00	0.02	0.27	0.04	0.00	0.00

*Based on RSE simulation analysis.

Analysis was also performed on the IRI measured using the automated devices, low-speed and high-speed profilers. The IRI values of the test sections are presented in Table 3. It should be noted that IRI measurements of the ICC Lightweight Profiler were not available.

Table 3 - *IRI results (m/km).*

Section Number	KJ Law Light	KJ Law Full	IRI m/km ARAN40	ARAN50	ARAN60
1	1.253	1.600	1.503	1.489	1.563
2	1.322	1.399	1.363	1.360	1.457
3	1.860	1.612	1.506	1.457	1.571
4	1.390	1.404	1.432	1.529	1.560
5	1.782	1.554	1.443	1.399	1.434
6	1.728	1.570	1.421	1.422	1.475
7	1.818	1.712	1.708	1.719	1.748
8	1.642	1.192	1.298	1.235	1.291
9	2.511	1.360	1.296	0.764	1.309
10	0.947	1.138	0.987	1.489	0.996
11	0.947	1.024	0.911	0.879	1.010
12	0.867	1.013	0.925	0.945	0.971

Statistical Analysis

Several statistical analyses were performed to investigate the equipment repeatability and the differences among devices, including the two RSEs. In these analyses, the F-Test and the Student T-Test were used *[2]*. The confidence level was always selected to be 90%. The analyses were performed on the %DL measured with the RSEs and that resulted from the simulation analysis as well as on the IRI measured using the automated profiles. In all cases, it was assumed that the %DL and IRI data are normally distributed. Some of the analyses were performed on the detailed section measurements (each section is sub-sectioned into a set of 50-ft [15.24 m] segments), while other analyses were performed on the overall section measurement (each section is represented as one unit).

The following section summarizes the performed statistical analyses. In each case, the purpose of the analysis, the hypothesis, the type of measurement (detailed or summary), the statistical test and the cases analyzed are presented.

Evaluate the Difference Among Devices

Purpose: To evaluate the significance of the difference among different devices by comparing the %DL of all devices/sections.
Null Hypothesis: No significant difference among devices for all test sections.
Type of Measurements: Summary measurements.
Statistical Test: F-Test with a 90% confidence level.
Cases Analyzed: Analysis was performed on the %DL for the average of the runs of ICC Light, KJ Law Light, ARAN (including ARAN40, ARAN50 and ARAN60), KJ Law Full, RSE I and RSE II.
Results: The analysis results indicate that the null hypothesis of no difference among devices is rejected.

Evaluate the Significance of Difference Between Pairs of Devices

Purpose: To evaluate the significant of the difference between pairs of devices by comparing the average %DL of different pairs of devices for all sections.
Hypothesis: No significant difference between the average %DL of the two devices for all test sections.
Type of Measurements: Summary measurements – analyses are performed on the difference between the measurements of the pair of devices under consideration.
Statistical Test: Two-Sided T-Tests with a 90% confidence level.
Cases Analyzed: See Table 4.
Results: Table 4 shows the results of the analysis. As can be seen, the difference between any two devices is significant.

Table 4 - *Significance of difference between pairs of devices.*

Device 1	Device 2	Calculated t	*Significant
ICC Light	KJ Law Light	4.06	Yes
ICC Light	KJ Law Full	3.76	Yes
ICC Light	ARAN	3.99	Yes
ICC Light	RSE I	3.47	Yes
ICC Light	RSE II	3.36	Yes
ICC Light	Average RSE I and RSE II	3.43	Yes
KJ Law Light	KJ Law Full	3.29	Yes
KJ Law Light	ARAN	2.96	Yes
KJ Law Light	RSE I	4.06	Yes
KJ Law Light	RSE II	4.12	Yes
KJ Law Light	Average RSE I and RSE II	4.12	Yes
KJ Law Full	ARAN	4.10	Yes
KJ Law Full	RSE I	3.33	Yes
KJ Law Full	RSE II	3.32	Yes
KJ Law Full	Average RSE I and RSE II	2.92	Yes
ARAN	RSE I	3.34	Yes
ARAN	RSE II	3.25	Yes
ARAN	Average RSE I and RSE II	2.58	Yes
RSE I	RSE II	3.14	Yes
RSE I	RSE II	3.14	Yes
RSE I	Average RSE I and RSE II	3.14	Yes
RSE II	Average RSE I and RSE II	3.14	Yes

t* = 1.798

Evaluate Equipment Repeatability

Purpose: To evaluate the repeatability of each device by comparing the repeated runs (pair-wise) for each test section.
Hypothesis: No significant difference among repeated runs on each test section.
Type of Measurements: Detailed measurements – analyses are performed on the difference between the measurements of the pair of devices under consideration.
Statistical Test: Two-Sided T-Tests with a 90% confidence level.
Cases Analyzed: See Table 5.

Results: Table 5 shows a summary of the analysis results. In this table, the total number of tests is presented for each device/parameter combination. Also, the percentage of tests that show no significant difference is presented, as well as the percentage of tests that show significant difference. For example, 36 tests were performed for the ICC Light/%DL combination. Fifty percent of these tests, i.e., 18 tests, show no significant difference, while the other 50% show significant difference.

Table 5 - *Summary of the equipment repeatability results.*

Device	Parameter	Total Number of Tests	No Significant Difference (Conclude H_o)	Significant Difference (Reject H_o)
ICC Light	%DL	36	50%	50%
KJ Law Light	%DL	36	44%	56%
ARAN 40	%DL	120	50%	50%
ARAN 40	IRI	120	0%	100%
ARAN 50	%DL	36	60%	40%
ARAN 50	IRI	36	0%	100%
ARAN 60	%DL	36	69%	31%
ARAN 60	IRI	36	0%	100%
KJ Law Full	%DL	120	74%	26%
KJ Law Full	IRI	120	5%	95%

Speed Effect on ARAN Measurements

Purpose: To evaluate the speed effect on ARAN measurements by comparing the repeated runs at different speeds (pair-wise) for each test section.
Hypothesis: No significant difference among repeated runs at different speeds on each test section.
Type of Measurements: Detailed measurements - analyses are performed on the difference between the measurements of the pair of speeds.
Statistical Test: Two-Sided T-Tests with a 90% confidence level.
Cases Analyzed: ARAN 40, ARAN 50 and ARAN 60.
Results: Table 6 shows a summary of the analysis results.

Table 6 - *Summary of ARAN-speed results.*

Device	Parameter	Total Number of Tests	No Significant Difference (Conclude H_0)	Significant Difference (Reject H_0)
ARAN40/ARAN50/ARAN60	%DL	36	61%	39%
ARAN40/ARAN50/ARAN60	IRI	36	0%	100%

Analysis of the Statistical Test Results

Several conclusions can be drawn from the results of the statistical tests as follows.

- The differences among %DL simulated from the measurements of different devices are found to be statistically significant. The same is also true for the IRI measurements. Also, the difference between the two RSEs is significant and cannot be ignored.
- The comparison between the two RSEs (RSE I and RSE II) indicated that RSE I always reads significantly higher deviations than RSE II (see Table 2). It should be noted that both RSEs were calibrated on the same bench by the same crew prior to the field inspection. The percentage difference between the two RSEs, calculated as the difference between the RSE readings divided by RSE II readings, ranges from 0% (no difference) to 1300% (the difference is 13 times the value of the RSE II measurement).
- Comparisons between the RSE I and RSE II measurements and the %DL simulated from the KJ Law Light and the ICC Light indicated that the differences between the RSE measurements and those of the lightweight profilers are significant and not consistent, except for ICC Light and RSE I. It was found that the numbers obtained from the ICC Lightweight profiler gives higher % DL than those measured using RSE I in most cases.
- Comparisons between the RSE I and RSE II measurements and the %DL simulated from the ARAN 40 mph (64.4 km/h), 50 mph (80.5 km/h) and 60 mph (96.6 km/h) measurements indicated that the differences between the RSE measurements and the ARAN measurements at all speeds are significant.
- The effect of speed on ARAN measurements is significant and not consistent. A portion of this difference might be related to the difference among repeated runs. However, since multiple runs were used for all speeds, this effect is randomized and should not have a significant impact on the difference among the results at different speeds.
- The comparison between the RSE results and the %DL simulated from the KJ Law Full measurements indicated that the difference between their measurements is significant and not consistent.
- Figure 2 shows a sample of the IRI measured using different devices, while Figure 3 shows the corresponding % difference among the IRI values. The % difference of a section is calculated as a percentage of the average IRI for the section. As can be seen, the percentage difference exceeds in some cases 140%, which is significant. However, the results of some of the tests, such as those performed using the KJ Law Full and ARAN at 40 and 50 mph (64.4 and 80.5 km/h), are very close.

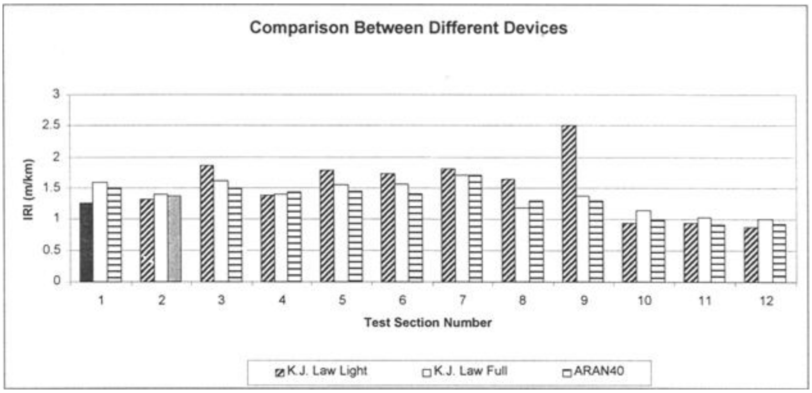

Figure 2 - *IRI comparison.*

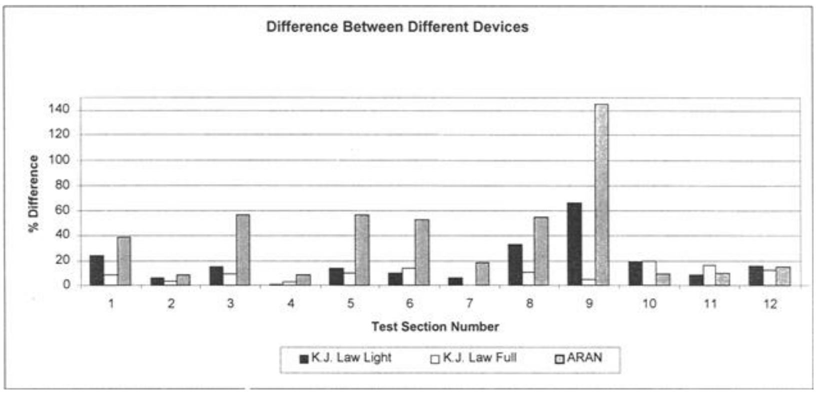

Figure 3 - *Percentage difference in IRI.*

Effect of Speed on the Lightweight Profilers

Test trials were performed at different speeds using the KJ Law and ICC Lightweight Profilers to evaluate the impact of speed on their measurements. The tests were performed on only one section of I-195 West, from Stations 100 to 600 ft (30.48 to 182.88 m). This section is a relatively rough section. Figure 4 shows the effect of speed on the simulated %DL for the KJ Law and ICC Profilers, respectively.

Results of the analysis performed to investigate the effect of speed on the measurements of the lightweight profilers indicated that the correlation between the % DL at different speeds is lower for the KJ Law Light (0.71) than that for the ICC Light (0.99). This implies that although there is a difference between the profiles measured at different speeds, this difference is consistent in the case of the ICC Light and can be eliminated by using a correlation model.

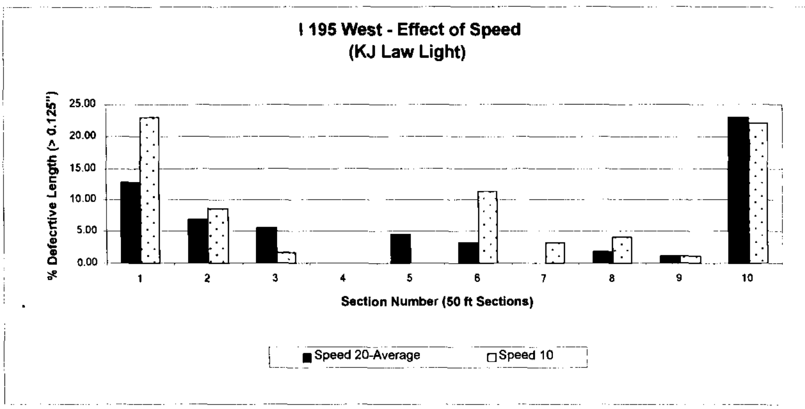

Figure 4a - *Effect of speed on lightweight profilers.*

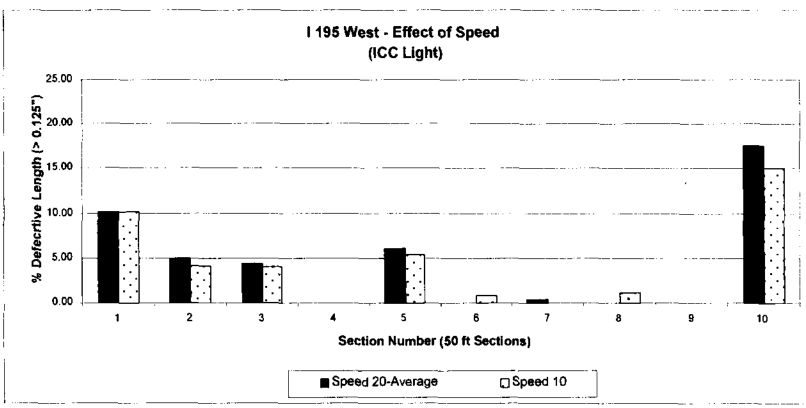

Figure 4b - *Effect of speed on ICC lightweight profilers.*

Correlation Analysis

Three correlation analysis studies were performed on the collected data. The objectives of these studies are to correlate the RSE measurements with the results of the simulation analysis performed on the profiles measured using the automated devices, to correlate the IRI measured with different devices and to correlate the IRI and %DL of the RSEs.

Correlation of %DL Between RSE and Automated Devices

- The objective of this correlation analysis is to correlate the %DL measured using the two RSEs with that resulting from the simulation analysis performed on the profiles measured using the automated devices. In this analysis, the measurements of both

RSEs as well as the average of their measurements, were correlated with the results of the simulation analysis performed on the profiles measured using the automated devices. Table 7 summarizes the results of the correlation analysis. As can be seen, RSE II correlated the best with the profilers, followed by the average of the two RSEs. ICC Lightweight Profiler was the device that correlated best with RSE I and the average of the two RSEs, while ARAN60 and KJ Law Lightweight Profiles were the devices that correlated best with RSE II.

Table 7 - *Summary of the % DL correlation analysis results.*

Profiler	RSE I	RSE II	Average (RSE I-RSE II)
ARAN40	0.55	0.72	0.69
ARAN50	0.78	0.83	0.92
ARAN60	0.79	0.89	0.89
KJ Law Light	0.79	0.88	0.92
ICC Light	0.89	0.84	0.96
KJ Law Full	0.63	0.86	0.8
Maximum	0.89	0.89	0.96
Minimum	0.55	0.72	0.69

Correlation Between IRI Measured Using Different Devices

Correlation analysis was performed on the IRI measured using the KJ Law Light, the KJ Law Full and the ARAN (40, 50 and 60 mph [64.4, 80.5 and 96.6 km/h]), Table 8 shows a summary of the correlation results. As can be seen, some devices correlate very well with each other, such as ARAN 40 and ARAN 60 and ARAN 40 and KJ Law Light. On the other hand, some devices correlated very poorly, such as ARAN 50 and KJ Full and ARAN 60 and KJ Full.

Table 8 - *Summary of the IRI correlation analysis results.*

Profiler	ARAN40	ARAN50	ARAN60	KJ Law Light	KJ Law Full
ARAN40	-	0.45	0.97	0.91	0.38
ARAN50		-	0.45	0.45	0.01
ARAN60			-	0.88	0.3
KJ Law Light				-	0.34
KJ Law Full					-

Correlation Between % DL and IRI

Correlation analysis was performed to correlate the RSEs %DL and the corresponding IRI values that were measured using the KJ Law Light, KJ Law Full and ARAN. The objective of this analysis is to evaluate the applicability of using IRI as a measure for initial smoothness, instead of % DL. Results of the correlation analysis are shown in Table 9. As can be seen, the R^2 value ranged from 0.01 (no correlation) to a maximum value of 0.48 (poor correlation). These results are expected because of the difference in the concept behind the % DL and IRI. In simple terms, IRI is calculated as the total vertical movement of the quarter-car model, divided by a selected base length.

On the other hand, the % DL is calculated as the total length, which has tolerance greater than 0.125 in. (3.175 mm), divided by the section length. Therefore, based on the analysis results, it can be concluded that IRI cannot be used to replace % DL in the smoothness acceptance testing.

Table 9 - *Summary of the % DL-IRI correlation analysis results.*

Profiler	RSE I	RSE II	Average (RSE I-RSE II)
ARAN40	0.42	0.2	0.33
ARAN50	0.21	0.08	0.15
ARAN60	0.46	0.24	0.38
KJ Law Light	0.48	0.2	0.36
KJ Law Full	0.05	0.01	0.02

Summary, Conclusions and Recommendations

Summary

A study was performed to evaluate the applicability of using automated profilers to replace the Rolling Straightedges (RSEs) currently used by New Jersey Department of Transportation (NJDOT) to implement the department smoothness specifications. Two categories of profilers were considered in the study, low-speed profilers (two devices) and high-speed profilers (three devices), in addition to two of NJDOT RSEs. The scope of the study was limited to asphalt surfaced pavements. A Design of Experiment (DOE) was prepared to select the test sections. Three levels of initial smoothness were considered in the DOE, very smooth, smooth and relatively rough pavements.

Field testing was performed in three phases. In Phase I, a laser-based profiler was used to scan several asphalt sections. Analysis was performed on the collected data to select test sections that satisfies the DOE.

In Phase II, the selected sections were tested using the two RSEs and the lightweight profilers (two devices). The testing with the lightweight profilers included data collection in the right and left wheel paths, three times each, at the speed recommended by the manufacturer. Also, three runs were performed at speeds of 10, 15 and 20 mph (16.1, 24.15 and 32.2 km/h) to investigate the impact of speed on the lightweight profiler measurements.

In Phase III, the sections were tested using the high-speed profilers. The number of runs varied among the devices. As a minimum, three repeated runs were performed on each of the test sections. Also, multiple runs at different speeds (40, 50 and 60 mph [64.4, 80.5 and 96.6 km/h]) were performed using one of the high-speed profilers.

Several analyses were performed on the collected data. These analyses included; preliminary analysis, RSE simulation, statistical analysis, effect of speed analysis and correlation analysis. The preliminarily analysis was performed on the results of the RSE inspection to select test sections that match the DOE requirements. RSE simulation analysis was performed on the collected profiles to simulate the RSE inspection. This analysis consisted of simulating a 10-ft (3.048 m) straightedge over the profile and calculating the tolerance at the mid-point of the straightedge. This analysis was summarized as the Percent Defected Length (%DL), the length of pavement out of tolerance divided by the total length tested.

Several statistical analyses were performed on the collected and simulated data to investigate the equipment repeatability and the differences among devices, including the two RSEs. In these analyses, the F-Test and the Student T-Test were used. The analyses were performed on the %DL measured with the RSEs and that resulted from the simulation analysis, as well as on the IRI measured using the automated profilers. The effect of speed analysis was performed on the data collected using the lightweight profilers and the high-speed profiler.

Three correlation analysis studies were performed on the collected data. The objectives of these studies are to correlate the RSE measurements with the results of the simulation analysis performed on the profiles measured using the automated devices, to correlate the IRI measured with different devices and to correlate the IRI and %DL of the same device.

Conclusions

Several conclusions can be made from the analysis results. The following are some of these conclusions.

- The differences among devices are significant. This includes the %DL and IRI measurements. Also, the difference among the RSEs is significant and cannot be ignored.
- The speed effect on ARAN measurements is significant and not consistent, for both %DL and IRI.
- The differences between the RSE measurements and those of the lightweight profilers are significant and not consistent.
- The differences among IRI values measured using different devices/speeds are significant.
- RSE simulation provides reasonably accurate estimate of the RSE %DL. Results of the correlation between the measured and simulated %DL are found to be as high as 99% in some cases.
- Results of the correlation between IRI and %DL indicated that IRI does not sufficiently correlate with %DL, and therefore should not be used to replace %DL.

Recommendations

- Since the results of the correlation studies indicated that the IRI does not correlate sufficiently with %DL, %DL is recommended to remain the primary indicator for smoothness evaluation of new and rehabilitated pavements. Further investigations are required to select an indicator that better represents the user's opinions.
- Since the RSE simulation results correlate very well with the actual RSE measurements, the following scenario is recommended as a step towards replacing the RSE with automated devices.
  - NJDOT selects a profiler as the official NJDOT profiler. Results of the simulation analysis performed on the profiles measured using this profiler will be considered as the official results based on which the construction quality will be evaluated.

As an example, ARAN may be considered as the NJDOT official profiler. Tests performed at 40 mph (64.4 km/h) speed will be considered as the official tests.
- Since the profiler measurements vary, correlation curves are required to correlated the %DL that is based on other profilers with that of the NJDOT official profiler.
- Using these correlation models will allow contractors to run tests using any profiler and correlate the results with the NJDOT official profiler.

Replacing the RSE with automated profilers is considered as the first step of improving the practice of evaluating pavement smoothness. This will give the contractors the opportunity to early detect problems and perform the remedial action in a timely manner. Automated profilers will eliminate the need for lane closure and will significantly reduce the inspection time. In addition, automated profilers will simultaneously provide roughness indices, such as IRI, which are required for the PMS group. Collecting as-built roughness indices will help the PMS group to improve their performance prediction models, as well as allow them to quantify the impact of initial roughness on the pavement life cycle cost.

As can be seen, replacing the RSE with automated profilers is very beneficial and has many positive impacts. However, it should be considered only as the first step in improving the construction quality control procedure. Replacing the RSE with automated devices and using a RSE Simulation Analysis will not solve some of the RSE inspection problems, such as the misleading results and the repeated waves problem.

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The Road to Smooth Pavements in Tennessee

Reference: Jackson, N. M., Jubran, A., Hill, R. E., and Head, G. D., " **The Road to Smooth Pavements in Tennessee,**" *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433*, M. S. Gardiner, Ed., American Society for Testing and Materials International, West Conshohocken, PA, 2003.

Abstract: Pavement smoothness directly affects the dynamics of moving vehicles, impacting the rate of deterioration of the pavement and the operation and safety of vehicles and occupants. Consequently, the FHWA and many state transportation agencies have taken measures to address pavement smoothness immediately following construction. The significance of smoothness is evidenced by the preliminary recommendations for the adoption of the International Roughness Index (IRI) in the forthcoming AASHTO 2002 Pavement Design Guide.

The State of Tennessee Department of Transportation (TDOT) adopted the Mays meter for the measurement of HMA pavement smoothness in the early 1980s. At that time, Mays meter measurements of 55 to 65 inches per mile (868 to 1026 mm per km) were not uncommon on pavements throughout the state. Through the implementation of increasingly stringent, incentive-based specifications, annual Smooth Pavement Awards for top-performing contractors, and advances in paving equipment, Mays meter measurements as low as 10 inches per mile (158 mm per km) are quite common today.

This paper documents the measures taken in Tennessee over the past 20 years to improve pavement smoothness.

Keywords: Smooth Pavement, Incentive-Based Specifications, Mays meter, Tennessee

Introduction

The Tennessee Department of Transportation (TDOT) is continuously working with the Federal Highway Administration (FHWA) and statewide paving contractors to improve pavement smoothness on state highways. These efforts have been underway

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since the early 1980s. The steps taken thus far in this pursuit include the implementation of a pavement smoothness incentive specification and competitive quality awards for outstanding contractors and projects. Incremental improvements to HMA paving equipment and enhanced training of contractor personnel have also contributed to this effort. The objective of this paper is to document some of the specific measures taken over the past 20 years to improve pavement smoothness and present summary data confirming the measured progress that has been made in Tennessee.

Background

Studies conducted at the AASHO Road Test in the late 50s and early 60s demonstrated that 95% of pavement serviceability is controlled by the smoothness of the surface [1]. Pavement smoothness affects the dynamics of moving vehicles, impacting the rate of deterioration of the pavement and the operation and safety of vehicles and occupants. Consequently, the FHWA and many state transportation agencies have taken measures to address pavement smoothness during and immediately following construction. The significance of smoothness is evidenced by the preliminary recommendations for the adoption of the International Roughness Index (IRI) in the forthcoming AASHTO 2002 Pavement Design Guide.

In an attempt to improve pavement smoothness, TDOT adopted the Mays meter for use in acceptance testing of HMA pavements in the early 1980s. The Mays meter is a Response Type Road Roughness Measuring System (RTRRMS). In other words, it measures the displacement between the body and axle of a standard vehicle. Test procedures, equipment specifications, test and calibration site set up, and precision and bias statements for RTRRMS, including the Mays meter, are described in ASTM Standard Test Method for Measurement of Vehicular Response to Traveled Surface Roughness (E 1082) and ASTM Standard Practice for Calibration of Systems Used for Measuring Vehicular Response to Pavement Roughness (E 1448). As implied, the Mays meter measures the response of the standard vehicle to the roughness of the road [1]. Due to its relatively low cost, simple design, and high operating speed, the Mays meter has been widely used by highway agencies, as reported by the published NCHRP Project 1-31, "Smoothness Specifications for Pavements, Final Report [2].

The Mays meter is still used in Tennessee today. TDOT operates the Mays meter in accordance with ASTM E 1082 and maintains calibration of the equipment in accordance with ASTM E 1448. ASTM Terminology Relating to Vehicle-Pavement systems (E 867) is also referenced by TDOT. Per ASTM E 1082, the "standard vehicle" is any vehicle capable of housing the equipment or a tow vehicle with a suspension system independent of the Mays meter equipment. TDOT uses a tow vehicle and a Mays meter trailer assembly, as manufactured by Rainhart Co. of Austin, Texas. Further, TDOT maintains calibration sections that are one mile in length, over an established roadbed. This eliminates the roadbed consolidation variable and its effects on the equipment and test results. There is a smooth, a medium, and a rough pavement section established for calibration purposes in each of TDOTs four construction regions. When the Mays meter was first introduced, typical ranges of roughness for these calibration sections were classified as: (1) Smooth, 35-45 inches per mile (ipm) [552-710 millimeters per kilometer (mm/km)]; (2) Medium, 55-65 ipm (868-1026 mm/km); and (3) Rough, > 100 ipm (> 158 mm/km). As the smoothness program evolved and gained greater acceptance in Tennessee, this scale was shifted down to reflect the smoother pavement surfaces common throughout the state. Today, the ranges of roughness for calibration sections are classified as follows: (1) Smooth, 15-25 ipm (237-395 mm/km); (2) Medium, 35-45 ipm (552-710 mm/km); and (3) Rough, 55-65 ipm (868-1026 mm/km).

In the early 1980s, it was not uncommon to record mean Mays meter output (M.O.) for newly paved highway surfaces in Tennessee in excess of 50 ipm (789 mm/km). For example, in 1985, a high profile interstate project on I-40 was measured to have an average M.O. of 50 ipm (789 mm/km) with individual lots tested as high as 63 ipm (994 mm/km). This project and others led to the development of the current project quality award system and the incentive-based smoothness specification currently used in Tennessee. The project quality award program was implemented in Tennessee in 1986. The incentive-based smoothness specification was implemented for selected interstate projects in 1993. These aggressive incentive measures, along with significant improvements to equipment and operator training programs have resulted in continued improvements in pavement smoothness throughout Tennessee. As an example, in 1999, Renfro Construction Company completed a resurfacing project in McMinn County Tennessee with M.O. as low as 5 ipm (79 mm/km) for a single lot tested and a project average of 11 ipm (174 mm/km). A marked improvement over work completed only a decade earlier, and smoothness values approaching the accuracy threshold of the Mays meter.

TDOT Smoothness Specifications

The TDOT requirements for acceptance of HMA pavement surfaces are outlined in Section 411 of the TDOT Standard Specifications for Road and Bridge Construction [3]. Section 411 of the Standard Specifications contains a straight edge requirement of no deviation greater than 1/2" in 12 feet (13 mm in 3.7 meters). In addition, TDOT currently maintains two different supplemental specifications regarding HMA rideability. These supplemental specifications are outlined in Special Provisions 411-B and 411-C. Special Provision 411-B is an "Incentive" Specification, and 411-C is now considered to be the "Standard" Specification. It should be noted that if one of these Special Provisions were not included in the contract, then the straight edge requirement, described above would prevail. The two TDOT Supplemental Provision Specifications, 411-B and 411-C are summarized in Tables 1 and 2, respectively.

The primary difference between these two specifications lies in the level of smoothness that is accepted for full pay. The Standard TDOT Specification allows up to 40 ipm (631 mm/km) whereas the TDOT Incentive Specification limits full pay to 30 ipm (473 mm/km). As noted, the TDOT Incentive Specification offers the monetary incentive of a 1% bonus for lots achieving less than 20 ipm (316 mm/km). The obvious objective of the TDOT Incentive Specification is to reduce the mean M.O. to less than 20 ipm (316 mm/km), whereas the Standard TDOT Specification essentially encourages contractors to construct HMA pavements with M.O. up to 40 ipm 631 mm/km).

TDOT Quality Awards Program

In addition to addressing HMA pavement smoothness through specification revisions, TDOT implemented a Quality Awards Program. Introduced in the mid 1980s, this program is a non-monetary, pride-based method of promoting pavement smoothness. It has generated healthy competition among local contractors with respect to pavement smoothness. The Quality Awards Program in Tennessee includes awards in each of the four TDOT regions of the state, including a large and small project category. A Top Quality Award for the entire state is also included. As previously noted, these awards are non-monetary. The awards are presented in April at the annual TDOT Transportation Symposium. The various categories of awards provided under this program are summarized in Table 3.

Equipment Improvements and Training

Many changes have occurred in the HMA paving industry since the early 1980s. Actually, the principles of laying a smooth pavement have not changed significantly in over 40 years. The early asphalt pavers required the same care of operation as the new, more modern machines of today [4]. The principles of placing smooth HMA include: 1) Keeping a constant head of hot mix in front of the paver screed, 2) Not allowing the haul truck to bump the paver, 3) Never stopping the paver, 4) Constructing transverse joints properly, and 5) Not allowing the HMA to segregate. These five basic principles, together will affect the ultimate smoothness of the pavement. A great deal of resources has also been expended in recent years to train contractor personnel in the proper method of placement and compaction of HMA.

Equipment improvements have been directed at two fundamental areas. These include: 1) Keeping the paver moving at a constant speed, and 2) Re-mixing of the HMA prior to transferring to the paver. In Tennessee, these have principally translated into increased capacity truck dump hoppers that allow trucks to be dumped into the paver at very high rates (400-500 tons per hour) and material transfer devices that also provide increased capacity as well as remixing capability [4]. Experience in Tennessee has also shown that significant gains in pavement smoothness have been made by educating roller operators not to make abrupt starts and stops during compaction of the hot mat. Abrupt roller motions result in what is known as "roller header." These undulations often cannot be rolled back out, and result in rough pavements. In summary, with the use of long-standing, and well-documented paving techniques and increased hopper capacity and re-mixing capability of the equipment introduced during the 1980s, pavements have been constructed in Tennessee with documented M.O. values less than 5 ipm (98 mm/km).

Table 1 - *TDOT incentive HMA smoothness specification, Special Provision 411-B.*

Mays meter Output		Consequences
ipm	**mm/km**	
< 20	< 316	1% Bonus
20-30	316-473	Full Pay
30-60	473-947	Penalty Zone
> 60	> 947	Remove and Replace

Table 2 - *TDOT standard HMA smoothness specification, Special Provisions 411-C.*

Mays meter Output		Consequences
ipm	**mm/km**	
< 40	< 631	Full Pay
40-50	631-789	3% Penalty Zone
50-60	789-957	10% Penalty Zone
> 60	> 947	Remove and Replace

Table 3 - *TDOT annual quality awards.*

Annual Award Category	Number Awarded
Small Project Award (< $1 Millon)	1 per Region (4 Statewide)
Large Project Award (> $1 Million)	1 per Region (4 Statewide)
Top Quality Award	1 Statewide

TDOT Smoothness Data

Pavement smoothness data for the state of Tennessee for the first five years of implementation of the TDOT Incentive Specification are summarized in Figures 1 through 4. Figure 1 presents the mean M.O. for each year from 1993 through 1997, including both the Standard TDOT Smoothness Specification, and the TDOT Incentive Specification. Figure 2 presents this same data for the projects contracted under the TDOT Incentive Specification only. Figure 3 exhibits the relative number of lots tested for each respective specification (Standard versus Incentive) during the same five-year period, and Figure 4 presents the percentage of lane miles contracted using the TDOT Incentive Smoothness Specifications only.

Analysis of TDOT Smoothness Data

As can be seen in Figure 1, there was about a 5 ipm (79 mm/km) improvement in mean pavement smoothness in Tennessee from 1993 to 1997, approaching a mean value of about 25 ipm (395 mm/km) in 1997. It can also be seen that the projects contracted under the provisions of the TDOT Incentive Specification were consistently measured to be smoother by about 2 to 5 ipm (32-79 mm/km) than those contracted under the provisions of the Standard TDOT Specification. The fact that all projects, regardless of contract method, demonstrated a reduction in M.O. suggests that the improvements in equipment and training exercised during this period were also realized in terms of improved pavement smoothness. It is also probable that the TDOT Quality Awards program contributed to this documented improvement, although the relative contributions of each of these programs cannot be quantified. The fact that the projects contracted under the provisions of the TDOT Incentive Specification also demonstrated a relative reduction in M.O., as displayed in Figure 2, validates that the TDOT Incentive Specification has ultimately contributed to improved pavement smoothness in Tennessee. It should be noted that the trend toward smoother pavements for the projects contracted under the provisions of the TDOT Incentive Specification is evident from the data, although there is a relatively poor correlation with the trend line, as evidenced by the coefficient of determination of 0.23 exhibited on Figure 2. For verification of this trend, additional data for subsequent years will have to be evaluated in a similar fashion.

As shown in Figures 3 and 4, less than 20% of the HMA placed in Tennessee was contracted under the provisions of the TDOT Incentive Specification in 1993, whereas in 1997, more than half of the HMA placed was contracted under the provisions of the TDOT Incentive Specification. This suggests that increasingly, projects are being let under the Incentive Specification. As more contractors gain experience with the more stringent smoothness specifications, it is anticipated that pavement smoothness will continue to improve in years to come. As previously noted, some contractors in Tennessee are currently achieving M.O. values as low as 5 ipm (79 mm/km), for individual lots, and approaching 10 ipm (158 mm/km), for the overall project.

The Future in Tennessee

Future efforts to improve pavement smoothness in Tennessee are uncertain at this time. TDOT recently purchased South Dakota Profilers for each of the four regions of the state. The South Dakota Profiler makes use of relatively low cost ultrasonic sensors and computing capability to measure the distance from the vehicle body to the pavement surface. An accelerometer is used to compensate for the vertical motion of the vehicle body.

It is anticipated that these devices will be used in conjunction with a future warranty specification to document and monitor pavement smoothness over the warranty life of the pavement. The advantages of the South Dakota Profiler include the ability to operate at highway speeds and at relatively low cost. The primary disadvantage is considered to be limited accuracy [1, 2]. However, the accuracy of the South Dakota Profiler has been documented to be as good or better than the Mays meter. If accuracy of the equipment is found to be a significant handicap, it is anticipated that TDOT will readily adopt a profilometer device, as currently used by others [2, 5, 6, 7]. Regardless, it is clear that a higher standard has been set throughout Tennessee with respect to pavement smoothness, resulting from the combined efforts of TDOT, FHWA, and participating contractors.

In keeping with the trend of increased participation with the pavement smoothness incentive specification, as shown in Figure 1, it is expected that more and more projects

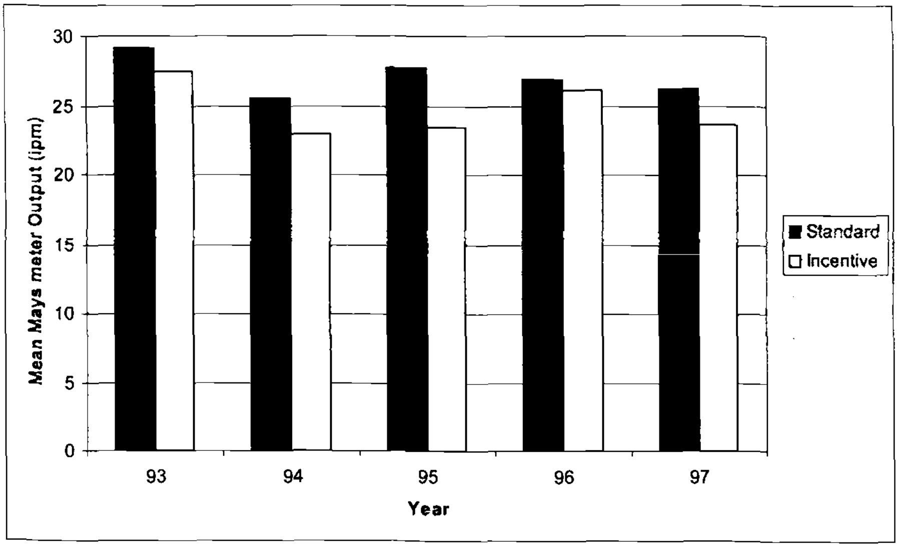

Figure 1 - *Mean M.O. for all projects tested in Tennessee, 1993 through 1997.*

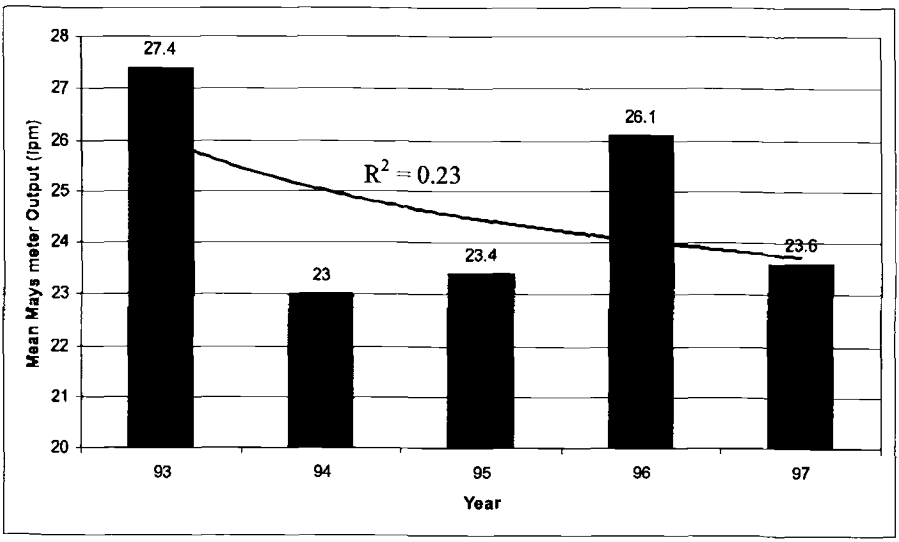

Figure 2 – *Mean M.O. for the projects contracted using TDOT Incentive Smoothness Specifications, 1993 through 1997.*

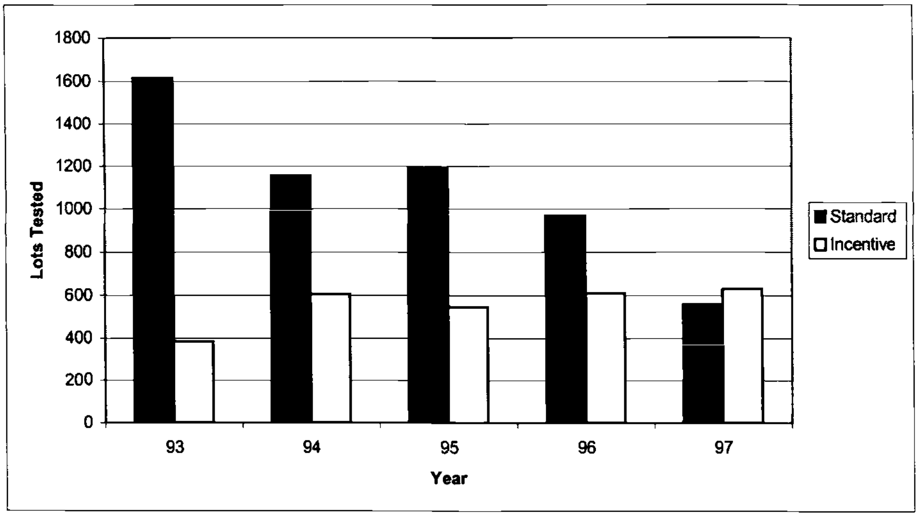

Figure 3 - *Number of lots tested for projects contracted using the Standard TDOT versus Incentive Smoothness Specifications, 1993 through 1997.*

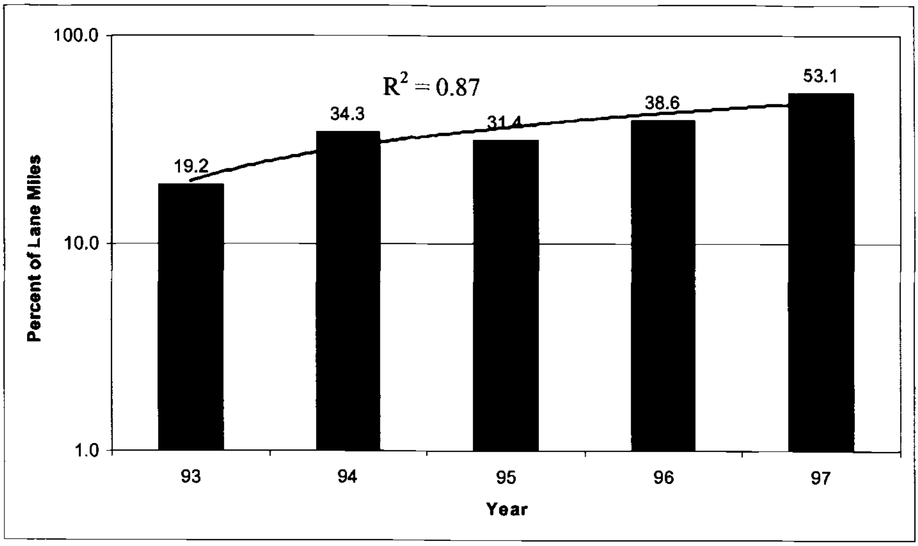

Figure 4 – *Percentage of lane miles contracted using the TDOT Incentive Smoothness Specifications, 1993 through 1997.*

will be contracted under the provisions of the TDOT Incentive Specification. As more and more contractors participate in the incentive-based program, the average or mean smoothness is expected to continue to improve. As quoted by one midwestern contractor, "... nothing motivates a contractor like a bonus" [5].

Summary Remarks

TDOT has worked with the FHWA and statewide paving contractors since the early 1980s to improve pavement smoothness on state highways. The steps taken thus far include the implementation of pavement smoothness Incentive Specification and competitive quality awards for outstanding contractors and projects. Improvements to HMA paving equipment and enhanced training of contractor personnel have also promoted smoother pavements. The results of Mays meter testing, as presented herein, demonstrate that there was about a 5 ipm (79 mm/km) improvement in mean pavement smoothness in Tennessee from 1993 to 1997. Further, projects contracted during this period, under the provisions of the TDOT Incentive Specification were consistently smoother by about 2 to 5 ipm (32-79 mm/km) than those contracted with the Standard TDOT Smoothness Specification. Based on the data presented herein, it is concluded that improvements in equipment and training as well as the implementation of the TDOT Quality Awards program and Incentive Specification have all together resulted in improved pavement smoothness on highways in Tennessee. Based on the documented past success of the TDOT incentive and awards approach, it is anticipated that these programs will continue into the foreseeable future, regardless of the methods and equipment used to measure smoothness.

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VDOT's IRI-Based Ride Quality Specification: From Inception to 2001

Reference: Clark, T. M. and McGhee, K. K., **"VDOT's IRI-Based Ride Quality Specification: From Inception to 2001,"** *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433,* M. S. Gardiner, Ed., American Society for Testing and Materials International, West Conshohocken, PA, 2003.

Abstract: In 1990, the Virginia Department of Transportation (VDOT) purchased its first South Dakota–type inertial road profiler. At that time, the profiler's primary function was to collect data required for the Federal Highway Administration's Highway Performance Monitoring System. By early 1995, however, the need to collect initial smoothness data safely, efficiently, and more accurately led VDOT to look to inertial profilers as a replacement for the California-type profilograph. In 1996, VDOT introduced a new special provision for smoothness, one that incorporated high-speed profilers and the International Roughness Index. In addition to chronicling those initial efforts, this paper discusses factors associated with achievable smoothness, as well as some deficiencies identified in the first generation of the new specification. The discussion then moves to a second generation of development, which focuses on enhancing the incentive/disincentive component of the specification, revising the smoothness targets, and reducing the pay lot size to combat excessive variability in ride quality.

Keywords: inertial road profiler, ride quality, smoothness specifications, roughness, International Roughness Index, incentives/disincentives

Introduction

In early 1995, the Virginia Department of Transportation (VDOT) began to develop the bid package for a rehabilitation project involving nearly 10 km (6 mi) and eight lanes of badly deteriorated interstate highway just southeast of the city of Richmond (the I-295 project). The existing continuously reinforced concrete pavement was to receive a new multiple-layered hot-mix asphalt concrete (HMAC) surface. This project was to be completed with minimum disruption to traffic and was to be constructed in accordance with the guidelines of a special provision for ride quality (as directed by VDOT's Chief Engineer).

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In 1995, VDOT's primary method for regulating the smoothness of highway surfaces used a specification built around the California-type profilograph [1]. The profilograph is a long (7.5 m, 25 ft) rigid frame assembly with several wheels at each end and a measurement wheel at the center. As the instrument moves along a surface, the center wheel travels up and down as it encounters variations in the surface. The amount of up and down movement is accumulated and reported as roughness. In some situations, a vehicle can tow the profilograph. More commonly, however, the instrument is pushed by hand.

VDOT's engineers had very good reasons for being reluctant to use the specification. First, administering the specification would involve manually propelling the profilograph for two passes over each of the eight lanes of the project, a total of nearly 155 km (96 mi) of profiling, if all went perfectly. A nearly universal trend toward fewer state force inspectors would make finding and devoting the necessary staff to such a formidable task difficult. Second, and perhaps more compelling, safety was an issue. According to statistics published by the Federal Highway Administration's (FHWA) Work Zone Safety Program, an average of 760 people are killed every year in work zone–related accidents [2]. Although most of these individuals are operating or traveling in motor vehicles, an average of 122 (16%) per year are not. Construction workers and inspectors make up the largest portion of the latter group. Thus, performing manual tests within several feet of interstate-speed traffic was unattractive, indeed.

A New Approach for Specifying Smoothness

Virginia's solution was a new special provision whereby testing could be conducted at highway speeds without directly exposing workers to traffic. In place of the California-type profilograph, an inertial road profiler was to be used. Inertial profilers are vehicle-mounted systems that measure longitudinal profiles in accordance with ASTM Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer Established Inertial Profiling Reference (E 950). These instruments typically combine accelerometers, height sensors, and electronic distance measuring equipment to collect two profiles with each pass, one representing the left and the other the right wheel-path. The conceptual difference between the inertial profiler and more traditional high-speed road roughness equipment is simple but important. Instead of measuring roughness as a response to the surface profile (e.g., Mays meter), the inertial profiler measures the profile directly.

To complement the inertial profiler and supplant the profile index from the profilograph, the new special provision defined smoothness in terms of the International Roughness Index (IRI).[3] The IRI, which is calculated using the ASTM Practice for Computing International Roughness Index of Roads from Longitudinal Profile Measurements (E 1926) is produced through a simulation that applies a "virtual" quarter-vehicle to an elevation profile such as that collected with the inertial profiler. To obtain the IRI, the suspension motion resulting from this simulation is accumulated and divided by the distance traveled [3]. Smaller values (less roughness) imply a smoother ride, and higher a rougher one.

The format of the new special provision resembled that of the profilograph-based specification. An average IRI was generated and reported for each 160-m (0.1-mi) pay lot. These values were then compared with a pay adjustment schedule that incorporated a target band for full payment and several pay ranges in which incentives or disincentives might be applied. In addition to the IRIs generated for each pay lot, IRIs were generated at 10 subintervals and these values were reviewed to identify localized roughness or bumps and dips. A threshold for allowable roughness (maximum IRI) was specified for both the pay lot and the subintervals. Roughness above these thresholds was subject to correction.

A "Maintenance Special Provision" for Smoothness

Although highly visible construction projects (such as the I-295 project) are important, they represent only a fraction of the HMAC pavement placed during a typical construction season. In Virginia, the annual maintenance resurfacing program is responsible for a much larger portion of new surface. Every year, VDOT's maintenance resurfacing program places 2 million metric tons of HMAC over almost 6 000 lane-km (3 600 lane-mi) of existing pavements. The real potential for a smoothness special provision of the type proposed would be realized only through its application to this program. Thus, the experimental smoothness specification was adapted specifically to maintenance resurfacing projects (the Maintenance Special Provision for Smoothness) and entered its initial pilot phase during the 1996 construction season.

Achieving and Measuring Smoothness of Asphalt Overlays

Virginia's adoption of high-speed inertial profilers and the IRI to regulate pavement smoothness was a significant departure from tradition. The established smoothness targets needed to be achievable yet appropriately challenging. It was also important that VDOT engineers understand how particular project variables affected the final surface ride quality. Recognizing these issues, the Virginia Transportation Research Council (VTRC) initiated a research project to accompany the debut of VDOT's special provision [4]. The foundation of the project was an extensive rideability survey covering 4 270 lane-km (2 650 lane-mi) of new HMAC paving, encompassing two full construction seasons (1996 and 1997) and the entire state.

Achieving Smoothness

A primary objective of the project was to identify the predominant factors affecting the achievable smoothness of asphalt overlays. The study examined variables that were subject to control by the contracting agency (VDOT, in this instance). Examples

[3]Virginia generally reports roughness as an average of two wheel-paths of IRI values, or a Mean Roughness Index (MRI), as defined in ASTM Terminology Relating to Vehicle-Pavement Systems (E 867). In most instances, IRI is used in lieu of the more technically correct MRI. For much the discussion included in this paper, MRI and IRI are used interchangeably. Virginia also departs slightly from the recognized standard for reporting metric IRI. Instead of meters per kilometer or millimeters per meter, Virginia reports metric IRI in millimeters per kilometer.

included the thickness and type of overlay material, the use of milling, the application of additional structural layers, and time-of-day restrictions on construction activities.

Factors Associated with Achievable Smoothness—Only three variables were notably associated with the achievable smoothness of an overlay: the roadway's highway system classification, the ride quality of the original (underlying) pavement, and whether the overlay was subject to the special provision for smoothness.

Factors Not Associated with Achievable Smoothness—Variables that were expected to be associated with achievable smoothness but were not included surface mix type, additional structural layers, milling, and the requirement to perform the work at night.

Measuring Smoothness

In addition to studying the characteristics contributing to achievable smoothness, the VTRC study provided a critical assessment of the non-traditional equipment and methods as used to administer the new special provision. This assessment examined sub-sections of paving projects that were typically exempt from the special provision. It also critiqued the effectiveness (or ineffectiveness) of the new special provision in identifying and addressing intra-project construction variability.

Exempt Sections—Typically, the beginning and end of overlay projects and areas immediately adjacent to any interior bridges are exempt from the requirements of smoothness specifications. Although it is generally accepted that the overlay will be rougher in these areas, it was not clear how much additional roughness (if any) should be expected and over what length it could be distributed. This research found that the traveling public should expect to encounter approximately 70% more roughness at the first pavement joint, 45% more at the last joint, and about 46% more at either end of bridges.

Table 1 summarizes the lengths necessary to address (gain/lose control of ride quality for) the respective features in each of the three highway system classifications. Although the length of overlay affected by the first and last joints were comparable, the length affected by the beginning joint was nearly universally longer than that at the end. The reported length of affected overlay at bridge approaches includes the total from before and after the structure. For the entire state, contractors were usually able to gain control of a surface within 84 m of the beginning of an overlay. In at least 32 cases (of 426) in the 1997 construction season, however, the contractor had established control over ride quality within the first 16-m (0.01-mi) subinterval.

Table 1—Average length to "equilibrium."

	Beginning Joint, m	Ending Joint, m	Bridge Approaches, m
Interstate	55	46	116
Divided Primary	84	73	172
2-Lane Primary	94	88	188

Intra-Project Variability— Many overlays placed on Virginia's highways were found to have a good average initial smoothness, even though they otherwise appeared ordinary to marginal in quality. Part of this inconsistency may be attributable to a uniform roughness of a wavelength that is discernible to the traveler but not significant to the IRI algorithm. However, the researchers theorized that this perceived roughness was due more often to objectionable levels of construction variability. In these cases, the averaging approach in the special provision tended to mask the resulting fluctuation in ride quality. In spite of the use of the short-interval (16-m, 0.01-mi) reports, which were generated specifically to identify locally rough spots, the relatively long pay lots (160-m, 0.1-mi) were effectively bridging significant surface events.

To illustrate, Figure 1, which plots the number of possible corrections per kilometer of paving versus the theoretical average percent paid (for 1997 resurfacing data), clearly shows the strong and desirable correlation between potential corrections and potential payment. However, it also shows that pavements with smoothness conforming to and exceeding the requirements of the special provision could exhibit frequent local roughness in excess of the limit at which corrections could be necessary. In fact, for projects that would have been eligible for 100% payment or better (as per the special provision), there was an average of just over one potential correction per kilometer. In the worst case, a project that would have fallen just short of achieving the maximum bonus for smoothness would also have been subject to nearly three potential corrections per kilometer.

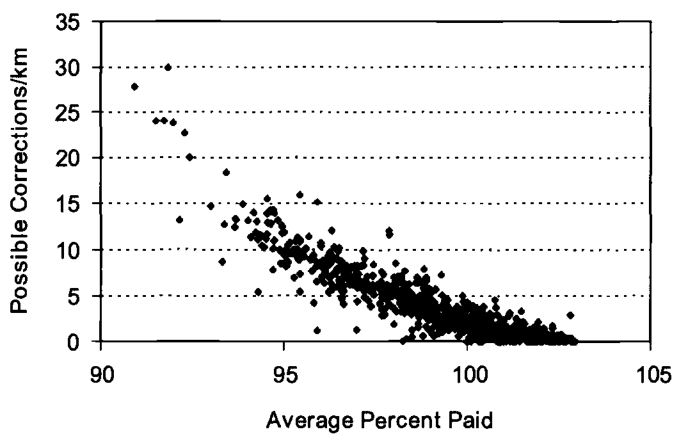

Figure 1—Possible corrections versus average percent unit price paid.

Early Evolution and Application of the Maintenance Special Provision (1996–1999)

After its introduction in 1996, the language in Virginia's special provision was revised slightly in each of the following three years. The following sections chronicle the major revisions and the extent of the provision's application during those early years.

1996

In its original form, the Maintenance Special Provision for Smoothness offered a single schedule of pay adjustments, regardless of highway system or other important peculiarities of a project. To receive 100% of the surface material bid price, a contractor needed to achieve a final surface IRI of 1 100 to 1 260 mm/km (70 to 80 in/mi) over the 160-m (0.1-mi) lot. The maximum allowable IRI for a 160-m pay lot was 1 580 mm/km (100 in/mi). Within that lot, the maximum allowable IRI of any 16-m subsection was 1 900 mm/km (120 in/mi). In 1996, the provision was applied to 81 lane-km of the HMAC resurfacing (plant mix) schedule.

1997

In 1997, the pilot of the Maintenance Special Provision was expanded to 611 lane-km (380 mi) in six of Virginia's nine construction districts. Although minimal, the special provision used in the second season of the pilot did undergo a couple of minor changes. The maximum incentives and disincentives were reduced and the pay bands were broadened slightly. The target smoothness range necessary to achieve 100% payment was not changed, but the maximum average IRI eligible for payment was increased to 1 700 m/km (110 in/mi). Perhaps the most significant change was acknowledgment of the influence of original surface ride quality (substantiated by the study conducted by VTRC). New language specified that a project was not eligible for an incentive if the final surface was rougher after completion of the work, regardless of the average achieved ride quality. Conversely, if a contractor effected at least a 25% improvement (over the original surface) in ride quality, he or she would not be subject to a disincentive, regardless of the degree of roughness remaining in the final surface.

1998

By late summer 1997, the specification revisions governing the 1998 construction season were complete. The 1998 version also reflected preliminary findings of the VTRC research by providing separate pay adjustment tables for interstate and non-interstate projects. Table 2 lists the 1998 pay schedule with accompanying target IRIs. In this schedule, contractors working within the special provision on an interstate highway were required to reduce the pavement roughness by an additional 160 mm/km (10 in/mi) with the new surface.

The 1998 version of the smoothness provision also addressed the VTRC findings relating to exemptions. Specifically, the length of exempted pavement section before and after bridges and at the beginning and end of a project was reduced from 160-m (0.1-mi) to 16-m (0.01-mi).

1999

For the 1999 construction season, VDOT applied the Maintenance Special Provision to approximately 100 plant mix projects. Some districts had as few as 6 projects, but the average number of projects per district was 15. The provision used in 1999 differed little

Table 2—Pay adjustment schedule for 1998 construction season.

IRI After Completion (mm/km)	Pay Adjustment (% pavement unit price)
Interstate System	
Under 710.0	104
710.1–790.0	103
790.1–870.0	102
870.1–950.0	101
950.1–1100.0	100
1100.1–1260.0	98
1260.1–1420.0	95
1420.1–1580.0	90
Over 1580.1	Subject to corrective action
Non-Interstate System	
Under 870.0	104
870.1–950.0	103
950.1–1025.0	102
1025.1–1100.0	101
1100.1–1260.0	100
1260.1–1420.0	98
1420.1–1580.0	95
1580.1–1740.0	90
Over 1740.1	Subject to corrective action

IRI units may be converted to in/mi by multiplying by 0.06336.

from that developed for the 1998 construction season. By 1999, however, VDOT maintenance and materials engineers were becoming more familiar and, to a certain extent, less forgiving of the details that made up the special provision. Although these engineers were generally satisfied with the achieved results, they collectively began to identify opportunities for improvement.

Formalizing the Evolution of the Special Provision for Smoothness

During its first few years of existence, the Maintenance Special Provision evolved arbitrarily. Although some changes were supported by documented research findings, others seemed simply to reflect anecdotal concerns expressed by district pavement engineers and/or industry representatives. In November 1999, a small group of experts (the Ride Spec Committee) was assembled and asked to formalize the continued development of all provisions relating to ride quality of pavements. This committee consisted of representatives from VDOT's materials and maintenance divisions, VTRC, and an individual from the FHWA's Division Office in Virginia.

Agency Issues

The first and major issue facing the committee was construction variability as it related to ride quality (see discussion on *Intra-Project Variability* from VTRC research). Because of the "averaging" nature of the specification, VDOT was paying 100% (even bonuses) for smoothness on many projects with obvious bumps and dips. A second issue concerned the penalties outlined in the special provision; they were not severe enough for a contractor to change paving operations. Many "ride spec" paving projects were part of a larger county or district-wide paving contract (within which few projects were subject to the special provision for smoothness). A small loss on one site because of smoothness disincentives would often have a negligible impact on the overall contract.

Industry Issues

The Ride Spec Committee also considered issues raised by the asphalt paving industry. As with VDOT personnel, two issues were presented. The first concerned incentives. The maximum incentive a contractor could receive on a project was 4% of the asphalt concrete surface price. In comparison to the requirements for full payment, the contractor needed to achieve at least another 27% reduction in roughness to qualify for the maximum incentive (see Table 2). For many contractors, the effort in terms of dollars exceeded the additional potential money from a 4% bonus. The second issue concerned site selection. For the majority of paving projects under the special provision, the roadway geometries provided the contractor few, if any, difficulties. Most projects were on a four-lane divided highway, with no curb and gutter, more than 0.8-km (0.5-mi) long, with limited intersections. However, some projects were placed on roads with numerous curves, steep grades, and multiple intersections.

Committee Issues

The Ride Spec Committee also addressed its own concerns. An example was modification of the "percent improvement clause," introduced in the 1997 version of the provision. Another example was the testing timeframe for "before" and "after" surveys.

Development of 2000 Maintenance Special Provision for Smoothness

In January 2000, the Ride Spec Committee began the most significant overhaul of the special provision since its inception in 1996. In addition to the input from VDOT field personnel and the Virginia paving industry, the committee gathered smoothness specifications from other states to gage current practice. Most states continued to use a profilograph-based specification, although an increased number were using or considering a special provision based on inertial profilers. Several of those states and transportation agencies paid contractors in accordance with a pay table using the IRI from a 160-m (0.1-mi) interval. However, the target IRIs used for payment were not consistent among agencies. Some used an average of both wheel-paths (VDOT's approach), others used the right wheel-path only, and still others used the half-car roughness index (HRI).

A few states, such as Maryland, were developing equations to determine payment as an alternative to pay tables/schedules.

A 2000 Pilot Special Provision

It soon became clear that no national approach to ride testing and payment existed. To minimize confusion, the Ride Spec Committee decided to begin developing the next generation profiler-based specification by revising the language and tables from VDOT's own 1998 special provision. Within that development, four of the issues introduced earlier were targeted.

Section Length to Base Payment—The Ride Spec Committee considered several methods to approach construction/smoothness variability. The first considered employing statistics for application to each 160-m (0.1-mi) pay lot. The revised special provision would base payment on a combined average IRI and standard deviation for each lot. Although the statistical concept had merit, the new special provision had to be completed by summer 2000. The committee agreed that investigating the use of statistics could be a long-term goal but could not be achieved by the deadline.

The second approach (and the one eventually incorporated into the pilot) involved revising the section length used for payment. Since its inception, the VDOT special provision had included a "bump/dip" clause, which set a maximum allowable IRI for any 16-m (0.01-mi) sub-section. If those limits were exceeded, the entire 160-m (0.1-mi) section containing the sub-section was not eligible for incentive payment. Too often, as a consequence, the 160-m pay lot either overlooked an undesirable amount of internal fluctuation in IRI or played a part in over-penalizing the contractor for more localized problems. This led to the adoption of the new base payment length of 16-m (0.01 mi). Since the calculation of pay adjustments (from the IRI reports) was nearly completely automated, the smaller pay lot size represented a negligible increase in effort.

Pay Tables and IRI Target Ranges—A concern raised by both VDOT personnel and industry was payment. Compared to many specifications reviewed by the committee, VDOT's 1998 special provision contained relatively small incentives and small disincentives. Using the incentives offered by other transportation agencies, the Ride Spec Committee conducted an exercise to determine what affect a spectrum of maximum incentives might have on a typical district's paving funds. The maximum bonus had to be large enough to encourage the contractors to improve on the paving processes but could not be so large that paving projects would be cancelled because of a lack of funds. For the maximum penalty, the committee resorted to expert opinion to set the value. Unfortunately, although the idea that initial roughness leads to shorter service life is widely accepted, there are little data supporting an exact mathematical relationship.

Unofficial tests conducted with VDOT profilers have demonstrated that IRI results for a 0.016-km (0.01-mi) section can vary between 2% and 10%. For the most part, this variability can be attributed to operator wander and longitudinal referencing. Although longitudinal referencing could be addressed using electronic triggering of the inertial profiler, some wander will always exist. Other research has confirmed that moving the laser footprint 20 or 30 mm left or right can affect the IRI results [5]. Recognizing that the variability in results can be reduced but not eliminated, the Ride Spec Committee compared the potential variability in IRI between two or more passes to the IRI ranges in the 1998 special provision. The provision contained nine IRI ranges, with several of the ranges having a span of 80 mm/km (5 in/mi). This comparison led the committee to combine the incentive ranges and expand the 100% payment band (see Table 3).

Table 3—Pay adjustment schedule for 2000 pilot special provision.

IRI After Completion (mm/km)	Pay Adjustment (% pavement unit price)
Interstate System	
Under 710	110
710.1–870	105
870.1–1100	100
1100.1–1260	90
1260.1–1420	80
1420.1–1580	60
Over 1580	Subject to corrective action
Non-Interstate System	
Under 870	110
870.1–1025	105
1025.1–1260	100
1260.1–1420	90
1420.1–1580	80
1580.1–1740	60
Over 1740	Subject to corrective action

IRI units may be converted to in/mi by multiplying by 0.06336.

Improvement Requirements—Each year VDOT paves roads where it is virtually impossible to achieve a final surface ride quality that would meet the requirements of the pay schedule for 100% payment. Anticipating that some of these roads would nonetheless be subjected to the special provision, the 1998 provision allowed 100% payment for a 25% reduction in IRI. However, even when limited to a single lift of HMAC, Virginia's better contractors consistently achieve a 30% improvement or more, depending on the existing ride quality. Although earlier VTRC research [*4*] indicated that multiple lifts and milling had a minimal affect on the final IRI, contractors were decreasing the IRI on rougher roads by a much larger percentage: 50% or more. Improved ride quality was attributed to improvements in paving equipment, e.g., material transfer devices and milling machines with skids. Therefore, realizing ride quality could be improved by a larger percentage while still protecting contractors from insurmountable initial conditions, the Ride Spec Committee proposed an increase in the minimum improvement percentage from 25% to 40%.

Allowable Testing "Windows" —Although not a significant issue for most VDOT field personnel and contractors, the time periods for "before" and "after" IRI testing were a major issue for VDOT's Non-Destructive Testing Unit, which provides the testing service. In the 1998 special provision, "before" testing must be performed no more than 60 days prior to paving. With the paving season starting April 1 and concluding November 1, and no required paving sequence for contractors, many sites had to be tested several times to meet the 60-day limit. Since analysis of historical IRI data showed little change from year to year, short of catastrophic failures, the testing window was expanded from 60 to 180 days.

The "after" testing in the 1998 special provision stated that testing must be performed within 14 days of completion of the final surface. For most projects, the days after paving are spent placing shoulder stone, painting lines, adjusting guardrail, etc., which makes access to the site very difficult. Since the change in IRI was observed to be minimal during the first few weeks after paving (typically gets smoother because of additional compaction), the "after" testing window was increased from 14 to 30 days.

A "Shadow" Application of the Pilot Provision

Traditionally, VDOT analyzes the implications of a proposed special provision through its mock (shadow) application on several projects. In this instance, VDOT selected three projects in the Staunton District to test the 2000 pilot special provision. One project was considered to have excellent ride quality, one project had average ride quality, and one project had poor ride quality. All three projects were located on interstates, where good ride quality after paving was essential. Table 4 compares the outcome of an application of the 2000 pilot special provision to that of the 1998 version.

Table 4—Results of "shadow" special provision.

Parameter	Project 1	Project 2	Project 3
Average IRI, mm/km (in/mi)	1215 (77)	1105 (70)	584 (37)
Percent Improvement	5	14	46
Payment, 1998 Special Provision	($10,700)	$700	$6,100
Payment, 2000 Pilot Special Provision	($20,900)	($2,950)	$13,100

Clearly, the 2000 pilot special provision would have affected the amount paid to the contractor. For Project 1, the penalty would be twice as much. For Project 2, bumps and dips (construction variability) that were de-emphasized by the 1998 special provision were isolated and penalized by the 16-m (0.01-mi) pay lots of the proposed 2000 pilot. For Project 3, the contractor was able to improve the ride quality by more than 40% and received a bonus. If the 2000 pilot special provision had been in effect, the contractor would have doubled his bonus.

2000 Pilot Special Provision Projects and Results

With the shadow application demonstrating reasonable and encouraging results, the Ride Spec Committee moved the special provision to a "live" pilot phase. To that end, five interstate paving projects in the Staunton District were subjected to the pilot special provision and constructed during the summer of 2000. The sites were selected because of the different scopes of work: $200,000 to $425,000 worth of HMAC; mill-and-replace; or mill, replace, and overlay with 38- to 50-mm (1.5 to 2 in.) surfaces. The intention was to ensure the pilot special provision could be applied to the projects routinely encountered on the annual plant mix schedules.

For each project, VDOT performed the same analysis performed on the shadow projects. Table 5 summarizes the results.

Table 5—Results of pilot special provision.

Parameter	Project 4	Project 5	Project 6	Project 7	Project 8
Average IRI, mm/km (in/mi)	710 (45)	900 (57)	947 (60)	979 (62)	710 (45)
Percent Improvement, %	34	33	27	26	48
Payment – 1998 Special Provision	$11,200	$1,000	($200)	$60	$13,900
Payment – 2000 Pilot	$24,400	$1,900	$1,200	($2,900)	$24,700

The contractors received a bonus for all but one project. For Project 6, the contractor would have been subject to a disincentive had the 1998 special provision been applied. The slight incentive (using the 2000 pilot) resulted from the higher bonus percentages and the elimination of the "bump/dip" clause. A few 16-m (0.01-mi) sections had very high IRIs. With the 1998 special provision, the entire 160-m (0.1-mi) section would have been penalized. In the pilot special provision, only the offending 16-m (0.01-mi) sections were affected. As with Project 2, Project 7 had numerous bumps and dips in the final pavement surface. With the increased penalty percentages and 16-m (0.01-mi) pay lot, the contractor was assessed a penalty.

Revisions to the 2000 Pilot Special Provision

After the completion of the pilot projects, several meetings were held with the asphalt paving industry to discuss the results and to make recommendations for the 2001 Plant Mix Schedules. For the development of many of VDOT's special provisions, VDOT and industry partner to resolve differences. In September and October 2000, meetings with the asphalt paving industry revolved around three main issues: the smaller (16-m) pay lot size, ride quality improvement requirements, and incentive/disincentives.

Regarding the 16-m (0.01-mi) pay lot, the asphalt industry wanted VDOT to return to the 160-m (0.1-mi) lot size. Although VDOT listened to their position, the experience with the shadow and limited pilot application suggested that the move to the 16-m (0.01-mi) pay lot would be worthwhile. For the percentage improvement clause, VDOT and the asphalt paving industry compromised on a value of 30%. However, for those projects involving two or more lifts of asphalt concrete, the percentage improvement clause would not apply. The contractor would be paid based on the achieved final surface IRI, regardless of original surface condition. On the last issue, the asphalt industry asked for a balanced scale. They contended that if the maximum penalty was 40%, the maximum bonus should be 40%. Although VDOT did not agree with the industry's position, the maximum penalty was reduced from 40% to 30%. These changes in the pilot special provision were incorporated in the 2000 Special Provision for Rideability.

Phasing in the 2000 Special Provision for Rideability

For the 2001 construction season, VDOT used two special provisions for rideability. Although the Ride Spec Committee recommended the 1998 special provision be eliminated and replaced with the 2000 special provision, the state construction engineer decided that a phased implementation plan would be more prudent. Within this plan, each construction district was assigned a special provision. Table 6 indicates how the special provisions were allocated. Three districts were assigned the 2000 special provision exclusively, and 5 were instructed to use the 1998 version. The Fredericksburg District was permitted to use the 2000 special provision on its interstate plant mix work, but the 1998 special provision was incorporated into the district's non-interstate plant mix schedules.

Table 6—Rideability projects in 2001.

District	Active Provision	Rideability Projects
Bristol	1998	10
Salem	1998	8
Lynchburg	1998	21
Richmond	2000	11
Hampton Roads	2000	45
Fredericksburg	Both	26
Culpeper	1998	14
Staunton	2000	15
Northern Virginia	1998	12

Preliminary Results from 2001

With approximately one third of the 2001 Plant Mix Schedule projects completed, the state materials engineer requested an analysis of the results in order to provide a recommendation to the chief engineer regarding the 2002 schedule. For this analysis, IRI data were summarized by highway system and special provision.

As expected, interstate projects were providing the lowest average IRI values and the U.S. and state routes, respectively, followed with slightly higher values. In all cases, average work was producing IRI values that would warrant 100% payment.

Table 7 summarizes the data by special provision. Overall, the ride quality resulting from work performed under the 2000 special provision was better than that done under the 1998 special provision.

Table 7—2001 average IRI by special provision.

Special Provision	Average IRI, mm/km (in/mi)
1998 Special Provision	1120 (71)
2000 Special Provision	1057 (67)

Based on the data collected and analyzed for the 2001 Plant Mix Schedules, the VDOT Materials Division recommended the use of the 2000 special provision for all schedules in 2002. Although a comparison showed little to no overall improvement in the IRI as a consequence of using the 2000 special provision, it does allow VDOT to reward more effectively contractors who provide better than average ride quality and to penalize those who provide less than average ride quality.

Conclusions

To be truly effective, any construction specification must evolve to complement and/or take advantage of the very latest equipment, techniques, and concepts. As illustrated through this paper, this is especially relevant for modern smoothness provisions for pavements.

Fortunately, today's capabilities for measuring ride quality will permit a much more functional "cradle to grave" tracking system than was possible with more traditional equipment (e.g., the profilograph). Measuring initial smoothness and conducting regular inventory assessments using a common index (the IRI in this case) will enable engineers to measure definitively how smoothness changes with time and loading. This will promote the development of credible relationships between initial smoothness and required levels of maintenance and service life. Ultimately, this understanding will result in incentives and disincentives that tie directly to increased and decreased agency costs.

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National and International Perspectives

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Smoothness Index Relationships for HMA Pavements

Reference: Evans, L. D., Smith, K. L., Swanlund, M. E., Titus-Glover, L., and Bukowski, J. .R., "**Smoothness Index Relationships for HMA Pavements**," *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433*, M. S. Gardiner, Ed., American Society for Testing and Materials International, West Conshohocken, PA, 2003.

Abstract: A recent Federal Highway Administration survey indicated that 48 states and Puerto Rico use smoothness specifications for hot-mix asphalt (HMA) pavement construction. As this is a relatively new concept, many states have adapted the Portland cement concrete pavement specifications of Profile Index using the standard 5-mm blanking band ($PI_{5.0}$) for use on HMA pavements. However, $PI_{5.0}$ may not provide a reproducible or portable smoothness measure for HMA pavements because of the technical limitations of the equipment and procèdures.

The International Roughness Index (IRI) or the Profile Index using a 0.0-mm blanking band ($PI_{0.0}$) seem to provide a more repeatable and portable smoothness standard. However, one barrier to more widespread implementation of these new smoothness standards is the lack of objective, verifiable correlation methods for use in establishing specification limits using the IRI or $PI_{0.0}$. Assistance in selecting appropriate IRI and $PI_{0.0}$ specification limits is needed to provide a basis for modifying current specifications to these more reproducible and portable smoothness indices.

This research effort has developed a relationship between IRI and PI that can assist in transitioning to a reproducible and portable initial IRI or $PI_{0.0}$ smoothness specification for HMA pavement.

Keywords: International Roughness Index, Profile Index, correlation, specification, smoothness

Introduction

Initial pavement smoothness is a key factor in the performance and economics of a pavement facility. All other things being equal, the smoother a pavement is built, the

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smoother it will stay over time (1). The smoother it stays over time, the longer it will serve the traveling public, thereby benefiting the public in terms of investment (initial construction and upkeep) and vehicular wear costs, as well as comfort and safety.

The importance of pavement smoothness has long been recognized, but it is only in the last 10 to 20 years that the degree of importance has been largely discerned. While early (late 1950s and 1960s) smoothness specifications generally reflected the levels of smoothness attainable in that period, the development of many new technologies over the years in the areas of materials and paving equipment and practices has resulted in the construction of increasingly smoother pavements. The beneficial effects of these smoother pavements, as identified in various national and state research studies, as well as a greater emphasis on the customer (i.e., the highway user), has resulted in a continual upgrading of smoothness specifications.

Whereas a 1981 survey by the American Association of State Highway and Transportation Officials (AASHTO) showed only 17 states having a rideability-type smoothness specification, a recent Federal Highway Administration (FHWA) survey indicates that 47 states utilize rideability specifications for new hot-mix asphalt (HMA), otherwise called asphalt concrete (AC), pavement construction (2). The majority of these specifications are based on tests performed with a profilograph.

The profilograph can be generally described as a 7.6-m (25-ft) rolling reference system capable of producing surface profile traces, which can be evaluated to identify severe bumps and to establish an overall measure of smoothness [i.e., the profile index (PI)]. Various types, makes, and models of profilograph are available, with the California-type systems being the most widely used. Also, various methods can be used to compute the PI smoothness index.

Although the profilograph has served the highway community fairly well as an easily understood index of initial pavement smoothness, concerns about its accuracy and its relationship with user response (fair to poor) have grown significantly in the last decade. For instance, because the device measures only wavelengths within the range of 0.3 to 23 m (1 to 75 ft) and because it amplifies wavelengths that are factors of its length [i.e., 7.6 m (25 ft)], the profile it produces is biased from a pavement's true profile. This can be seen in Figure 1, where a true profile would be represented by a gain of 1.0. Also, the PI statistic attenuates longer pavement wavelengths and amplifies shorter wavelengths. Coupled with these facts, a 2.5- or 5-mm (0.1- or 0.2-in) blanking band is often applied when computing PI, thereby masking short wavelength roughness. It is, therefore, understandable how correlation with user response is generally deemed inadequate.

Over the last six years, a handful of state agencies have moved toward using a zero blanking band ($PI_{0.0}$) statistic for construction acceptance testing. This has reportedly improved the ability to control initial smoothness and bettered the relationship between profilograph PI and user response. However, the fact that the same biased profiles are being used to compute $PI_{0.0}$ does not fully alleviate the major concerns with the profilograph.

Among many agencies, the belief persists that inertial profilers are the best available means for specifying and evaluating initial smoothness. This equipment consists of an integrated set of vertical displacement sensors, vertical accelerometers, and analog computer equipment mounted in a full-sized vehicle (usually a van or large automobile) equipped with a distance-measuring instrument (DMI). Inertial profilers can produce a more definitive profile of a pavement, from which the widely accepted International Roughness Index (IRI) can be computed.

Inertial profilers are used extensively in pavement management for monitoring pavement smoothness over time. In recent years, they have seen an increased use in construction acceptance testing of asphalt pavements, and the profiling instruments have been adapted to lightweight vehicles (e.g., utility carts, all-terrain vehicles [ATVs]) for testing of concrete

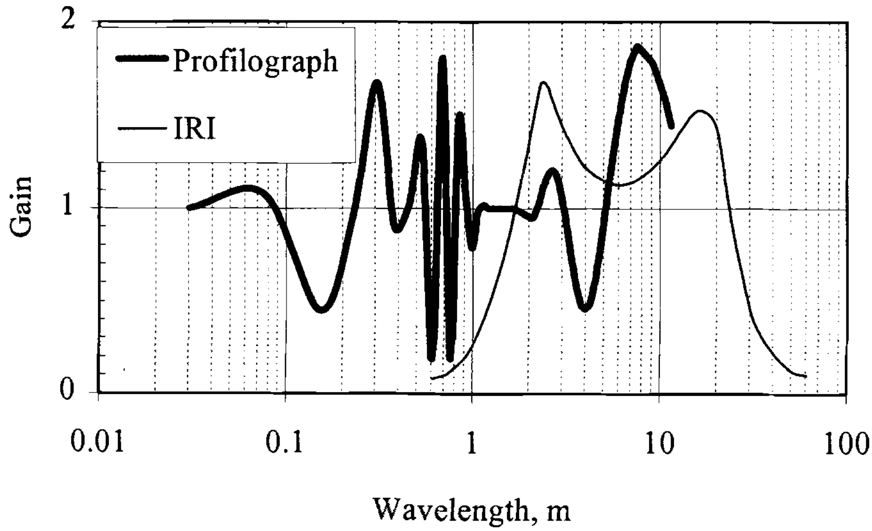

Figure 1 – *Sensitivity of IRI and simulated profilograph to wavelength.*

pavements. These lightweight profilers enable testing personnel to obtain timely and highly definitive measurements of surface profiles at rates of speed significantly higher than profilographs [4 km/hr ([15 mi/hr) versus 5 km/hr (3 mi/hr)]. The profilers are capable of producing IRI and other indices [e.g., simulated PI and Mays output, ride number (RN)] commonly used in controlling and monitoring pavement smoothness.

With the trend being toward using inertial profilers in construction acceptance testing and with high interest among agencies for a "cradle-to-grave" smoothness index, it is quite apparent that the PI-based smoothness specifications so prevalent today will be transformed in coming years to IRI specifications. This switch will not be easy. Agencies will need to assess current PI limits and determine the levels of IRI that best reflect those limits, given the type of profilograph and PI computation procedures currently used. They may also choose to transition to PI using a tighter blanking band.

An FHWA study was recently undertaken to help agencies in the transition from PI- to IRI-based specifications. The study, entitled "Development of Smoothness Relationships," involves the development of PI–IRI relationships that can be used to formulate supportable IRI smoothness limits. These relationships are being derived from an analysis of comprehensive time history smoothness data collected by high-speed inertial profilers under the Long-Term Pavement Performance (LTPP) program. Using advanced computer simulation algorithms, it is possible to compute PI values from the surface profile data, thereby allowing detailed comparisons between IRI and PI.

It should be noted that the IRI model focuses on relatively the same pavement wavelengths as the PI model—0.3 to 23 m (1 to 75 ft). However, the IRI statistic amplifies and attenuates different pavement surface wavelengths than the PI statistic. This can be seen in Figure 1, which indicates that for IRI there is a significant gain for wavelengths at about 16.1 m (52.5 ft) and 2.2 m (7.1 ft). Compared with the profilograph effect, the IRI model amplifies and attenuates profile features at different wavelengths than the profilograph, making it difficult to obtain an exact correlation between the two statistics. Nevertheless, small data sets have indicated that the correlation is relatively good.

Objectives

The specific objectives of the FHWA Smoothness Relationships study include the following.

- Analyze LTPP profile data from General Pavement Studies (GPS) and Specific Pavement Studies (SPS) test sections for IRI and PI using the 0.0-mm (0.0-in), 2.5-mm (0.1-in), and 5.0-mm (0.2-in) blanking bands. This includes profile data from AC and Portland cement concrete (PCC) test sections in the four LTPP climatic zones: dry freeze (DF), dry nonfreeze (DNF), wet freeze (WF), and wet nonfreeze (WNF).

- Compile and provide recommendations for smoothness specification acceptance limits for new and rehabilitated PCC and hot-mix asphalt (HMA) pavements, based upon IRI and PI.

Data Collection and Database Development

Since the time the LTPP program was initiated in 1989, several hundred pavement test sections throughout the United States and Canada have been surveyed for smoothness on an annual or biennial basis using full-sized, high-speed inertial profilers. In each test, the longitudinal surface profile of each wheelpath was measured and recorded, and from those profiles the IRI of each wheelpath was computed and recorded for inclusion in the LTPP Information Management System (IMS) database. The sections below describe the collection of LTPP data and the development of the project database used to examine the relationship between IRI and PI.

To retrieve the profile and smoothness data required for this study, all 1996-2001 archived profile data contained in the LTPP Ancillary Information Management System (AIMS) and IRI data contained in the LTPP IMS were obtained. This data includes only that collected using the 1995 version of the K.J. Law Engineers, Inc. Model T-6600 inertial profiler. Four such profilers were purchased by LTPP in 1996 for use in all profiling operations.

The 1995 T-6600 profiler is considered a Class I accelerometer-established inertial profiling reference based on the ASTM Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer-Established Inertial Profiling Reference (E-950-98). It is a van-mounted system containing two infrared sensors spaced 1 680 mm (66 in.) apart. The system collects longitudinal profile data at 25-mm (1-in.) intervals, and these data are later processed and the IRI is computed. Because current automated profilographs record profile traces on 33-mm (1.25-in.) intervals, the 25-mm profile data represents a close match.

To model profilograph traces and generate simulated PI values from the 25-mm profile data, the models currently used by K.J. Law in computing PI from data collected by their lightweight profilers were employed. These lightweight profilers use the same vertical sensors and sampling intervals as the high-speed T-6600, ensuring that the models are compatible with LTPP profile data. In this study, the 25-mm (1-in.) profile data were processed into 0.0-, 2.5-, and 5-mm (0.0-, 0.1-, and 0.2-in.) blanking band PI values (herein designated as $PI_{0.0}$, $PI_{2.5}$, and PI_5) for each profile data set using the K.J. Law software. These simulated PI values were computed using a standard 0.76-m (2.5-ft) moving-average filter, along with standard minimum height, maximum height, and rounding scallop filters settings of 0.9, 0.6, and 0.25 mm (0.035, 0.024, and 0.01 in.), respectively.

IRI, simulated $PI_{0.0}$, $PI_{2.5\text{-mm}}$, and $PI_{5\text{-mm}}$ values, pavement type, and climatic data for a total of 1 793 LTPP test sections in 47 states and 8 Canadian provinces were used to populate the project database. The sections represent a variety of pavement types, including original and restored AC and PCC pavements, asphalt overlays of both AC (AC/AC) and PCC (AC/PCC) pavements, and concrete overlays of PCC pavements. They also span all four climatic zones as defined by mean annual precipitation [wet being greater than 508 mm (20 in) of precipitation per year] and mean annual freezing index (FI) [freeze being more than 83 °C-days per year (150 °F-days per year)]. Each test section in the database includes IRI and simulated PI values corresponding to individual profiler runs made between 1996 and 2001. The range of IRI and PI values is typical of highway pavements—IRI between 294 and 6 200 mm/km (18 and 393 in/mi) and PI between 0 to 1 700 mm/km (0 and 108 in/mi).

Development of LTPP-Based PI–IRI Relationships

Following the database compilation, the data was examined to determine its general properties, remove anomalies, and prepare it for further analysis. Next, the data's general trends were preliminarily evaluated to observe trends in plots of IRI versus PI and to identify possible effects of pavement type and climatic region on IRI–PI models. Suitable models were then selected based on the results of the preliminary evaluation. These tentative models were developed and refined to allow for the selection of the final IRI–PI and PI–PI models.

Preliminary data evaluation consisted of evaluating plots of IRI versus PI to observe general trends in the plots and performing an analysis of variance (ANOVA) to determine the effects of climate and pavement type on the slope of the IRI–PI and PI–PI relationship. The plots evaluated are presented as Figures 2 through 6. Figures 2 through 4 show that a linear relationship exists between IRI and PI for all three blanking bands evaluated. To assess the inference space of the data, a typical "full pay" range of pavement smoothness specification based upon PI is indicated in Figures 2 through 4. Slopes and intercepts increased from blanking bands of 0.0 to 2.5 to 5.0 mm. Variability also increased as the blanking band increased. Figures 5 and 6 show no clear trends regarding the effect of climate and pavement type on either the IRI–PI or PI–PI relationship.

For ANOVA the data sets were grouped according to the test section from which they were collected. Each data group included PI and IRI indices collected from the left and right wheelpaths, in multiple runs, on multiple dates, averaging about 22 data sets in a data group. The IRI–PI and the PI–PI slopes from the data sets in each data group was then computed. The slopes were then grouped according to pavement types or climatic regions. Pavement types evaluated included AC, AC/AC, and AC/PCC, and the climatic regions analyzed were dry-freeze, dry-nonfreeze, wet-freeze, and wet-nonfreeze.

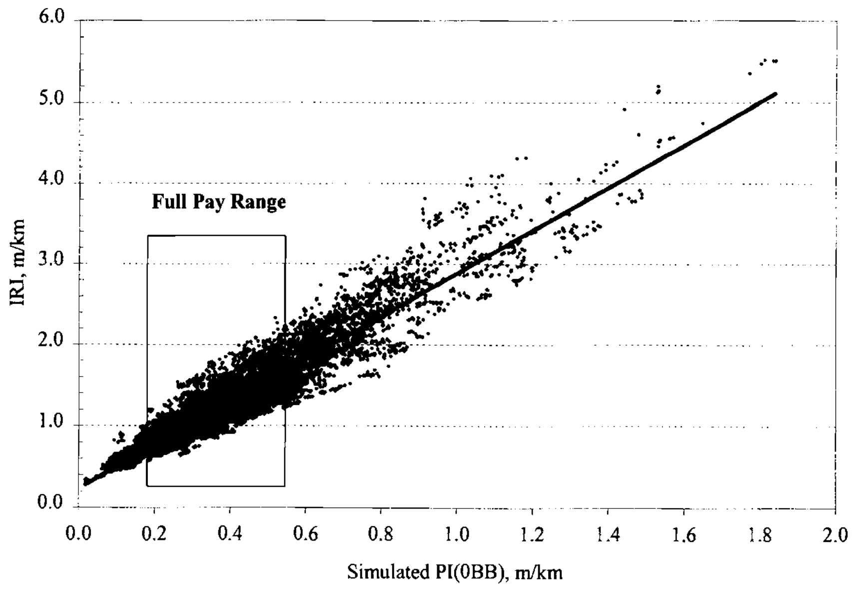

Figure 2 – *IRI vs.* $PI_{0.0}$ *for all AC pavements and climates*

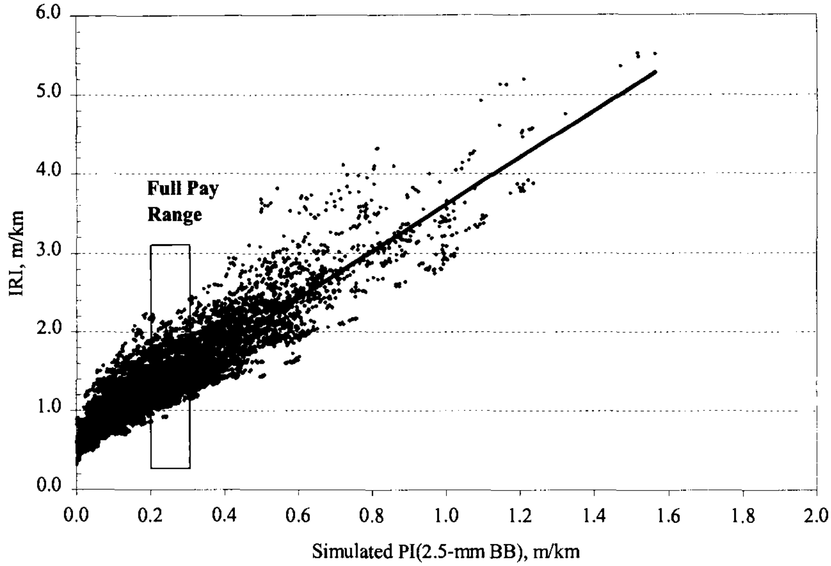

Figure 3 – *IRI vs.* $PI_{2.5}$ *for all AC pavements and climates*

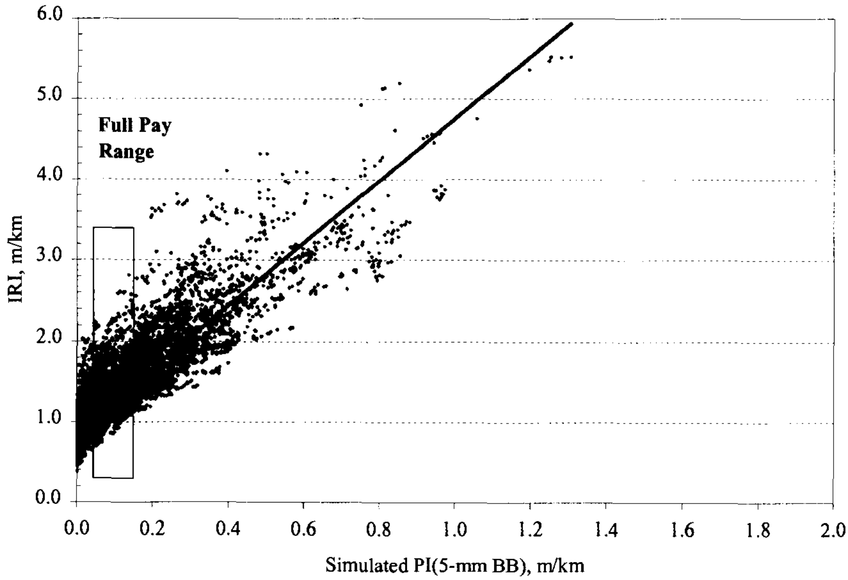

Figure 4 – *IRI vs.* $PI_{5.0}$ *for all AC pavements and climates*

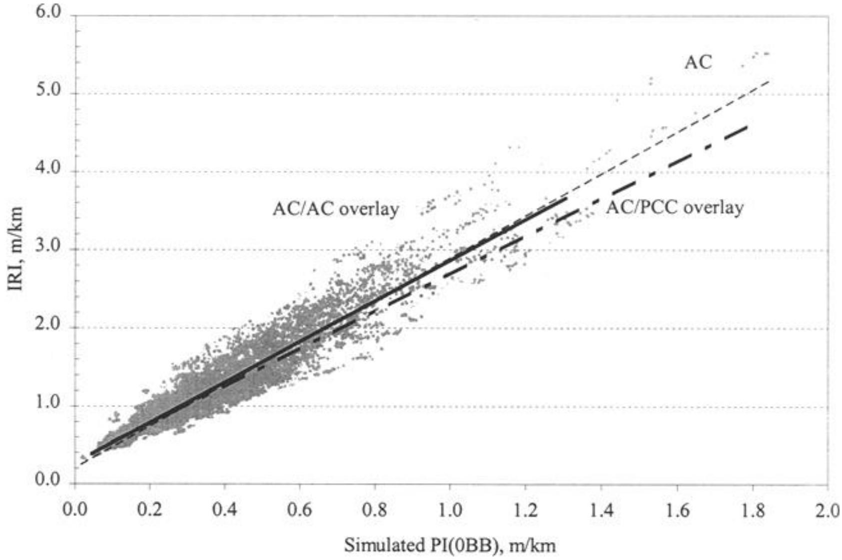

Figure 5 – *IRI vs. $PI_{0.0}$ for each pavement type*

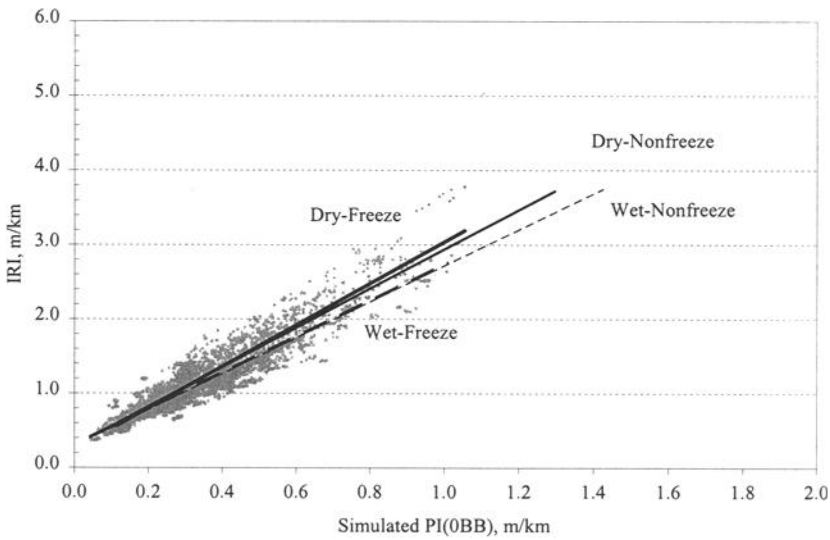

Figure 6 – *Effect of climate on the IRI vs. $PI_{0.0}$ relationship*

ANOVA was then used to determine if there were significant differences in the mean slope of the each pavement or climate type grouping. Statistical differences in the slopes of pavement or climate groupings were determined by making inferences about the mean slope of each grouping (e.g., mean IRI–PI relationship slope of AC, AC/AC, and AC/PCC groupings are equal) and using ANOVA to check the reasonableness of the inferences at a predetermined level of significance. For this study, the inferences (also called the null and alternative hypothesis) were as follows:

- H_o: Mean slopes of the data groups (grouped according to pavement type or climatic region) were all equal.
- H_a: Mean slopes of the data groups were not all equal.

The decision-making process and interpretation of ANOVA test results as to reasonableness of the null and alternative hypotheses is presented in Table 1.

Table 1 – *Summary of analysis and interpretation of ANOVA test results.*

Analysis Type	**Test Statistic**	**Null Hypothesis**	**Alternative Hypothesis**	**Results**	**P-Value**	**Significance?**	**Decision**
Analysis of variance (ANOVA)	F	$\mu_1 = \mu_2 = \mu_3 = \ldots = \mu_k$	$\mu_1 = \mu_2 = \mu_3 = \ldots = \mu_k$	Large F	Small p (< 0.05)	Yes (significant difference among means)	Reject H_o, Accept H_a
				Small F	Large p (> 0.05)	No	Accept H_o Reject H_a

As Table 1 shows, a large F-test statistic value (the test statistic used in ANOVA) with a corresponding small p-value (the probability that the mean slope of a data group would assume a value greater than or equal to the observed value strictly by chance) discredits the null hypothesis. Hence, it can be concluded that the mean slopes of the data groups are significantly different (or not equal). The converse (i.e., a small F-test statistic value with a corresponding large p-value) indicates that they are not significantly different (or equal).

It is conventional in statistics to reject the null hypothesis at the 5 percent significance level. That is, we reject H_0 when there is a one in twenty chance, or less, of the event (in this case the sample mean slopes being equal) occurring. When the p-value is less than 0.05, the event that has occurred is said to be statistically significant at the 0.05 level.

The results of the ANOVA, presented in table 2, show that differences in both pavement type and climate had a significant effect on the PI–IRI and PI–PI relationships. That is, the ANOVA F test results indicate that one or more of the mean slopes for the different groupings of climate and pavement type are significantly different.

Duncan's multiple comparison method in ANOVA was used to identify differences and similarities among the mean slopes of climate and pavement type data groups at a 95 % significance level. Table 3 provides a summary of the grouping based on the Duncan's multiple comparison tests. The final groupings were based not only on the results of the statistical analysis but also on the practicality of the groupings and engineering judgment.

Table 2 – ANOVA results on effect of pavement type and climate on PI–IRI relationship.

Dependent Variable[1]	Grouping Variable[2]	N	F-Statistic Value	Probability > F
Slope of IRI-$PI_{0.0\text{-mm}}$ linear relationship	Pavement Type	2395	2.24	0.1061[3]
	Climate	2395	7.61	0.0001[4]
Slope of IRI-$PI_{2.5\text{-mm}}$ linear relationship	Pavement Type	2395	5.91	0.0028[4]
	Climate	2395	3.87	0.0089[4]
Slope of IRI-$PI_{5.0\text{-mm}}$ linear relationship	Pavement Type	2395	0.72	0.4870[5]
	Climate	2395	0.95	0.4177[5]

[1] Computed for each wheelpath within a given pavement section within a uniform construction period.
[2] Pavement type considered—AC, AC/AC, and AC/PCC and climate types —DF, DNF, WF, and WNF.
[3] Borderline significance at the 10 % significance level.
[4] Significant at the 5 % significance level.
[5] Not significant.

Table 3 – *Summary of groupings for model development.*

Pavement Type	Climatic Region	IRI-PI Grouping	PI-PI Grouping
AC	DF, WF	A	A
AC	DNF, WNF	A	B
AC/AC	DF	A	A
AC/AC	DNF	B	B
AC/AC	WF, WNF	C	C
AC/PCC	ALL	A	A

Model Development

A linear model was selected for the IRI–PI regression analysis based on the trends shown in Figures 2, 3, and 4. Similar trends for the PI–PI relationship also resulted in linear models. Regression models for all of the groupings in Table 3 are presented in Tables 4 and 5. Each model was verified for accuracy and reasonableness. Standard diagnostic statistics, such as the standard estimate of the error (SEE), coefficient of determination (R^2), and the number of data points used were reviewed to check model suitability.

In general, the models appear reasonable. Coefficient of determination (R^2) was typically greater than 70 %, with only 3 out of 33 models having reported R^2 values less than 70%. SEE ranged from 178 to 308 mm/km (11 to 19 in/mi) for IRI and 21 to 79 mm/km (1.3 to 5.0 in/mi) for PI. These models contain the largest number of data points to date for modeling the IRI–PI relationships, ranging from 1 800 to 14 170 data points per model.

The models presented in Tables 4 and 5 predict the mean smoothness index (IRI or PI)

Table 4 – *PI to IRI index conversion equations and variability indices*

Pavement Type	Climate[1]	Blanking Band (mm)	Correlation Equation (IRI = mm/km, PI = mm/km)	N	SEE	R^2	Eqn. No.
AC	1,2,3,4	0.0	$IRI = 2.66543*PI_{0.0\text{-mm}} + 213.009$	14170	200.166	0.89482	1
AC	1,2,3,4	2.5	$IRI = 2.97059*PI_{2.5\text{-mm}} + 638.738$	14160	231.687	0.85896	2
AC	1,2,3,4	5.0	$IRI = 3.78601*PI_{5.0\text{-mm}} + 887.507$	13775	292.259	0.77349	3
AC/AC	1	0.0	$IRI = 2.74599*PI_{0.0\text{-mm}} + 265.419$	1854	191.973	0.90691	4
AC/AC	2	0.0	$IRI = 2.68169*PI_{0.0\text{-mm}} + 274.674$	1494	184.642	0.8096	5.
AC/AC	3,4	0.0	$IRI = 2.42295*PI_{0.0\text{-mm}} + 301.897$	5126	178.808	0.8404	6
AC/AC	1	2.5	$IRI = 3.12622*PI_{2.5\text{-mm}} + 708.557$	1854	230.03	0.86635	7
AC/AC	2	2.5	$IRI = 3.33564*PI_{2.5\text{-mm}} + 655.672$	1494	246.644	0.66026	8
AC/AC	3,4	2.5	$IRI = 2.68324*PI_{2.5\text{-mm}} + 660.343$	5126	216.981	0.76498	9
AC/AC	1	5.0	$IRI = 4.25316*PI_{5.0\text{-mm}} + 957.795$	1824	288.165	0.78851	10
AC/AC	2	5.0	$IRI = 4.39478*PI_{5.0\text{-mm}} + 883.203$	1345	308.232	0.44966	11
AC/AC	3,4	5.0	$IRI = 3.42671*PI_{5.0\text{-mm}} + 876.799$	4906	265.845	0.63287	12
AC/PCC	1,2,3,4	0.0	$IRI = 2.40300*PI_{0.0\text{-mm}} + 292.93$	4156	205.577	0.78699	13
AC/PCC	1,2,3,4	2.5	$IRI = 2.78217*PI_{2.5\text{-mm}} + 716.867$	4156	229.678	0.73412	14
AC/PCC	1,2,3,4	5.0	$IRI = 3.94665*PI_{5.0\text{-mm}} + 939.216$	4052	259.576	0.65344	15

[1] Climatic zones: 1=DF, 2=DNF, 3=WF, 4=WNF.

Table 5 – *PI to PI index conversion equations and variability indices*

Pavement Type	Climate[1]	Correlation Equation (PI = mm/km)	N	SEE	R^2	Eqn. No.
AC	1,3	PI0.0-mm = 1.08722*PI2.5-mm + 174.418	5744	47.73	0.96	16
AC	1,3	PI0.0-mm = 1.35776*PI5.0-mm + 275.476	5684	83.58	0.88	17
AC	1,3	PI2.5-mm = 1.28213*PI5.0-mm + 87.7861	5684	46.62	0.95	18
AC	2,4	PI0.0-mm = 1.12338*PI2.5-mm + 152.837	8418	45.23	0.95	19
AC	2,4	PI0.0-mm = 1.46417*PI5.0-mm + 240.094	8093	71.73	0.86	20
AC	2,4	PI2.5-mm = 1.34055*PI5.0-mm + 73.1258	8093	38.64	0.95	21
AC/AC	1	PI0.0-mm = 1.14153*PI2.5-mm + 160.701	1856	43.41	0.96	22
AC/AC	1	PI0.0-mm = 1.56038*PI5.0-mm + 250.888	1826	73.74	0.88	23
AC/AC	1	PI2.5-mm = 1.39462*PI5.0-mm + 75.5486	1826	40.47	0.95	24
AC/AC	2	PI0.0-mm = 1.28067*PI2.5-mm + 138.152	1496	52.26	0.86	25
AC/AC	2	PI0.0-mm = 1.75837*PI5.0-mm + 222.837	1347	79.32	0.66	26
AC/AC	2	PI2.5-mm = 1.52523*PI5.0-mm + 56.5960	1347	34.14	0.89	27
AC/AC	3,4	PI0.0-mm = 1.11926*PI2.5-mm + 145.849	5128	44.86	0.93	28
AC/AC	3,4	PI0.0-mm = 1.45876*PI5.0-mm + 233.588	4908	71.53	0.81	29
AC/AC	3,4	PI2.5-mm = 1.36739*PI5.0-mm + 71.1735	4908	38.12	0.93	30
AC/PCC	1,2,3,4	PI0.0-mm = 1.15412*PI2.5-mm + 177.077	4158	44.46	0.93	31
AC/PCC	1,2,3,4	PI0.0-mm = 1.61123*PI5.0-mm + 271.113	4054	71.07	0.81	32
AC/PCC	1,2,3,4	PI2.5-mm = 1.44895*PI5.0-mm + 76.8267	4054	36.99	0.93	33

[1] Climatic zones: 1=DF, 2=DNF, 3=WF, 4=WNF.

for the sample LTPP data used in model development. In this case, the sample means are probably a reasonable estimate of means of the population of pavements within the limits of the reference data. However, they do not necessarily indicate the range of values within which the true population means lies. The range of values within which the true population mean lies can be obtained by computing a confidence interval around the predicted sample mean. The *confidence interval* for the mean provides a range of values around the mean where one can expect the "true" (population) mean to be located (with a given level of certainty). Confidence interval can be computed using the following equation.

$$CI = mean \pm t_{\alpha/2}\sigma \tag{34}$$

where

CI	=	Confidence interval,
$mean$	=	Predicted smoothness index,
t	=	Value of "t" statistic at a given significance level,
α	=	Significance level (usually 90 or 95%), and
σ	=	Model standard error of estimate (SEE).

For example, if the predicted mean IRI (computed using models based on the LTPP data sample) is 1 000 mm/km (63 in/mi), and the lower and upper limits at a significance level of 95% are 900 mm/km and 1 100 mm/km, respectively, then it can be concluded that there is a 95% probability that the population mean is between 900 mm/km and 1 100 mm/km. If the significance level is set to a smaller value (say 99%), then the interval would become wider thereby increasing the "certainty" of the estimate, and vice versa.

In essence, the larger the sample size, the more reliable will be its mean, and the larger the variation (SEE) the less reliable will be the mean. Sample size used for development of both the LTPP IRI-PI and PI-PI models ranged from 1 347 to 14 170 data points. These numbers are greater than the generally required minimum of 100 and should provide reliable results. SEE values noted for Equations 1 to 33 range from 179 to 292 mm/km (11 to 18 in/mi) for the IRI-PI models and from 25 to 58 mm/km (1.6 to 3.7 in/mi) for the PI-PI models. These SEE values are reasonable, considering the differences between IRI and PI in the weighting of pavement wavelengths (see Figure 1).

Use of LTPP-Based Models to Update Current Smoothness Specifications

Many agencies are not familiar enough with the IRI, $PI_{2.5}$ and $PI_{0.0}$ statistics to set specification limits. To assist in this process, these new LTPP models provide a generalized method for transitioning between PI and IRI specifications. Each of the above models was used, together with the most recently reported agency smoothness specification limits, to compile the transition Tables 6, 7, and 8. These tables list the currently reported agency AC smoothness indexes and full-pay limits, noted in bold print. They also provide an estimate of the $PI_{0.0}$, $PI_{2.5}$, and IRI limits that could be used as a starting point for developing specifications based on these indices.

Because the IRI and PI indices are not exactly correlated, the transition table provides a 90% standard error of the estimate range for the projected specification limit. This error rating should assist specification writers in defining their limits. It also can be used as a basis for refining the specification on an ongoing basis.

For example, if the state of Maryland (a wet-freeze climatic zone state) is considering switching from $PI_{5.0}$ to IRI for AC/AC, it can use Equation 12 from Table 4 and its current specification limits to estimate the IRI values that correspond with those limits. Otherwise, using the full-pay smoothness limits of 64 to 110 mm/km (4 to 7 in/mi) in Table 7, the state can estimate that the comparable IRI range would be 1 096 to 1 254 mm/km. With a standard error estimate (SEE) of 266 for this relationship, the specification writer can assume that variability within the relationship results in a reasonable range for comparable IRI values of 1 096 to 1 254 mm/km (69 to 79 in/mi). If the agency is considering transitioning to a $PI_{0.0}$ specification, it can use Equation 17 from Table 5 or note from Table 7 that a comparable $PI_{0.0}$ range is 321 to 403mm/km (20 to 26 in/mi).

Direct state-to-state comparisons of derived specification limits may not be appropriate due to individual agencies' implementation practices. Factors that may impact the specification limits for a specific agency include segment length, whether an agency aggregates segments, scope of application (new pavements or overlays, and type facilities), and method of index computation (Half-car roughness index, individual wheelpath IRI, or average IRI). More standardized testing and reporting procedures are currently under development by an FHWA task force (3).

Conclusions

This analysis of 29 000 LTPP profile data sets ranging in IRI from 294 to 6 212 mm/km indicates that a reasonable correlation can be developed between IRI and PI. Supplemental correlation between $PI_{0.0}$, $PI_{2.5}$, and $PI_{5.0}$ can also be developed. Conclusions that can be drawn from the results of this study include the following.

- Pavement type (AC, AC/AC, or AC/PCC) is a significant factor in the correlation between IRI and PI.
- Climatic conditions have the effect of increasing the slope of the PI–IRI relationship for AC/AC pavements in dryer climatic regions. As a result, models for AC/AC vary according to the climatic conditions (precipitation and mean annual freezing index) of the pavement.
- Equations listed in Tables 4 and 5 can be used to assist agency personnel in transitioning smoothness specification limits from PI to IRI or to PI with a tighter blanking band.
- The estimated standard error in the IRI–PI equations allows agency specification writers to quantify the variability in the transition specification levels. It also provides input for the local research needed to refine the specification cutoff levels.

Table 6 – *Smoothness specifications for AC pavement (mm/km).*

State [1,2]	Climate [3]	IRI	SEE [4]	$PI_{0.0\ mm}$	SEE [4]	$PI_{2.5\ mm}$	SEE [4]	$PI_{5.0\ mm}$	SEE [4]
AL	4	1009-1126	292	274-327	72	112-156	39	**32 - 63**	-
AK	3	1130-1247	292	349-397	84	166-207	47	**64 - 95**	-
AR	3,4	1062-1171	232	321-366	84	141-180	47	**46 – 75**	-
CA	2,3,4	≤ 1190	292	≤ 356	72	≤ 180	39	**≤ 80**	-
CO	1,3	1295-1381	232	413-446	48	**221 - 250**	-	105-127	35
CT	3	**950-1260**	-	276-393	71	105-209	72	17-98	68
GA	3,4	**≤ 750**	-	≤ 201	71	≤ 37	72	0	68
ID	1,3	≤ 1190	292	≤ 374	84	≤ 187	47	**≤ 80**	-
IL	3	933-1493	292	264-498	84	92-295	47	**9 – 160**	-
IN	3	≤ 1595	292	≤ 540	84	≤ 331	47	**≤ 187**	-
IA	3	1073-1304	292	326-421	84	145-227	47	**49 – 110**	-
KS	1,3	642-1479	200	**161 – 475**	-	0-276	43	0-145	35
LA	4	≤ 1065	292	≤ 300	72	≤ 133	39	**≤ 47**	-
ME	3	**946-1105**	-	275-335	71	103-157	72	15-57	68
MD	3	1130-1304	292	349-421	84	166-227	47	**64 - 110**	-
MA	3	**< 1500**	-	< 483	71	< 290	72	< 162	68
MI	3	1130-1486	292	349-495	84	166-292	47	**64 – 158**	-
MN	3	1035-1187	292	311-373	84	132-186	47	**39 - 79**	-
MS	4	1190-1304	292	356-407	72	180-222	39	**80 - 110**	-
MO	3	973-1266	200	**285 - 395**	-	108-205	43	23-93	35
NE	1,3	1175-1304	292	368-421	84	182-227	47	**76 – 110**	-
NV	1	≤ 1190	292	≤ 375	84	≤ 187	47	**≤ 80**	-
NM	1,2	1137-1190	292	352-374	84	168-187	47	**66 – 80**	-
OH	3	1130-1304	292	349-421	84	166-227	47	**64 – 110**	-
OR	1,4	1194-1304	292	376-421	84	188-227	47	**81 – 110**	-
PA	3	1394-1642	200	**443 - 536**	-	247-329	43	124-184	35
PR	4	1561-1664	292	522-568	72	319-357	39	**178-205**	
SD	1,3	**869-1105**	-	246-335	71	78-157	72	0-57	68
TX	1,2,3,4	847-1503	200	**238 – 315**	-	66-134	43	0-42	35
UT	1,3	≤ 1304	292	≤ 421	84	≤ 227	47	**≤ 110**	-
VT	3	**950-1090**	-	276-329	71	105-152	72	17-53	68
VA	3,4	**869-1105**	-	246-335	71	78-157	72	0-57	68
WA	1,3,4	**947-1500**	-	275-483	71	104-290	72	16-162	68
WI	3	≤ 1486	292	≤ 490	84	≤ 295	35	**≤ 158**	-
WY	1	**845-1081**	-	237-326	71	69-149	72	0-51	68

[1] Non-IRI or PI specification in AZ, DE, FL, HI, KY, NH, NJ, NC, NY, ND, RI, SC, TN, WV, and WY.
[2] Unknown specifications for DC and MT.
[3] Climatic zones: 1=DF, 2=DNF, 3=WF, 4=WNF (Bolded zone used as standard specification).
[4] Range of values with 90 % confidence.

Note: 1 mm/km = 0.001 m/km, 1 mm/km = 0.06336 in/mi

Table 7 – *Smoothness specifications for AC /AC pavement (mm/km).*

State [1,2]	Climate [3]	IRI	SEE [4]	$PI_{0.0\,mm}$	SEE [4]	$PI_{2.5\,mm}$	SEE [4]	$PI_{5.0\,mm}$	SEE [4]
AL	4	986-1093	266	263-319	72	110-155	38	**32 - 63**	-
AK	3	1096-1202	266	321-376	72	157-202	38	**64 - 95**	-
AR	3,4	784-862	217	288-340	72	130-173	38	**46 - 75**	-
CA	2,3,4	≤ 1235		≤ 399	79	≤ 186	34	**≤ 80**	-
CO	1,3	1399-1490	230	413-447	43	**221 - 250**	-	104-124	28
CT	3	**950-1260**	-	267-395	68	108-223	71	21-112	62
GA	3,4	**≤ 750**	-	≤ 185	68	≤ 33	71	0	62
ID	1,3	≤ 1151	266	≤ 349	72	≤ 180	38	**≤ 80**	-
IL	3	908-1425	266	222-493	72	76-298	38	**9 – 160**	-
IN	3	≤ 1518	266	≤ 542	72	≤ 338	38	**≤ 187**	-
IA	3	1045-1254	266	294-403	72	135-224	38	**49 – 110**	-
KS	1,3	708-1570	191	**161 – 475**	-	9-273	37	0-139	44
LA	4	≤ 1038	266	≤ 290	72	≤ 132	38	**≤ 47**	-
ME	3	**946-1105**	-	266-331	68	106-166	71	20-67	62
MD	3	1096-1254	266	321-403	72	157-224	38	**64 - 110**	-
MA	3	**< 1500**	-	< 494	68	< 313	71	< 182	62
MI	3	1096-1418	266	321-490	72	157-295	38	**64 – 158**	-
MN	3	1010-1148	266	276-348	72	120-179	38	**39 - 79**	-
MS	4	1151-1254	266	349-403	72	180-224	38	**80 - 110**	-
MO	3	992-1259	179	**285 - 395**	-	128-219	39	44-105	44
NE	1,3	1281-1426	288	363-423	74	179-229	40	**76 - 110**	-
NV	1	≤ 1298	288	≤ 370	74	≤ 185	40	**≤ 80**	-
NM	1,2	1239-1298	288	346-370	74	165-185	40	**66 – 80**	-
NY	3	1063-2015	217	311-739	45	**150 - 505**	-	59-301	27
OH	3	1096-1254	266	321-403	72	157-224	38	**64 – 110**	-
OR	1,4	1302-1426	288	372-423	74	187-229	40	**81 - 110**	-
PA	3	1375-1601	179	**443 - 536**	-	259-337	39	132-184	44
PR	4	1487-1579	266	525-574	72	324-364	38	**178-205**	-
SD	1,3	**869-1105**	-	220-306	67	51-127	68	0-35	60
TX	1,2,3,4	919-1130	191	**238 – 315**	-	74-139	37	5-49	44
UT	1,3	≤ 1426	288	≤ 423	74	≤ 229	40	**≤ 110**	-
VT	3	**950-1090**	-	267-325	68	108-160	71	21-62	62
VA	3,4	**869-1105**	-	234-331	68	78-166	71	0-67	62
WA	1,3,4	**947-1500**	-	248-450	67	76-253	68	0-127	60
WI	3	≤ 1418	266	≤ 490	72	≤ 295	38	**≤ 158**	-
WY	1	**845-1081**	-	211-297	67	44-119	68	0-29	60

[1] Non-IRI or PI specification in AZ, DE, FL, HI, KY, NH, NJ, NC, NY, ND, RI, SC, TN, WV, and WY.
[2] Unknown specifications for DC and MT.
[3] Climatic zones: 1=DF, 2=DNF, 3=WF, 4=WNF (Bolded zone used as standard specification).
[4] Range of values with 90% confidence.

Note: 1 mm/km = 0.001 m/km, 1 mm/km = 0.06336 in/mi

Table 8 – *Smoothness specifications for AC /PC pavement (mm/km).*

State [1,2]	Climate [3]	IRI	SEE [4]	$PI_{0.0\ mm}$	SEE [4]	$PI_{.2.5\ mm}$	SEE [4]	$PI_{5.0\ mm}$	SEE [4]
AL	**4**	1066-1188	260	307-369	71	118-167	37	**32 - 63**	-
AK	**3**	1192-1314	260	371-432	71	168-217	37	**64 - 95**	-
AR	**3,4**	1121-1235	260	335-393	71	140-186	37	**46 - 75**	-
CA	**2,3,4**	≤ 1255	260	≤ 403	71	≤ 193	37	**≤ 80**	-
CO	**1**,3	1332-1412	230	436-472	45	**221 - 250**	-	98-116	25
CT	**3**	**950-1260**	-	273-402	76	84-195	71	3-81	53
GA	3,4	**≤ 750**	-	≤ 190	76	≤ 12	71	0	53
ID	1,3	1255	260	≤ 403	71	≤ 193	37	**≤ 80**	-
IL	**3**	975-1571	260	261-562	71	82-319	37	**9 – 160**	-
IN	**3**	≤ 1677	260	≤ 616	71	≤ 361	37	**≤ 187**	-
IA	**3**	1133-1373	260	341-462	71	145-240	37	**49 – 110**	-
KS	**1**,3	680-1434	206	**161 – 475**	-	0-252		0-116	40
LA	**4**	≤ 1125	260	≤ 337	71	≤ 142	37	**≤ 47**	-
ME	**3**	**946-1105**	-	272-338	76	82-140	72	2-42	53
MD	**3**	1192-1373	260	371-462	71	168-240	37	**64 - 110**	-
MA	**3**	**< 1500**	-	< 502	76	< 281	71	< 142	53
MI	**3**	1192-1563	260	371-558	71	168-316	37	**64 – 158**	-
MN	**3**	1093-1251	260	321-401	71	129-192	37	**39 - 79**	-
MS	**4**	1255-1373	260	403-462	71	193-240	37	**80 - 110**	-
MO	**3**	978-1242	206	**285 - 395**	-	100-188		21-76	40
NE	**1**,3	1239-1373	260	395-462	71	187-240	37	**76 - 110**	-
NV	**1**	≤ 1255	260	≤ 403	71	≤ 193	37	**≤ 80**	-
NM	**1**,2	1200-1255	260	375-403	71	172-193	37	**66 – 80**	-
OH	**3**	1192-1373	260	371-462	71	168-240	37	**64 – 110**	-
OR	**1**,4	1259-1373	260	405-462	71	195-240	37	**81 - 110**	-
PA	**3**	1357-1581	206	**443 - 536**	-	227-301		100-147	40
PR	**4**	1642-1748	260	598-652	71	347-389	37	**178-205**	-
SD	**1**,3	**869-1105**	-	240-338	76	55-140	71	0-42	53
TX	**1**,2,3,4	865-1050	206	**238 – 315**	-	62-124		0-36	40
UT	**1**,3	≤ 1373	260	≤ 462	71	≤ 240	37	**≤ 110**	-
VT	3	**950-1090**	-	273-332	76	84-134	71	3-38	53
VA	**3**,4	**869-1105**	-	240-338	76	55-140	71	0-42	53
WA	**1**,3,4	**947-1500**	-	272-502	76	83-281	71	2-142	53
WI	**3**	≤ 1563	260	≤ 558	71	≤ 316	37	**≤ 158**	-
WY	**1**	**845-1081**	-	230-328	76	46-131	72	0-36	53

[1] Non-IRI or PI specification in AZ, DE, FL, HI, KY, NH, NJ, NC, NY, ND, RI, SC, TN, WV, and WY.
[2] Unknown specifications for DC and MT.
[3] Climatic zones: 1=DF, 2=DNF, 3=WF, 4=WNF (Bolded zone used as standard specification).
[4] Range of values with 90% confidence.

Note: 1 mm/km = 0.001 m/km, 1 mm/km = 0.06336 in/mi

Recommendations

The major goal of this research was to develop a practical tool to assist in the transition between PI and IRI specifications. Correlation and error estimates have been provided to allow agencies to estimate the level of IRI and PI smoothness that is associated with their current specifications. To make this research useful, agencies are asked to do the following.

- Evaluate the validity of the research results based on agency conditions and experiences.
- Use the correlation equations and variability information to estimate the required level of smoothness for a specification that transitions to IRI or to PI with a tighter blanking band. The authors recommend adjusting the derived specification limits to reflect agency implementation practices such as segment length, segment averaging, scope of application, and index computation method.
- Track the results of the new smoothness specification and adjust the smoothness requirements to meet the increasing abilities of contractors and the smoothness levels desired by the agency.

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Chui-Te Chiu,[1] Ming-Gin Lee,[2] and Dar-Hao Chen[3]

Study of Profile Measurements Using Six Different Devices

Reference: Chiu, C. T., Lee, M. G., and Chen, D. H., "**Study of Profile Measurements Using Six Different Devices**" *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433*, M. S. Gardiner, Ed., American Society for Testing and Materials International, West Conshohocken, PA, 2003.

Abstract: One of the top priorities for transportation authorities is to build a smooth pavement. A smoother pavement provides a better ride and reduces damage from dynamic traffic loading. Many different profilers have been selected by various transportation agencies to measure roughness. Roughness has been used in many specifications to determine if the contractor should obtain a bonus or penalty. Thus, a reliable device is needed to collect repeatable and defendable roughness values. The purpose of this study is to review, evaluate, and analyze six existing devices. Repetitive tests were conducted on both flexible and rigid pavements to determine the repeatability, correlation and limitations of these devices. Excellent correlations among devices have been developed with high R^2. The 0.1 in. (2.54mm) blanking band yielded approximately 20% higher profile indexes than the 0.2 in. (5.08mm) band, and was more sensitive to roughness. The ARRB walking profiler yielded the highest precision, and is easy to operate and transport. Thus, the ARRB walking profiler was recommended for use on newly constructed pavement. The ARRB multiple laser profiler can be used for collecting profiles on existing pavements for management purposes.

Keywords: Straightedge, Profiler, Profilograph, Roughness, Smoothness

Introduction

It has been reported that the long-term performance of a pavement is related to the initial, as-built smoothness [*1, 2*]. Based on the NCHRP survey done under project 1-31 in 1994, more than 50% of state Departments of Transportation (DOTs) have adopted smoothness requirements in their specifications for flexible and rigid pavements [*1*]. A smoother pavement provides a better ride and reduces damage from dynamic traffic loading. The traveling public also express that the most important aspect of the highway system is the smoothness of the pavements [*3*]. Although smoothness is important, the smoothness definitions are inconsistent among transportation agencies.

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For example, Kansas state specification enforces a standard of no more than 30 in. of roughness per mile (473mm/km) with a zero blanking band on the surface of a new road for full pay. Under similar condition, Colorado state requires less than 18 in. of roughness per mile (284mm/km) with a 0.1 inch blanking band. Oklahoma state has a standard of no more than 7 in. of roughness per mile (110mm/km) with a 0.2 inch blanking band [*3*].

It is not always easy to measure smoothness in an objective way, especially in the early years when tools and knowledge were very limited. With advances in technology, many automated devices were invented so that more precise profiles can be made. The fundamental operations, cost, and appropriateness to address specific needs vary considerably among different devices. Certain devices are far better suited for the quality control on the smoothness of newly constructed pavements [*4, 5*]. Therefore, knowing the suitability of each machine is important to those who are concerned with evaluating the smoothness of roads. Although there are more than 15 different types of smoothness-measuring devices used in new pavement construction, they can be categorized into four categories as straightedges (static and rolling), profilographs, response-type road roughness measuring systems, and inertial profilers.

It is essential to investigate different devices and methods of measuring surface profiles and to develop smoothness specifications based on profilers that offer the greatest precision and production rates. In pursuing its goal of providing smoother pavements, the Taiwan Area National Expressway Engineering Bureau (TANEEB) initiated a research project with the Chung Hua University to evaluate six existing devices. They include the 3m straightedge, 3m rolling straightedge, California profilograph, Rainhart profilograph, ARRB walking profiler, and ARRB Multiple Laser Profiler (MLP).

These six devices were applied on more than 20 different pavement sections with various levels of roughness to evaluate their limitations. Most of the sections were newly constructed pavements. The other few were old, rough pavements. Repeated tests were conducted to determine the repeatability and the coefficients of variation of each device. It is important to note that the results to be presented may not be representative of current device performance because equipment designs may have changed. In fact, several devices used in this study were purchased more than ten years ago. The equipment vendor(s) should be contacted for updated information. Depending on an agency's needs, new tests may be required to be conducted before purchases are made.

Objectives

Over the years, transportation agencies in Taiwan have acquired different types of profiling devices. The information on what scopes can these devices be applicable to is thus very important for these agencies. This study evaluates and compares measurements from these devices over different levels of roughness. The objectives of this study are outlined as follows:

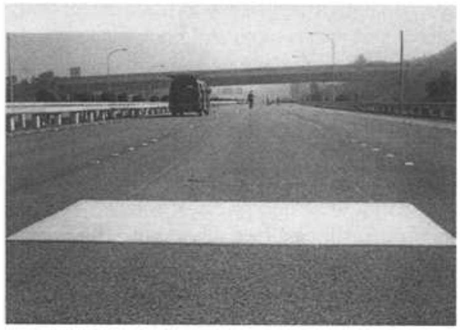
- Determine the limitations of the devices investigated.
- Determine the effects of blanking band on profile index measurements.
- Develop the correlations among different devices.
- Determine the pavement qualities in Taiwan to see if they meet the criteria suggested by State DOTs.

- Select and recommend devices for routine pavement applications.

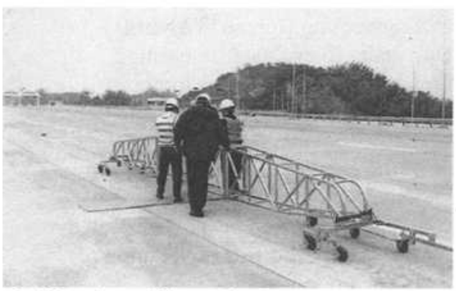
Six Devices Used in This Study

The six devices used in this study are shown in (Figure 1), including two straightedges, two profilographs, and two profilers.

ARRB MLP and other five devices



calibration plate



California profilograph

Rainhart profilograph

rolling straightedge

ARRB walking profiler

Figure 1 – Devices employed in this study

Straightedges

Both static and rolling straightedges were used in this study. The rolling straightedge is equipped with an automated recorder. These straightedges are all three meters long. The measurement is often taken at center (1.5m) reporting a depth in mm. With the obvious disadvantage of the effort to move the static straightedge numerous of times (and the wear and tear that resulted) it was not surprising to see development of the rolling straightedge. ELE in UK manufactured the rolling straightedge. Typical data analysis for rolling straightedge data is given in (Figure 2). The standard deviation (SD) of all recorded elevation differences is calculated and used as the smoothness index for the tested pavement. Instead of using SD as the smoothness index, current TANEEB specifications require that new pavement be built with less than 3mm in elevation difference within any three meter length. Any location that exceeds the 3mm should be repaired at the contractor's expense. The most common way to repair this is to grind the uneven surface. Contractors often complain that the 3mm criterion is too strict. This study provides an opportunity to study this criterion by using different devices on different classes of pavements.

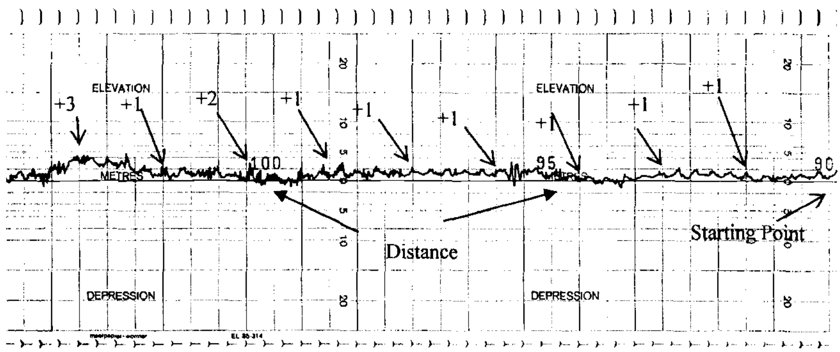


Figure 2 -- Data computation for rolling straightedge

California Profilograph

Because the straightedge contacts the road surface at three points, bumps of certain wavelengths may result in erroneous results. To overcome this problem, the rolling concept was subsequently improved by adding an array of wheels to establish a reference plane from which to measure deviations. The elevation is averaged for the whole array of wheels and roughness is measured as the deviation of the center wheel from this reference. Most state smoothness specifications currently call for the use of the original California profilograph [*1, 2, 4, 5*]. The California profilograph is the original 7.6m (25 foot) rolling straightedge. The California profilograph type C-990 was used in this study. A typical data analysis for the California profilograph is given in (Figure 3). Under normal circumstances, a 0.2-inch blanking band is currently used for California profilograph data.

The profile index (PI) obtained from the California profilograph is expressed in inches per mile (or millimeters per kilometer, mm/km). The ASTM Test Method for

Measuring Pavement Roughness Using a Profilograph (E1274) was used to compute the PI value. Note that a lower PI represents a smoother surface than a higher PI.

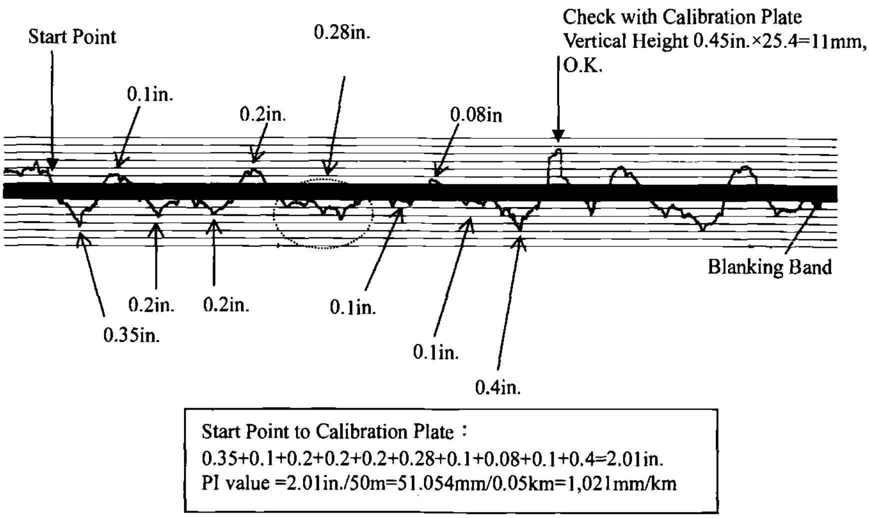


Figure 3 -- Data computation for California profilograph

Rainhart Profilograph

Similar to the California profilograph, Rainhart profilograph also has an array of rolling wheels. Rainhart profilograph No. 860 was used in this study. Normally, a 0.1-inch blanking band is used with the Rainhart Profilograph. Like the California profilograph, results are expressed in inches per mile (or mm/km).

The Rainhart profilograph consists of 12 wheels equally spaced at 0.686m. Thus, the total length is greater than 8m. The horizontal scale can be 1:120 or 1:300. The latter was selected to be consistent with the California profilograph. Although the Rainhart profilograph is more cumbersome than the California profilograph in transport and assembly, its measurements are more representative of the true profile. The increased accuracy is due to two substructures added to the main structure to reflect any roughness encountered.

ARRB Walking Profiler

The ARRB Walking profiler is produced by the Australian Road Research Board (ARRB) Ltd. It is a portable hand-operated device for precision measurements of pavement surfaces, and meets the World Bank Class 1 profiler requirements. The ASTM Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer Established Inertial Profiling Reference (E950) requires an average absolute discrepancy of 1.25 mm or less for Class 1 devices, and between 1.25 and 2.50 mm for Class 2 devices. The ARRB walking profiler consists of a small wheeled unit about the size of a standard lawnmower.

With the ARRB walking profiler, it is possible to accurately record profile measurements. The unit is simply pushed over the surface to be surveyed as the onboard computer handles all calculations. The operation speed is approximately 1km per hour. Measurement is reported as International Roughness Index (IRI) in m/km.

ARRB Multiple Laser Profiler

A high-speed laser profiler from ARRB was compared to the other devices. The Multiple Laser Profiler (MLP) is mounted to a vehicle and operated at highway speed. The output from the MLP is IRI in m/km. The IRI is unique among roughness indices in that it is easy to obtain and widely used. The World Bank experiments have validated that it can be measured by an extensive range of equipment. Thus, the IRI today provides highway engineers with a proven and robust basis for comparing roughness information across institutional boundaries. However, the IRI is sometimes criticized because it is not quantifying specific road roughness qualities, does not conform to local preferences for a roughness index, and it may not be related to ride [*6*].

Testing and Results

Because of the higher data collection rate, the ARRB MLP was used initially to select pavement sections with different levels of roughness. To determine the capability of these devices, a few old and rough pavements were also selected. A total of 21 test locations shown in (Figure 4) include PI of 0 to 1100mm/km and IRI of 0.57 to 4.65m/km. Due to equipment breakdowns, only 11 locations were tested with all six devices.

Precisions of Devices

To evaluate the precisions of different devices, a few locations were selected to perform repeatability test. The typical testing layout is presented in (Figure 5). There are three calibration stops located at the starting, mid-length, and end points of the testing section of 200 meters long. Four repeated tests were conducted on both left and right wheel paths. The 1.2m wide plywood plates with known thickness were used for calibration during testing. According to the calculated indices, SD, PI, and IRI, the six devices may be categorized into three groups, i.e., Straightedges, Profilographs, and Profilers. The data from the analysis on the variance of repeatability tests at different locations for the three smoothness indices are summarized in (Table 1). As shown in (Table 1), the coefficients of variance for the indices SD, PI, and IRI range from 10% to 18%, 8% to 34%, and 5% to 8%, respectively. It can be seen that higher precision may be achieved with the two Profilers than the other four devices. It can also be seen that profilograph-type devices exhibit higher variation in precision.

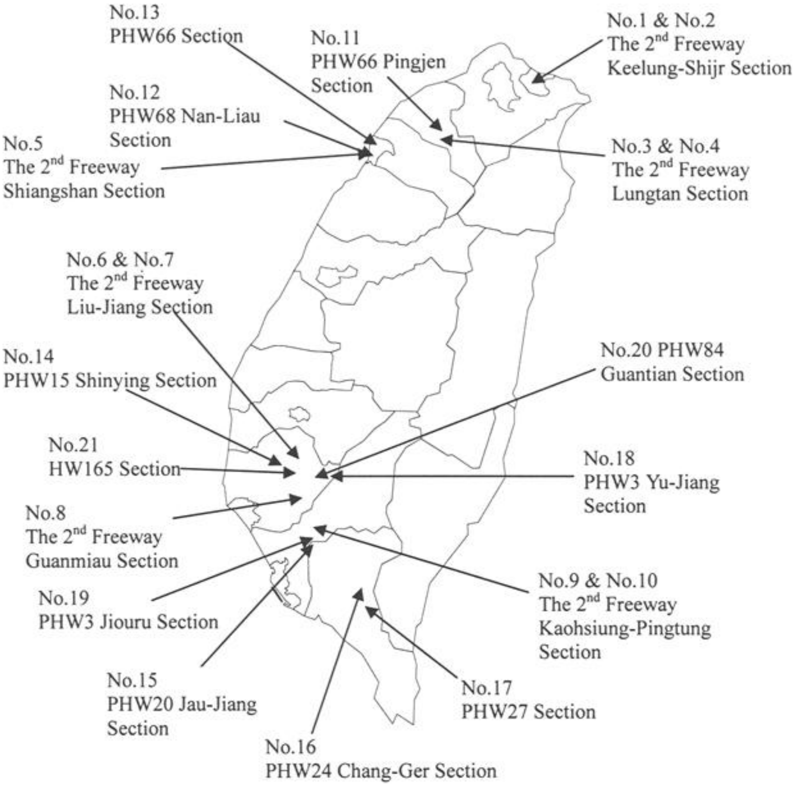


Figure 4 -- Test locations in this study

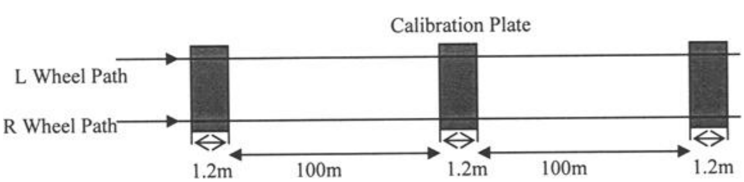


Figure 5 – Typical test layout for repeatability tests

Table 1 – Summarized data from the analysis of variance for repeatability tests

Smoothness Index	Location	Overall Mean	Error Term		Coefficient of Variance (%)
			Degree of Freedom	Sum of Squares	
PI, mm/km	No. 6	20.25	12	278.50	23.79
	No. 7	71.125	4	279.50	11.75
	No. 12	78.25	4	1298.00	23.02
	No. 5	111.16	24	33373.25	33.55
	No. 8	340.45	48	55570.25	11.26
	No. 2	677.04	36	103500.50	7.92
SD, mm	No. 6	0.458	12	0.0459	13.50
	No. 7	0.5875	4	0.0287	14.42
	No. 12	0.7883	6	0.0760	14.28
	No. 5	0.8375	24	0.5431	17.96
	No. 8	1.053	48	0.6750	9.99
IRI, m/km	No. 5	0.9606	24	0.1260	7.54
	No. 2	3.229	36	0.9689	5.08

Correlations Among Devices

With different agencies owning and operating different devices, it is important to develop correlations among devices. This allows different agencies to share meaningful test results. The relationships between California profilograph and rolling straightedge, California and Rainhart profilographs, California profilograph and ARRB MLP, and Rainhart profilograph and ARRB MLP are presented in (Figure 6) and (Figure 7). It is interesting to see R^2 values are all very high (0.948, 0.945, 0.972, and 0.978) when the linear trend lines were used. The 0.1 inch blanking band was used in the data collection for both California and Rainhart profilographs. The correlations are outlined below:

California Profilograph (CP) and Rolling Straightedge (RSE)

$$CP\ (mm/km) = 407.71\ RSE\ (mm) - 236.77,\ R^2=0.948$$

California (CP) and Rainhart (RP) Profilographs

$$RP\ (mm/km) = 0.9232\ CP\ (mm/km) + 7.3697,\ R^2=0.945$$

California Profilograph (CP) and ARRB MLP (MLP)

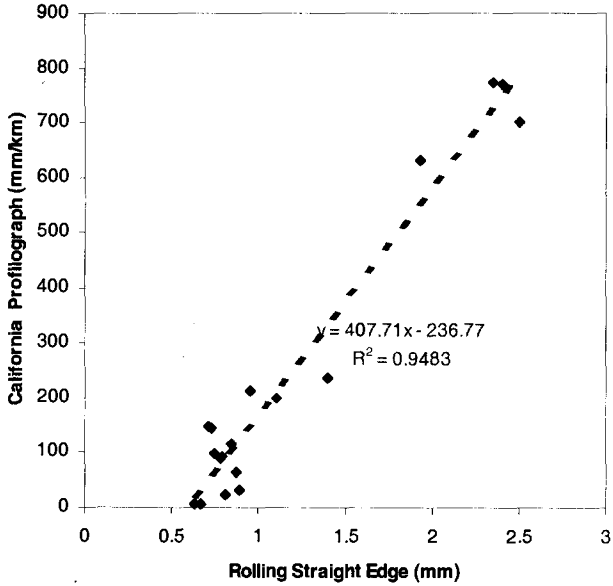
$$CP\ (mm/km) = 258.76\ MLP\ (m/km) - 178.93,\ R^2=0.972$$

Rainhart Profilograph (RP) and ARRB MLP (MLP)

$$RP\ (mm/km) = 257.87\ MLP\ (m/km) - 173.63,\ R^2=0.978$$

Effects of Blanking Band

Based on an NCHRP project conducted in 1994, 41% and 64% of State DOTs adopted the profilograph in their smoothness specifications for flexible and rigid pavements, respectively [*1*]. Two different blanking bands (0.1 inch and 0.2 inch) have been suggested and used for the two different type of profilograph in the past. The effects of the blanking band on profile index measurements were investigated. It was found that a 0.1 inch blanking band is more sensitive to roughness than the 0.2 blanking band; that is, 0.1 inch blanking band yielded a higher profile index (mm/km), as shown in



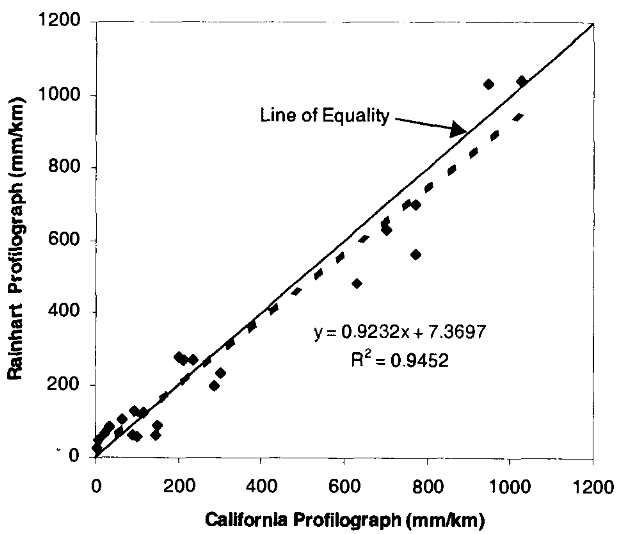
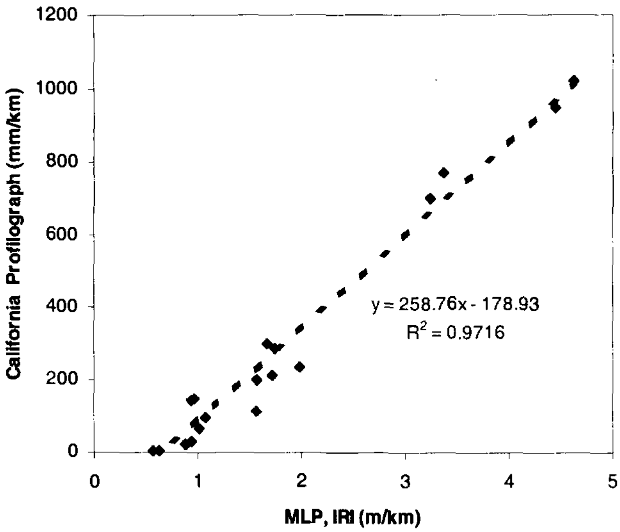


Figure 6 -- Corrections between rolling straightedge and California profilograph, California and Rainhart profilographs



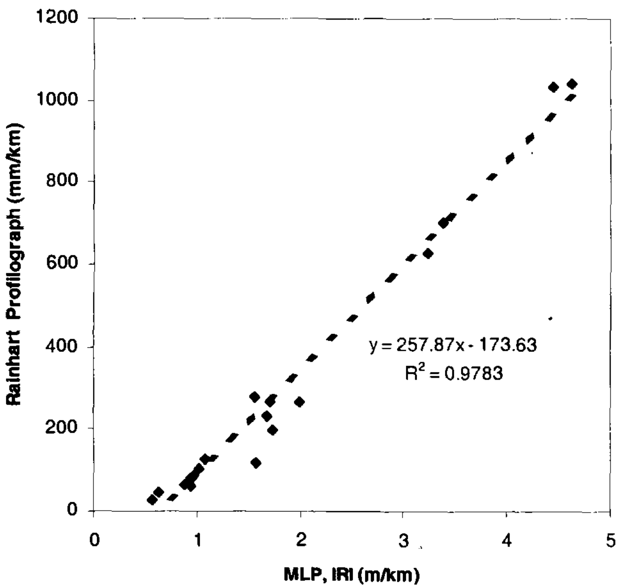


Figure 7 -- Correction between the ARRB MLP and California profilograph, ARRB MLP and Rainhart profilograph

(Figure 8). This conclusion is based on tests conducted on two flexible and two rigid pavements. Noted that only the California profilograph was used and the results are presented. Recommendations to highway agencies in Taiwan were made to use the 0.1-inch blanking band for both California and Rainhart profilographs. The relationship between the 0.1-inch and 0.2-inch blanking bands is presented in (Figure 8). A linear trend line ($R^2 = 0.89$) can be obtained. On average, the 0.1-inch blanking band yields approximately 20% higher PI values than the 0.2-inch blanking band. This is derived by fitting a linear trendline with the intercept set to 0. The R^2 value in this case was equal to 0.83.

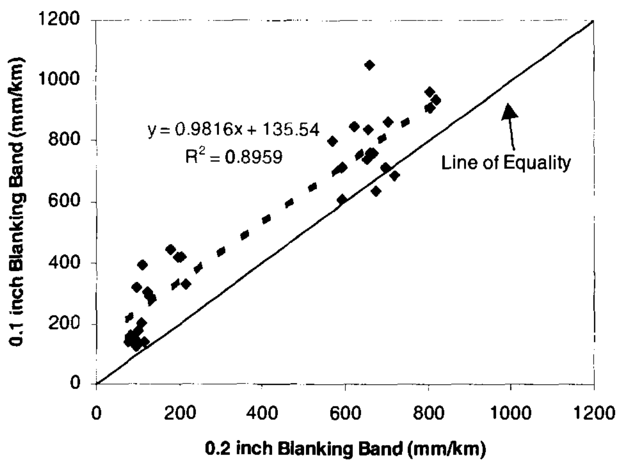


Figure 8 -- Effects of blanking band using California profilograph

Findings

Important findings are identified as follows:

- The two profilers yielded lower Coefficient of Variance (COV) among the six different devices, and the two profilograph-type devices yielded higher variation.
- Based on NCHRP 1-31 report [1], allowable PI values for flexible pavement adopted by state DOTs range from 110-240 mm/km. Most of the time, state DOTs have incentive programs to encourage PI values less than 50-110 mm/km. It is found that most of the newly constructed flexible pavements in Taiwan are able to meet the criteria given above. Due to lack of construction experience in rigid pavement, the roughness is much higher than those set by state DOTs.
- Based on the literature survey and the data collected in this study, the current 3mm criteria adopted by TANEEB is too strict and can be increased to 5mm.
- The size of the new project in Taiwan is relatively small as compared to most US projects, thus the ARRB walking profiler is a good choice for controlling smoothness of initial pavements.
- The IRI should be used for flexible pavement smoothness specifications, with a threshold of 2.0m/km in effect over the next several years. A target of 1.5m/km can be achieved after additional construction experience has been gained.

Conclusions

This study was aimed to evaluate the accuracy and consistency of different equipment for the development of smoothness specifications. For this purpose, comparative evaluations were made by using six different devices at 21 different pavement sections with different levels of roughness. These comparisons showed that the devices are able to collect reliable profile data and evaluate the acceptability of the finished project. Observations and conclusions are given as follows:

- Correlations among different types of devices have been developed with high R^2 values.
- For California profilograph, the 0.1-inch blanking band criterion yielded approximately a 20% higher profile index than the 0.2-inch blanking band, and was more sensitive to roughness.
- The two profilers yielded higher precision than the other four devices and the measurements accurately reflect different levels of roughness. The ARRB walking profiler is much easier to transport, assemble and operate than the other devices. Although the operation rate of the ARRB MLP is much higher than walking profiler, it is more expensive and may not be suitable for newly constructed concrete pavement due to its heavy weight. Thus, the ARRB walking profiler is recommended for use as a smoothness control tool on newly constructed pavements. The ARRB MLP can be used for speedily collecting roughness on existing pavement for management purposes.
- Based on the literature survey and the data collected in this study, the current 3mm criteria used by the TANEEB of Taiwan is too strict and can be increased to 5mm. The IRI should be used for flexible pavement smoothness specifications, with a threshold of 2.0m/km in effect over the next several years. A target of 1.5m/km can be achieved after additional construction experience has been gained.

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Equipment Comparisons, Materials Considerations, and Analyses

Christopher T. Wagner[1]

A Comparison of Devices Used to Measure Smoothness of Newly Constructed HMA Pavements

Reference: Wagner, C. T., "**A Comparison of Devices Used to Measure Smoothness of Newly Constructed HMA Pavements,**" *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433*, M. S. Gardiner, Ed., ASTM International, Conshohocken, PA, 2003.

Abstract: The construction of the National Center for Asphalt Technology (NCAT) track was utilized to evaluate the possibility of using the automated walking Australian Road Research Board (ARRB) profiler, the McCracken (a California style profilograph), and the South Dakota Profiler for analyzing pavement smoothness. Results indicate that there was a poor correlation between the ARRB unit and the McCracken profilograph. The data from these two units should be evaluated independently. There was a fair correlation between the ARRB and the South Dakota profiler. Because the ARRB unit uses an inclinometer for determining the profile, use of this profiler should be limited to sections without severe superelevations.

Keywords: Smoothness, ARRB, South Dakota Profiler, Profilograph

Introduction

There has always been a concern about the smoothness of pavements because smoothness, or conversely roughness, is one of the primary gauges of how the traveling public perceives pavement quality. The driving public generally equates smoother roads as being better roads. Considerable research has indicated a direct correlation between smoother pavement (immediately after construction) to extended pavement life [1]. Also, research shows a decrease in vehicle performance (fuel efficiency and vehicle wear) with a decrease in pavement smoothness [2].

Smoothness criteria are an important construction control parameter for pavements due to the effect on pavement life and public perception of pavement quality. Proper construction practices ultimately lead to a smooth, higher quality pavement. Conversely, initial roughness in a pavement can be an indicator of poor construction. Pay adjustment factors are increasingly used to ensure new pavements meet smoothness requirements.

Smoothness data collection has evolved from the use of the simple 8 to 16 foot straightedge to the high-speed (55 - 70 mph) inertial profilers. High-speed profilers use state of the art technology, computers, accelerometers, and lasers to collect pavement

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profile data. Application of these newer technologies combined with recent research will lead to smoother, higher quality pavements.

To achieve proper pavement smoothness it is important that proper construction practices are used. Various measurement devices are currently used to assure that new pavements are built to meet specifications. A summary of the types of devices used by state highway agencies for quality assurance is included in Table 1. A number of states allows the use of different profiling devices depending on the type of pavement.

Table 1 - *Smoothness Measurement Devices Used by State Highway Agencies [3]*

Profilograph	41
Inertial Profile Device	15[1]
Mays Ride Meter or Similar Device	6
Rolling Straight Edge	3
Rolling Dipstick	1

[1] 2 or more states plan to implement inertial profilers in the near future

The profilograph is still the device used by most State Highway Agencies to measure smoothness, but inertial profilers are continuing to gain use as a quality control and acceptance tool.

Background

Research on the more traditional methods of road profiling (straightedge, profilographs, and response-type road roughness meter systems) has been the subject of many publications; however, many of the newer profile devices (high-speed inertial profilers, lightweight inertial profilers, and inclinometer devices) have not been fully researched by independent agencies. Additional research of the newer devices is needed in order to evaluate their use for pavement quality acceptance.

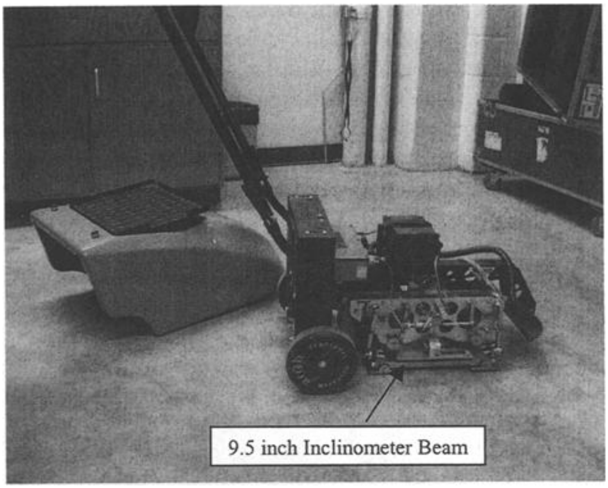

Figure 1 - *ARRB Walking Profiler Without Cover*

Research was conducted at the National Center for Asphalt Technology (NCAT) Pavement Test Track, in order to study construction practices and the effect on smoothness. Three different profile devices were used to collect the profile data: an inclinometer device, a profilograph, and a high-speed inertial system.

The inclinometer device used was the Australian Road Research Board Walking Profiler (ARRB) as shown in Figure 1. Inclinometer based profilers use a small straightedge beam up to 12 inches in length to measure profile. This beam is placed on the pavement surface and its inclination is measured and recorded. The beam is then moved its length and placed on the pavement surface where another measurement is made. This process is repeated for the length of the section being tested. The ARRB Walking Profiler then uses all of the measurements to create a longitudinal profile from which an International Roughness Index (IRI) is calculated.

The IRI is defined as the reference average rectified slope ($RARS_{50}$) of a standard quarter car at a traveling speed of 50 mph. A specific set of quarter car parameters was established in creating the IRI index. This set of parameters has come to be called the Golden Car, referencing the 1.0000 ft golden bar used to calibrate length [4].

The IRI is calculated from a single longitudinal profile assumed to have a constant slope between sample points. The gathered profile is then smoothed with a moving average having a base length of 10 inches. The smoothed profile is then filtered using the "Golden Car" parameters simulating the suspension motion, which is then linearly accumulated and divided by the length of the profile being tested [5].

The profilograph used was the McCracken model. The McCracken profilogragh consists of a rigid frame supported by a system of wheels; at the midpoint of the frame is a profile wheel that rests on the pavement surface and is linked to a strip recorder or computer. The profiling wheel is at the center of a straight plane and measures vertical deviations from that plain. A strip recorder is connected by a bicycle chain to the profile wheel that turns at a rate of 1 inch per 25 feet of horizontal movement of the profilograph. The vertical movement of the profile wheel is recorded on the strip recorder or in newer models a computer records the vertical and horizontal movements. The profilograph generally spans a length of 25 feet and records the pavement profile continuously. The profile trace is then analyzed to produce a roughness statistic called a Profile Index (PI).

The PI is determined by summing the deviation from a smooth plane. The sum of the deviations are then divided by the length of the tested section to produce the PI.

The inertial system used was the South Dakota Profiling System. Inertial systems use an accelerometer to measure the vertical motion of the vehicle and double integrates the signal to cancel vehicle motion to establish an inertial reference plain in space. Utilizing the inertial reference plain, a non-contact sensor (laser) determines the vehicle to road displacement for computation of longitudinal profile. This profile is then analyzed to produce an IRI.

The comparison of each system in collecting and measuring profile data on newly constructed hot-mix asphalt (HMA) pavements is evaluated in this report.

Scope

Research fieldwork began at the National Center for Asphalt Technology (NCAT) Pavement Test Track in conjunction with the beginning of paving in February 2000. There are a total of 46 sections on the test track. The last pavement section was completed in July 2000 and delineation and pavement markings were completed in August 2000. The completion of data collection for this research coincided with the completion of the paving of the test track in July 2000. Ongoing research on pavement smoothness and many other areas is scheduled through the life of the Test Track.

Profile measurements were taken of the base lift directly underneath the wearing course and the wearing course of the test sections. Profiles of the base lift were collected using both a McCracken California Profilograph and the ARRB Walking Profiler. Inside wheelpath and outside wheelpath profile measurements were taken on both the binder and wearing courses. The South Dakota Profiler was not available to profile the binder layer of the Test Track. Profile data collection on the East and West curve sections of the test track was limited by the superelevation. The steep superelevation of the curved sections made accurate profile measurements impossible to achieve with the McCracken model. For this reason, profile measurements of this section were made with the ARRB unit only. Further evaluation presented shows the effect of the superelevations on the ARRB unit.

Paving of the final test section was completed on July 20, 2000. Traffic loads were applied to the pavement test track starting in September of 2000. The accelerated traffic loading will eventually place a total of ten million Equivalent Single Axle Loads (EASLs) on the track test sections in two years. Factors that effect pavement performance such as weather, traffic loading, mix properties, and others will be studied in an effort to increase pavement performance and life span.

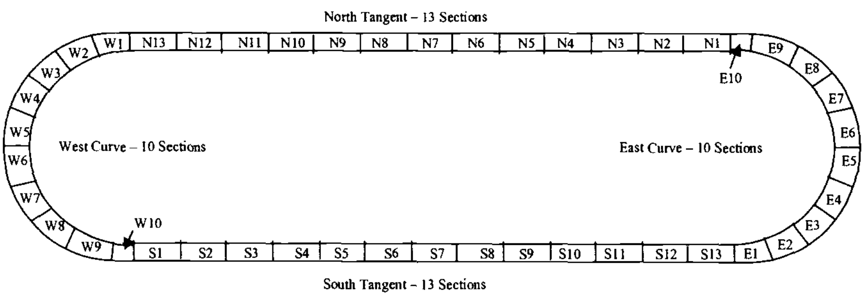

Figure 2 - *NCAT Pavement Test Track*

Correlation Between Units

The data obtained by the three devices was evaluated for comparison of the models. The data obtained by the ARRB unit and the profilograph on the base lift and wearing course of the test track was used to evaluate the similarity between the two units. The South Dakota Profiling System was not used to obtain data on the base lift; therefore, only data obtained from the wearing course was used to compare the different units.

Traces obtained from the profilograph and the ARRB units indicate that the devices capture similar profile data. An example of the profile traces with the smoothness irregularities circled is shown for a section of the pavement test track in Figure 3.

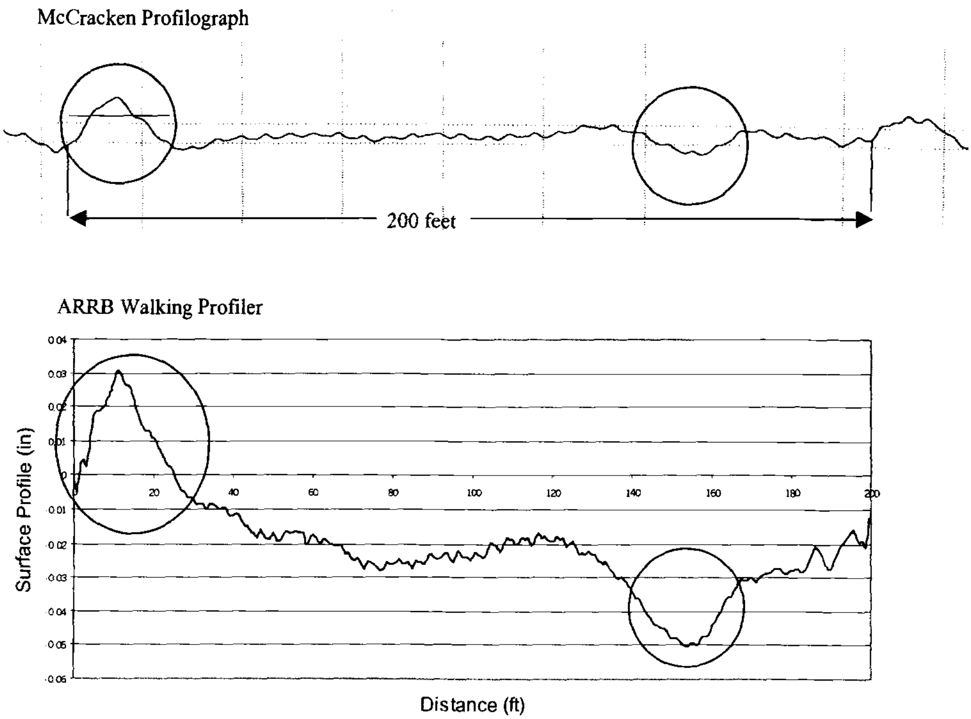

Figure 3 - *Profile Traces From ARRB Unit and Profilograph Unit*

The IRI data from the ARRB walking profiler was compared to the Profile Index data from the profilograph for both the inside and outside wheelpaths of the base and wearing course lifts. The profile index was calculated using a zero blanking band. The results are shown in Figure 4.

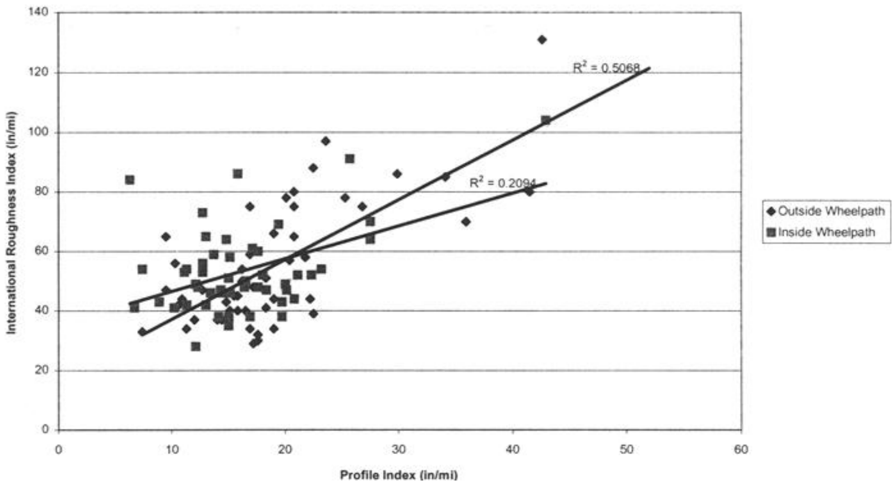

Figure 4 - *ARRB IRI Compared to Profilograph PI*

The data indicates that the outside wheelpath data had a fair correlation between the units with a R^2 value of 0.51 while the inside wheelpath data shows a poor correlation with a R^2 value of 0.21. This analysis indicates that the data from each of the units should be evaluated independently. There was no conclusive explanation found to explain the difference in correlations between the inside and the outside wheelpaths.

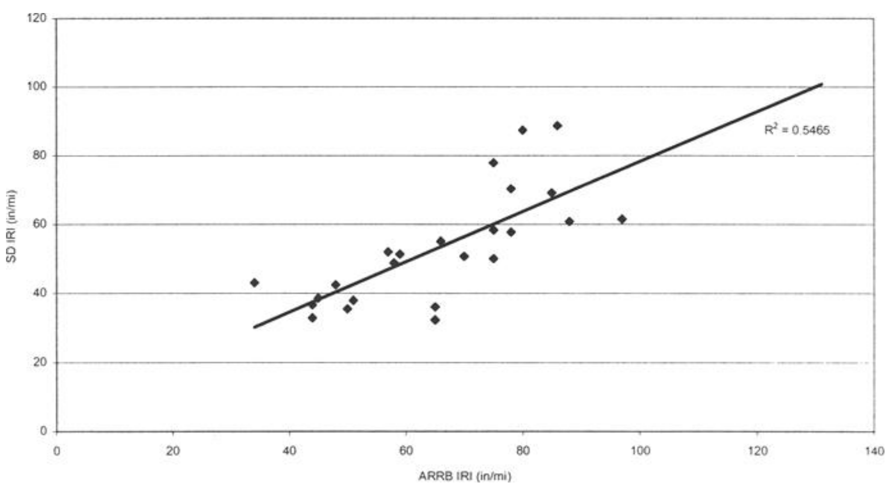

Figure 5 - *Comparison between ARRB and South Dakota Profiler*

A comparison of the ARRB unit and the South Dakota unit was made based on data collected from the inside wheelpath of the wearing course. Data was not collected with the South Dakota Profiler for the outside wheelpath or the base lift of the sections. The analysis between the ARRB unit and the South Dakota unit also indicates a fair correlation between the units with a R^2 value of 0.55. The results are shown in Figure 5.

The comparison between the South Dakota unit and the Profilograph indicates a poor correlation between the two units with a R^2 vale of 0.24. This analysis also indicates that the data from these two units should be used independently.

Operational Differences Between Units

There were several operational limitations of the ARRB unit and the Profilograph. Operations of these units were limited to the tangent sections of the test track. The IRI data from the ARRB unit indicates that there is a significant increase in roughness in the superelevations of the east and the west quadrants. This appears to indicate that the geometry of the pavement effects pavement roughness. However it was discovered that because the superelevations were measured with an inclinometer-based profile device, these increases are due to the "side to side" inclination of the inclinometer beam. The increase in IRI on the superelevated curved sections can be seen in Figure 6.

Collection of smoothness data on the east and west quadrants with the profilograph was limited due to the 25 foot length of the unit, the tight turning radius of the curves, and the superelevations on these sections. Data was obtained on the curved sections with the South Dakota Profiling System. This data did not show a significant increase in roughness values due to the effect of the superelevations.

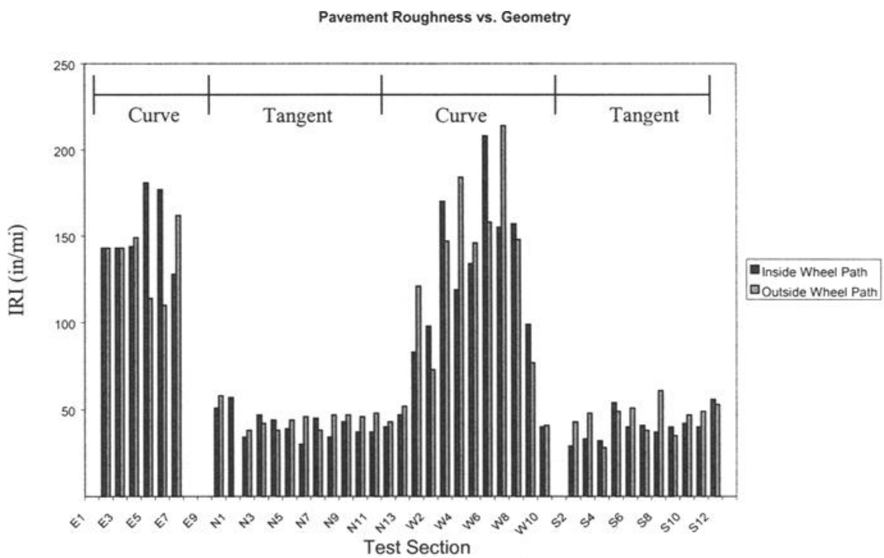

Figure 6 - *Effect of Superelevation on ARRB Unit*

The speed of these units should also be taken into consideration when determining the proper application of these devices. The speed of the ARRB Walking Profiler is about one-half walking speed (1.5 mph). Obviously this unit would not be suitable for network analysis and would be very cumbersome to use for quality acceptance. The profilograph system operates at walking speed (3-5 mph). The profilograph is currently the most used system for quality acceptance, but also has limited application to network management. The South Dakota Profiling system is also used for quality acceptance and is the preferred method of smoothness data collection for a network.

While different systems can be used in the same application (i.e. quality acceptance) the previous data analysis shows that caution should be used when comparing the data obtained from the different units.

Conclusions

The following conclusions can be readily observed from the analysis of the profile data collected by the three different units.

1. There was a poor to fair correlation between the ARRB walking profiler and the McCracken profilograph. Data from these two units should be evaluated independently.
2. There was a fair correlation between the ARRB walking profiler and the South Dakota profiler. Care should be taken when comparing data from these two devices.
3. The ARRB walking profiler should be limited to use on pavement sections without severe superelevations due to the influence of the side to side orientation of the inclinometer beam.
4. The ARRB walking profiler is limited to research applications due to its slow operating speed (1.5 mph).
5. The profilograph and the South Dakota profiler can be used for quality acceptance. The South Dakota profiler is applicable to network analysis due to its speed of operation.

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Effect of Temperature Differentials on Density and Smoothness

Reference: Stroup-Gardiner, M., Wagner, C. T., Hodgson, D. T., and Sain, J., **"Effect of Temperature Differentials on Density and Smoothness,"** *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433*, M. S. Gardiner, Ed., American Society for Testing and Materials International, West Conshohocken, PA 2003.

Abstract: A Roadware ARAN inertial profiler was used to calculate the International Roughness Index (IRI) for the Auburn University National Center for Asphalt Technology (NCAT) test track tangent sections as well as three paving projects in Michigan. IRI values were calculated for short intervals [4.6 to 7.6 meters (15 to 25 feet)] rather than for the traditional 160 meters (0.1 mile). These short intervals were able to identify easily localized anomalies in the pavement smoothness due to changes in the construction activities. They were also useful in evaluating relationships between IRI and mix variables in the short NCAT test track test sections [60 meters (200 feet)].

IRI repeatability (standard deviation for three replicates) immediately after construction was found to be 0.063 m/km (3.99 inches/mile) in the right wheel path for mixes at the test track. The left wheel path tended to have higher and more variable IRI values. This was attributed to the left laser path being close to potentially segregated mix at the edges of the screed extension. The IRI variability increased with traffic. Using this initial estimate of variability, some statistically significant differences due to mix variables such as aggregate source and/or binder grade can be seen.

Results from the test track indicate that the finer mixes tended to have IRI values that increase slightly as density increases. This suggests that working the fine mixes to obtain higher densities results in a higher initial roughness. This observation was also generally seen in the data from the Michigan field projects. IRI values decreased significantly within 6 months of trafficking for the majority of the mixes at the test track. This suggests that traffic can have a smoothing affect on the pavement for up to 6 months after the start of trafficking.

All results from this research were based on the construction of test track sections. These results should be verified on actual paving projects.

Keywords: roughness, smoothness, International Roughness Index

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Introduction

It is generally accepted that initially smoother pavements result in longer lasting pavements [1]. This concept is the driving force for most states to implement pay incentive/disincentive programs for pavement smoothness immediately after construction. While there is a considerable amount of experience with a wide range of devices for measuring smoothness, the number of different devices has led to a correspondingly wide range in how smoothness is quantified. For example, profilographs produce a mathematically calculated smoothness statistic called the profile index (PI) [2]. This statistic is derived from a rolling 7.6 meter (25-foot) base length profile trace using a zero, 2.54 or 5.08 mm (0.1 or 0.2) inch blanking band. Inertial profilers use a combination of lasers and accelerometers to record the pavement profile then calculate the International Roughness Index (IRI). This last smoothness statistic is quickly becoming the standard value for representing the driving publics' perception of rideability.

However, there has been relatively little work done that indicates the repeatability of replicate IRI measurements, or the influence of hot mix asphalt (HMA) mix variables and construction practices on the initial IRI. Estimates of repeatability will become increasingly important as more specifications are developed for inertial profiler generated IRI values. As warranties become more wide-spread, contractor decisions on aggregate source selection, gradation, binder type, compaction temperatures, and general construction practices will need to consider the effects of these variables on the initial IRI values because this will ultimately indicate the anticipated long-term pavement performance. This research project provides a preliminary evaluation of these topics.

Research Program

Objective

The objectives of this research program were to:

- Develop a precision estimate for repeatability for IRI measurements.
- Evaluate changes in IRI with time and traffic for a range of HMA mixtures.
- Identify any relationships between temperature differentials during construction, in-place densities and changes in smoothness.

The estimate of within-laboratory standard deviation associated with replicate IRI measurements was a required first step so that statistically different IRI values could be identified for the remainder of the analysis.

Scope

The National Center for Asphalt Technology (NCAT) closed loop test track consists of 46 test sections constructed using a wide range of aggregate sources, gradations, binder types, and additives (e.g., RAP, antistrip additives). Of these 46 test sections, there are 13 in each of the two tangent sections and 10 in each of the two super elevated curves. Only the tangent sections were evaluated for this study due to construction problems encountered in the curves.

The evaluation of the NCAT tangent test sections during construction included obtaining infrared images, hand held infrared temperatures immediately behind the paver, and the final in-place density. After construction, three replicate IRI values were obtaining for each of the six 7.62 m (25-foot) sublots in each section (Figure 1) once a week since the trafficking started at the test track. Three sets of three IRI replicates (one each from October 2000, January 2001, and May 2001) were selected from the main database for analysis. While a number of relationships between construction practices, density and smoothness were identified, the atypical construction associated with these short test sections made it difficult to confirm that these observations would be seen under more normal construction conditions.

A limited evaluation of three projects in Michigan was used to determine if there were a reasonable expectation that any of these initially identified relationships between temperature differentials, density and smoothness could occur during typical HMA construction. Infrared images were obtained during construction when temperature differentials were evident. Nuclear densities were measured on 10-foot longitudinal intervals in the outside wheel path 15.24 m (50 feet) before and after any area with temperature differentials. The IRI measurements were determine for each wheel path then averaged for every 6 meters (20 feet) of test section.

Project Information

NCAT Test Track

Each of the 46 test sections was divided into three lots, each with two sublots (Figure 1). There was a 15.06 m (25-foot) transition section at the beginning and end of each section that is reserved for periodic destructive testing. Table 1 summarizes the general variables included in the north and south tangent test sections. This table shows that while there was a range of gradations, aggregates, and binders (PG 64-22 and PG 76-22) were included; the gradations could be generally grouped as coarse, fine intermediate, and SMA. The PG 67-22, as classified by Georgia, grades as a standard Superpave PG 64-22.

During construction, the weather was generally hot, humid, and mostly clear. The paving operations were continuous throughout the short lengths of each test section (approximately 200 ft). Both a hand held infrared gun and an infrared camera were used to record the mat temperature prior to rollers starting compaction. Nuclear density measurements, correlated to cores taken from the inside lane, were used to establish the final in-place density for each test section.

The construction process used at the test track was not typical of normal construction practices because of the very short length (200 feet) of sections being paved. The inside (non-traffic) lane was paved first; the paver backed up, and then paved the outside (traffic) lane. There was no evidence of temperature segregation in any of the track sections. As with the other projects with uniform temperature, the only anomaly seen was a slightly cooler longitudinal center stripe due to the gearbox. This limited temperature differential was not considered significant.

Because there was no evidence of temperature segregation in the infrared camera images and the rollers started compaction immediately after the paver finished pulling the 200 foot test section, it was assumed that the mat temperature immediately behind the paver was indicative of the temperature at which compaction was started. These temperatures are shown in Table 1 for each of the test sections. The corresponding final nuclear densities (corrected based on core densities) and the IRI over the center 150 feet of each test section are also shown in this table.

Table 1 - *General information for NCAT test track sections.*

	Binder	Aggregate Source[1]	Gradation	Mat Temp. °C	% Max. Density	IRI in Right Wheel Path, in/mile[2]		
						Oct 2000	Jan 2001	May 2001
S9	PG 67-22	Granite	Coarse	166.7	93	28.26	26.08	21.95
S7	PG 67-22	Lms/RAP	Coarse	156.1	93.3	29.95	34.48	40.33
N5	PG 67-22	Lms/slag	Coarse	138.9	92.8	43.25	34.74	31.73
N6	PG 67-22	Lms/slag	Coarse	142.8	95.8	25.37	25.78	24.48
S1	PG 76-22	Granite	Coarse	168.3	94.1	46.03	51.95	50.02
S8	PG 76-22	Granite	Coarse	166.1	91.5	43.10	46.53	47.71
S11	PG 76-22	Granite	Coarse	148.9	93.3	51.32	50.99	34.14
S2	PG 76-22	Gravel	Coarse	162.8	94	32.28	33.62	34.52
S3	PG 76-22	Lms/gravel	Coarse	157.8	92.8	45.26	36.50	32.33
N7	PG 76-22	Lms/slag	Coarse	161.7	94.1	26.98	30.46	23.32
N8	PG 76-22	Lms/slag	Coarse	159.4	95.3	51.94	34.45	31.64
N9	PG 76-22	Lms/slag	Coarse	156.7	94.3	50.59	43.87	32.97
N10	PG 76-22	Lms/slag	Coarse	162.2	94	26.30	25.74	28.14
S10	PG 67-22	Granite	Fine	166.1	93.4	31.48	24.19	22.28
S6	PG 67-22	Lms/RAP	Fine	166.7	94.1	40.35	32.77	31.65
N3	PG 67-22	Lms/slag	Fine	146.1	95.1	66.09	56.99	49.56
N4	PG 67-22	Lms/slag	Fine	142.2	94.7	31.40	28.76	27.75
S13	PG 76-22	Granite	Fine	158.9	93.9	137.11	130.46	78.71
S4	PG 76-22	Lms	Fine	161.1	93.4	46.87	39.09	32.38
N1	PG 76-22	Lms/slag	Fine	152.2	94.3	37.92	36.05	36.20
N2	PG 76-22	Lms/slag	Fine	157.8	92.9	37.52	28.63	33.56
N11	PG 76-22	Granite	Intermediate	163.9	93.1	51.51	49.08	33.78
S5	PG 76-22	Gravel	Intermediate	161.7	94.7	29.48	30.04	32.52
S12	PG 76-22	Lms	Intermediate	167.8	94.3	78.35	80.36	50.24
N12	PG 76-22	Granite	SMA	172.8	94.6	45.59	37.01	35.84
N13	PG 76-22	Granite	SMA	158.9	92.5	45.63	36.61	34.91

1:Lms = limestone

2: average of two 25-foot (7.6 meter) sublots in the center 50 feet (15.2 meters) of each test section.

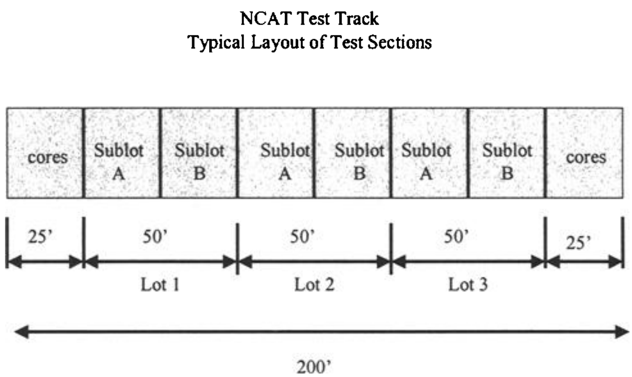

Figure 1 – *NCAT Test Track Layout.*

Michigan

All three Michigan projects were evaluated at the end of September 2000. Each project is briefly described in the following sections.

M 52 near Perry, Michigan - This project was a conventional daytime paving project on M 52 that is a two-lane rural highway near Perry, Michigan. A portion of the northbound lane was used for the test section. The HMA was a 12.5 mm Superpave mix. The job mix formula (JMF) gradation and asphalt content are shown in Table 2. The binder was a PG 58-28. The weather was overcast, cool and breezy during the paving operation.

There were a number of stoppages in the paving operations during the construction of this test section. Rollers consistently remained at least 152 to 304 m (500 to 1 000 feet)behind the paver.

US 27 and M 57 - This project evaluated the outside southbound upper lift of US 27 at the junction of M 57. The JMF for this project is also shown in Table 2; the binder was a PG 64-28. Weather conditions were sunny, cool, and calm.

The paving operations were consistent; there were no stoppages during the construction of the test section. The rollers stayed immediately behind the paver although there was some lag in rolling due to a railroad crossing in the middle of the test section.The length of the test section was limited due to equipment problems early in the day. The only available area for a test section was before and after the railroad tracks immediately prior to the junction with M 57.

Table 2 - *JMF for Michigan and selected NCAT test track sections.*

Sieve Size, mm	US27 and M57	M 52	N2	N11	N8	N12
			Fine	Intermediate	Coarse	SMA
Cumulative Percent Passing, %						
19	100	100	100	100	100	100
12.5	100	100	99	97	99	96
9.5	99.6	99.6	90	80	85	73
4.75	82.2	80	66	52	54	32
2.36	58.9	55	50	37	37	23
1.18	42.9	41	33	30	24	21
0.6	32.3	31	22	24	17	19
0.3	19.4	19	16	18	12	17
0.15	9.7	9.5	11	11	9	14
0.075	5	5	7.6	7.2	6.6	11.8

*SMA job mix formula information not available for Michigan.

US 27 and M 57 - This project evaluated the outside southbound upper lift of US 27 at the junction of M 57. The JMF for this project is also shown in Table 2; the binder was a PG 64-28. Weather conditions were sunny, cool, and calm.

The paving operations were consistent; there were no stoppages during the construction of the test section. The rollers stayed immediately behind the paver although there was some lag in rolling due to a railroad crossing in the middle of the test section.The length of the test section was limited due to equipment problems early in the day. The only available area for a test section was before and after the railroad tracks immediately prior to the junction with M 57.

I 94 near Detroit, Michigan - This was a nighttime project that paved the outside lane of east bound I 94 near Detroit. The mix was an SMA (no JMF information was obtained). Windrow paving with an Acupave mixer was used.

The paving operations were consistent; no stoppages in the paving operations were noted. The rollers stayed immediately behind the paver.

Testing Program

Density Measurements

NCAT Test Track –One random location was selected in each lot (see Figure 1) for density testing. While both the inside and outside lanes were tested using the nuclear gauge, cores for correlation purposes were only obtained from the inside (non-traffic) lane. Standard one-minute readings in each of four positions (90° rotation around the point) were averaged. The density reported in Table 1 represents the average nuclear density for both lanes in the top lift corrected to the average core density.

Michigan - Only relative nuclear densities were used to evaluate the effect of temperature differentials on in-place density. Testing was limited to one position with one-minute readings due to the large number of density tests needed and the limited length of time that traffic control was available in the construction zone.

IRI Measurements

A Roadware ARAN profilometer was used to measure the IRI in both wheel paths for all of the projects. This profilometer meets the requirement for Class 1 equipment as described in the American Society for Testing and Materials Standard Practice for Measuring the Longitudinal Profile of Traveled Surfaces withan Acclerometer Established Inertial Profiling Reference (E 950)]. The testing reported for the test track was conducted at 70 kph (45 mph) . IRI values were obtained for 7.6 meter (25-foot) increments. Because of the large increase in IRI values seen at each of the construction joints between each test section, one IRI average (i.e., one 7.6 meter (25-foot) section) on either side of the joint was eliminated from the database. The IRI values reported represent the average IRI value over the center 45.6 meter (150-foot) - of each test section.

The testing for the Michigan project was conducted at 32 kph (20 mph) . IRI values were obtained every 6 meters (20 feet) for the length of each test section.

Results and Discussion

IRI Repeatability

The variance within a set of three replicate IRI measurements was determined for every 7.6 meter (25-foot) sublot on the test track; each wheel path was evaluated independently. The average within-laboratory variance was calculated for each 7.6 meter (25-foot) sublot and used to estimate the within-laboratory standard deviation associated with a set of three IRI replicate measurements. Outliers were removed from the database by establishing an acceptable within-replicate range of four standard deviations. Once these outliers were identified and removed, the process was repeated until no more than 5 percent of the data sets exceeded this range. This usually took about three to four iterations. Data sets that were removed from the database were consistently from within the first and last 15.2 meters (50 feet) of each test section. This suggests that start and stop construction practices substantially increased the IRI variability in these areas.

A closer examination of the raw profile data helps explain this increased variability. A swell in the profile within the first 7.6 meters (25 feet) can be seen, this is attributed to the roller moving from the cold mat of the previously placed section onto the hot mix in the new section. It appears that this construction procedure generates an initial hump in the fresh mix. . There are also small ripples in the last 25 feet of the section. This is attributed to the roller turning slightly so that it can back up for a return pass over another part of the section [3].

The IRI database for each month evaluated was processed as described above. Table 3 shows the final estimates of repeatability for each wheel path for each month data was evaluated. In both October 2000 at the beginning of the trafficking, and in May after about 3 million equivalent single axle load applications, the right wheel path shows a lower IRI variability than the left wheel path. This difference is especially obvious for the May data. There appears to be an increase in IRI variability with increasing traffic load applications. This is logical since the transverse profile within the wheel path can be expected to be more variable as pavement distresses increase due to traffic loads. It also implies that repeatability of IRI measurements will be difficult to estimate after initial construction because of the traffic-related and/or distress related impact on variability.

Table 3 – *IRI Repeatability as developed from NCAT test track database.*

Statistics	October		January		May	
	Left Wheel Path	Right Wheel Path	Left Wheel Path	Right Wheel Path	Left Wheel Path	Right Wheel Path
IRI Standard Deviation within a set of three replicates, m/km (in/mile)	0.079 (4.99)	0.054 (3.39)	0.079 (5.00)	0.085 (5.37)	0.239 (15.12)	0.099 (6.24)
Acceptable range of two test results, m/km (in/mile[1])	0.223 (14.12)	0.152 (9.58)	0.223 (14.13)	0.240 (15.18)	0.676 (42.77)	0.279 (17.65)

1: estimated as 2 times square root of 2 times the standard deviation

An examination of the infrared images obtained during construction also suggests why there is a difference in the IRI between wheel paths. Figure 2 shows the infrared images for two sections in the south tangent. Subtle differences between the two types of pavers can be seen as thin longitudinal cold stripes in the area of the auger gearboxes. Based on previous experience, these areas indicate areas of slightly coarser HMA [4]. Because these anomalies occur between the wheel paths, they would not be expected to influence the IRI measurements. However, both pavers used screed extensions to pave the full 4.8 meter (16-foot) width of the outside lane (driving lane plus shoulder). A longitudinal stripe indicating cold, most likely slightly segregated mix at the start of the screed extension can be seen about 0.6-m (2-ft) in from the construction joint [4]. Another one can be seen about 4-foot (1.2 meters) in from the shoulder leaving about 3.0 meters (10 feet) between the stripes. The construction joint was used as a sight line for the ARAN van driver that resulted in the left laser path approximately 0.91-m (3-ft) off of the joint. In other words, the laser profile was within about 0.3-m (1-ft) of the slightly segregated area. The right laser would then be more than 0.6-m (2-ft) away from the other extension stripe. The difference in the mix in or near the left wheel path may account for the increased variability when compared to the right wheel path.

Based on these results, further comparisons will use the averaged IRI data for the center two sublots for the right wheel path only. This will provide the least variable estimate of IRI for each mix type used at the test track.

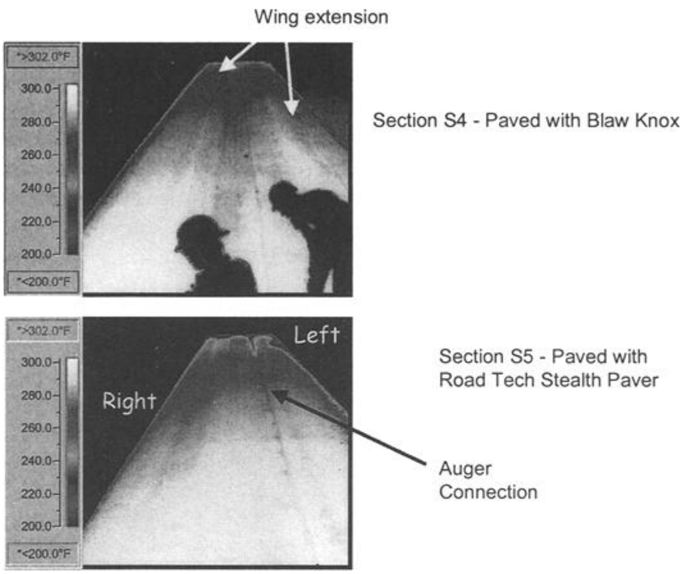

Figure 2 – *Infrared images from selected NCAT test sections.*

Changes in IRI with Time

Figure 3 indicates that 4 of the 13 coarse graded sections had IRI values that decreased significantly within the first three months of trafficking (N5, S3, N8, and N9). This suggests that some of the initial roughness may be smoothed out by traffic loadings. The granite mixes with the PG 76-22 tended to have a consistent IRI of around 40 in/mile. The one mix with the same gradation and aggregate but with the PG 67-22 binder had an IRI of about half that value. After about six months of traffic, all of the limestone blends had IRI values between 0.32 and 0.63 m/km (20 and 40 in/mile)while the granite mixes with the PG 76-22 binder had values of between 0.55 and 0.79 m/km (35 and 50 in/mile).

Figure 4 shows that five of the eight fine graded sections had IRI values that decreased significantly after six months of traffic. The granite with the PG 76-22 had a much higher IRI value, regardless of time after trafficking than any of the other mixes.

Figure 5 shows that four of the five intermediate and SMA mixes showed a significant decrease in IRI values after six months of traffic. Both the intermediate and SMA granite gradations with the PG 76-22 had similar IRI values. The gravel mix had the lowest IRI values while the limestone blend had the highest in these comparisons.

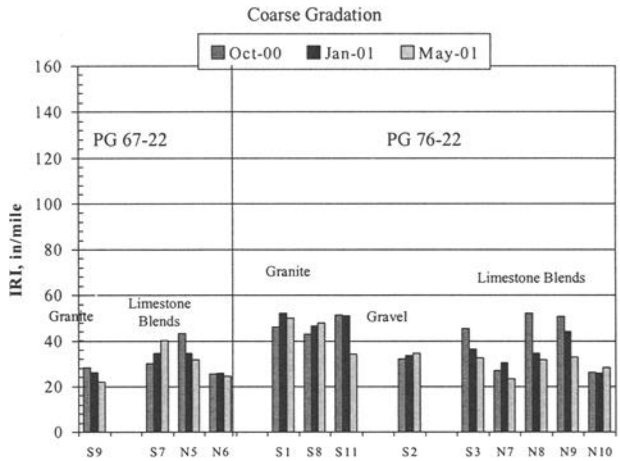

Figure 3 – *IRI with time for coarse graded NCAT tangent test sections.*

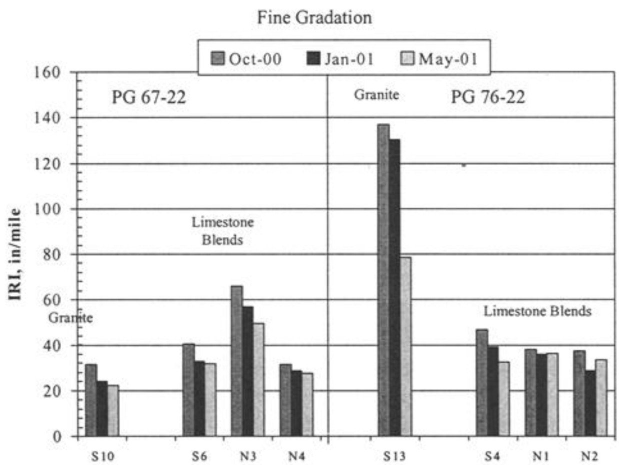

Figure 4 – *IRI with time for fine graded NCAT tangent test sections.*

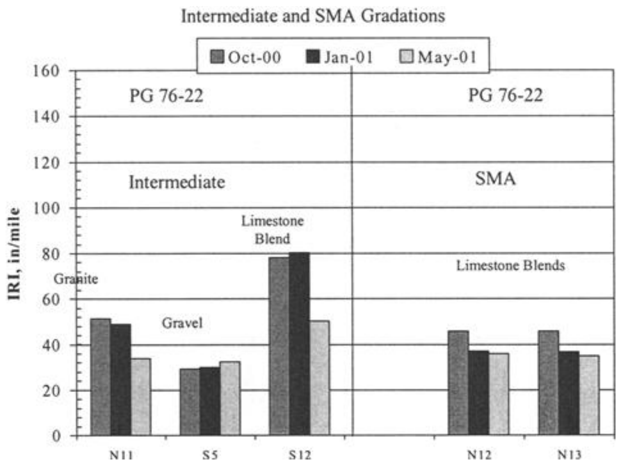

Figure 5 – *IRI with time for intermediate and SMA graded NCAT tangent test sections.*

Relationships Between Temperature, Density, and Smoothness

Figure 6 suggests that the finer gradation mixes that had higher densities also tended to have increasing IRI values. That is, density may be obtained at the cost of smoothness. No relationship was seen between mat temperature, density and initial IRI values for the coarse, intermediate or SMA mixtures. This hypothesis should be confirmed on actual paving projects; the test track data is too limited to draw solid conclusions.

M 52 near Perry, Michigan - Approximately 305 meters (1 000 feet) of paving were used as a test section for this project. Figure 7 shows an example of the typical image obtained when the construction process stopped for any length of time. There is a distinct transverse line that is indicative of the HMA cooling behind the screed. The HMA was about 68°C (155°F) in this area. The temperature increased to about 120°C (278°F) once the fresh mix started to move through the paver. There was also a colder region in the center of the lane in the foreground of the picture that is from the cold mix in the paver hopper. The temperature in this region was 87°C (176°F).

Given these infrared images, it would be expected that there should be lower densities in areas with evidence of temperature segregation. However, nuclear density measurements in selected areas with and without temperature differences showed no clear evidence of lower densities in the areas associated with cooler infrared images. This is most likely a function of how far the rollers were behind the paver. That is, the mat was being rolled at temperatures of 95°C (205°F) or less in areas with continuous paving operations. It is likely that density differences due to initial temperature differentials may be reduced once the overall mat temperature cools below a certain limit. The average densities on this project ranged from 2 156.1 to 2191.3 kg/m^3 (134.6 to 136.8 pcf) (uncorrected nuclear density gauge readings).

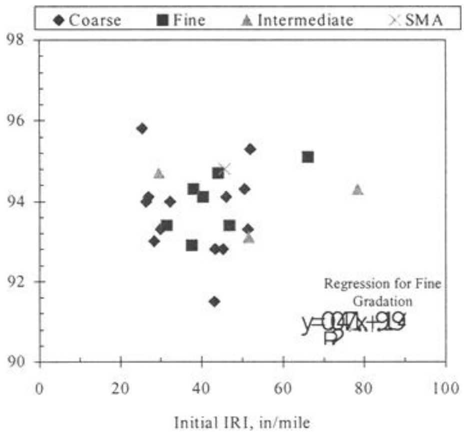

Figure 6 – *Relationship between mat density and initial IRI for NCAT test track.*

Figure 8 shows areas with uniform temperatures but rolled at temperatures below 96°C (205°F) also had IRI values around 0.47 to 0.79 m/km (30 to 50 in/mile). The IRI more than doubled in areas where paver stops were noted. In general, the IRI values tended to fluctuate throughout the project; this is assumed to reflect the varying temperatures at which the mat was rolled.

US 27 and M 57 – This project used essentially the same mix except that the asphalt content was one-tenth of a percent lower than for the M 52 project. The mat temperatures immediately behind the paver in the uniform temperature areas were similar to those seen in the hotter areas for the M 52 project. However, paving operations were continuous for this project and the roller stayed consistently close to the paver. The only exception was at the railroad crossing in about the center of the test section.

There was no thermal evidence of temperature segregation and densities were consistent throughout the test section. The average nuclear density of 2 221.8 kg/m^3 (138.7 pcf) for this project was very consistent (within about 2 pcf range). The IRI values for this section were between 0.63 and 0.95 m/km (40 and 60 in/mile). The slight increase in the IRI values for fine mixes compacted to higher densities as seen for the test track was also seen when comparing the results for these projects.

Perry, Michigan

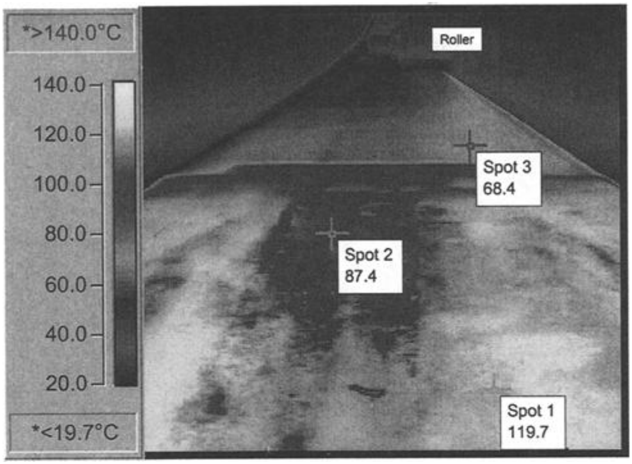

Figure 7 – *Infrared image typical of after a paver stop (Michigan).*

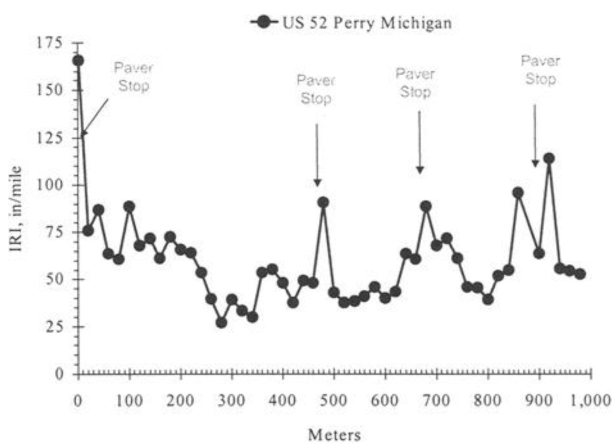

Figure 8 –*Effect of paver stops on IRI (Michigan).*

I 94 near Detroit, Michigan **- The infrared images for this SMA project showed consistent uniform temperature other than slightly cooler longitudinal stripes due to the**

auger boxes. This was a windrow paving operation with an Acupave mixer and there were no stoppages in the operations. The only anomaly in the IRI is at the joint at the start of the paving (Figure 9). In general, the IRI values were consistently between 0.40 and 0.63 m/km (25 and 40 in/mile) for this SMA project. Spikes in the IRI due to the construction joint are easily identified with the short 4.76-meter (15-foot) intervals for calculating the IRI. Both the magnitude of the IRI and the consistency of the values were similar to those seen for the NCAT SMA mixes.

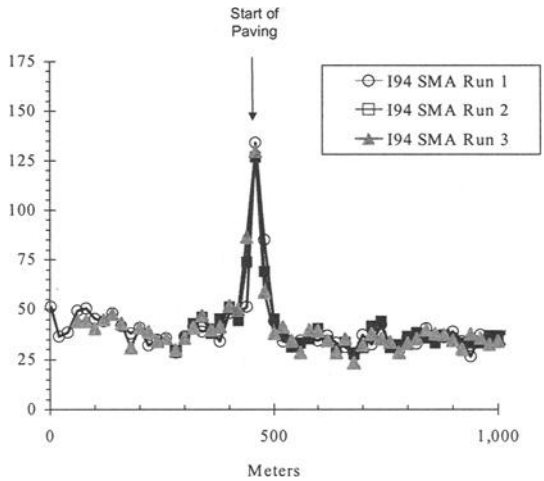

Figure 9 – *Influence of construction joint on IRI (Michigan).*

Conclusions

The following conclusions can be drawn from this research project

1. Construction practices created variability in IRI measurements. Rollers moving onto a hot mat from a cold one produce a "hump" in the profile for at least the first 7.6 meters (25 feet). Ripples in the pavement profile are seen when the rollers turn slightly prior to backing up for a return pass. There appears to be some slight segregation of the mix at the inside edge of the screed extension. If the left wheel path is close to this region, the variability and the mean IRI increase due to the mix variations.
2. Repeatability of three replicate IRI measurements for pavements tested immediately after construction was 3.99 in/mile in the right (least variable) wheel path.
3. IRI variability tended to increase with traffic. This may make it difficult to get good estimates of statistically significant changes in IRI on pavements evaluated after a significant number of traffic load applications.

4. Traffic loads of up to 3 million ESALs with no clear evidence of pavement distress resulted in lower IRI values when compared to the initial values. This would suggest that testing programs that allow initial IRI values to be obtained after some trafficking results in lower initial values in a number of cases.
5. The data suggest that there may be some differences in IRI values due to the material and/or gradation selections used in the NCAT test track. This implies that initial IRI values may be mix-dependent.
6. Coarser mixes tended to have more IRI values more consistently below 40 in/mile than the fine graded mixes.

All results from this research were based on the construction of test track sections. These results should be verified on actual paving projects.

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Characterizing Pavement Profile Using Wavelets Analysis

Reference: Attoh-Okine, N. O. and Mensah, S., **"Characterizing Pavement Profile Using Wavelets Analysis,"** *Constructing Smooth Hot Mix Asphalt (HMA) Pavements, ASTM STP 1433,* M.. S. Gardiner, Ed., American Society for Testing and Materials International, West Conshohocken, PA, 2003.

Abstract: Pavement smoothness can be described by the magnitude of the profile measurements and their distribution over measurement intervals. Surface smoothness, especially on newly constructed pavements, is a major concern for the highway industry. The smoothness is a measure of the quality of the constructed pavements. Measured data from profilographs are inherently multiscale in nature owing to different contributions from events occurring at different locations and with different localization frequency. Therefore, a data analysis method that can represent the measured data at multiple scale is better suited for extracting information and making inferences. This paper presents a mathematical tool, a wavelets analysis, that has the potential to extract information at different scales. The multiscale property and structure of the wavelet algorithm can lead to a method of analysis and display that highlights changes in the profile measurements.

Keywords: Pavements Profile, Profilograph, Wavelets, Smoothness

Introduction

Wavelets have proven to be very useful in many areas of engineering and science. Wavelets have been used in denoising [1], constructing regression models [2], reduction of distributed parameter system [3], particle shape analysis [4] and pavement profile evaluation and assessment [5].

Pavement smoothness is one of the important indicators of pavement ride quality. It is extensively used in both pavement management decision making and the acceptability of new and reconstruction. Pavement smoothness can be described by the magnitude of profile regularities and their distribution over measurement intervals. The surface smoothness, especially on a newly constructed pavement, is a major concern for highway agencies. This affects the road user directly. A recent study [5], which includes data from more than 200 pavement projects in 10 states, for most pavement types, found a 25 percent increase in initial smoothness produced about 9% increase in life. A 50% increase in smoothness yielded a minimum 15% increase in pavement life.

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A profilograph is a basic instrument for characterization, evaluation, specification, and quality control of pavement smoothness during pavement construction. The profilograph measures the vertical deviations from a moving fixed length and reference plane. The procedure generates graphical charts known as profilograms. A process known as trace reduction is used to derive a profile index [6]. The profile index provide the quantitative measure of the smoothness of the pavement.

The objectives for smooth measurement include:

- Tracking construction quality control,
- Location of abnormal changes in the pavement profile,
- Establishment of a basis for allocation of resources for road maintenance and rehabilitation, and
- Determination of pavement roughness that can be used in pavement deterioration modeling.

Sconfield [1992] lists the problems regarding smoothness measures and interpolation of the test results. These include:

- Effect of surface type,
- Trace reduction,
- Interpretation of traces (profile), and
- Identification of grinding locations (maintenance spots).

Attoh-Okine [1999] used Dubachies-5 (classes of wavelets) with level 5 to assess and evaluate pavement profile. The mathematical explanation of level is discussed in the next section. Attoh-Okine [1999] investigated the denoising of the original profilograph, identification of abnormal behavior of the profilograph and multiscale feature detection. The aim of this paper is to extend further the [5] studies, by presenting more concise and easy to understand description of the wavelet technique; and comparing different types of wavelets, and an attempt to develop a unify framework of the application of wavelets in pavement smoothness assessment. The paper used data from LTPP (Long Term Pavement Performance) GPS (General Pavement Studies) [8] pavement studies for the analyses.

Wavelets

A wavelet transform involves the decomposition of a signal function or vector into simpler, fixed building block at different scales and positions [9]. The decomposition is a successive approximation method that adds more and more projections to the detail spaces spanned by the wavelets and their shifts at different scales. The wavelet transform characterizes the fine component of nonstationary signal. This fine component implies high frequency or small scale. Compared to Fourier transform, the advantages of wavelets lie in their localization in both time and frequency [10]. "Signal" as used in the paper refers to pavement profile.

In wavelet analysis, the low frequency content is called the *approximation* and the high frequency content is called the *detail*. The filtering process uses *lowpass* and *highpass* filters to decompose an original signal into the approximation and details of the signal.

Mathematical Representation

The natural framework for the construction of wavelets is given by multiresolution analysis (MRA) which consists of a successive decomposition in an hierarchical scheme of approximations and different levels [11]. The basic idea of MRA is shown in Figure 1. The MRA involve the following:

- The approximation and detail signal are computed from the original signal at the first scale.
- In the second stage twice-as-large features are extracted from the approximated signal of the first scale and another coarse approximation is computed.

Figure 2 shows the mathematical representation of the MRA. Figure 2 shows four steps or four scales. In the first scale, the original profile data is split into *approximation* A_x^1 and detail D_x^1. The detail D_x^1 is supposed to be noise components of the original profile. A_x^1 is further decomposed into *approximation* A_x^2 and *detail* D_x^2. The same process is used to construct all the remaining steps. In each step the extrema of the detail are found. As the scales are increased, the noise extrema will be gradually removed while the extrema of the noise free profile remains. Many different wavelet transforms have been proposed in the literature. The most common is the Haar wavelet.

Discrete wavelet transform (DWT), which will be used in assessing the smoothness, proceeds as follows: two related convolutions on the profile with one being the low-pass filter **H** (={h_k}) and the other a high-pass filter **G** (={g_k}). The profile is then converted into two bases with equal size

(1) $$c_k^{(j)} = \sqrt{2}\sum_n c_n^{(j-1)} h_{n-2k}.$$

and

(2) $$d_k^{(j)} = \sqrt{2}\sum_n c_n^{(j-1)} g_{n-2k}$$

The variables h_k and g_k are coefficients of low-pass and high pass filters, with the following properties

$$g_k = (-1)^k h_{1-k}$$

$$\sum_k h_k = 1$$

$$\sum_k g_k = 0$$

$\{c_k^{(j)}\}$ is called the wavelet coefficients and $\{d_k^{(j)}\}$ are the detail information.

Computational Example

Data for the profile analysis was obtained from of the General Pavement Studies

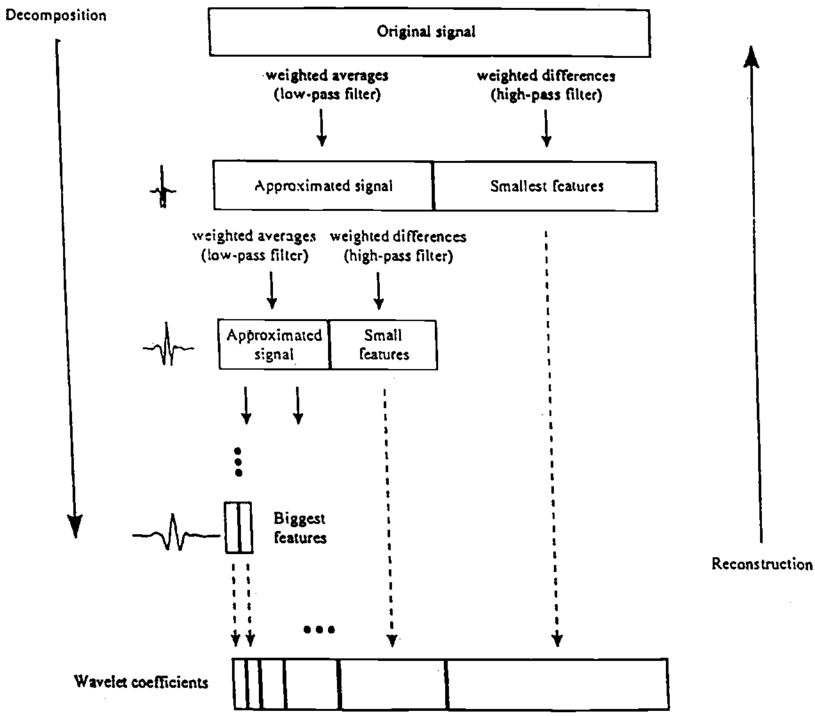
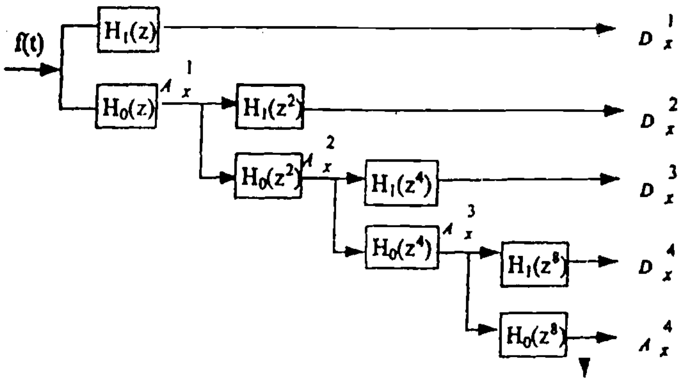


Fig. 1—Multiresolution analysis.



D_x^i -- detail on the ith decomposition

A_x^i -- approximation on the ith decomposition

H_0, H_1 -- low-pass and high-pass filters

Fig. 2 —Mathematical representation.

(GPS) of LTPP database. The profilograph values were in inches. The data consist of 1000 discrete elevation points of a pavement section in inches. The pavement section is about 1000 ft long. Table 1 shows the basic statistics of the profile data.

The Matlab Wavelet Toolbox [12] was used to perform the analysis. Three different wavelets were investigated: a) Haar with level 5; b) Daubechies - 1 wavelet with level 5; and c) Daubechies- 2 wavelet with level 5. Figure 3 shows the Haar wavelets with the various details and approximation. *S* is the original profile and *a* is the approximated signal and *d's* are the details. Figure 4 is the corresponding tree decomposition of the *Haar* wavelet. Figure 5 shows the wavelet transformation using the *Daubechies-1 level 5* wavelet and Figure 6 shows *Daubechies-2 level 5* wavelet decomposition of the profile. Based on the transformation the following statistics were generated for the approximated profile using the wavelets.

Table 1. *Statistical comparison of the original profile and reduced profile**

Wavelets	Mean	Median	Mode	Max	Min	Range	Std. Dev
Original Profile	0.348	0.686	1.966	18.29	-17.98	36.27	5.454
Haar-5 Wavelet	0.346	0.699	1.966	18.17	-17.84	36.01	5.449
Daubechies 1-5	0.346	0.699	1.966	18.17	-17.84	36.01	5.449
Daubechies 2-5	0.347	0.668	1.911	18.29	-18.11	36.40	5.452

* *The profile measurements are in inches*

Table 1 indicates that there is virtually no change in *Haar* and *Daubechies*-1 level 5 wavelets since they provide the same statistical result. The same approach can be used to analyze a selected range of pavement sections. In pavement profile analysis, pavement engineers often are faced with the problem of recovering a true profile from incomplete, indirect, or noisy data. This can be achieved by thresholding, that is, if details are small they can be omitted without substantially affecting the main features of the profile [5].

In most profile studies, the measurements are taken from different wheel paths on the same road network, the above approach can be used to compare the profile of the section and deduce the "true" profile measurement of the section. The different wavelets and corresponding levels represent the form the profile was initially composed in terms of approximations and details. Therefore depending on the characteristics of the profile different wavelet forms will better describe the profile. Therefore the effect of surface type, trace reduction techniques and identification of anomalies can be detected based on the outcome of the wavelet analysis. For example two pavement sections with the same traffic and material properties can have different wavelet outcomes. This can be interpreted as the presence of maintenance spots in one section.

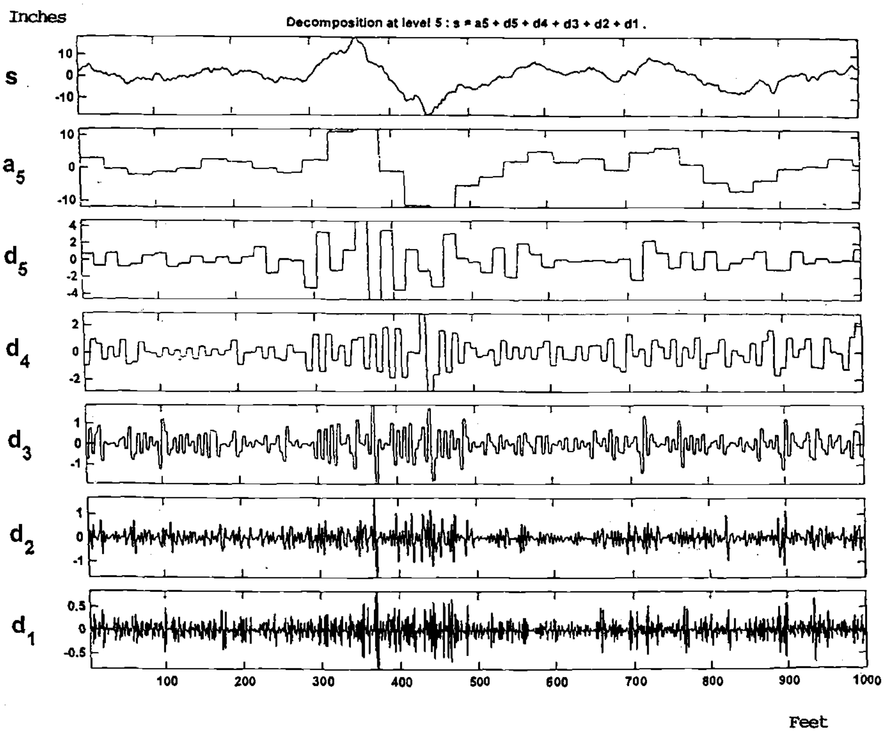


Fig. 3—Haar wavelet decomposition of pavement smoothness profile.

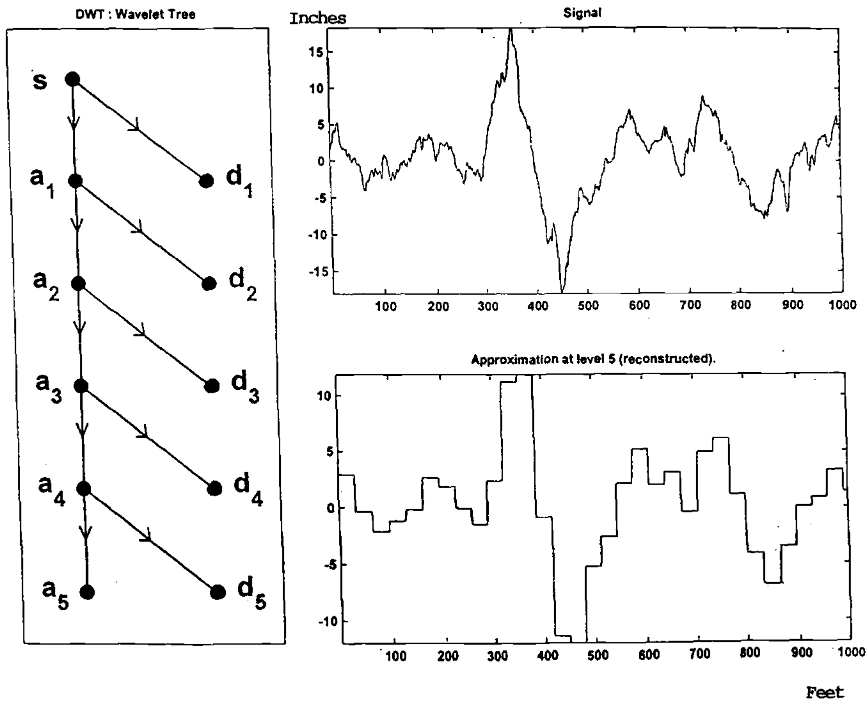


Fig. 4—Tree decomposition of Haar wavelets shown in Fig. 3.

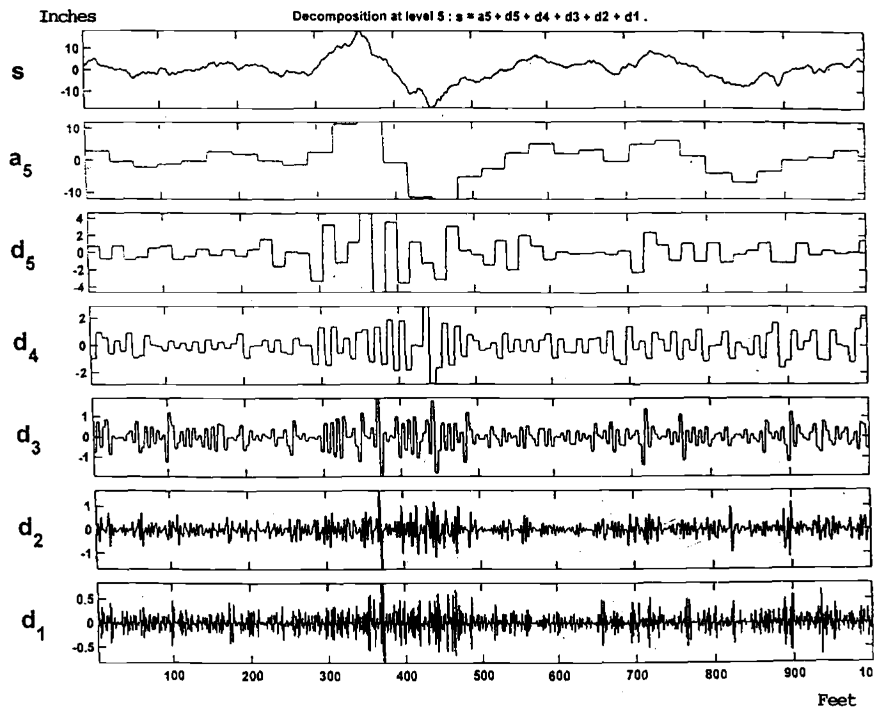


Fig. 5—Daubechies 1 level 5 wavelets decomposition of pavement smoothness.

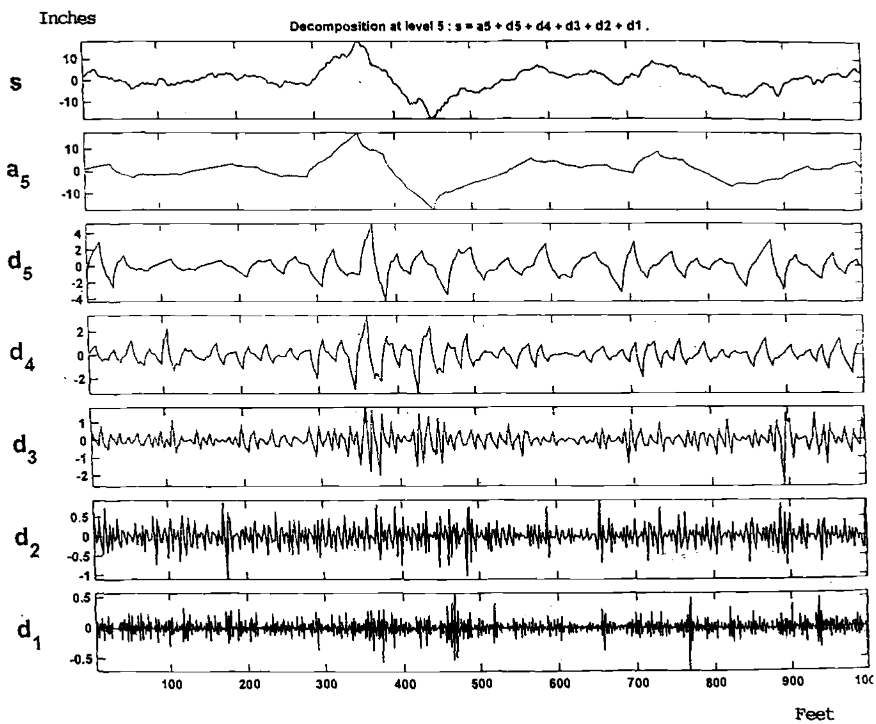


Fig. 6—Daubechies 2 level 5 wavelets decomposition of pavement smoothness.

Summary

Smoothness is a very important measure of quality of both newly and reconstructed pavements. The profile measurement are multiscale and therefore a correct tool is needed to analyze information in the profile data. This analysis demonstrates that a discrete wavelets analysis can be used in profile data analysis. Using the decomposition approach (discussing the profile in terms of approximate and detail signal) of the wavelet analysis, one can reduce both the noise and incomplete information in the profile data, thereby leaving the "correct information and result" for interpretation and further input into pavement performance models.

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