

# Issues in Pavement Smoothness:

## A Summary Report

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**Submitted by:**

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

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# ISSUES IN PAVEMENT SMOOTHNESS: A SUMMARY REPORT

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## **ABSTRACT**

Pavement smoothness has been recognized as one of the measures of pavement performance. Several Long-Term Pavement Performance (LTPP) studies have addressed certain aspects of pavement smoothness, including roughness development, measurement methods, and modeling. To advance the state of practice and knowledge of pavement smoothness, a workshop on pavement smoothness was held from August 26-28, 2001, in Irvine, California as part of NCHRP Project 20-51(01). Participants included individuals from state highway agencies, the Federal Highway Administration (FHWA), asphalt concrete and portland cement concrete paving industries, academia, consulting firms, and research organizations.

The workshop began with a series of presentations that covered a variety of topics related to pavement smoothness. These topics included findings from LTPP data analysis studies, equipment used for measuring smoothness, findings from a FHWA survey of state practices, contractors' perspective on pavement smoothness, and state highway agencies perspective on issues related to pavement smoothness. After the presentations, workshop participants reviewed in facilitated group discussions, the subjects of equipment and measurements, data analysis, and specifications and use. Through these discussions and a consensus-building process, workshop participants identified and prioritized the primary issues related to pavement smoothness that require concerted efforts for advancement. The top nine issues (in order of priority) were:

- Accuracy and repeatability of equipment;
- Reproducibility of equipment;
- Use of profile data for corrective actions;
- Knowledge and understanding of equipment and measurements;
- Relating smoothness to cost and performance;
- Identifying an appropriate index for smoothness;
- Standard guide specification;
- Future use of profile data; and
- Use of roughness index for monitoring pavement performance during service life.

Workshop participants then recommended strategies to address each of the nine issues. These strategies require an extensive effort that involves research, training, specialized development, and demonstration activities to improve use of profile/smoothness information. Workshop participants also identified groups within the private and public sectors that could play an active role in implementing these strategies. The information provided in this document should serve as a guide to those concerned with pavement smoothness in identifying, sponsoring, or pursuing parts of this extensive effort and thus help achieve the expected benefits from such measurements.

## INTRODUCTION

Pavement smoothness has been recognized as one of the measures of pavement performance. Several Long-Term Pavement Performance (LTPP) studies have addressed certain aspects of pavement smoothness, including roughness development, measurement methods, and modeling. To advance the state of practice and knowledge of pavement smoothness, there is a need to provide to review the most recent information on this subject, identify issues of concern, and recommend strategies for addressing these concerns. A workshop on pavement smoothness was convened as part of NCHRP Project 20-51(01) to accomplish this goal.

The workshop was held on August 26-28, 2001, at the National Academies' Arnold and Mabel Beckman Center in Irvine, California. Participants included individuals from state highway agencies, the Federal Highway Administration (FHWA), asphalt concrete and portland cement concrete paving industries, academia, consulting firms, and research organizations; a list of participants is provided in Appendix A.

The workshop included a series of presentations and facilitated group discussions; workshop program is provided in Appendix B. To facilitate the discussions, workshop participants were provided several weeks prior to the workshop with a report titled, "Pavement Smoothness Measurement and Analysis: State of the Knowledge." The report is provided in Appendix C. It covers several related topics, including user perception of ride quality, benefits of smooth pavements, equipment for smoothness measurement, profile indices, operational characteristics of profilographs, factors affecting measurements, specifications, application of smoothness data, and findings from related studies.

The workshop began with presentations that covered a variety of topics related to pavement smoothness, including findings from LTPP data analysis studies, equipment used for measuring smoothness, FHWA survey of state practices, contractors' perspective, and state departments of transportation interests. After the presentations, workshop participants reviewed in facilitated group discussions the subjects of equipment and measurements, data analysis, and specifications and use. Through these discussions and a consensus-building process, workshop participants identified and prioritized the primary issues related to pavement smoothness that require concerted efforts for advancement. Through further review and discussions, workshop participants then recommended strategies to address each of the nine issues, and identified groups within the private and public sectors that could play an active role in implementing these strategies.

The report provides a summary of the presentations and workshop findings. It includes discussions of the identified issues related to pavement smoothness and the strategies recommended for addressing them, and a listing of the groups that are expected to play a major role in implementing these strategies.

## **SUMMARY OF PRESENTATIONS**

As part of the workshop, several presentations were made to address topics related to pavement smoothness. A brief summary of each of these presentations follows.

### **Introduction and Welcoming Remarks**

#### **Eric E. Harm, Illinois Department of Transportation**

This presentation reviewed the purpose of the workshop and highlighted the following key points.

- One of the objectives of the workshop is to advance the state of knowledge by reviewing the findings from LTPP and other data analysis studies and discussing the issues related to the measurement and use of smoothness data.
- State and contractor perspectives on current practices on initial smoothness measurements should be discussed to identify issues of concern.
- Major issues related to pavement smoothness should be reviewed to identify and prioritize those of need for advancement and to recommend strategies for addressing them. The goal should be to advance the state of practice of pavement smoothness.
- Relevant issues should be identified with consideration to the shortcomings of the state of practice and should not necessarily be limited to those requiring a specific research effort.

### **NCHRP Studies Related to LTPP**

#### **Amir N. Hanna, NCHRP/Transportation Research Board**

A background on National Cooperative Highway Research Program (NCHRP) was presented. The NCHRP was started in 1962 by AASHTO and is supported by state highway departments. The NCHRP is currently sponsoring several projects on the analysis of LTPP data addressing several aspects of pavement performance. One such projects is NCHRP Project 20-50(08/13), *LTPP Data Analysis: Factors Affecting Pavement Smoothness*. Also, this workshop is being conducted as part of and NCHRP Project 20-51(01), *LTPP Project Development: Workshop on Pavement Smoothness*. These two projects focus on pavement smoothness issues; some of the other projects consider pavement smoothness among the factors being studied.

### **LTPP Role in Promoting Smoother Pavements**

#### **Mark Swanlund, Federal Highway Administration**

FHWA's goal is to have the roughness (measured by the International Roughness Index, IRI) of less than 170 in./mile on 95 percent of the National Highway System (NHS) and less than 85 in./mile on 60 percent of the system by 2008. The state of Georgia has some of the smoothest roads in the United States. The approach used by the State of Georgia to achieve this level of pavement smoothness includes the repair of pavements before they reach a very poor condition, use of preventive maintenance, and adoption of strict smoothness specifications for new construction with no incentives. The following key points were also made:

- FHWA has conducted demonstrations of several lightweight profilers in various states.
- FHWA has developed a “Best Construction Practice” video that describes procedure for constructing smooth pavements.
- The FHWA expert task group on pavement smoothness is currently developing guide specifications for inertial profilers.
- Lightweight profilers are gaining wide acceptance in the asphalt industry, primarily due to the length of pavement that can be constructed in one day. Lightweight profiler can collect smoothness data in less time than required for profilographs. Some contractors are now interested in purchasing high speed vans because of the shorter travel time, and the shorter profiling time when compared to lightweight profilers.
- FHWA has awarded a contract to develop a profiler viewer software that can be used to view profiles collected by different profiling equipment.

### **LTPP Data Collection for Smoothness Measurement**

#### **Larry Wisner, Federal Highway Administration**

In the LTPP program, profile data are collected for the General Pavement Studies (GPS) and Specific Pavement Studies (SPS) sections, including the Seasonal Monitoring Program sections, using four profilers. K.J. Law Model DNC690 profilers, equipped with optical sensors that recorded data at 6-inch intervals, were used from the start of the LTPP program until late 1996. New K.J. Law T-6600 profilers, equipped with infrared sensors that record data at 25-mm intervals, were purchased in 1996. The LTPP Manual for Profile Measurements describes the procedures for equipment calibration, daily checks, and for data collection. The collected profile data are processed using the Proqual software. The profiler height sensors, accelerometers, and distance measuring system are calibrated monthly, whenever problems are suspected, or when major vehicle or equipment repairs are performed. A bounce test and a height sensor check is performed prior to data collection each day to determine if the equipment is functioning properly. Because data from five error free runs are required at a test section, the Profscan program is used in the field to determine if five repeat profile runs that satisfy specified criteria were collected. If profile runs meeting the specified criteria were not collected, additional runs (up to nine runs) are performed. Other quality control checks of data include evaluating profile repeatability by comparing IRI and profile elevation data with those obtained at an earlier visit. Profile data are subjected to further quality control checks prior to uploading into the LTPP database. Roughness indices [e.g., IRI, Root Mean Square Vertical Acceleration (RMSVA), and Slope Variance] computed from the profile data are also uploaded into the LTPP database. A comparison between the four profilers used in the LTPP program is conducted annually to ensure accurate data collection. In this comparison, several test sections are profiled and analyses are performed to evaluate accuracy of the distance measurement system, compare IRI and profiles obtained by the four profilers, and compare IRI values obtained from profilers to that IRI value obtained from a reference measurement.

## **Findings from NCHRP Data Analysis for General Pavement Studies**

### **Starr D. Kohn, Soil and Materials Engineers**

The findings of NCHRP Project 20-50(08/13), *Factors Affecting Pavement Smoothness*, related to the General Pavement Studies (GPS) were presented. In this project, data available in the LTPP database were used to determine the effect of factors such as design and rehabilitation parameters, climatic conditions, traffic levels, material properties, and extent and severity of distress on changes in pavement smoothness. Data from GPS Experiments 1 through 7 were analyzed to determine the rate of change of roughness and IRI trends, relationships between IRI and the parameters that affect roughness, and to develop models to predict roughness. Parameters selected for evaluation were pavement age, traffic level, pavement thickness, structural number, AC properties (e.g., air voids, bulk specific gravity, and AC content), environmental parameters (e.g., wet days, mean temperature, annual days above 32°C, annual days below 0° C, freeze index, and freeze thaw cycles), base properties (e.g., moisture content and percent material passing No. 200 sieve), subgrade properties (e.g., plasticity index, moisture content, silt content, clay content, and percent material passing the No. 200 sieve). For each GPS experiment, relationships between IRI and these parameters were evaluated for all sections in each environmental zone (i.e., wet-freeze, wet no-freeze, dry freeze, dry no-freeze).

The GPS-1 experiment deals with the performance of AC pavements on granular base. The effect of subgrade type on performance was evaluated with consideration to the percent material passing the No. 200 sieve in three ranges (i.e., less than 20 percent, between 20 and 50 percent, and greater than 50 percent). For each range, IRI trends were generally different for the different environmental zones. However, when considering the entire data set, material in base passing No. 200 sieve, freezing index, and plasticity index (PI) of subgrade were found to have a strong effect on roughness; higher values resulted in a higher roughness values. In the wet no-freeze zone, higher IRI values were associated with higher values of days above 32°C, PI of subgrade, moisture content of subgrade, fines content in subgrade, and fines content in base. In the wet freeze zone, higher IRI values were associated with higher values of freezing index, fines content in base, annual precipitation, and silt content in coarse-grained subgrade.

The GPS-2 experiment deals with the performance of AC pavements on asphalt and cement stabilized bases. Asphalt stabilized base types include hot mix AC, AC treated mixtures, sand asphalt, and cold-laid mixtures. Cement stabilized bases include cement aggregate mixtures, soil cement, and lean concrete. Relationships between IRI and evaluated parameters were reviewed for the entire data set and for each base type. Very few relationships could be observed, probably because of the variety of stabilization types. When all sections were considered, an indication of higher IRI values was observed for higher air voids. For cement-stabilized bases, higher IRI values were observed for higher number of days above 32° C.

The GPS-3 experiment deals with the performance of jointed plain concrete pavements (JPCP), either doweled or non-doweled. Because few doweled sections were located in the dry zone (dry-freeze and dry no-freeze), an evaluation of the effects of dowels on roughness in this zone could not be made. However, in the wet-freeze zone the change in IRI over the monitored period (average of seven years) was of less than 0.1 m/km for 60 percent of sections with dowels and 18 percent of sections without dowels. Also, a change in IRI of over 0.5 m/km was observed for 6 percent of all sections and 36 percent of sections without dowels; a similar observation was



noted in the wet no-freeze zone. Higher values of PCC elastic modulus, annual precipitation, faulting, moisture content of subgrade, clay content of subgrade, and PI of subgrade, and lower values of mean temperature have contributed to an increase in roughness of non-doweled pavements. For doweled pavements, higher IRI values were associated with higher values of wet days, freezing index, and pavement age.

The GPS-4 experiment deals with the performance of jointed reinforced concrete pavements (JRCP). All test sections are located in the wet-freeze and wet no-freeze zones. Higher IRI values were associated with higher values of moisture, clay content, and PI of the subgrade; annual precipitation; mean temperature; number of wet days; slab thickness; joint spacing; and PCC modulus and Poisson's ratio.

The GPS-5 experiment deals with the performance of continuously reinforced concrete pavements (CRCP). Most of the GPS-5 sections showed little change in IRI over the monitored period (average of seven years); a change in IRI of less than 0.1 m/km was recorded for 64 percent and 75 percent of the sections in the wet-freeze and wet no-freeze zones, respectively. Higher IRI values were associated with higher values of PCC modulus, mean annual temperature, and fine material in subgrade, and lower values of water cement ratio of PCC mix, and steel content.

The GPS-6 experiment deals with the performance of AC overlays of AC pavements and includes GPS-6A and GPS-6B; pavement condition prior to overlay is available for GPS-6B sections but not for GPS-6A sections. No relationship between IRI before and immediately after overlay was observed. Thin overlays have shown to reduce roughness by a large amount. Factors that were found to contribute to an increased IRI of the overlaid pavements were higher values of IRI prior to overlay, annual days per year  $< 0^{\circ}$  C, moisture content and PI of subgrade, and fines content in subgrade and lower values of structural number and AC bulk specific gravity.

The GPS-7 experiment deals with the performance of AC overlays of PCC pavements (JPC, JRCP or CRCP) and includes GPS-7A and GPS-7B; pavement condition prior to overlay is available for GPS-7B sections but not for GPS-7A sections. No relationship between IRI before and immediately after overlay was observed. The IRI values obtained for all sections after overlay fell within a relatively narrow band. The factors affecting the roughness development of the specific PCC pavement type (i.e., JPC, JRCP or CRCP) are expected to influence the roughness progression of the overlaid sections. A general trend of higher IRI values was observed for higher values of PCC modulus.

Longitudinal data analysis methods were used to develop models to predict roughness for each of the GPS experiments. This analysis method differs from the traditional regression analysis methods; it takes into account the time-sequence nature of the data at the test sections.

### **Findings from NCHRP Data Analysis for Specific Pavement Studies** **Rohan W. Perera, Soil and Materials Engineers**

The findings of the recently completed NCHRP Project 20-50(08/13) related to Specific Pavement Studies (SPS) were presented. In this project, data available in the LTPP database for

SPS-1 (Flexible Pavements), SPS-2 (Rigid Pavements), SPS-5 (Rehabilitation of AC Pavements), and SPS-6 (Rehabilitation of PCC Pavements) experiments were used to determine the effect of certain factors on pavement smoothness.

In the SPS-1 experiment, 12 test sections were constructed at each project location. The pavement factors studied in this experiment were AC thickness (100 and 175 mm) and base type [aggregate base (AB), asphalt treated base (ATB), permeable asphalt treated base (PATB) over AB, and ATB over PATB]. Profile data were available for 16 SPS-1 projects; most of which were relatively new (less than 3 years: 10 projects, 3 to 5 years: 2 projects, and greater than 5 years: 4 projects). The projects were generally profiled within one year after construction; the IRI obtained at this time was referred to as the early-age IRI. The average early-age IRI for the 100 mm and 175 mm AC pavements were 0.88 and 0.82 m/km, respectively with standard deviations of 0.21 and 0.18 m/km for the 100 mm and 175 mm AC pavements, respectively. An IRI of less than 1.0 m/km was obtained for 70 percent of the 100 mm AC sections and 85 percent of the 175 mm AC sections. The average early-age IRI value obtained for AC pavements placed on different base types were 0.94, 0.82, and 0.84 m/km for the AB, ATB, and PATB bases, respectively. Most sections of the projects in Iowa, Kansas, and Ohio showed an increase in IRI of over 20 percent. Material test data were not available to investigate the cause of roughness increase at these projects. The increase in roughness for the Iowa project appears to be related to transverse cracking and longitudinal cracking in the wheel paths.

In the SPS-2 experiment, 12 test sections were constructed at each project location. All test sections were jointed PCC with dowels and a joint spacing of 4.6 m. The factors studied in this experiment were PCC thickness (200 and 275 mm), base type [AB, lean concrete base (LCB), and PATB over AB], concrete flexural strength (3.8 and 6.2 Mpa), and lane width (3.66 and 4.27 m). Profile data were available for 12 SPS-2 projects, most of which were relatively new (less than 3 years: 4 projects, 3 to 5 years: 5 projects, and greater than 5 years: 3 projects). The projects were generally profiled within one year after construction; the IRI obtained at this time was referred to as the early-age IRI. The average early-age IRI for the 200 mm and 275 mm PCC pavements were 1.27 and 1.30 m/km, respectively, with standard deviations of 0.28 and 0.30 m/km for the 200 mm and 275 mm PCC pavements, respectively. The average early-age IRI values obtained for PCC pavements placed on different base types were 1.27, 1.40, and 1.25 m/km for the AB, LCB, and PATB bases, respectively. Over the monitored period, 23 percent of the sections with 200 mm PCC thickness and 9 percent of the sections with 275 mm PCC thickness showed an increase in IRI of over 20 percent. The projects in Nevada showed the largest increase in IRI; 9 sections showed an increase of over 20 percent of which 5 sections showed an increase of over 40 percent. This increase in roughness occurred within 2 years and is attributed to curling of the slabs. An analysis of profile data indicated a change in curvature over time for the sections that showed an increase in IRI of greater than 20 percent.

The SPS-5 experiment deals with the performance of rehabilitated AC pavements. Each SPS-5 project consists of eight test sections, each of which is rehabilitated with an AC overlay. The factors studied in this experiment were overlay thickness (50 and 125 mm), AC type (virgin and recycled), and surface preparation prior to overlay placement (minimum and intensive). The minimum surface preparation consisted of patching distressed areas of the pavement while the intensive surface preparation included milling the existing AC surface (38 mm) and patching distressed areas. Profile data were available for 17 SPS-5 projects. The IRI after overlay was less

than 1.0 m/km for 80 percent of the sections that had an IRI of less than 1.5 m/km prior to overlay. The IRI after overlay ranged from 0.8 to 1.2 m/km for most sections that had an IRI of more than 1.5 m/km prior to overlay. A 50 mm overlay was shown to reduce the IRI of the pavement in some cases by 2.5 to 1.0 m/km. An analysis of all data available for the SPS-5 projects indicated that the IRI after overlay placement did not depend on the IRI before the rehabilitation, overlay thickness, milling prior to overlay, or AC type. An analysis of the data from the projects that had IRI values greater than 1.5 m/km indicated that milling prior to overlay placement results in a smoother pavement with an IRI value on the average 0.07 m/km less than that for a non-milled section. Generally, for each SPS-5 project, the IRI of all test sections in the project fell within a relatively narrow band irrespective of the IRI prior to overlay of the test sections. A statistical analysis indicated that the progression of roughness over time of the overlaid pavements depended on the pre-overlay IRI of the section and overlay thickness. When all projects were considered, the average rates of increase of roughness were 0.042, 0.050, 0.025, and 0.028 m/km/year for the 50 mm overlay with milling prior to overlay, 50 mm overlay without milling prior to overlay, 125 mm overlay with milling prior to overlay, and 125 mm overlay without milling prior to overlay, respectively.

The SPS-6 experiment deals with the performance of rehabilitated jointed concrete pavements. The rehabilitation treatment studies in this experiment were minimum restoration (joint sealing, crack sealing, partial depth and full depth patching), minimum restoration and a 100 mm AC overlay, minimum restoration and a 100 mm AC overlay (with sawed and sealed joints), intensive restoration (including diamond grinding) without an overlay, intensive restoration and a 100 mm AC overlay, crack/break and seat and a 100 mm AC surface, crack/break and seat and a 200 mm AC surface. The average rates of increase of IRI for the different treatment types were 0.058 m/km/year for minimum restoration and 100 mm overlay, 0.057 m/km/year for minimum restoration and 100 mm overlay with sawed and sealed joints, 0.200 m/km/year for intensive restoration with diamond grinding, 0.054 m/km/year for intensive restoration with a 100 mm overlay, 0.032 m/km/year for crack/break seat with a 100 mm AC surface, and 0.013 m/km/year for crack/break seat with a 200 mm AC surface. The rate of increase of IRI for the diamond ground sections was statistically different from that for the other sections; it was generally higher for sections that had higher IRI values prior to rehabilitation.

### **Findings from Other LTPP Data Analyses** **Harold L. Von Quintus, Fugro - BRE, Inc.**

The findings from the analysis of LTPP data related to smoothness performed as part of NCHRP Project 1-37A, *Development of 2002 Design Guide for the Design of New and Rehabilitated Pavement Structures*, and other studies (e.g., *Characteristics of Good and Poorly Performing Pavements*, *Effect of Rehabilitation on Pavement Performance*, *Evaluation of SPS-1 Experiment*, and *Evaluation of SPS-5 Experiment*) were presented.

In NCHRP Project 1-37A, IRI is used as a measure of pavement performance (i.e., an incremental increase in distress causes an incremental increase in IRI). A generalized model to predict IRI was developed. The model considers the initial IRI and the changes in IRI due to distress, frost heave of the subgrade, and shrink-swell of the subgrade. Different models were developed for AC pavements with unbound, asphalt-treated, and cement-treated bases; AC

pavements with AC overlays, and PCC pavements with AC overlays. In the models, most of the change in IRI was attributed to changes in surface distress; transverse cracks had a detrimental effect on IRI for all AC pavement types.

In terms of IRI, most AC surfaced GPS sections have shown good performance characteristics. The top ten factors that were identified as having a major influence on roughness development were traffic, asphalt viscosity, annual days with temperature greater than 32 °C, AC thickness, base thickness, freeze index, material in subgrade less than 0.075 mm, air voids in AC, base compaction, annual precipitation, daily temperature range, and freeze thaw cycles. Higher values of AC thickness, base thickness, and days with temperature greater than 32°C resulted in lower values of IRI, while higher values of other parameters resulted in higher IRI values.

In terms of IRI, SPS-1 sections with ATB/PATB and DGAB bases have shown the best and worst performance, respectively. The sections with ATB and PATB/DGAB bases have shown the second and third best performance, respectively. The sections with a ATB base layer were built smoother and exhibited a lower rate of roughness development over time than the other sections. Sections with an aggregate base and a drainage layer exhibited lower IRI values over time than the sections with an aggregate base but without a drainage layer.

Analysis of data for SPS-5 projects indicated that sections with greater amounts of distress were rougher. Resurfacing with thin (50 mm) or thick (125 mm) overlays substantially reduced the IRI of the pavement. Milling of the pavement prior to overlay placement had no significant effect on the IRI that was obtained immediately after overlay placement. The condition of the existing pavement had little to no effect on the IRI that was obtained immediately after overlay placement, or on roughness development over time. While there is a benefit of using thicker overlays on ride quality over time, milling of the surface prior to placing an overlay has only a slight effect on ride quality over time. The sections with virgin and recycled AC mixes have shown similar performance.

## **Equipment Types and Applications**

**Steven M. Karamihas, University of Michigan.**

An overview of the different equipment types that have been used in the past and those currently being used to measure pavement smoothness was presented. This equipment included straightedge, profilograph, response type profilers, reference profilers, and high-speed inertial profilers.

The deficiencies in using straightedges for measuring smoothness (e.g., missing a recurring wavelength) were described. Profilographs have varying response to the wavelengths present on the roadway; some wavelengths are measured correctly, others are amplified, and others are attenuated. Because of the operational characteristics of the profilograph, certain wavelengths that affect ride quality can be totally missed resulting in an acceptable profile index for a pavement that provides a poor ride quality.

Maysmeters are response type devices that were widely used to measure pavement roughness from 1960s to 1980s. The Maysmeter measures the vehicle's suspension motion by recording the

relative movement between axle and body. The roughness measurements obtained by Maysmeters are influenced by the characteristics of the mechanical system and the speed of travel; they are not transportable and cannot be compared between different units. For these reasons, response type devices are not currently used.

Reference profilers, such as rod and level and Dipstick, are used to obtain a reference measurement at a test section to judge the performance of other profile measuring devices.

Inertial profilers are widely used to measure pavement profiles. The three main components of a profiler are the height sensors, accelerometers, and distance measuring system. Inertial profilers should be capable of accurately measuring the wavelengths present on the roadway without amplification or attenuation. The measurements obtained from inertial profilers could provide a basis for smoothness specifications for new and rehabilitated pavements, and for providing a roughness index for monitoring pavement smoothness during service life.

Two of the ride quality indices that are currently being used are the International Roughness Index (IRI) and the Ride Number (RN). These two ride quality indices are influenced by different wavelengths. The IRI is influenced by wavelengths in the 1.2 to 30.5 m range, with the gain function of IRI being different for the different wavelengths. The IRI has maximum sensitivity to wavelengths of 2.4 m and 15.4 m; the maximum sensitivity of the RN occurs at a wavelength of 6.1 m.

### **FHWA Survey of State Practices** **David B. Law, Federal Highway Administration**

The results of a survey conducted by FHWA to identify state DOT practices related to smoothness measurements were presented. The following is a summary of the results (number of states responding to each question is provided in parenthesis).

*Equipment Used For New AC Pavements:* Profilograph (24), Rolling Straightedge (5), Straightedge (7), Profiler (16), Mays Meter (3), Lightweight Profiler (3), Rolling Dipstick (1), Hearne Straightedge (1).

*Equipment Used for AC Overlays:* Profilograph (14), Rolling Straightedge (4), Straightedge (6), Profiler (12), Mays Meter (3), Lightweight Profiler (3), Rolling Dipstick (1), Hearne Straightedge (1).

*Equipment Used for New PCC Pavements:* Profilograph (39), Rolling Straightedge (1), Straightedge (5), Profiler (7), Lightweight Profiler (3), Rolling Dipstick (1).

*Equipment used for Concrete Rehabilitation (grinding, with and without full depth patching):* Profilograph (19), Rolling Straightedge (4), Straightedge (3), Profiler (7), Lightweight Profiler (3).

*Providers of Equipment:* Contractor (21), DOT (6), Both (1), Either (1).

*Unit of Measurement for New HMA and HMA Rehabilitation:* Profile Index (16), IRI (4), Straightedge Variability (6), Other (6).

*Unit of Measurement for New PCC:* Profile Index (25), IRI (1), Straightedge Variability (3), Other (2).

*Unit of Measurement for PCC Rehabilitation (grinding, with and without full depth patching):* Profile Index (21), IRI (1), Straightedge Variability (3), Other (3).

*Width of Blanking Band:* 0 inch (6), 0.1 inch (10), 0.2 inch (27), 0.3 inch (1).

*Pay Adjustment Factors for New HMA:* Penalty Only No Bonus (1), Bonus Only No Penalty (2), Both Bonus and Penalty (22), Neither Bonus Nor Penalty (2).

*Adjustment Factors for HMA Overlays:* Penalty Only No Bonus (1), Bonus Only No Penalty (2), Both Bonus and Penalty (21), Neither Bonus Nor Penalty (2).

*Pay Adjustment Factors for New PCC:* Penalty Only No Bonus (3), Bonus Only No Penalty (4), Both Bonus and Penalty (17), Neither Bonus Nor Penalty (1).

*Pay Adjustment Factors for PCC Rehabilitation:* penalty only no bonus (2), bonus only no penalty (3), Both bonus and penalty (8), neither bonus nor penalty (2).

## **Issues in Pavement Smoothness: Contractor's Perspective**

### **Gary Fick, Duit Construction**

This presentation focused on the contractor's perspective related to smoothness issues of PCC pavements. There are a variety of design considerations that can influence pavement smoothness including base and subbase considerations (e.g., pad lane width and stability), pavement obstructions (e.g., manholes and inlets), pavement details (e.g., dowels and expansion joints), horizontal alignment (e.g., superelevations and transitions), vertical alignment, traffic control (e.g. consistent supply of concrete to the paving operation), and string line maintenance. There are also differences among states on pavement smoothness specifications for blanking band (0, 0.1, and 0.2 inches), bump height (0.3, 0.4, 0.5, and 0.6 inches), number of traces required, locations and averaging, correction procedures, exclusions and special conditions, and incentives. Because of these differences, from a contractor's perspective there is a need for standardizing both measurement and analysis procedures. Variability between equipment is also a concern that needs to be addressed as was demonstrated in a study conducted on a test section using 24 profilographs. The profile index obtained from the profilographs ranged from 5.40 to 8.00 in./mile, with an average value of 6.48 in./mile and a standard deviation of 0.71 in./mile; the value obtained by the "standard" profilograph was 6.22 in/mile. The following remarks were also made:

- Incentives do not increase the total cost of the project.
- Incentives encourage quality construction practices.

- Contractors should be able to reasonably assess risk and potential incentives associated with a smoothness specification.
- Specifications do not recognize the imprecision of the profilographs; pay factors should not be stepped but should be “smoothened out” to avoid potential conflicts (e.g., whether a value is 2.99 in./mile or 3.01 in./mile).

### **Issues in Pavement Smoothness: Contractor’s Perspective** **Gary Fick, Duit Construction**

This presentation focused on contractor’s perspective related to smoothness issues of AC pavements. Incorporating incentives in the contracting process provides the benefits of defining the minimum quality, allowing private sector to be innovative, stimulating innovations as bonus specifications allow contractors to risk money on new methods and equipment, and providing successful contractors a means for recovering costs of innovation. Incorporating disincentives without incentives contribute to adverse features such as the owner’s payment for perceived risk at bid time, apparent difference in contractor’s interests, owner’s possible overlooking subtle quality defects, lack of incentive on part of contractor to exceed owner’s expectations. Quality does not have to cost more because high quality contractors succeed through market competition. It was stressed that improvements in smoothness can be more costly to achieve if higher levels of smoothness are specified.

The Marana I-10 project, constructed in 1994, was described as a case study. In this project, the contractor earned a bonus of \$18,000 but the cost for of equipment modification was also \$18,000. In earlier years, the bonus was not sufficient to compensate for the risks taken by the contractor; the specifications were modified later to provide more incentive dollars.

The key factor to building a smooth road is ensuring continuous uninterrupted processes. Consistency is required in production, quality, heat of mix, and personnel. During paving, it is necessary to keep the paver moving, avoid bumping the paver or allowing a truck to touch the paver, sensing off the smoothest part of the pavement (reference the new mat or the adjacent mat), averaging the bumps out over the longest possible length, avoiding transfer of adjacent bumps into new mat by rollers straddling it, and avoiding roller stopping on the mat.

The personnel involved in the project are the key element in achieving a smooth pavement. It is important to provide sufficient education to ensure that everyone understands the specification also and to share the bonuses with all the workers involved in the project.

### **Issues in Pavement Smoothness: States’ Perspective** **Kenneth W. Fults, Texas Department of Transportation**

A chronological evolution of smoothness specifications in Texas was presented. Texas DOT currently uses a zero blanking band and manufactures high-speed profilers for states’ use. The Texas DOT has implemented a certification procedure for profilers; two test sections (smooth and a medium smooth) have been established for evaluating profilers. For the certification process, profilers have to collect data on these two test sections and submit the data to the Texas

DOT for evaluation against a set of standard criteria. To date two devices have taken part in the certification process; one of which has passed.

Among the key issues that have to be addressed are equipment standardization to provide “apples to apples” comparison between devices, developing certification procedures based on the true profile and not based on an index value such as IRI, training of transportation department personnel and contractors, correlating smoothness level to pavement performance, and establishing bonus/penalty based on sound research.

The FHWA has developed a set of protocols for roughness, rut depth, and distress measurements. A meeting with equipment vendors was held at the Road Profiler User Group (RPUG) meeting in Auburn, Alabama in 2000 to discuss these protocols. The FHWA Expert Task Group (ETG) on pavement smoothness will address the issue of equipment specifications later this year.

### **Role of LTPP in Improving Current Practice** **Starr D. Kohn, Soil and Materials Engineers, Inc.**

The first gathering of LTPP profiler operators was held in 1989, to discuss data collection and calibration procedures. This meeting gave rise to Proqual software that was developed used for processing profile data, and to the LTPP Profile Manual that documents procedures for calibration, data collection, and field data quality review. The first comparison of the four LTPP profilers was held in 1991 in Ann Arbor, Michigan; the results were presented to the RPUG in 1992. This introduction helped the RPUG organize other profiler comparisons in 1993 and 1994. The results from these comparisons have shown that profilers with ultrasonic sensors do not collect accurate profile data; they produce substantially high roughness values for chip-sealed pavements. States have now replaced the ultrasonic sensors in profilers with laser sensors.

Analysis of profile data collected by LTPP profilers revealed problems due to spikes in the data, lost signals, and incorrect profiling locations. Improved field quality control procedures were developed to address these problems; profiler owners can use this information to develop specific quality control procedures. The current LTPP Profiler Manual describes procedures for equipment calibration and daily checks and for data quality control; profiler owners can use this manual use as a guide for developing manuals for specific profile operations. A direct comparison of the output from a state’s profiler to the output from an LTPP profiler can be made by profiling LTPP test sections in this state using the state’s profiler at the time when the sections are profiled by the LTPP profiler.

### **State’s Approach to Pavement Smoothness** **J. Patrick Gardiner, Pennsylvania Department of Transportation.**

The Pennsylvania DOT approach to pavement smoothness was reviewed in this presentation. In 1995, Pennsylvania was identified as one of the states having the roughest pavements in the United States. To address this issue, PennDOT adopted an approach that involves determining the current performance level, identifying a desired goal for performance level, implementing strategies to achieve this goal, and periodically evaluating the achieved progress and introducing



improvements. A team composed of representatives from Penn DOT policy and field personnel, contractors' organizations, contractors, and equipment manufacturers was assigned the tasks of stressing the importance of smooth pavements, establishing policies and goals for specific classes of roads, proposing methods for determining current performance levels, developing and modifying strategies for achieving the established goals, and coordinating the implementation schedule. Implementation activities included developing a pilot program for a specific class of highways, developing contract language that balances risks, and defining measures of performance. This plan was evaluated periodically to determine if the expected level of performance was achieved, suggested policies were effective, and contract language was easy to understand and follow, and to review the costs associated with these improvements and to modify the program for future work.

Smoothness specifications for bituminous pavements were intended to improve smoothness on interstate highways to at least the national median. Specifications that use IRI as the smoothness parameter were developed and reviewed with the industry groups, and were initially used on interstates and expressways. In addition to addressing specifications deficiencies, other actions such as providing bonus with no penalties and using the data to determine a fair scale for penalties and corrective work were introduced. The industry used lightweight profilers to collect profile data and compute IRI, and PennDOT developed a certification process for equipment and operators. A periodic policy evaluation is being done; it includes an annual analysis of smoothness performance, cooperation with industry to improve and expand the program, and identification of steps for addressing smoothness of non-interstate roads and bridges. The most important issue currently facing the department relates to the use of profile data to determine locations requiring grinding.

The data for 1995 indicated that the national median value for smoothness was 100 in./mile and the median value for Pennsylvania was 113 in./mile. In 2000, the national median was 89 in./mile and the median value for PennDOT was 88 in./mile. These results clearly show that the policies adopted in Pennsylvania resulted in obvious improvements in smoothness levels.

## ISSUES IN PAVEMENT SMOOTHNESS

Workshop participants focused on identifying issues and knowledge gaps in the areas of equipment and measurement, data analysis, and specifications and use through facilitated group discussions. The participants were divided into three groups; each group reviewed these subject areas and identified the issues requiring further consideration. Identified issues were then compiled in a preliminary list that was then reviewed by all participants to ensure consistency and to eliminate duplication. As a result, the following 15 issues emerged as those requiring concerted efforts for advancement to enhance the understanding and use of smoothness.

Issues related to equipment and measurement:

- Lack of knowledge/understanding.
- Accuracy and repeatability of the equipment.
- Reproducibility of equipment.
- Characteristics of the ideal equipment.
- Use of roughness index to track pavement performance during service life.

Issues related to specification and use:

- Lack of knowledge and understanding.
- Identifying an appropriate index for smoothness.
- Relating smoothness to cost and performance.
- Lack of standard guide specification.
- Lack of implementation plan.
- Use of profile data for corrective actions.
- Acceptance procedures for contractor/vendor collected data.

Issues related to data analysis:

- Lack of knowledge and understanding.
- Future use of profile data.
- Lack of uniform analytical procedures (for HPMS).

Workshop participants then rated these 15 issues according to importance; the relative ranking shown in Table 1 was determined. The nine top ranked issues, in order of priority, were:

1. Accuracy and repeatability of equipment,
2. Reproducibility of equipment,
3. Use of profile data for corrective actions,
4. Lack of knowledge/understanding of equipment and measurement,
5. Relating smoothness to cost and performance,
6. Identifying an appropriate index for smoothness,
7. Lack of standard guide specification,
8. Future use of profile data, and
9. Use of roughness index to track pavement performance during service life.

Table 1. Issues in Pavement Smoothness

Ranking	Issue
1	Accuracy and repeatability of equipment
2	Reproducibility of equipment
3	Use profile data for corrective actions
4	Lack of knowledge/understanding (equipment and measurement)
5	Relating smoothness to cost and performance
6	Identifying an appropriate index for smoothness
7	Lack of standard guide specification
8	Future use of profile data
9	Use of roughness index to track pavement performance during service life
10	Lack of uniform analytical procedures (for HPMS)
11	Lack of knowledge and understanding (specifications and use)
12	Characteristics of ideal equipment
13	Acceptance procedures for contractor/vendor collected data
14	Lack of knowledge/understanding - data analysis
15	Lack of implementation plan (specifications and use)

Workshop participants also recommended strategies for addressing these issues and identified groups within the private and public sectors that could play an active role in implementing these strategies. Brief descriptions of these issues and recommended strategies together with listings of the proposed action groups are provided in this section.

### **Issue 1: Accuracy and Repeatability of Equipment**

Because of their lack of accuracy and repeatability, inertial profilers have not been widely adopted for construction smoothness measurement. However, these profilers could better serve the pavement community if accuracy and repeatability has been sufficiently demonstrated.

Mandatory certification of high-speed and lightweight profilers is perceived as necessary for ensuring equipment accuracy. This certification requires the development of procedures for testing and evaluation through comparisons to reference measurements or testing with simulated (known) inputs. Data from verification sections, when used, must be processed in a manner that directly evaluates a device's ability to measure specific roughness properties.

Standards for equipment "configuration" may also help evaluate the profilers' ability to provide accurate and repeatable measurements. Aspects of profiler design that should be considered for standardization include data processing algorithms, filter settings, sampling and recording frequency, and sensor performance. Because most aspects of profiler design and operation that affect accuracy and repeatability are well understood, it appears that standardization is only a matter of building consensus among involved parties. However, studies may be required to address some related factors such as the effect of daily and seasonal changes on profile repeatability, profiling at low speeds (when starting and stopping), and benefits of automated error checking.

The certification process could also be structured to cover equipment standards. In addition, operator certification should be considered to ensure that the operator is capable of maintaining proper equipment calibration, operating speed, lateral position, and maintenance.

### ***Recommended Strategies***

Several action items were identified to address the issue of equipment accuracy and repeatability.

- Develop standards for equipment.
  - Specify minimum resolution and hardware requirements (e.g., height sensor, accelerometers, and distance measuring system).
  - Specify sampling and recording interval of profile data.
  - Specify procedures for automated warning and detection of signal loss and out-of-range measurements by sensors.
  - Familiarize equipment manufacturers with developed standards.
- Develop Standard procedures for computation of profile data.
  - Specify algorithms for profile computation.

- Specify filters used in computation of profile (e.g., lead-in filter, anti-alias, and upper cut-off filters).
- Develop measurement standards for equipment.
  - Specify data collection procedures (e.g., number of profiler runs, sensor spacing, identification of wheel path, operating speed, and data recording interval).
  - Outline procedures for controlling transverse sensor location.
- Develop procedures for certification of equipment.
  - Investigate the use of testing equipment (e.g., shake table) for certification.
  - Establish national and regional test sites.
- Develop procedures for certification of operators.
- Develop procedures and guidelines for equipment calibration.
  - Develop procedures for calibrating height sensors, accelerometers and distance measuring system.
- Develop procedures and guidelines for daily checks of equipment to ensure proper functioning.
- Develop precision and bias statements for profile and smoothness indices (e.g., IRI and RN).
- Develop standard method for data analysis (e.g., IRI algorithm parameters/filters).
  - Determine implications of pre-processing of data on analysis results.
- Develop better means for dealing with location referencing.
- Conduct research on related topics
  - Develop a procedure to determine if measurements reflect the true profile.
  - Determine the effect of lateral wander and number of runs on profile data and computed smoothness indices.
  - Determine the effect of daily temperature variations on profile data and computed smoothness indices for consideration in specifications.
  - Investigate the sensitivity of profile and computed indices to driver and operational speed.
  - Investigate the effect of seasonal variations on profile measurements.
  - Determine the relationship between the number of height sensors and the understanding of pavement profile.
  - Develop procedures to update the repeatability and bias requirements provided in ASTM Standard E 950.
  - Determine the effect of sensor footprint on measured profile, profile data, and smoothness indices.

### ***Proposed Action Groups***

The Road Profiler User Group (RPUG), AASHTO (to deal with equipment standards), AMRL/ASTM (to deal with certification procedures), and FHWA Pavement Smoothness Expert Task Group (ETG) were identified as candidates for leading efforts to deal with the strategies recommended for addressing the issue of equipment accuracy and repeatability.

### **Issue 2: Reproducibility of Equipment**

When operated on the same pavement section, the different profiling devices are expected to provide the true profile and the same value of a smoothness statistic. However, the profiling devices currently available for network monitoring and construction control do not yield reproducible results. There is a need for an effort to ensure that these profiling devices produce the same (i.e., profile and computed smoothness indices) results when operated on the same pavement section.

### ***Recommended Strategies***

Several action items have been identified to address the issue of reproducibility of equipment.

- Develop an equipment standard.
  - Specify requirements for resolution and hardware (i.e., height sensor, accelerometer, and distance measuring system).
  - Familiarize equipment manufacturers with developed standards.
  - Specify procedures for automated warning and detection of signal loss and out-of-range measurements by sensors.
  - Specify an automated trigger for data collection.
  - Familiarize state highway agencies with the design and features of equipment used by other state agencies.
  - Specify automated executable functions to configure the system, load programs, and startup for regular operations.
  
- Develop standardized procedures for use of profile data.
  - Specify algorithms for profile computation.
  - Specify the filters used in computation of profile (e.g., lead-in, anti-alias, and upper cut-off filters).
  
- Develop operational standards for equipment.
  - Specify data collection procedures (e.g., number of profiler runs, sensor spacing, identification of wheel path, operating speed, and data recording interval).
  - Identify procedures for controlling transverse sensor location.
  - Identify procedures for automated marking of beginning and end of data collection to facilitate skipping specific segments (i.e., exclusion zones).

- Develop provisions to eliminate data discrepancies at the start and end of runs (i.e., minimum lead in, distance required after end of section for stopping, dealing with braking or reducing speed prior to end of section, and data to be excluded or deleted).
- Develop procedures and methods for certification of equipment.
  - Investigate the use of testing equipment (e.g., shake table) for certification.
  - Establish national and regional test sites.
- Develop procedures for certification of operators.
- Develop procedures and guidelines for equipment calibration.
  - Develop procedures for calibrating height sensors, accelerometers and distance measuring system.
- Develop better means for dealing with location referencing.
- Conduct research on related topics.
  - Develop procedure to determine if measurements reflect the true profile.
  - Determine the effects of sensor footprint on measured profile and computed smoothness indices.
  - Investigate the sensitivity of profile and computed indices to driver and operational speed.

### ***Proposed Action Groups***

ASTM, AASHTO, State DOT's, FHWA, equipment manufacturers, and RPUG were identified as candidates for leading efforts to deal with the strategies recommended for addressing the issue of reproducibility of equipment.

### **Issue 3: Use of Profile Data for Corrective Actions**

Procedures are not currently available for using profile data to reliably identify the locations of the defects and quantify the impact of these defects on the computed smoothness parameter. There is a need for developing a method that identifies defective areas in a pavement profile so that corrective actions can be taken. Also, an approach for identifying the specific corrective action to be taken need to be developed to facilitate implementation.

### ***Recommended Strategies***

Several action items have identified to address the issue of use of profile data for corrective actions.

- Develop procedures and methods (a research effort may be required).
  - Develop procedures for using profile data to identify in near real time the locations in need of corrective action.

- Develop procedures to model the effects of alternative corrective actions pavement smoothness.
  - Identify methodologies for evaluating alternative corrective actions.
  - Develop procedures for using profile data to identify anomalies in pavement surface.
- Develop training material to familiarize personnel with the procedures for identifying pavement anomalies and selecting corrective actions for implementation.

### ***Proposed Action Groups***

AASHTO Highway Subcommittee on Construction and AASHTO Joint Task Force on Pavements were identified as candidates for leading efforts to deal with the strategies recommended for addressing the issue of using profile data for corrective actions.

### **Issue 4: Knowledge and Understanding of Equipment and Measurements**

Accurate, reproducible, and transportable pavement profile data is needed for use in smoothness specifications for construction and for network condition assessment. With the transition currently occurring from the use of profilographs to inertial profilers for construction acceptance, there is a potential for inappropriate or incorrect collection and use of profile data. Increasing the knowledge of the personnel involved in the selection, operation, and maintenance of profiling equipment and those involved in the analysis of profile data should help reduce the potential for inappropriate use of data. Comprehensive education and training programs on related topics are not readily available; there is a need to develop such training material.

### ***Recommended Strategies***

Several action items have identified to address the issue of the knowledge and understanding of equipment and measurements.

- Develop an increased awareness of available resources (e.g., NCHRP research and LTPP Profiling Manual)
- Develop “Best Practices” guides.
- Develop a training course for data collection personnel.
- Establish a web-based forum for discussion of related issues.
- Conduct regional workshops on pavement smoothness.

### ***Proposed Action Groups***

FHWA (to develop best practices guide and conduct regional workshops), NHI (to develop training courses), and RPUG (to establish a web-based forum for discussion of related issues) were identified as candidates for leading efforts to deal with the strategies recommended for addressing the issue of knowledge and understanding of equipment and measurements.



## **Issue 5: Relating Smoothness to Cost and Performance**

There is insufficient data to quantify the relationship between initial smoothness and long-term pavement performance. While specifications often provide incentives for initial smoothness, the point at which the cost associated with constructing a smoother pavement exceeds the expected long-term benefits has not been established. To justify the increased costs associated with constructing very smooth pavements, the relationship between costs and benefits need to be quantified. How smooth is good enough from ride quality and pavement performance perspectives is unknown; there is a need to determine the point at which increasing pavement smoothness provides no additional long-term benefit. Also, models to predict the effect of increased initial smoothness on long-term pavement performance need to be developed. This information will contribute to improved pavement management and construction decisions and help achieve better ride quality, reduced user costs, and improved pavement performance.

### ***Recommended Strategies***

A research effort was considered necessary for addressing the issue of relating smoothness to cost and performance. Specifically, this research effort should encompass the following tasks:

- Quantify the relationship between initial smoothness and long-term pavement performance, establish relationship between cost of constructing smoother pavements and expected benefits, determine optimum smoothness levels, and refine the understanding of cost-effectiveness of different levels of initial smoothness.
- Investigate if smoothness indices other than IRI would better address the different uses of smoothness (e.g., in determining bonus/penalties, evaluating serviceability, and measuring customer satisfaction).
- Establish the relationship between vehicle dynamics and roughness indices.
- Determine the feasibility of using profile data to develop different roughness indices for the different functional classifications of roads (e.g., interstates and urban streets), and for adjusting the simulation speed of quarter car in IRI depending on the functional classification.
- Determine if future pavement smoothness can be reliably predicted using the current and historical data for the pavement.
- Determine if profile data can reliably be used for programming future construction and maintenance.
- Determine if pavements that are ground when they were new will maintain their smoothness over time.
- Investigate the rationale for current QA/QC acceptance practices for smoothness and recommend improvements.

### ***Proposed Action Groups***

NCHRP (to conduct of research), State DOTs and FHWA (to conduct pooled-fund projects), Innovative Pavement Research Foundation (IPRF), National Center for Asphalt Technology (NCAT), PIARC (World Road Association), and FHWA Expert Task Group on Smoothness

were identified as candidates for leading efforts to deal with the strategies recommended for addressing the issue of relating pavement smoothness to cost and performance.

### **Issue 6: Identifying an Appropriate Index for Smoothness**

A number of indices are currently available to describe pavement smoothness (e.g., IRI, PI, RN, and RQI). However, there is no consensus among pavement managers, construction personnel, and researchers that these indices accurately relate the effect pavement profile on driver comfort, cargo damage, and vehicle wear and tear for all vehicle types. There is a need to identify or develop a consensus index or indices for use in network and project level analyses.

#### ***Recommended Strategies***

Several action items have identified to address the issue of identifying the most appropriate smoothness index.

- Investigate vehicle response (e.g., dynamic force and acceleration of the body) to different profiles, and develop criteria for defining pavement smoothness based on vehicle dynamics.
- Investigate the relationship between existing smoothness indices and other indices that relate public perception of ride quality.
- Investigate the relationship between smoothness indices (e.g., PI, IRI, and RN) and pavement performance.
- Identify appropriate parameter (e.g., PI, IRI, RN, or RQI) for use in smoothness specifications.
- Develop statistical parameters (i.e., precision and bias statements) for different summary statistics (e.g. IRI, and PI).
- Develop relationships between profile index and smoothness parameters (e.g., IRI or RN) for use by states agencies when converting from profilographs to inertial profilers.
- Investigate the use of a moving average IRI as a means for defining roughness.
- Investigate the merits of an overall smoothness statistic for a pavement segment versus smoothness statistics based on a short interval (e.g., IRI for 0.1 mile segment versus IRI for shorter interval) for use in specifications.
- Investigate the effects on of variations of blanking band, variations of bump height, time of measurement, method of averaging, number of repeat runs, speed of operation, segment length, and other factors on smoothness indices.
- Investigate the relationship between smoothness indices and pay factors (i.e., step and continuous).
- Investigate the need for different smoothness specifications for different pavement types (e.g., AC and PCC), construction type (e.g., new and rehabilitated), pavement location (e.g., urban and rural), and road functional classification (e.g., interstate and local).

### ***Proposed Action Groups***

State DOTs, NCHRP, FHWA, and World Bank were identified as candidates for leading efforts to deal with the strategies recommended for addressing the issue of identifying appropriate smoothness indices.

### **Issue 7: Lack of Standard Guide Specification**

In addition to smoothness specifications for construction quality control, many guide specifications for profiling equipment are currently available from several equipment manufactures. Not all of these guide specifications are current and some of them do not even apply to the equipment being used today. There is a need to develop comprehensive guide specifications that cover equipment purchase, certification of operator and equipment, network operations (monitoring), and project operations (construction).

### ***Recommended Strategies***

Several action items related to equipment, operator, measurement, and construction were identified to address the issue of lack of guide specifications.

- Equipment
  - Develop guidelines for equipment purchase.
  - Develop procedures for equipment certification.
  - Develop guidelines for calibration of equipment.
  
- Operator
  - Develop certification procedures for operators.
  
- Measurement
  - Develop operational guidelines for network level profiling (e.g., operational speed, sampling rate, sensor spacing, wheel path identification, and reporting interval for smoothness parameter).
  - Develop operational guidelines for project level profiling (e.g., operational speed, sampling rate, sensor spacing, wheel path to be profiled, number of runs, and reporting interval for smoothness parameter.)
  - Develop specifics for measurements required for acceptance testing of new construction (e.g., number of runs, computation of smoothness parameter obtained by replicate runs, and reporting of smoothness parameter based on an average value only versus an average value and standard deviation).
  - Develop guidelines for controlling the transverse sensor location.
  - Investigate sensitivity of collected profile data and computed smoothness indices data to profiled path, driver, and operational speed.
  - Develop procedures for profiling a pavement section for construction acceptance that address related factors (e.g., lead-in distance required prior to test section, distance required beyond the end of section, and effect of braking or reducing speed at end of

section), and identify the data that should not be included in the computation of the smoothness parameter because of the non-ideal conditions at start or end of section.

- Construction
  - Develop guidelines on the limitations and applicability of specifications (e.g., exclusions, minimum length, and speed requirements).
  - Develop a national standard for construction acceptance.
  - Investigate suitability of providing an overall statistic for a pavement segment and another statistic for a short interval (e.g., an IRI value for 0.1 mile segment and another value for 30 ft intervals within the segment).
  - Develop procedures for dealing with short segments (e.g., shorter than 0.1 mile).
  - Investigate the effects of variations in blanking band, bump height, time of measurement, measuring one wheel path or both wheel paths, speed, summary reporting interval, and other related factors on smoothness parameters.
  - Address the issue of pay factors (e.g., continuous or stepped, and their advantages and disadvantages).
  - Investigate the need for different specifications for different pavement types (e.g., asphalt and concrete), location (e.g., rural and urban), functional classification of highway (e.g., interstate and local), and type of construction (e.g., new and rehabilitation).
  - Develop procedures for QC/QA of collected data.

### ***Proposed Action Groups***

AASHTO, ASTM, FHWA, State DOT's, and NCHRP were identified as candidates for leading efforts to deal with the strategies recommended for addressing the issue of standard guide specification.

### **Issue 8: Future Use of Profile Data**

Profile information has not yet been effectively or fully used to improve the highway system. There is a need to explore new and innovative ways for using profile data in pavement analysis, pavement management, and construction acceptance, and also to establish a clear relationship between pavement profile and pavement performance parameters. New technologies and analytical procedures were developed in the last 3 to 5 years but many of these applications have not been used or tested on a large scale. Applications that show promise should also be demonstrated and pursued for implementation.

### ***Recommended Strategies***

Several action items related to the interaction of pavement profile and vehicles, use of profile data for pavement management and forensic analysis, and specification development were identified to address future use of profile data.

- Interaction of pavement profile and vehicles
  - Develop profile indices based on vehicle response parameters for different vehicle types (e.g., cars, trucks, campers, and motorcycles) and for different speed limits.
  - Conduct research to establish the relationships between vehicle dynamics and various roughness indices.
  - Conduct research to develop analytical methods to identify dynamic loading problems from profile data.
  - Investigate vehicle response (e.g., dynamic force and acceleration of the body) to different profiles and develop smoothness criteria based on vehicle dynamic response.
  
- Use of profile data for pavement management
  - Investigate use of new analytical methods to identify surface distresses and features (e.g., faulting, joints, texture, and cracking) from profile data.
  - Investigate use of profile data for pavement management (e.g., different guidelines or standards for different functional classifications).
  - Investigate use of profile data for programming rehabilitation activities.
  - Identify or develop procedures for obtaining better information on rutting (and transverse profile) from profile data.
  
- Use of profile data for forensic analysis and pavement performance
  - Investigate the effect of slab curvature on pavement performance by using data obtained from profile measurements.
  - Investigate use of profile data to identify problems related to construction practices and construction equipment.
  
- Use of profile data to develop specifications
  - Develop specifications for use of profile data for bridges and railroad crossings.
  - Investigate use profile data to interpret other specifications (e.g., simulated straightedge versus IRI).

### ***Proposed Action Groups***

TRB LTPP Expert Task Group on Distress and Profile, FHWA, AASHTO, and States DOTs were identified as candidates for leading efforts to deal with the strategies recommended for addressing the issue of future use of profile data.

### **Issue 9: Using Smoothness Index to Monitor Pavement Performance During Service Life**

Monitoring pavement smoothness from initial construction through rehabilitation cycles should enhance understanding of pavement performance. One approach for assessing pavement performance involves the collection and evaluation of pavement profile data. However, different pavement profile indices are often used for initial and long-term evaluations (e.g., PI from profilograph for construction acceptance and IRI from inertial profiler for monitoring). The use of different indices makes it impossible to obtain a direct comparison between initial smoothness and subsequent performance. There is a need for establishing a consistent means for evaluating

pavement performance by identifying or developing a single index for characterizing pavement smoothness for both construction acceptance and monitoring. However, if a single index cannot be identified, initial smoothness should also be characterized using the index proposed for long-term assessment to provide a reference.

### ***Recommended Strategies***

Several strategies were recommended to address the issue of using smoothness index to monitor pavement performance during service life.

- Identify or develop an appropriate smoothness index for use over the pavement life.
- Develop specifications for profiling equipment.
- Develop standard procedures for data collection.
- Develop guidelines for uniform data storage and data access.

### ***Proposed Action Groups***

AASHTO Joint Task Force on Pavements and AASHTO Highway Subcommittee on Materials were identified as candidates for leading efforts to deal with the strategies recommended for addressing the issue of using smoothness index to monitor pavement performance during service life.

## CONCLUDING REMARKS

Through facilitated working groups and consensus-building process, about 40 pavement professionals from state highway agencies, the FHWA, academic institutions, consulting firms, and the paving industry identified and prioritized the primary issues related to pavement smoothness requiring a concerted effort for advancement. These issues pertain to equipment and measurement, data analysis, and specifications and use. The top-ranked issues were:

- Accuracy and repeatability of equipment;
- Reproducibility of equipment;
- Use of profile data for corrective actions;
- Knowledge and understanding of equipment and measurements;
- Relating smoothness to cost and performance;
- Identifying appropriate index for smoothness;
- Standard guide specification;
- Future use of profile data; and
- Use of smoothness index for monitoring pavement performance during service life.

Strategies for addressing these issues were recommended. These strategies require an extensive effort that involves research, training, specialized development, and demonstration activities to improve use of profile/smoothness information. Groups from the private and public sectors were proposed as candidates for leading efforts to pursue these strategies. The information provided in this document should serve as a guide to those concerned with pavement smoothness in identifying, sponsoring, or pursuing parts of this extensive effort and thus help achieve the expected benefits from such measurements.

## APPENDIX A - Workshop Participants

### NCHRP Project 20-51(01) *Workshop on Pavement Smoothness* (Irvine, California, August 26 - 28, 2001)

#### Participants:

Mr. Mark Belshe  
Vice President  
FNF Construction, Inc.

Dr. Christopher R. Byrum\*  
Project Engineer  
Soil and Materials Engineers, Inc.

Dr. James K. Cable  
Associate Professor  
Iowa State University

Mr. Jagjit S. (John) Dade  
Transportation Branch Manager  
Kentucky Transportation Cabinet

Dr. Tahar El-Korchi  
Professor of Civil Engineering  
Worcester Polytechnic Institute

Mr. Lynn D. Evans  
Senior Engineer  
ERES Consultants/ARA

Mr. Gary Fick  
Chief Estimator  
Duit Construction

Mr. Kenneth W. Fults  
Director of Pavements Section  
Texas Department of Transportation

Mr. J. Patrick Gardiner  
Chief, Quality Assurance  
Pennsylvania Department of Transportation

Mr. Max G. Grogg  
Program Director  
Applied Pavement Technology, Inc.

Mr. Eric E. Harm  
Engineer of Materials and Physical Research  
Illinois Department of transportation

Mr. Steven M. Karamihas  
Senior Research Associate  
University of Michigan  
Transportation Research Institute

Dr. Starr D. Kohn\*  
Vice President  
Soil and Materials Engineers, Inc.

Mr. David B. Law  
Technology/Systems Engineer  
Federal Highway Administration

Mr. Daris W. Ormesher  
Research Project Engineer  
South Dakota Department of Transportation

Dr. Tom Papagiannakis  
Associate Professor  
Washington State University

Mr. William H. Parcels, Jr.  
Pavement Surface Research Engineer  
Kansas Department of Transportation

Dr. Rohan W. Perera\*  
Project Engineer  
Soil and Materials Engineers, Inc.

Ms. Joy F. Portera  
State Research Engineer  
Mississippi Department of Transportation

Dr. Gonzalo R. Rada  
Assistant Vice President  
LAW PCS



**Participants (continued):**

Mr. Gary L. Robson  
Director of Paving Services  
American Concrete Pavement Association

Mr. Charles H. Van Deusen  
President  
Charles H. Van Deusen, Inc.

Mr. Brian L. Schleppe  
Researcher, Office of Pavement Engineering  
Ohio Department of Transportation

Mr. Harold L. Von Quintus  
Senior Research Engineer  
Fugro-BRE, Inc.

Mr. Mark Swanlund  
*Concrete Pavement Engineer*  
Federal Highway Administration

Mr. Jon S. Watson  
Pavement Engineer  
Montana Department of Transportation

Mr. Alton Treadway  
Bureau of Research and Development  
Alabama Department of Transportation

Mr. Larry Wisner  
Research Highway Engineer  
Federal Highway Administration

Mr. Peter Vacura  
Chief, Structural Section  
Design and Rehabilitation Branch  
Caltrans

Dr. Shie-Shin Wu  
Pavement Engineer  
North Carolina Department of Transportation

\* Member of the research team

---

**Facilitators:**

Ms. Carolyn D. Goodman  
Technology Transfer Director  
Virginia Transportation Research Council

Mr. Kevin K. McGhee  
Senior Research Scientist  
Virginia Transportation Research Council

Ms. Cathy C. McGhee  
Senior Research Scientist  
Virginia Transportation Research Council

Mr. Wallace T. McKeel, Jr.  
Research Manager  
Virginia Transportation Research Council

---

**NCHRP Responsible Staff Officer:**

Dr. Amir N. Hanna  
Senior Program Officer  
NCHRP/Transportation Research Board

## **APPENDIX B – Workshop Agenda**

**NCHRP Project 20-51(01)  
*Workshop on Pavement Smoothness*  
Sunday-Tuesday, August 26-28, 2001**

**Arnold and Mabel Beckman Center  
100 Academy Drive  
Irvine, CA 92614**

### **Sunday, August 26, 2001**

- **1:00 to 2:00 p.m.: *Opening Session*** (Huntington Room)  
*Eric E. Harm, Illinois Department of Transportation – Moderator*
- Introduction and Welcoming Remarks – Eric E.Harm
- NCHRP Studies Related to LTPP – Amir N. Hanna, NCHRP/Transportation Research Board
- LTPP Role in Promoting Smoother Pavements – Mark Swanlund, Federal Highway Administration
- **2:00 to 5:30 p.m.: *Understanding LTPP Findings and Procedures*** (Huntington Room)  
*Eric E. Harm – Moderator*
  - LTPP Data Collection for Smoothness Measurement – Larry Wisler, Federal Highway Administration
  - Findings from NCHRP Data Analysis for Specific Pavement Studies – Starr D. Kohn and Rohan W. Perera, Soil and Materials Engineers, Inc.
  - Findings from NCHRP Data Analysis for General Pavement Studies – Rohan W. Perera and Starr D. Kohn, Soil and Materials Engineers, Inc.
  - Findings from Other LTPP Data Analysis – Harold L. Von Quintus, Fugro-BRE, Inc.

## Workshop Agenda (Continued)

### Monday, August 27

- **8:00 to 11:30 a.m.: Current Practices and Issues** (Huntington Room)  
*Eric E. Harm – Moderator*
- Equipment Types and Applications – Steven M. Karamihas, University of Michigan Transportation Research Institute
- FHWA Survey of State Practices – David B. Law, Federal Highway Administration
- Issues in Pavement Smoothness: Contractors’ Perspective – Gary Fick, Duit Construction and Mark Belshe, FNF Construction, Inc.
- Issues in Pavement Smoothness: States’ Perspective – Kenneth W. Fults, Texas Department of Transportation
- Role of LTPP in Improving Current Practice - Starr D. Kohn
- State’s Approach to Pavement Smoothness – J. Patrick Gardiner, Pennsylvania Department of Transportation
- **11:30 a.m. to 12:00 noon: Plan for Afternoon Sessions** (Huntington Room)  
*Carolyn D. Goodman, Virginia Transportation Research Council – Moderator*
- **1:00 to 5:00 p.m.: Strategies to Improve Practice: Identification of Gaps** (Balboa, Newport, and Crystal Cove Rooms)  
*Facilitated Working Groups\**

Group A:	1:00 – 2:15 p.m.	Equipment/Measurement Issues
	2:30 – 3:45 p.m.	Data Analysis
	3:45 – 5:00 p.m.	Specifications and Use
Group B:	1:00 – 2:15 p.m.	Data Analysis
	2:30 – 3:45 p.m.	Specifications and Use
	3:45 – 5:00 p.m.	Equipment/Measurement Issues
Group C:	1:00 – 2:15 p.m.	Specifications and Use
	2:30 – 3:45 p.m.	Equipment/Measurement Issues
	3:45 – 5:00 p.m.	Data Analysis

- \* Group A (Red): Balboa Room
- Group B (Blue): Newport Room
- Group C (Green): Crystal Cove Room)

## Workshop Agenda (Continued)

**Tuesday, August 28**

- **8:00 to 9:45 a.m.: Plenary Session: Analysis and Prioritization of Identified Gaps** (Huntington Room)  
*Carolyn D. Goodman – Moderator*
- **10:00 a.m. to 12:00 noon and 12:45 – 1:45 p.m.: Strategies to Improve Practice: Dealing with Gaps** (Balboa, Newport, and Crystal Cove Rooms)  
**Facilitated Working Groups\***
  - Group D: Means for dealing with top priority gaps
  - Group E: Means for dealing with top priority gaps
  - Group F: Means for dealing with top priority gaps
  - \* Group D: Balboa Room
  - Group E: Newport Room
  - Group F: Crystal Cove Room)
- **2:00 to 3:00 p.m.: Summary Reports: Gaps, Needs, and Roles.** (Huntington Room)  
**Carolyn D. Goodman – Moderator**
  - Group D: Summary and Issues
  - Group E: Summary and Issues
  - Group F: Summary and Issues
- **3:00 to 3:30 p.m.: Closing Session** (Huntington Room)  
**Eric Harm – Moderator**
  - Next Steps – Amir N. Hanna and Eric E. Harm
  - Closing Remarks – Eric E. Harm

**APPENDIX C**

**PAVEMENT SMOOTHNESS MEASUREMENT  
AND ANALYSIS: STATE OF THE KNOWLEDGE**

Prepared for  
National Cooperative Highway Research Program  
Transportation Research Board  
National Research Council

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

July 2001

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# CHAPTER 1

## INTRODUCTION

It is believed that the public perceives a good road as one that provides a smooth ride. Studies at the road test sponsored by the American Association of State Highway Officials (AASHO) showed that the subjective evaluation of a pavement, based on mean panel ratings, was primarily influenced by roughness. State highway agencies have recognized pavement smoothness as an important measure of pavement performance.

A variety of profiling equipment has been used over the years to measure roughness of pavement networks. Until the mid 1980's, many highway agencies used some type of Response Type Road Roughness Measuring System to measure roughness of their pavement networks. High-speed road profiling technology began in the 1960's when Elson Spangler and William Kelly developed an inertial profiler at the General Motors Research Laboratory (1). The number of States that have adopted high-speed profilers to collect roughness data on their highway networks has increased dramatically in the past decade. The recently completed NCHRP Project 10-47, presented an in-depth analysis of factors influencing the longitudinal profile measurements made by inertial profilers (2) and recommended guidelines for measuring longitudinal pavement profiles (3).

Profilographs are widely used to evaluate the smoothness of new construction. The Profile Index that is computed from the profilograph trace is used as the basis of acceptance of new construction, and for payment of incentives. During recent years, lightweight profilers have been used increasingly to evaluate the smoothness of new pavements.

The Long Term Pavement Performance (LTPP) program has been collecting roughness data at General Pavement Studies (GPS) and Specific Pavement Studies (SPS) test sections located throughout the United States over the past ten years. This data provides information on roughness development over time for a variety of pavements. Several data analysis studies that analyzed the roughness data collected at LTPP sections have been performed during the past several years (4,5). The ongoing NCHRP Project 20-50(08/13) is utilizing roughness data collected at LTPP test sections to investigate factors affecting pavement smoothness.

This report provides a summary of the information available on a variety of topics related to pavement smoothness, including the following:

- Pavement roughness and user perception of ride quality;
- Benefits of smooth pavements;
- Equipment for roughness measurement;
- Profile indices;
- Operational characteristics of profilographs;



- Factors affecting inertial profiler measurements;
- Smoothness testing of new construction;
- Application of roughness data at network and project level;
- Findings from LTPP data analysis studies; and
- Issues related to pavement smoothness measurement.

## **CHAPTER 2**

### **ROAD ROUGHNESS AND USER PERCEPTION OF RIDE QUALITY**

#### **ROAD ROUGHNESS**

Road users judge the quality of a road primarily based on its ride quality. From a user's point of view, rough roads mean discomfort, decreased speed, potential vehicle damage, and increased vehicle operating cost.

The American Society of Testing and Materials (ASTM) standard E867 (6) defines roughness as the deviations of a pavement surface from a true planer surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and drainage. Road roughness can also be defined as the distortion of the road surface that imparts undesirable vertical accelerations in the vehicle that contribute to an undesirable or uncomfortable ride.

There are several factors that contribute to pavement roughness: built-in construction irregularities, traffic loading, environmental effects, and construction materials. Construction irregularities can cause variations in the pavement profile from the design profile, and this can cause a pavement that has not been opened to traffic to exhibit roughness. The roughness of a pavement increases with increased traffic loading. Repeated traffic loading can cause pavement distresses such as cracking that result in increased roughness. Environmental effects such as frost heave and volume changes due to shrink and swell of subgrade can also cause the roughness of a pavement to increase over time. Non-uniformities in the materials that are used for pavement construction as well as non-uniform compaction of pavement layers and subgrade can also contribute to roughness.

#### **USER PERCEPTION OF RIDE QUALITY**

Studies at the road test sponsored by the American Association of State Highway Officials (AASHO) showed that the subjective evaluation of the pavement, based on mean panel ratings, was primarily influenced by roughness (7).

During the AASHO road test, rating panels were used to rate pavement sections on a scale of 0 to 5. A rating of 5 indicates a perfect pavement, whereas a rating of 0 represents an exceedingly poor pavement. A rating panel consisted of several members, who were driven over several pavement sections that had different levels of roughness. Each panel member rated each of the roadway segments, and the mean value of the panel rating was called the present serviceability rating (PSR) for the road segment. Thereafter, regression analysis was performed between mean panel ratings and pavement roughness and distress to develop relationships. The objective of this analysis was to develop relationships that could be used to predict the serviceability rating for a pavement section from objective measures such as roughness and distress.

The roughness of the pavement sections at the AASHO road test was measured by a device called the CHLOE profiler, and the values recorded by this device were used to compute a roughness index called slope variance. The roughness of the sections expressed as the slope variance was used in the regression analysis. Separate relationships to predict the serviceability rating for flexible and rigid pavements were developed. The estimate of the PSR obtained from the developed equations was called the present serviceability index (PSI). The following equations were developed in the regression analysis to obtain the PSI.

Flexible Pavements:

$$\text{PSI} = 5.03 - 1.91 \text{ Log } (1+\text{SV}) - 1.38 \text{ RD}^2 - 0.01 \sqrt{C+P}$$

Rigid Pavements:

$$\text{PSI} = 5.41 - 1.80 \text{ Log } (1+\text{SV}) - 0.09 \sqrt{C+P}$$

where,

PSI = present serviceability index

SV = slope variance

RD = mean rut depth in inches

C = lineal feet of major cracking per 1000 ft<sup>2</sup> area

P = patching in ft<sup>2</sup> per 1000 ft<sup>2</sup> area

The PSI is based upon the concept of correlating user opinions with measurements of road roughness, cracking, patching and rutting. This analysis indicated that about 95 percent of the information about the serviceability of a pavement is contributed by the roughness of its surface profile (7). That is, the correlation coefficients in the present serviceability equations improved only about 5 percent when other factors were added.

## CHAPTER 3

### BENEFITS OF SMOOTH PAVEMENTS

#### INTRODUCTION

Road users judge the quality of a road by its smoothness (lack of roughness). Therefore, highway agencies strive to achieve smooth pavements for both new construction and rehabilitated pavements. Many highway agencies believe that pavements that are constructed smoother will remain smooth over time, and provide a longer service life. In recent years, many highway agencies have been offering incentives to contractors who achieve higher levels of smoothness than a specified value.

Recent research studies have indicated that when compared to rough pavements, smooth pavements exhibit the following features:

1. Result in lower dynamic loads on pavements;
2. Remain smooth over time;
3. Provide a longer service life; and
4. Result in a decrease in fuel consumption and vehicle maintenance costs.

#### DYNAMIC LOADS ON PAVEMENTS

Dynamic motions of vehicles are caused by road surface unevenness. These dynamic motions cause the load that is applied by the vehicle on the pavement to fluctuate about the static load of the vehicle. The load applied to the surface of a road by a truck tire will be the sum of the static load carried by the tire and a continuously varying load, which can be either positive or negative.

Figure 1 illustrates the variations in the load applied along a section of a roadway due to variations in roughness ( $\delta$ ). The y-axis of figure 1 represents  $E_{dyn}$ , which is defined as:

$$E_{dyn} = \frac{(\text{Actual Applied Load} - \text{Static Load}) \times 100}{\text{Static Load}}$$

This section of the roadway has a present serviceability rating of 2.9. At some locations of the roadway, the applied load is greater than the static load, while at other locations it is less.

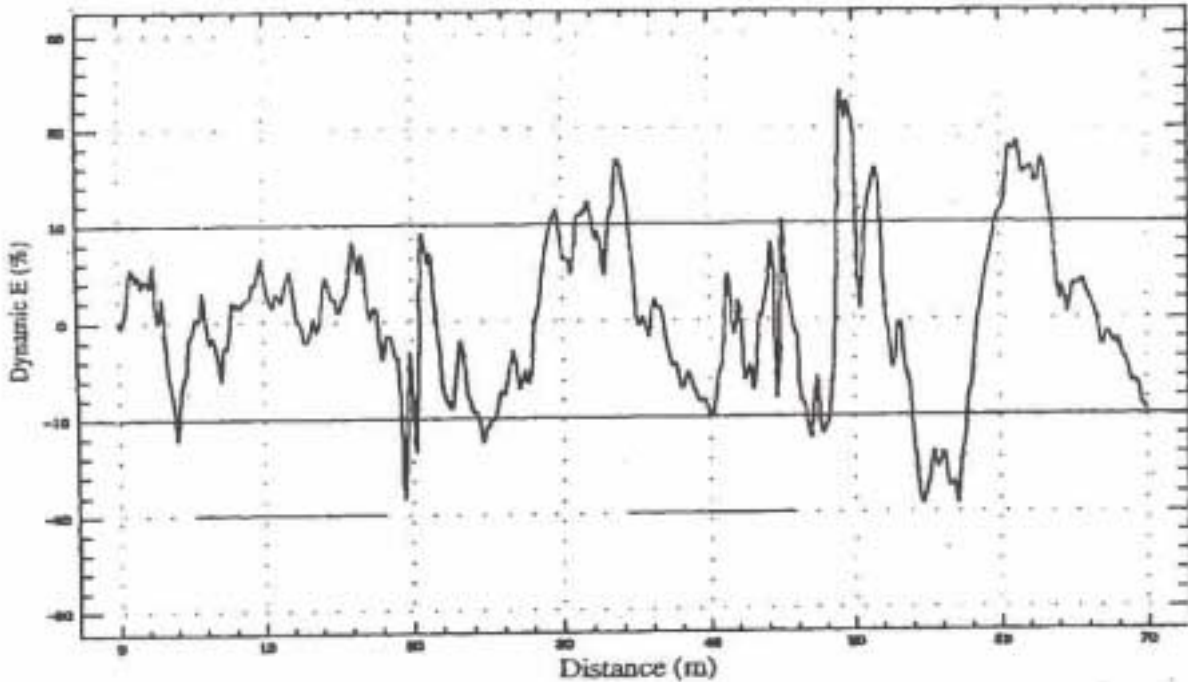


Figure 1. Variation of dynamic load along a pavement (8).

Ma and Caprez (8) performed dynamic load simulations for pavements having different PSI values to determine dynamic load effects. Vehicle simulations were carried out for different simulation speeds. Figure 2 presents the maximum and minimum  $E_{dyn}$  values that were obtained for the different pavement sections. Figure 2 shows that dynamic load effects increase with decreasing PSI values (i.e. increasing pavement roughness).

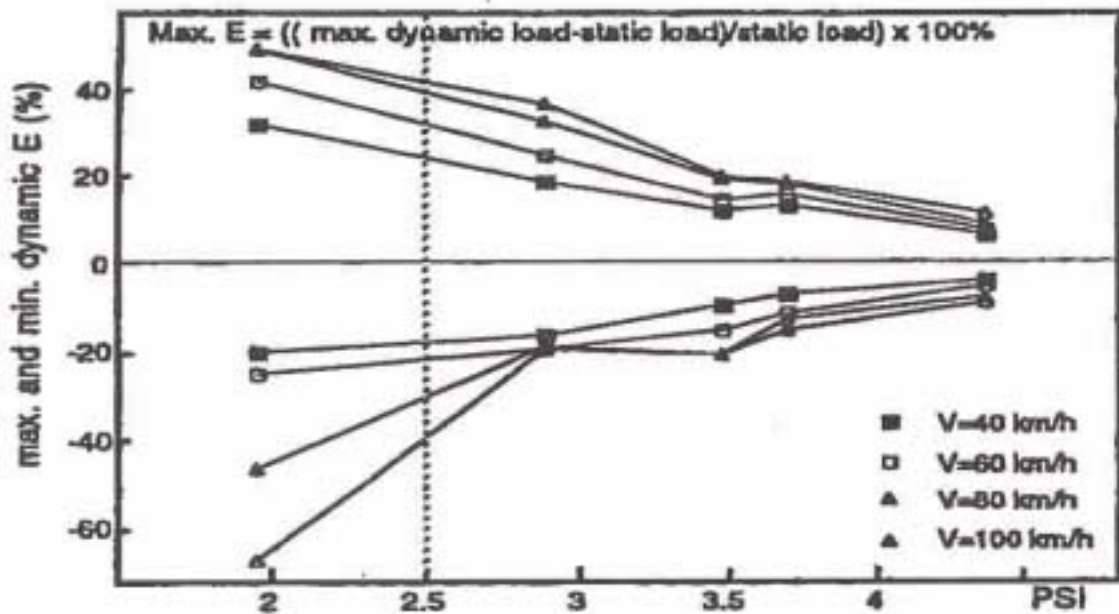


Figure 2. Variation of dynamic load with PSI (8).

Figure 3 shows the relationship between roughness and dynamic load expressed as a Dynamic Load Index (DLI) (9). These results were obtained from a research project in which the dynamic loads were measured on trucks with three different suspension systems. The DLI is defined as the standard deviation of the load normalized by the static load. Thus, an index of zero implies the load is its static value, while an index of 0.25 represents a load variation for which the standard deviations is 25 percent of the static load. The results in figure 3 show that higher roughness levels cause higher dynamic loading on the pavement.

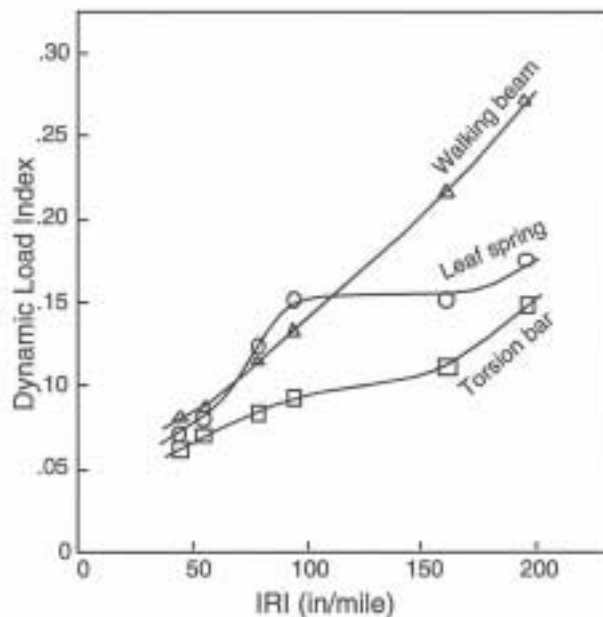


Figure 3. Relationship between IRI and truck dynamic loads (9).

The results from both studies show the magnitude of dynamic loads induced on smoother pavements is substantially less than that induced on rougher pavements. An increase in dynamic loads will contribute to deterioration of the pavement. Therefore, a pavement that has a higher smoothness level initially is expected to have a longer service life than a rougher pavement, all other factors being equal.

### EFFECT OF INITIAL SMOOTHNESS ON FUTURE SMOOTHNESS

Smith et al. (10) performed a study to analyze the effect of initial pavement smoothness on future pavement smoothness. Over 200 pavement projects from 10 States were analyzed in this study. Each project was divided into two or more adjacent “replicate” sections along a highway in an effort to isolate the effect of initial pavement smoothness. An example of these adjacent replicate sections is illustrated in figure 4.

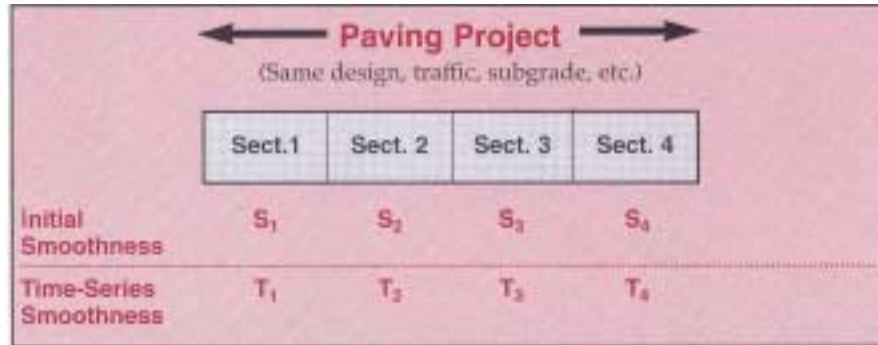


Figure 4. Replicate sections along a highway pavement project (10).

For each project, a regression analysis was conducted relating the smoothness at any time  $t$  to the initial smoothness and to the age, as expressed in the following general form:

$$S_t = a_0 + a_1 S_i + a_2 t$$

where,

$S_t$  = pavement smoothness at time  $t$

$a_0, a_1, a_2$  = regression coefficients

$S_i$  = initial pavement smoothness

$t$  = time (age) in years since construction or overlay to time of smoothness testing

The analysis determined the regression coefficients and information on the statistical significance of the independent variables (initial smoothness and age) on the dependent variable (smoothness at time  $t$ ). Figure 5 shows the physical significance of the  $a_1$  coefficient for four different cases, assuming four sections of a pavement project with different initial smoothness values. Many projects included in the analysis had  $a_1$  values close to 1, indicating a long-term effect of initial smoothness.

The regression analysis results showed initial smoothness was significant in 80 percent of portland cement concrete (PCC) pavement projects, 80 percent of asphalt concrete (AC) pavement projects, 77 percent of AC overlay of AC projects (AC/AC), and 63 percent of AC overlay of PCC projects (AC/PCC). The effect of initial smoothness on future smoothness was lower for overlay projects because the performance of AC overlays is strongly influenced by the condition of the underlying pavement. For AC/PCC pavements, the future smoothness is strongly influenced by the development of reflection cracking. This factor resulted in the lower significance for AC/PCC projects. Although initial smoothness has an effect on future smoothness of the pavement, there are other factors influencing pavement performance that in some instances may overwhelm or negate the effects of initial smoothness. Such factors include: variability in materials and construction, subgrade settlement and heaves, variations in topography, presence of bridges, culverts, and other structures along the highway.

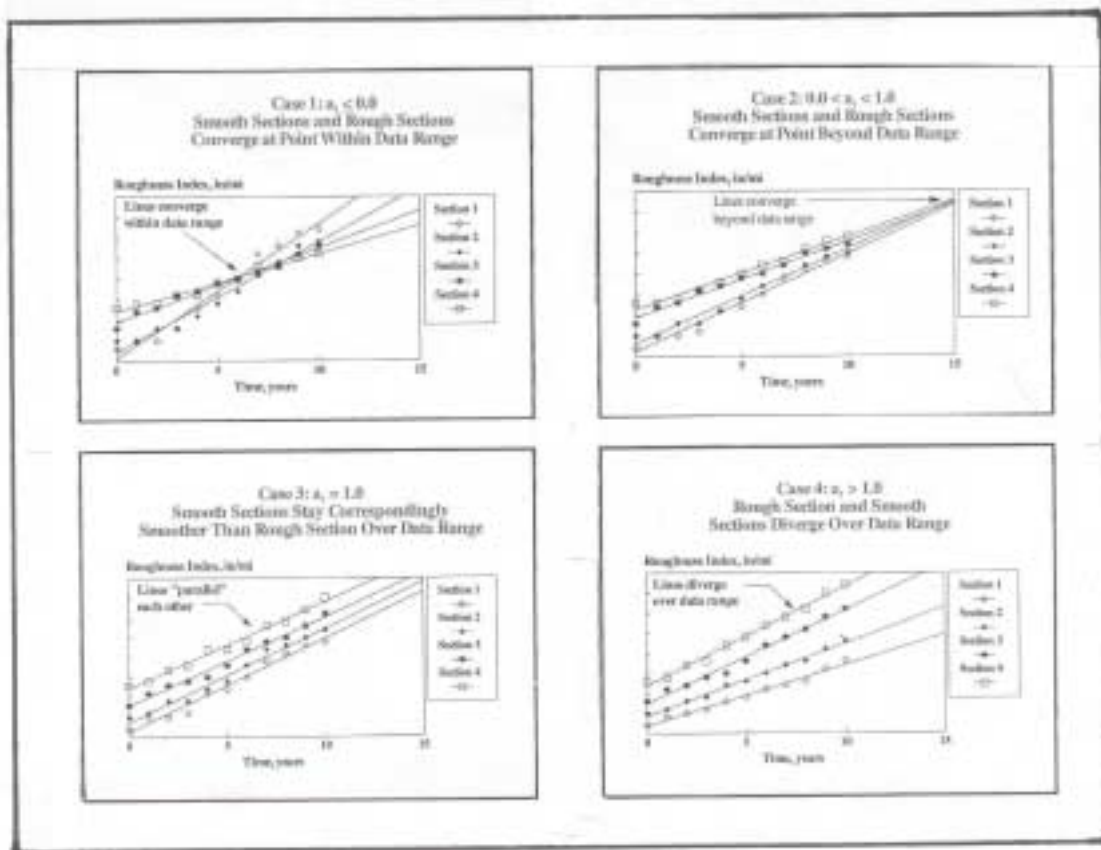


Figure 5. Effect of  $a_1$  coefficient in the relationship between time and future roughness (10).

An analysis of AASHO road test data by Smith et al. (10) indicated smoother sections stayed smoother over time, provided that there was a difference of more than 0.3 serviceability units in initial smoothness. Thus, results from this research project indicate that all other things remaining equal, pavements constructed smoother remain smoother over many years.

### EFFECT OF INITIAL SMOOTHNESS ON PAVEMENT LIFE

The AASHO pavement design equations are widely used in the design of both flexible and rigid pavements. These design equations indicate a pavement with a higher initial serviceability has a longer life when compared to a pavement having a lower initial serviceability, all other factors being equal.



The initial smoothness of a pavement is an indicator of the overall quality of construction of the pavement. If the pavement is constructed smooth, the contractor is likely to have provided good quality workmanship in other aspects of construction. A survey of contractors indicated obtaining smoother pavements requires attention to other aspects of pavement construction thereby improving the overall quality of the pavement (10). Therefore, a pavement that has a higher initial smoothness is expected to provide a longer service life than a pavement that has a lower initial smoothness, all other factors being equal.

Smith et al. (10) used two different analytical techniques to investigate the effect of initial pavement smoothness on pavement life. The two analytical techniques that were used were: project specific regression models and analysis of failure curves. Project specific regression models were used for predicting pavement life to trigger roughness levels. The failure curve method was used to develop failure curves relating the percentage of failed projects (overlaid projects) as a function of time. Both analytical methods indicated that initial pavement smoothness has a significant effect on pavement life. It was shown that added pavement life could be obtained by achieving higher levels of initial smoothness. Combined results of both roughness model and pavement failure analyses indicated at least a 9 percent increase in life can be achieved by increasing the smoothness 25% from a target profile index values of 7 to 5 in/mile for PCC pavements, and from 5 to 3.5 in/mile for AC pavements. A 50 percent increase in smoothness from these target values was found to increase pavement life by at least 15 percent in many cases. Thus, an approximate increase in smoothness from 7 to 3.5 in/mile based on profile index for PCC, and 5 to 2.5 in/mile based on profile index for AC pavements could conceivably yield at least a 15 percent increase in service life.

## **EFFECT OF PAVEMENT SMOOTHNESS ON FUEL COST AND VEHICLE MAINTENANCE COST**

Researchers at the Westrack pavement testing facility near Reno, Nevada have found that smoother pavements cause improved fuel efficiency and reduce vehicle maintenance costs (11). At the Westrack facility, from 1997 to 1999, four driverless trucks traveled an average of 15-hours a day, around the 2.9 km oval track, simulating more than 10 years of interstate level traffic loads. Their runs were designed to evaluate how variations in hot-mix asphalt construction properties affect pavement performance and to validate the Superpave mix design and analysis procedures. During this time, some sections of the track developed varying amounts of roughness, rutting, and fatigue cracking, requiring major rehabilitation. The rehabilitation consisted of milling 4 inches from the AC surface and resurfacing.

To determine the effect of pavement quality changes on fuel economy, data from two identical Westrack vehicles were examined for periods just before and after March 1998 track rehabilitation. Prior to the rehabilitation, the track was in a rough condition with fatigue cracking of various test sections and deterioration of areas that had been patched after core sampling. The improvement resulting from the rehabilitation was evident in the International Roughness Index (IRI) values for the track, which showed that the average IRI had been reduced by at least 10 percent.

As part of the study of fuel economy, the fuel rate, fuel temperature, torque and engine speeds of the trucks were analyzed, as were fuel use data from daily inspections and refueling. The data showed that the average fuel mileage over an 8-week period before rehabilitation was 1.79 km/l. After rehabilitation, the average fuel mileage over a 7-week period was 1.86 km/l, indicating a 4.5 percent improvement. All other factors such as truck geometry, air temperature, and wind speed were either identical before and after rehabilitation or were compensated for within the comparison calculations. For a trucking company with a fleet operation of 1.6 million km, driving on smoother pavements would thus mean a saving of 46,660 l of fuel.

The increased pavement roughness at Westrack also increased the frequency of failures in truck and trailer components. For example, during the weeks just before pavement rehabilitation, trailer frames began to fracture and required reinforcing welds, and steering motors and other components loosened more frequently. During the 2.5 years of traffic loading at the track, 8 of 17 trailer spring failures occurred within the 2 months prior to the March 1998 rehabilitation. Over these 2 months, 265,000 equivalent single axle loads (ESALs) were applied to the track. In contrast, the 350,000 ESALs applied in the 7 weeks after rehabilitation resulted in only one spring failure. Figure 6 shows the number of trailer spring failures per million ESALs during different trafficking periods. Substantial savings in vehicle maintenance cost for a smooth pavement could be inferred from the decrease in number of failures after rehabilitation.

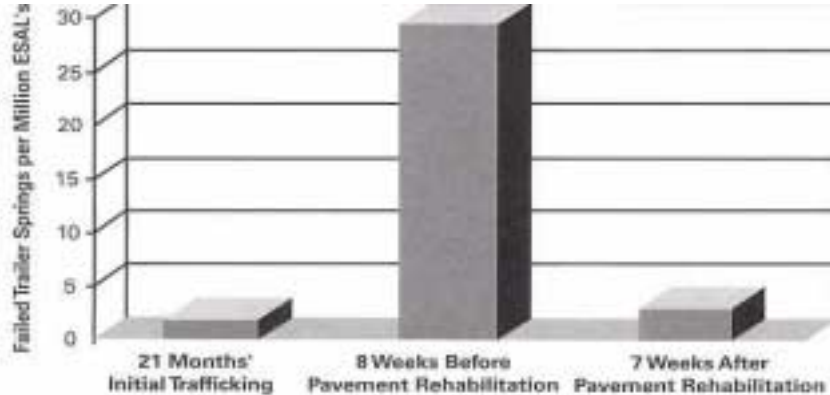


Figure 6. Effect of pavement condition on truck spring failures (11).

## CHAPTER 4

# EQUIPMENT FOR ROUGHNESS MEASUREMENT

### INTRODUCTION

A variety of equipment has evolved over the years to measure the roughness of pavements. Roughness measurements of roadways are performed to monitor the condition of a road network for use in a pavement management system (PMS), or to evaluate the ride quality of newly constructed or rehabilitated pavements. Profile data obtained from inertial profilers can also be used to diagnose the condition of specific sites and determine appropriate remedies, and to study the condition of specific sites for research.

The equipment that are used to measure roughness of pavements can be divided into the following five categories:

1. Response type road roughness measuring systems;
2. High speed inertial profilers;
3. Profilographs;
4. Lightweight profilers; and
5. Manual devices.

Until the mid 1980s, many highway agencies used some type of Response Type Road Roughness Measuring System (RTRRMS) to measure roughness of their pavement networks. The response type devices measured the response of the road on the vehicle. These devices typically had a transducer that accumulated the relative motion between the axle and the vehicle frame. A variety of RTRRMS has been developed over the years. Some of the popular response type devices were the BPR roughometer, PCA meter, and Mays ride meter. With the advent of inertial profilers, the use of RTRRMS has declined.

High speed road profiling is a technology that began in the 1960's when Elson Spangler and William Kelly developed an inertial profiler at the General Motors Research Laboratory (1). The number of States that have adopted high-speed profilers to collect roughness data on their highway networks have shown a dramatic increase in the past decade. Inertial profilers collect pavement profile data at highway speeds, and generate the true profile of a roadway.

Profilographs are widely used to evaluate the smoothness of new construction as well as overlays. Most States use the Profile Index (PI) that is obtained from the profile trace measured by the Profilograph as the basis for acceptance of new pavements as well as overlays. Incentive and disincentives for new construction are also based on the PI value.

During recent years, lightweight profilers have been increasingly used to evaluate the condition of new construction. The lightweight profilers collect the true profile of a roadway. The profile data can be used to simulate a profilograph over the pavement section, and generate a PI for the roadway and to identify bump locations. The profile data can also be used to

compute roughness indices such as International Roughness Index (IRI) and Ride Number (RN) of the pavement.

Manual devices such as the Dipstick, walking profiler, and rod and level are generally used to collect profile data at a section in order to verify the data collected by road profilers. The general procedure that is used today to verify the output from road profilers is to collect profile data at test sections using a manual device, then compute a roughness index such as IRI from that data and to compare that value with the output from the road profiler.

This chapter presents a description of the equipment that fall into each of the five different equipment categories that were described previously.

## **RESPONSE TYPE ROAD ROUGHNESS MEASURING SYSTEMS (RTRRMS)**

### **Types of Response Type Road Roughness Measuring Systems**

Response Type Road Roughness Measuring Systems measure the response of the road on the vehicle or a special trailer using a transducer. Automobiles or standardized trailers have been used to house response type devices. The vehicle-mounted systems accumulate the vertical movement of the rear axle of the automobile with respect to frame, while the trailer mounted systems accumulate the movement of the trailer with respect to the frame. One of the earliest response type devices was the Bureau of Public Roads (BPR) Roughometer. Thereafter, a variety of response type devices such as Cox Roadmeter, PCA Roadmeter, and Maysmeter were developed. These devices measure roads at speeds up to 80 kph (50 mph). The American Society for Testing and Materials has developed ASTM Standard E 1082 (6), “Standard Test Method for Measurement of Vehicle Response to Traveled Surface Roughness” that specifies procedures to be followed for measuring roughness with vehicle mounted response type systems. The ASTM Standard E 1215 (6) “Standard Specifications for Trailers Used for Measuring Vehicle Response to Road Roughness” specifies standards and procedures to be used for measuring roughness with trailer mounted roughness measuring systems. Although there are problems involving reproducibility and portability of data taken with response-type systems, one reason that they have been so popular in the past is that they do provide an economic way of obtaining the roughness of roadways. The measures they produce have been viewed by engineers as matching their experience for determining pavement quality in a meaningful way.

A brief description of the measurement principles and operating procedures for the BPR Roughometer, Maysmeter and PCA meter are presented next.

#### *Bureau of Public Roads (BPR) Roughometer*

The Bureau of Public Roads (BPR) roughometer was first introduced in 1925 and was recognized as the best high-speed roughness device available at that time. Figure 7 shows a sketch of the BPR Roughometer. The BPR Roughometer consists of a single wheel trailer that is towed by a car or a light truck. The wheel mounted on the trailer is supported by leaf springs.

Variations in the pavement surface cause the wheel to move with respect to the frame of the trailer. These vertical movements are accumulated and the roughness for the pavement is given in terms of inches per mile. This device was operated at a test speed of 32 km/h (20 mph) to collect data.

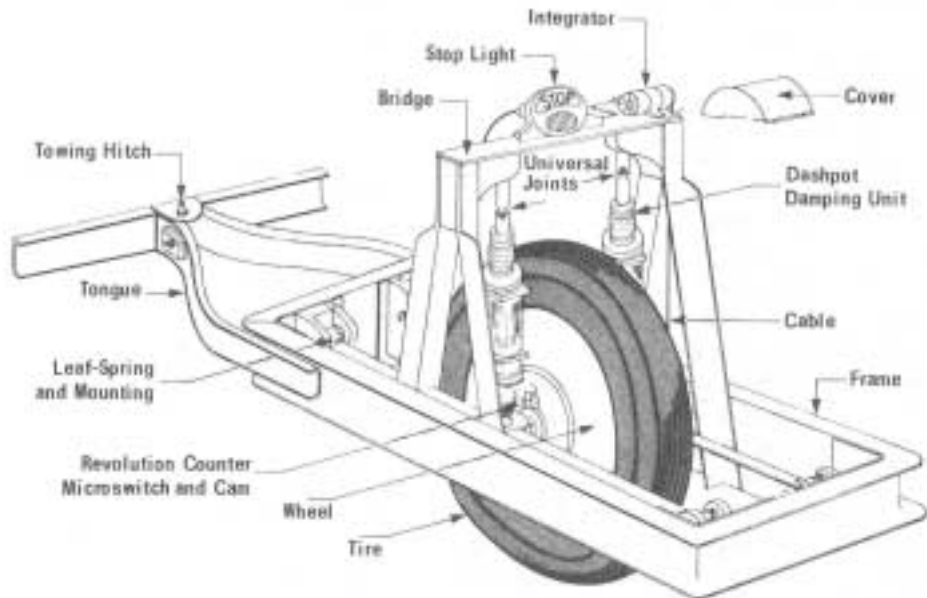


Figure 7. Bureau of Public Roads (BPR) Roughometer (12).

The measurements obtained by this device were susceptible to temperature, condition of bearings and mechanical components. This device also had a resonant frequency effect that produced incorrect results. Because of the slow operating speed of this equipment, many equipment modifications were made to increase the operating speed. However, the basic operational characteristics were altered at high speeds, and the use of the device gradually declined over time.

### *Maysmeter*

The Maysmeter is commercially manufactured by Rainhart Company of Austin, Texas. Maysmeters were widely used to measure pavement roughness from 1960s to early 1980s. This device was mounted on an ordinary passenger car or a light truck. Figure 8 shows a sketch of a Maysmeter.

The Maysmeter measures the suspension motions of the vehicle by recording the relative movement between the axle and the body. The roughness measure that is obtained by this device is “inches” of accumulated suspension stroke, divided by the distance traveled, and is reported in units of inches per mile. The measure of vehicle response measured by the Maysmeter is very similar in its frequency content to the accelerations on the vehicle body, so it is highly correlated to ride vibration.

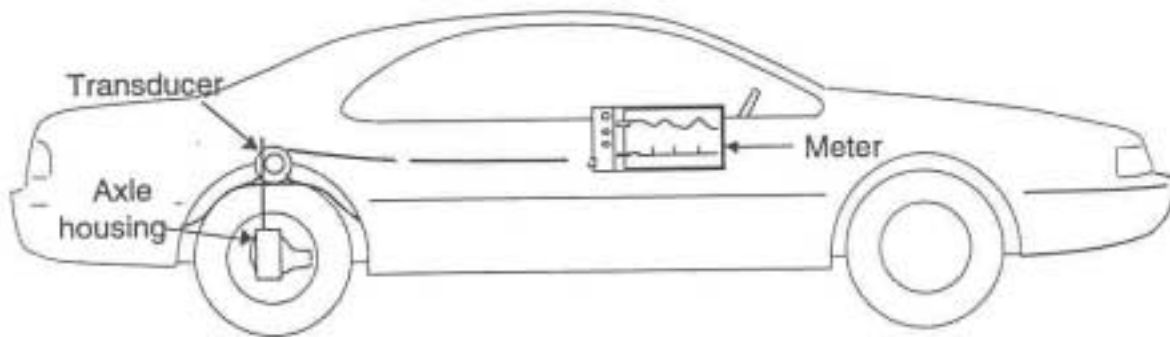


Figure 8. Car mounted Maysmeter (13).

### *PCA Roadmeter*

The PCA Roadmeter was developed by the Portland Cement Association in 1965 (14). This device measures the number and the amplitude of the vertical deviation between the body of an automobile and the center of the rear axle. The deviations are recorded in 1/8 inch increments up to a maximum excursion of  $\pm 1 \frac{1}{2}$  in from the neutral or null position. This device is used to take readings at a speed of 80 kph (50 mph).

### **Disadvantages of Response Type Road Roughness Measuring Systems**

Response type road roughness measuring systems have features that affect the accuracy of data collection. The following are some of the problems and disadvantages associated with response type devices.

#### 1. Characteristics of the mechanical system affect measurements

The measurements obtained from response type systems are influenced by the properties of the vehicle such as suspension system characteristics, tire conditions, tire pressure, and vehicle weight. If the properties of the vehicle change over time, the response that is measured will vary. This raises concerns regarding accuracy and repeatability of RTRRMS systems. As the roughness measurements may not be stable with time, measurements obtained by a RTRRMS cannot be compared with confidence to those made previously.

#### 2. Roughness measurements are not transportable

The measurements obtained by a RTRRMS are seldom reproducible by another device. The output obtained from a RTRRMS that is installed in a particular vehicle will be different from the output obtained from the same RTRRMS that is installed in another vehicle. Even if the vehicle is standardized (i.e., RTRRMS installed in same vehicle make), differences remain

between vehicles that one might think are identical. These differences occur due to differences in the suspension system, tire pressure and tire conditions between the vehicles.

### 3. Speed of travel affects measurements

The speed of travel affects the response of the system. If the same section is measured by the same device at two different speeds, the outputs that are obtained will be different. When roughness measurements are made with response type systems, a standard speed is used to obtain measurements. However, the vehicle may not be able to always maintain this standardized speed due to existing traffic conditions.

### 4. Lack of a standard roughness scale

A major disadvantage of using data obtained from a response-type system has been the lack of a standard roughness scale. The lack of a standard measure was at first not seen as a serious problem by many of the users of roughness instruments. Roughness data for a city, county, or State could have arbitrary units, as long as the database was internally consistent. If the vehicle properties did not change during the period in which the roughness measurements were obtained, the roughness measurements could be compared. However, if the vehicle properties has changed from the previous year, the roughness measurements obtained during the current year cannot be compared with roughness measurements obtained previously.

### 5. Comparison cannot be made between different units

The roughness measurements obtained for a highway network using different RTRRMS are not comparable. This situation can arise if a State had used different RTRRMS to measure roughness of different portions of their highway network. A method to overcome this problem was to calibrate the RTRRMS systems to a standard roughness measurement using several test sections. This involves selection of several test sections that have roughness levels varying from smooth to very rough. Thereafter, the longitudinal profile of the test sections is measured using an inertial profiler or a manual device, and a roughness index (such as IRI) is computed from profile data. Thereafter, a regression analysis is performed between the output obtained from each response type device and the roughness index of the test sections. The regression equation that is developed for each device provides a method to convert the roughness value measured by the response type device to a standard roughness scale. As the roughness value measured by different devices can be converted through regression equations to a standard roughness scale, this procedure provides some confidence in obtaining a uniform roughness measurement for a pavement network that is measured by different response type devices.

### 6. Inability to locate rough features

As the response type devices record only the response of the vehicle, no information regarding the location of rough spots in the pavement can be obtained from the recorded data.

## HIGH SPEED INERTIAL PROFILERS

Inertial profilers record the true profile of a pavement surface. Inertial profilers collect profile data on pavements at highway speeds. The first high-speed inertial profiler was developed by Elson Spangler and William Kelley at the General Motors Research Corporation (1). Early inertial profilers sensed the height of the vehicle relative to the ground using an instrumented follower wheel that traversed along the wheel path. The follower wheels were fragile, and required testing at low speeds to avoid bouncing. All profilers that are sold today use non-contacting sensors located on the vehicle instead of follower wheels.

A schematic diagram of an inertial profiler is shown in figure 9. The principal components of an inertial profiler are height sensors, accelerometers, distance measuring system, and computer hardware and software for computation of the road profile. The height sensors record the height to the pavement surface from the vehicle. The accelerometers that are located on top of the height sensors record the vertical acceleration of the vehicle. Data from the accelerometers are used to determine the height of the vehicle relative to an inertial reference frame. The distance measuring system keeps track of the distance with respect to a reference starting point. Using the data recorded by the distance measuring system, height sensor and the accelerometer, a computer program computes the profile of the pavement surface. The non-contact height sensor types that are used in profilers today are either laser, ultrasonic, optical or infrared. Ultrasonic sensors were the most common type of sensors used in the 1980s. However, because of problems with this type of sensors, their use has declined over the past several years. Currently laser sensors are the most commonly used height sensors used in profilers.

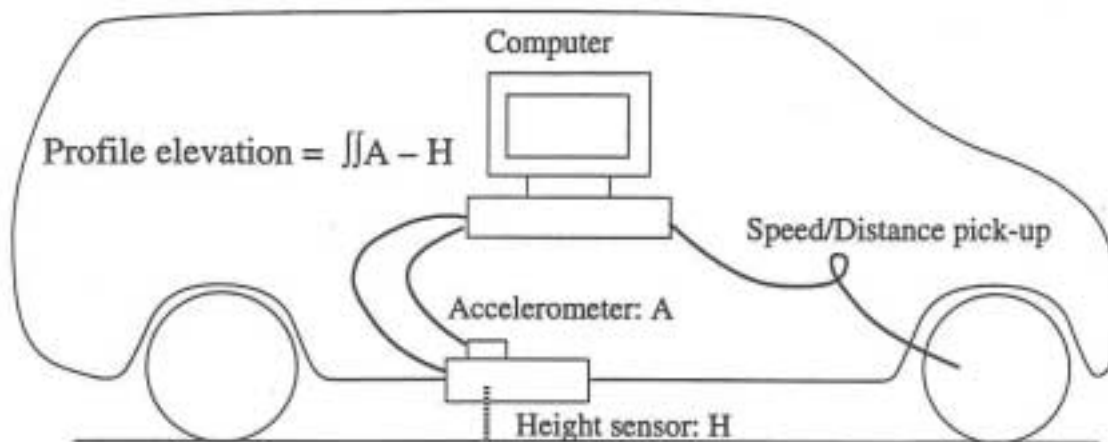


Figure 9. Components of an inertial profiler (13).

The profilers can also be equipped with a photocell that can be used to automatically initiate data collection. Two types of photocells, vertical and horizontal, are available. The vertical photocell can be used to automatically initiate data collection when the photocell detects a white reflective tape that is placed on the center of the travel lane. The horizontal photocell is used with a traffic cone that has reflective markings to initiate data collection. The traffic cone is



placed on the shoulder at the location where data collection has to be initiated. When the horizontal photocell detects the cone, data collection is initiated.

The first inertial profiler was commercially manufactured by K.J. Law Engineers. The South Dakota Department of Transportation developed a profiling system in 1984 (15). This profiling system was mounted in a small van and used ultrasonic sensors, and was adopted by several highway agencies. Currently a variety of inertial profilers are available. Manufacturers that currently manufacture inertial profilers include: Dynatest, International Cybernetics Corporation (ICC), Infrastructure Management Services (IMS), K.J. Law, Pathway and Roadware. Many of these profiling systems include a center sensor, whose elevation is used in conjunction with the elevations measured along the wheel paths to compute a rut depth. The procedure that is used to compute rut depth from a three sensor profiling system is shown in figure 10

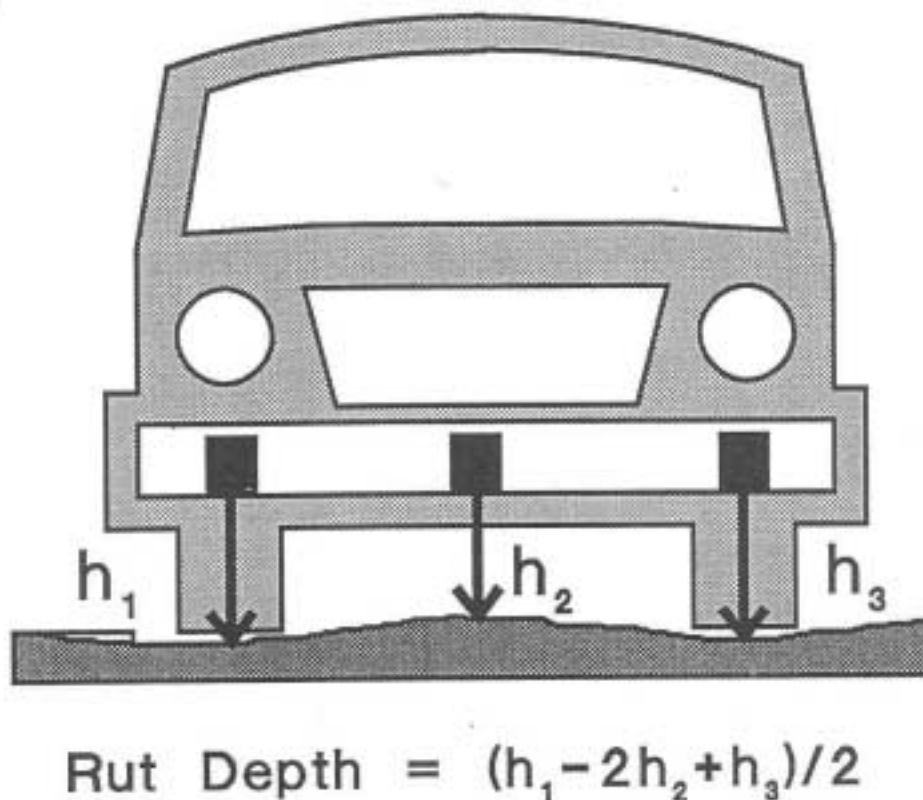


Figure 10. Rut depth computation from a three sensor profiling system

Some of the equipment manufacturers offer a five sensor profiling system to compute rut depths. In these systems, two additional sensors are mounted outside the left and the right wheel paths. Most of the profile equipment manufacturers can build customized systems that will collect a variety of data in addition to profile data. Some of these profiling systems are equipped with video cameras that can record several perspectives of the road, and these videos can be used to assess the condition of road, shoulder, signs as well as other roadway features.

Brief descriptions of inertial profiling systems that are currently available commercially are presented next, listed according to the alphabetical order of the manufacturer.

## **Dynatest**

The Dynatest Road Surface Profiler (RSP), shown in figure 11 performs continuous highway-speed measurements of longitudinal and transverse profile, including real-time roughness and rut depth evaluation. This product line is available in several levels of sophistication, ranging from a 21-laser top of the line version down to a single wheel path version for longitudinal profile and IRI evaluation. The transducer unit is located on the front bumper of the vehicle and can hold up to 21 laser sensors and 1 to 3 accelerometers.



Figure 11. Dynatest Road Surface Profiler.

## **International Cybernetics Corporation (ICC)**

The International Cybernetics Corporation manufactures a variety of inertial profilers. The following are the model types that are available.

Model MDR 4084: Equipped with three height sensors and one accelerometer. Obtains profile and IRI in one wheel path, and the average rut depth of two wheel paths.

Model MDR 4085: Equipped with two accelerometers and three height sensors. Obtains IRI and profile along two wheel paths, and the average rut depth of two wheel paths.

MDR 4087: Equipped with two accelerometers and five height sensors. Obtain IRI and profile along two wheel paths, and obtains the rut depth using data from the five height sensors.

Figure 12 shows the three-sensor ICC road profiler used by Ohio DOT.



Figure 12. ICC laser profiler - Ohio DOT.

### **Infrastructure Management Services (IMS)**

The IMS Laser RST Profiler is a road surveying system developed by the Swedish National Road Administration. Figure 13 shows a version of the Laser RST system. These vehicles can be equipped with cameras to collect pavement condition as well as perspective views of the roadway.



Figure 13. Infrastructure Management Services Laser RST profiler.

## **K.J. Law Engineers**

K.J. Law Engineers currently manufactures two inertial profiling devices called Model T6500 and T6600. Both these devices are equipped with infrared height sensors. The K.J. Law Model T6600 profilers are used to collect profile data for the Long Term Pavement Performance (LTPP) program. These profilers have been collecting profile data for the LTPP program since 1996. From the inception of the LTPP program in 1989 up to 1996, profile data collection at the LTPP test sections was performed by K.J. Law Model DNC 690 profilers.

The DNC690 profilers were equipped with optical height sensors, which were mounted on the body of the vehicle between the front and the rear axles. A shroud enclosed the height sensors to prevent sunlight from affecting the height sensor readings. Common problems with these profilers were the inability of the sensors to collect data on dark pavements, and contamination of profile data from sunlight. Figure 14 shows a photograph of the K.J. Law DNC 690 profiler.



Figure 14. K.J. Law DNC 690 profiler.

In the T6600 profilers, the height sensors are mounted in the bar that is located at the front of the vehicle. This device can collect data at 25 mm intervals. The infrared height sensors in this device have a footprint that covers an area of 6 mm x 37 mm, with the 37 mm dimension being perpendicular to the direction of travel. Figure 15 shows a photograph of the K.J. Law T6600 profiler.

## **Pathway**

Profilers manufactured by Pathway are being used by several State agencies to obtain profile data as well as videos that are used to assess the pavement condition. Figure 16 shows a photograph of the Pathway profiler used by Minnesota DOT.



Figure 15. K.J. Law T6600 profiler.



Figure 16. Pathway profiler -Minnesota DOT.

## **Roadware**

Roadware offers a variety of devices to collect road profile data and to collect pavement condition data. Figure 17 shows a photograph of the Laser SDP profiler manufactured by Roadware.





Figure 17. Roadware laser SDP profiler.

## PROFILOGRAPHS

Numerous models of profilographs have been used since 1900. A profilograph consists of a rigid beam or frame with a system of support wheels at either end, and a center wheel. The support wheels at the ends establish a datum from which the deviations of the center wheel can be evaluated. The center wheel is linked to a strip chart recorder or a computer that records the movement of the center wheel from the established datum. The profilograph is pushed along the pavement, and 3.2 to 4.8 km (2 to 3 miles) can be measured in an hour.

Mechanical profilographs record the data on a strip chart recorder. The output from the strip chart recorder has to be analyzed in order to obtain the smoothness results. The evaluation of the output from the strip chart recorder can be done either manually or electronically. In the manual method, a technician evaluates the profilograph output to determine smoothness results and bump locations. In the electronic method, the output of the strip chart recorder is scanned, and the data reduction is performed by a computer program and the results can be printed.

In the mid 1980s, James Cox and Sons introduced a computerized profilograph that recorded the measurements in a computer. These computerized profilographs can analyze the data using computer programs and generate the Profile Index of the section, and indicate the high points in the profile. The computerized profilograph eliminated the need for analyzing the profile in the office after the test was conducted in the field. The computerized profilograph was able to perform the trace reduction in the field immediately after the test was performed. This significantly increased productivity and reduced variability in trace reduction since the trace would always be interpreted the same.

The profilographs that are currently in use can be categorized into either California profilographs or Rainhart profilographs based on the support wheel configuration. Less than 10 percent of the profilographs in use are Rainhart profilographs (16). Of the California type profilographs, almost two thirds are truss type California profilographs while one third are beam type Ames profilographs (16).

## California Profilograph - Truss Type

The California Profilograph has been in use for over a half a century. The first California profilograph was constructed in 1940 by the materials and research division of the California Division of Highways. Since then, it has ranged in length from 4 m (13 ft) to 9.9 m (32.5 ft), and has had as few as four wheels and as many as sixteen (16). The twelve-wheel profilograph that is in common use today was first produced in 1961 (16). Figure 18 shows a photograph of the twelve-wheel California profilograph.



Figure 18. Truss type California profilograph.

The profilograph consists of frame segments, wheel assemblies, steering mechanism, and a recorder. The frame can be disassembled for transportation in a pick up or a van. The assembly time for the unit is reported to be 10 minutes, with the components being assembled using quick connect features and toggles. A plan view of the California profilograph is shown in figure 19.

As shown in this figure, the beam length of the profilograph is 7.6 m (25 ft), while the overall length is 9.9 m (32.5 ft). There are two support wheel systems at either end of the profilograph. Each wheel system consists of six wheels, with four wheels on one side of the truss and the other two wheels on the other side of the truss. The profilograph is pushed along the pavement by an operator who steers the front support wheels in the unit using a steering wheel located at the center of the unit. Profile traces are recorded to a horizontal scale of 1:300 (i.e., 1 in. = 25 ft) and to a vertical scale of 1:1.

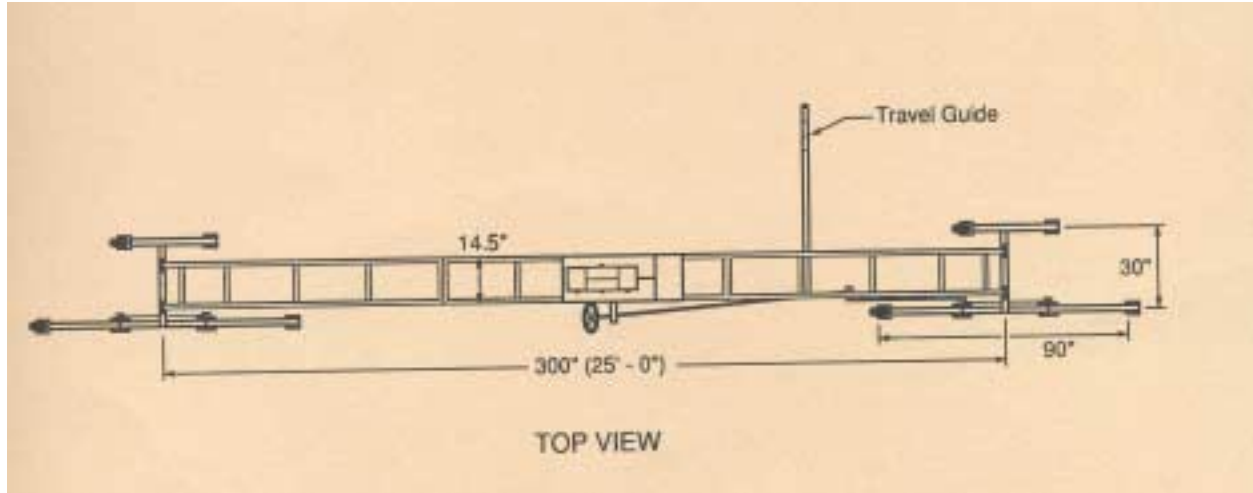


Figure 19. Plan view of California profilograph (17).

The California profilograph is commercially manufactured by James Cox and Sons, McCracken Concrete Pipe Machinery Company, and Soiltest. James Cox and Sons Inc. of California have been manufacturing profilographs since 1960 (16). In the mid 1980s they developed a computerized profilographs. It is reported that they currently manufacture only computerized profilographs. The McCracken Concrete Pipe Machinery Co of Sioux City, Iowa has marketed mechanical profilographs since 1984 and computerized models since 1990 (16).

### Ames Profilograph

The Ames profilograph manufactured by Ames Engineering is designed to produce the same profile trace as the truss type California profilograph, but it is considerably different in basic construction. Figure 20 shows a photograph of the Ames profilograph. Instead of a truss type of framework, the 7.6 m (25 ft) long portion in the Ames profilograph consists of an aluminum box beam. However, the wheel assembly of the Ames unit is similar to the truss type California profilograph in that six wheels are used to support the unit at each end, and a single wheel is located at the mid point of its 7.6 m (25 ft) length. Computerized models that are battery operated as well as mechanical models are manufactured by Ames Engineering. The unit can be disassembled and transported in a pickup truck.

The operator pushes the profilograph from one end of the unit. A steering wheel located at this end can steer the front wheels. The recorder in the profilograph is located at the rear of the unit. The movement of the wheel at the middle of the device is transmitted to the recording device at the rear of the device. A high-resolution ultrasonic vertical transducer is used to measure the vertical displacement of the center measuring wheel. The Central Direct Federal Division of FHWA conducted a study in 1987 in which the Ames Profilograph was compared with two truss type California profilographs manufactured by McCracken Concrete Pipe Machinery Company (18). The



traces produced by Ames device were found to be virtually identical to those produced by the two McCracken units.

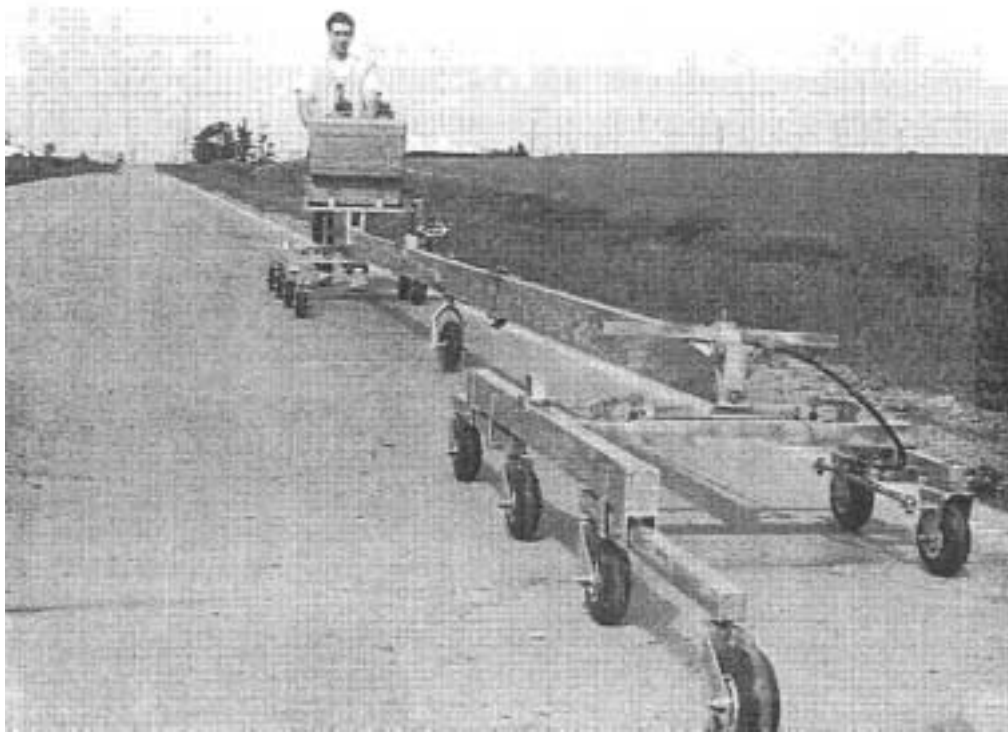


Figure 20. Ames profilograph.

### **Rainhart Profilograph**

The Rainhart profilograph was developed by Rainhart Company in Austin, Texas in conjunction with Texas Highway Department in 1967. Their studies served to establish the parameters under which the device was designed and constructed. The Rainhart Profilograph is commercially manufactured by Rainhart Company of Austin, Texas. Figure 21 shows a schematic sketch of the Rainhart profilograph. The apex of each tripod is attached with a ball joint to the ends of two minor trusses. The two minor truss centers are pivoted on the ends of a major truss, which supports a recorder at the center.

The overall length of the Rainhart profilograph is 7.5 m (24.75 ft). As in the California profilograph, the Rainhart profilograph has 12 wheels. However, the arrangement of these twelve wheels is different from the California profilograph. In the Rainhart profilograph the wheels operate in groups of three, with the front two groups and the rear two groups supporting minor truss, which in turn support a major truss as shown in figure 21. Each wheel traverses a separate path. The wheels in the Rainhart profilograph are evenly spaced along its 7.5 m (24.75 ft) span. Each of the 12 wheels has its own longitudinal path, spaced at 100 mm (4 in) intervals. The datum that is

established by the twelve wheels covers a pavement area 7.5 m (24.75 ft) long by 1.1 m (3.67 ft) wide. This is the major difference between the California and Rainhart profilographs, as the datum for the California profilograph is established based on the ends of the 7.6 m (25 ft) beam. The front support wheels of the Rainhart profilograph can be steered by the operator.

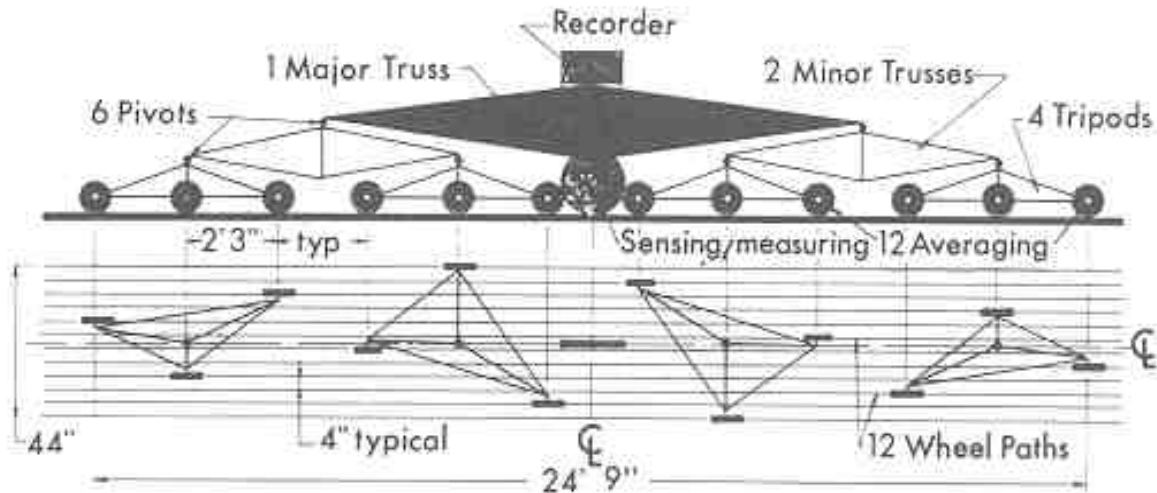


Figure 21. Rainhart profilograph (19).

The wheel located at the center of the unit is linked to a strip chart recorder, which records the movement with respect to the datum established by the support wheels. The profile trace in the Rainhart profilograph is recorded with a horizontal scale that is 10 ft per inch or 25 ft per inch, and the vertical scale is 1:1. The Rainhart profilograph cannot be disassembled for transport, although the recorder can be removed. The profilograph is equipped with a trailer hitch for towing and also possesses two wheels (not shown in figure) that are used for transport. The transport wheels are raised or lowered with respect to the truss system for profiling or transport mode. Upon arriving at the job site the unit can be disconnected from the towing vehicle and positioned for measurement. Rainhart profilographs are reported to be heavier and less maneuverable than the California type profilographs (19).

## LIGHTWEIGHT PROFILERS

Lightweight profilers were primarily developed to record the profile of newly placed PCC pavements. The term lightweight profiler is used to refer to devices in which a profiling system has been installed in a light vehicle, such as a golf cart or an all terrain vehicle. Lightweight profilers have gained popularity during the last several years, with many State highway agencies as well as contractors purchasing these systems. The profiling system in a lightweight profiler is similar to the profiling system of an inertial high-speed profiler and consists of height sensor(s), accelerometer(s), and a distance measuring system. The light weight of these devices makes it possible to profile freshly placed PCC as soon as the pavement is able to support the weight of the profiler. The profile

recorded by these devices can be used to generate a roughness index such as IRI, or to use the profile data to simulate a profilograph and get the Profile Index (PI). The vendors who are currently marketing lightweight profilers include Ames Engineering, International Cybernetics Corporation, and K. J. Law Engineers.

### **Ames Engineering - Lightweight Inertial Surface Analyzer (LISA)**

The lightweight inertial surface analyzer (LISA) was developed by the Materials and Technology Division of the Michigan Department of Transportation (20). Figure 22 shows a photograph of this device. This device is currently marketed by Ames Engineering. The profiling equipment is mounted on a John Deere 4-wheeler weighing about 364 kg (800 lb). The accelerometer on the vehicle has a resolution of 0.0001g. It is reported that the profiler can be operated at speeds between 8 to 24 km/h (5 to 15 mph), and records pavement profile values at 75 mm (3 in) intervals.



Figure 22. Lightweight Inertial Surface Analyzer (LISA).

### **International Cybernetics Corporation (ICC)**

International Cybernetics Cooperation (ICC) manufactures single track as well as dual track lightweight profilers. Figure 23 shows a single-track device, while figure 24 shows a dual track device. ICC reports that the profiling system can be mounted on an all terrain vehicle or on any specified vehicle. The device can be equipped with an optional photocell that can be used to automatically initiate and terminate data collection.



Figure 23. ATV configured single-track profiler – ICC.



Figure 24. Dual track profiler – ICC.

## **K. J. Law Engineers**

K.J. Law Engineers manufactures the T6400 lightweight profiler, which is a single sensor system designed to profile new pavements. The device is equipped with a digital encoder that provides the distance pulses for calculating the spatial profile. Figure 25 shows a photograph of this profiler.



Figure 25. K.J. Law T6450 lightweight profiler.

## **Comparison Study between Devices**

Fernando and Leong (21) performed a comparison study using rod and level data, K.J. Law T6450 lightweight profiler, and LISA. This comparison was conducted in 1996, and equipment currently available may have undergone changes since the time of comparison. Three sites were evaluated in this study. Multiple measurements were obtained with rod and level and the lightweight profilers. The IRI of the evaluated sites ranged from 0.9 to 2.1 m/km. The difference with the Rod and Level IRI ranged from 2 to 7 percent for both profilers with an average absolute difference of 5 percent.

## **MANUAL DEVICES**

A variety of manually operated equipment are available that can be used to obtain the profile of a pavement. Such methods/devices include: rod and level, Dipstick, and ARRB walking profiler. The primary application of these devices in road profile measurements is to obtain elevation measurements on calibration sections that are used to check the accuracy of

profilers. The profile of a test section obtained from these devices can be used to compute a roughness index such as IRI. This roughness index can be compared with the roughness index obtained by an inertial profiler to check the output of the profiler. Performing such a procedure on several test sections having a range of roughness values can be used to verify that the profiler is collecting valid data. In addition, filtering techniques can be used to compare the profiles obtained by the manual device and the profiler.

## **Rod and Level**

The rod and level is perhaps the most accurate method of obtaining the true elevations along a pavement surface. The ASTM standard “Test Method for Measuring Road Roughness by Static Level Method (E 1364)” describes the procedures to be followed to collect rod and level data at a test section (6). The most important factor when collecting rod and level data is to make sure that the resolution of the level meets the requirements outlined in the ASTM standard.

## **Dipstick**

The Dipstick is a hand held device manufactured by Face Technologies. Figure 26 shows a sketch of the Dipstick. The Dipstick stands on two feet and has a stand to support it when it is not in use. The distance between the two feet are adjustable. The equipment has two liquid crystal displays at each end that shows the elevation difference between the two feet. The operator walks the unit along the path to be measured, by rotating the unit. At each position of the Dipstick, the reading displayed in the front display is recorded. An option available with the Dipstick is to use a computer to record the displayed data. The data collected by the Dipstick can be used to obtain the elevation profile. Due to operator bias, the elevation profile obtain from the Dipstick has to be adjusted in order to get the correct elevation profile. However, no adjustments to the data are necessary if the data is used to compute either the IRI or the Ride Number.

## **Walking Profiler**

The walking profiler manufactured by the Australian Road Research Board (ARRB) is marketed in USA by Trigg Industries International. Figure 27 shows a photograph of the ARRB walking profiler. This device is a multi-wheeled inclinometer based system that is pushed by an operator at walking speed. The typical operation speed is 0.8 km (0.5 miles) per hour. The device continuously records the relative elevation of successive points at 241 mm (9.5 in) intervals, and stores the readings in an on-board lap top computer. All incremental changes are totaled giving the height of every measured point and its surface distance relative to the starting point. The device is reported to be capable of measuring elevations to an accuracy of  $\pm 2$  mm (0.0787 in) of height over a 100 m (328 ft) of horizontal distance.

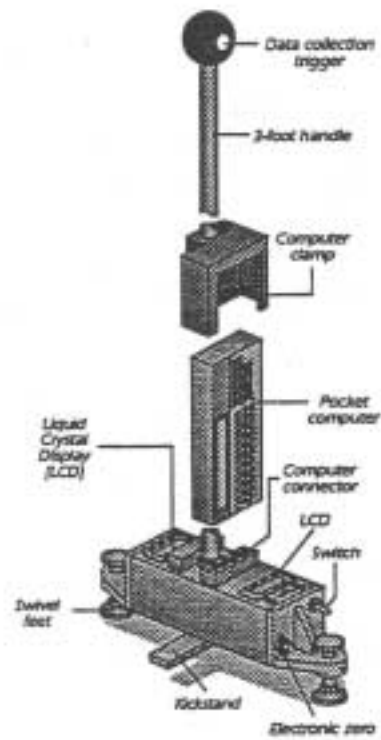


Figure 26. Face Technologies Dipstick.

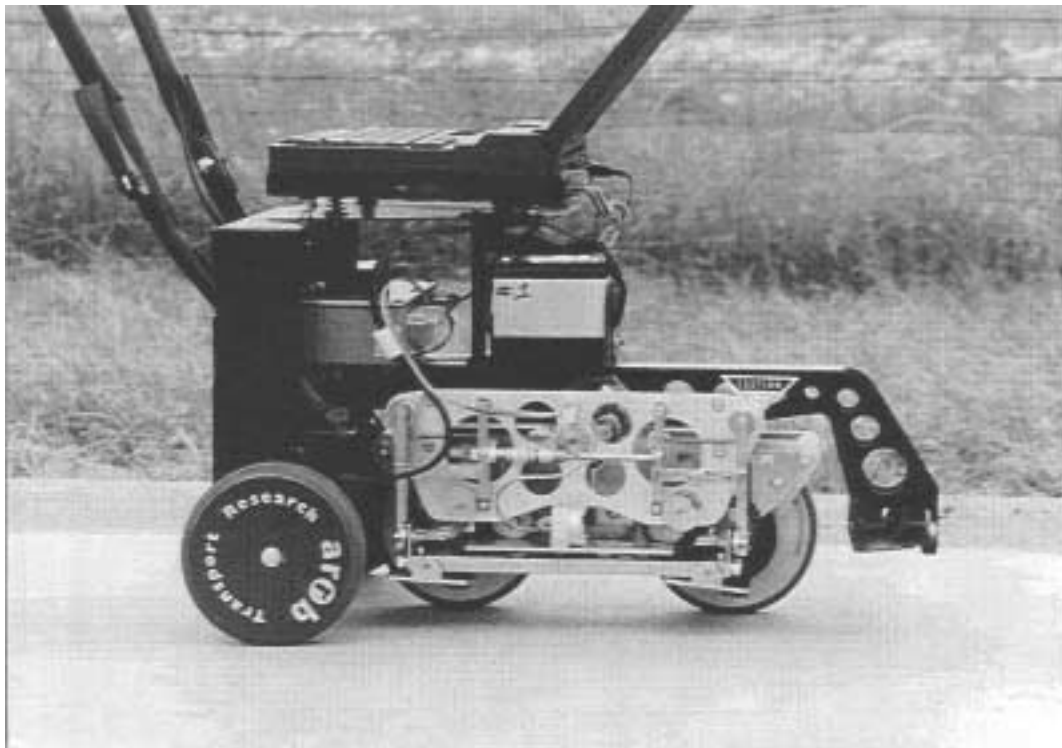


Figure 27. Walking profiler – ARRB.

## CHAPTER 5

### PROFILE INDICES

#### INTRODUCTION

The three most common profile indices that are currently in use are the International Roughness Index (IRI), Ride Number (RN) and Profile Index (PI).

The IRI is the most widely used profile index in use today to assess the roughness of highway networks. The IRI was developed in the early 1980s as a part of a study that was sponsored by the World Bank (22). Many States have adopted IRI as the parameter for monitoring the roughness of their highway network. The Federal Highway Administration (FHWA) uses data collected at the Highway Performance Monitoring System (HPMS) sections that are located throughout the United States to track the condition of the highway system in the United States. The FHWA requires the States to submit the roughness of the HPMS sections in IRI.

The Ride Number is a profile statistic that was first developed during the early 1980s as a result of research conducted by Janoff for two NCHRP studies (23, 24). In 1995, the University of Michigan developed a revised version of RN (25). The RN is a parameter that is closely related to the rating that is given by road users to a section of roadway.

Many States have specifications for acceptance of new pavements. The most common equipment that is currently used for measuring new pavements is the profilograph. The profile trace measured by the profilograph is used to compute the Profile Index (PI) of the pavement. The PI is then used as the parameter for acceptance of new construction. The PI is also used to determine incentives and disincentives. In recent years, many States as well as contractors have adopted lightweight profilers to measure new pavements. These devices record the profile of the pavement. A computerized profilograph simulation can be performed on the profile data to obtain the PI of the pavement section.

Currently, most highway agencies use the PI that is obtained by profilograph measurements or simulation to judge the quality of a new pavement. Thereafter, they use a profile statistic such as IRI to monitor the condition of their pavement network. It has been shown that there is little correlation between PI and the IRI. As two different profile indices are used for measuring the smoothness of new pavements and to subsequently monitor the roughness of the pavement, it is difficult to relate the roughness of the pavement at some point in time with its as-constructed smoothness. Currently, some State agencies are moving towards adopting a consistent measure of pavement smoothness that can be used throughout the life cycle of a roadway. This involves using the same index for measuring new pavements for acceptance and for subsequent monitoring for pavement management purposes. Some States have adopted IRI as this index. Moving towards such a procedure would provide a roughness statistic that can be used to monitor the performance of a pavement from “cradle to grave”.



## **INTERNATIONAL ROUGHNESS INDEX**

### **Background**

Almost every automated road profiling system includes software to calculate the IRI. Since 1990, the FHWA has required the States to report roughness on the IRI scale for HPMS sections. The world bank sponsored several large-scale research programs in the 1970's that investigated some basic choices facing developing countries: should the governments borrow money to build good quality expensive roads, or should they save money with roads of lesser quality that are cheap? Road roughness was identified as a primary factor in the trade-off involving road quality vs. user cost. The study found that roughness data from different parts of the world could not be compared. Even data from the same country were suspect because the measurements were based on hardware and methods that were not stable over time.

In 1982, the World Bank initiated a correlation experiment in Brazil to establish correlation and a calibration standard for roughness measurements (25). In processing the data, it became clear that nearly all roughness measuring instruments in use throughout the world were capable of producing measures on the same scale, if that scale was suitably selected. From that point on, an objective of the researchers was to develop the IRI, which would be such a scale. The IRI is the first widely used profile index where the analysis method is intended to work with different types of profilers. It is defined as a property of the true profile, and therefore it can be measured with any valid profiler. At the time of its development, response type road roughness measuring systems were common, so the index was tailored to correlate well with the output of these systems.

### **Properties of the IRI Analysis**

The computation of the IRI is based on a mathematical model called a quarter car model. The mathematical model calculates the suspension deflection of a simulated mechanical system with a response similar to a passenger car. The simulated suspension motion is accumulated and then divided by the distance traveled to give an index with units of slope (m/km or in/mi). The mathematical simulation that is carried out by the computer program is shown schematically in figure 28. The quarter car model used in the IRI algorithm is just what its name implies: a model of one corner (a quarter) of a car. As shown in figure 28, the quarter car is modeled as: one tire that is represented with a vertical spring, the mass of the axle supported by the tire, a suspension spring and damper, and the mass of the body supported by the suspension for that tire.

When the IRI was developed, the quarter car model was tuned to maximize correlations with response type road roughness measuring systems. The quarter car simulation was meant to be a theoretical representation of the response type systems in use at the time the IRI was developed, with the vehicle properties of the quarter car adjusted to obtain maximum correlation to the output of those systems.

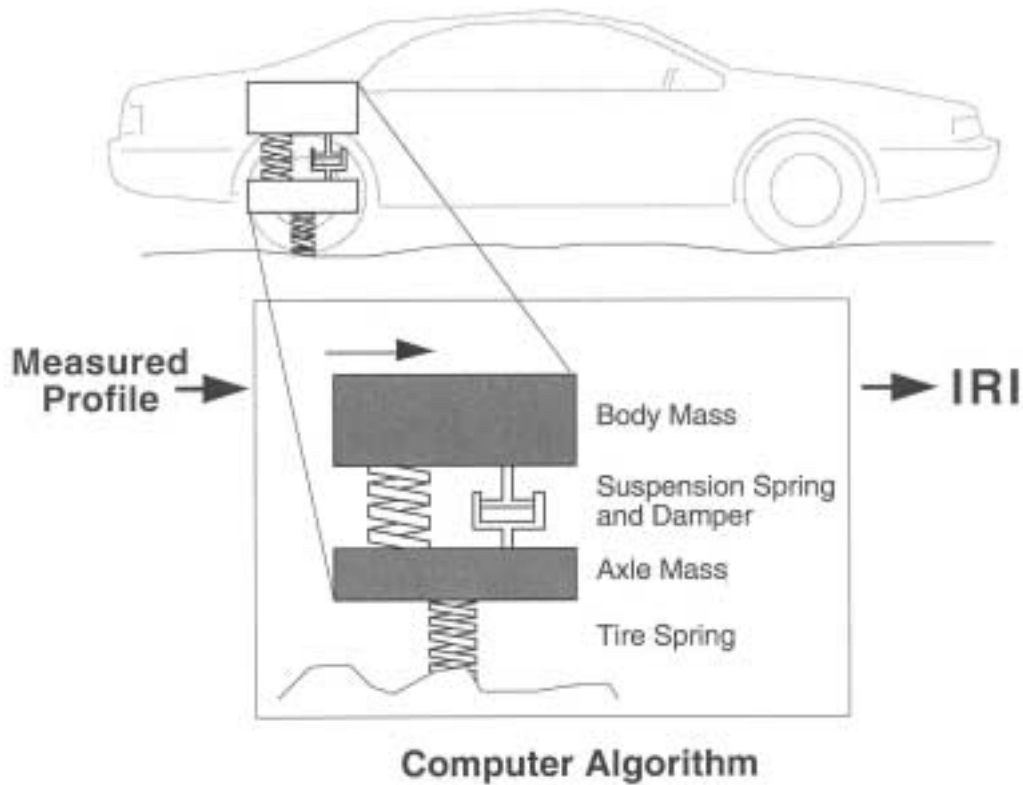


Figure 28. Illustration of computer algorithm used to compute IRI (13).

The response of the IRI to sinusoids is intentionally very similar to measured physical response of highway vehicles. It was mainly developed to match the responses of passenger cars, but subsequent research has shown good correlation with light trucks and heavy trucks. The IRI has become recognized as a general-purpose roughness index that is strongly correlated to most kinds of vehicle response that are of interest. Specifically, IRI is highly correlated to three vehicle response variables that are of interest: road meter response (for historical continuity), vertical passenger acceleration (for ride quality), and tire load (for vehicle controllability and safety).

The wave number response of the IRI quarter car filter is shown in figure 29. The amplitude of the output sinusoid is the amplitude of the input, multiplied by the gain shown in the figure, which is dimensionless. The IRI is influenced by wavelengths ranging from 1.2 to 30.5 m (4 to 100 ft). The IRI filter has maximum sensitivity to slope sinusoids with wave numbers near 0.065 cycle/m (a wavelength of 15.4 m) and 0.42 cycle/m (a wavelength of 2.4 m). The response is down to 0.5 for 0.033 and 0.82 cycle/m wave numbers, which correspond to wavelengths of 30.5 m and 1.2 m, respectively. However, there is still some response for wavelengths outside this range.

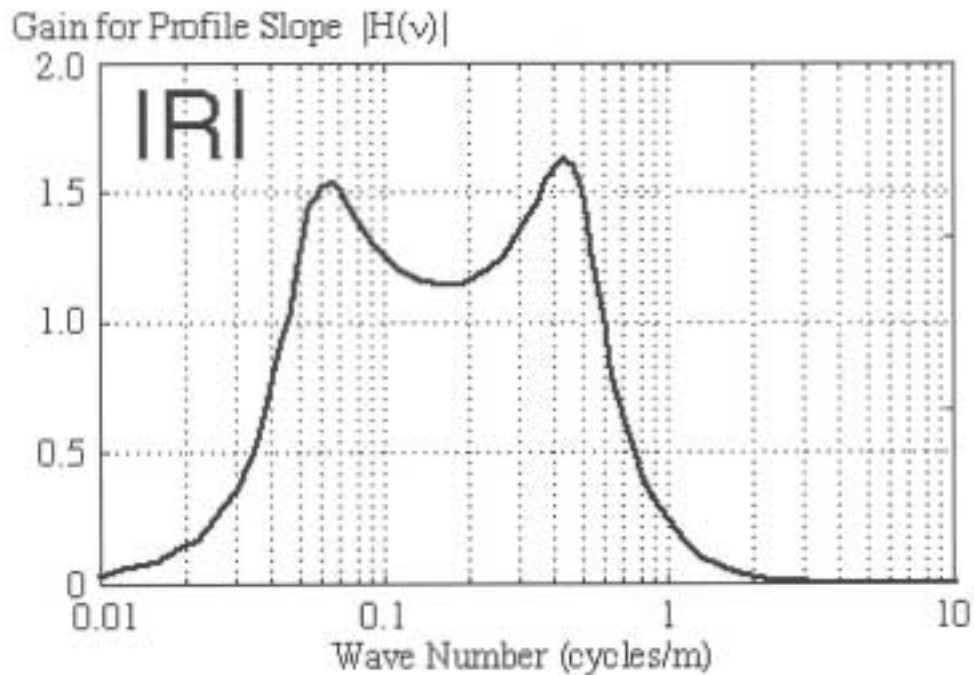


Figure 29. Response of IRI filter (13).

### Computation of IRI

The computation of IRI from profile data is performed by a computer program. The ASTM standard E 1926-98, “Standard Practice for Computing International Roughness Index of Roads from Longitudinal Profile Measurements” presents the computer program that should be used to compute IRI (16). The specific steps that are taken in the computer program to compute IRI are:

1. The IRI is calculated for a single profile: The IRI computation is performed for profile data that is obtained for a specific profile path. Most profilers collect data along the two wheel paths. The IRI computation should be carried out separately for each wheel path.
2. The profile is filtered with a moving average having a 250 mm (10 in) base length: The moving average is a low pass filter (it attenuates short wavelengths) that smoothens the profile. The moving average filter is applied by the computer program. The 250 mm (10 in) moving average filter should be omitted if the profile has already been filtered by a moving average or with an anti-aliasing filter whose cut-off attenuates wavelengths shorter than 0.6 m (2 ft). For example, there are some K.J. Law profilers in operation that obtain profile measurements at 25 mm (1 in) intervals, then apply a 300 mm (12 in) moving average and then store the results at 150 mm (6 in) intervals. It is important to skip the 250 mm (10 in) moving average filter when processing profiles from such systems.

3. Quarter car simulation is performed on the profile: The parameters of the quarter car are defined in the IRI program. The quarter car simulation on the profile is performed for a simulated speed of 80 km/h (50 mph), and the suspension motions of the quarter car are accumulated.
4. Compute IRI: The absolute values of the suspension motion that is obtained from the simulation are summed and then divided by the profile length to obtain the average suspension motion over the simulated length. The value that is computed is the IRI, and has units of slope with the most common units being inches per mile or mm per km.
5. Mean IRI: The IRI is obtained for a specific profile path. Most profilers collect profile data along two wheel paths. The IRI value can be computed for each wheel path. The average of these two values is referred to as the mean IRI, and presents an overall view of the roughness of the roadway.

## **Equipment Requirements**

Analysis has shown that for computation of IRI, the sample interval of a profiler must be 167 mm (6.7 in) or less (2). If the profile data is sampled at an interval of 167 mm (6.7 in) or less, and are then averaged prior to saving, the recording interval must be 250 mm (10 in) or less in order to accurately compute the IRI (2).

## **Half-Car Roughness Index**

The half-car roughness index (HRI) is the IRI algorithm applied to the average of the left and right wheel path profiles. The half-car analysis more closely matches the way road meters are installed in passenger cars. The roughness as calculated with an HRI analysis is less than or equal to the result obtained from the IRI analysis. When the HRI values, calculated for the average of the left and right profiles were compared to the mean IRI (average IRI of left and right wheel paths), there was a very high correlation between the HRI and mean IRI (13). This indicates that little or no additional information is provided by the HRI. The relationship between the IRI and HRI is shown in figure 30.

## **RIDE NUMBER**

### **Background**

For decades, highway engineers have been interested in estimating the opinion of the traveling public on the roughness of roads. The PSI scale from the AASHO Road Test has been of interest to engineers since its introduction in the 1950s. Ride number is a profile index intended to indicate rideability on a scale similar to PSI. The longitudinal profile measurements taken with a profiler are processed using a computer program to obtain the RN, which matches the mean panel rating of a rating panel.

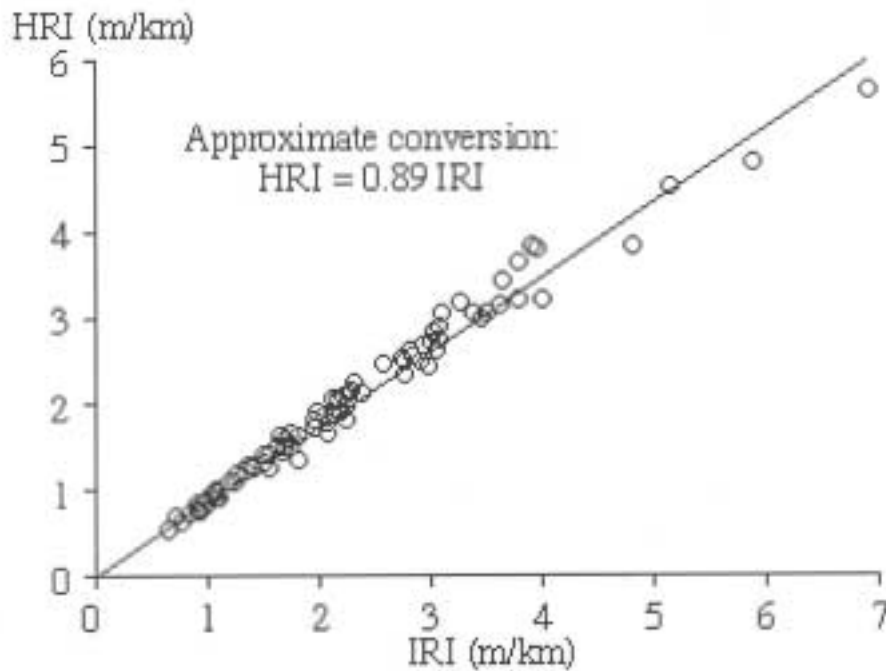


Figure 30. Relationship between IRI and HRI (13).

The NCHRP sponsored two research projects by Dr. Michael Janoff in the 1980's that investigated the effect of road surface roughness on ride comfort (23, 24). The objective of that research was to determine how features in road profiles were linked to subjective opinion about the road from members of the public. During two studies, spaced at about a 5-year interval, mean panel ratings (MPR) were determined experimentally on a 0 to 5 scale for test sites in several States. Longitudinal profiles were obtained for the left- and right-hand wheeltracks of the lanes that were rated. Profile-based analyses were developed to predict MPR. A method was developed in which power spectral density (PSD) functions were calculated for two longitudinal profiles and reduced to provide a summary statistic called PI (profile index). The PI values for the two profiles were then combined in a nonlinear transform to obtain an estimate of MPR. The mathematical procedure developed to calculate RN is described in NCHRP Report 275 (23), but not in complete detail. Software for computing RN with the PSD method was never developed for general use. It should be noted that there is no relationship between the PI used in Ride Number computations with the PI that is obtained from the reduction of profilograph traces.

In 1995, some of the data from these two NCHRP projects and a panel study conducted in Minnesota were analyzed again by University of Michigan Transportation Research Institute (UMTRI) for a pooled-fund study initiated by FHWA (25). The objective was to develop and test a practical mathematical process for obtaining RN. The profile data in the original NCHRP research were obtained from several instruments. Most measurements were made with a K.J. Law profiler owned by the Ohio Department of Transportation, and were thought to be accurate. A few other test sites were profiled with instruments whose validity has been questioned. The new analyses were limited to 138 test sites that had been profiled with the Ohio system, and the

data from the Minnesota study. Based on the analysis of this data, a new profile analysis method to compute the RN was developed (25). This procedure predicts MPR slightly better than previously published algorithms. More importantly, it is portable. The software was tested on profiles obtained from different systems on the same sites, and similar values of RN were obtained.

### **Properties of the Ride Number Analysis**

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

Figure 31 shows the sensitivity of PI for a slope sinusoid. When a sinusoid is given as an input, the PI filter produces a sinusoid as the output. The amplitude of the output sinusoid is the amplitude of the input, multiplied by the "gain" shown in the figure. The maximum sensitivity is for a wave number of 0.164 cycle/m, which is a wavelength of about 6.1 meters (20 ft).

The content of a road profile that affects RN is different from the content that affects IRI. The IRI has the greatest sensitivity to sinusoids with a wave number of 0.065 m/cycle, which corresponds to a wavelength of 15.4 m. Figure 31 shows that the ride number has a low sensitivity to that wavelength and even lower sensitivity for longer wavelengths. The IRI and RN will not always correlate the same way, and do not have the same meaning. Thus, they each provide unique information about the roughness of the road. The RN values for adjacent sections of profile cannot be averaged. For example, if one mile has an RN value of 3 and the next has a RN of 4, the RN for the two-mile segment is not 3.5, but 3.37.

### **Computation of Ride Number**

The ASTM standard E 1489-98 (6), "Standard Practice for Computing Ride Number from Longitudinal Profile Measurements Made by an Inertial Profile Measuring Device" presents the computer program that should be used to compute RN. The specific steps that are taken in the computer program to compute RN are:

1. The RN is calculated for a single profile: The RN computation is performed for profile data that is obtained for a specific profile path. Most profilers collect data along the two wheel paths. The procedure for computing an overall RN for the roadway from the profile data obtained from two wheel paths is described later in this section.
2. The profile is filtered with a moving average having a 250 mm (10 in) base length: The moving average is a low pass filter (it attenuates short wavelengths) that smoothens the profile. The moving average filter is applied by the computer program. The 250 mm (10 inch) moving average filter should be omitted if the profile has already been filtered by a

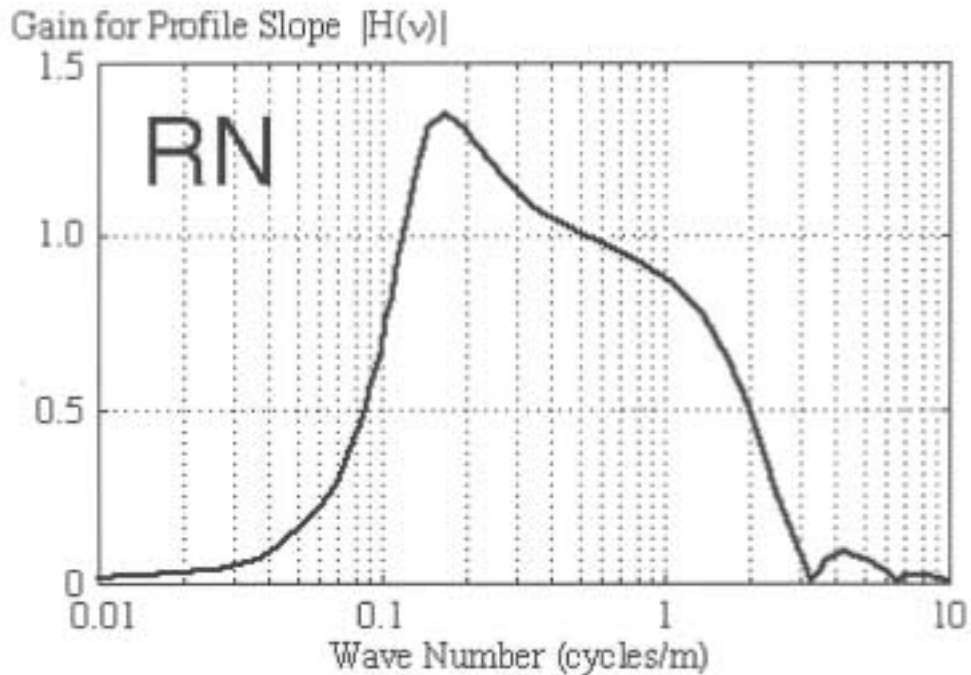


Figure 31. Sensitivity of PI for a slope sinusoid (13).

moving average or with an anti-aliasing filter whose cut-off attenuates wavelengths shorter than 0.6 m (2 ft). For example, there are some K.J. Law profilers in operation that obtain profile measurements at 25 mm (1 in) intervals, then apply a 300 mm (12 in) moving average and then store the results at 150 mm (6 in) intervals. It is important to skip the 250 mm (10 in) filter when processing profiles from such systems.

3. Profile is further filtered with a band pass filter: The filter uses the same equations as the quarter car model in the IRI. However, different coefficients are used to obtain the sensitivity to wave number as shown in figure 31.
4. Filtered Profile is reduced to give PI: The filtered profile is reduced to yield a root mean square (RMS) value called the Profile Index (PI), that has units of dimensionless slope (ft/ft, m/m etc.)
5. PI is transformed to RN: The RN is defined as an exponential transform of PI according to the equation:  $RN = 5e^{(-160PI)}$

If a single profile is processed, its PI is transformed into RN as shown in the above equation. If two profiles for both left and right wheel paths are processed, the PI values for the two wheel paths are combined according to the following equation:

$$PI = \left[ \frac{(PI_L^2 + PI_R^2)}{2} \right]^{1/2}$$

$PI_L$  and  $PI_R$  are the PI for the left and the right wheel paths, respectively. Thereafter, the PI values are transformed into the Ride Number using the following equation:

$$RN = 5e^{(-160PI)}$$

## Equipment Requirements

Analysis has shown that for computation of RN the profile data has to be obtained at a sample interval of 50 mm (2 in) or less (2). If the profile data are sampled at an interval of 50 mm (2 in) or less and then averaged prior to saving, the recording interval must be 75 mm (3 in) or less in order to accurately compute the RN (2). Profiles obtained by ultrasonic profilers are not valid for obtaining RN. Research results have shown that most profiles obtained with ultrasonic systems give incorrect results (2). For these profiles, the computed PI values are too high, leading to RN values that are too low.

## PROFILE INDEX

Profilographs have been widely used to measure the smoothness of new pavements. The profilograph provides a trace of the pavement profile, which is reduced to obtain a parameter called the Profile Index (PI), which is used to judge the smoothness of the pavement. It should be noted that the PI obtained from a profilograph trace has no relation to the PI that is used in the calculation of RN.

Up to about mid 1980's all profilographs in use were mechanical profilographs. The trace obtained by mechanical profilographs was reduced manually by a technician. As the reduction of the profilograph trace is subjective depending on the rater, there can be variability in the results that are obtained. In the mid 1980's computerized profilographs were introduced. These profilographs recorded the profile data, and reduce the data using a computer program to compute PI. This eliminated the subjectivity of profilograph trace reduction that occurs when raters are used to reduce profilograph traces. The next major development in profilograph trace reduction occurred when the Proscan system was developed by Devore (26). In the Proscan system, the trace recorded by a mechanical profilograph is scanned and the data reduction is performed by a computer program that computes the PI. This procedure eliminates the subjectivity that occurs with raters.

It should be noted that there is no universal standard that is followed by all State agencies in reducing profilograph traces. Each State agency will have their own standardized procedures for reducing the profilograph trace. Therefore, comparisons of PI values between States may not be meaningful, as there could be differences in procedures that are used to reduce profilograph traces. California, which have had extensive experience with the use of profilographs use the procedures described in California Test Method 526 for reducing profilograph traces.

Following is the general procedures that are followed in reducing the profilograph trace to obtain the PI. In computerized profilographs and Proscan system, this procedure is performed by a computer program.



## 1. Outline the Profile Trace

The first step in trace reduction is outlining of the trace. A medium point red pen or other contrasting color is typically used to outline the trace. Outlining consist of drawing a new profile line through the midpoint of the spikes of the field trace as shown in figure 32. The purpose of outlining is to average out spikes and minor deviations caused by rocks, texture, dirt or transverse grooving. Outlining is one of the very subjective aspects of trace reduction, both in the width of the line drawn and the manner in which it averages the field trace. Outlining the trace expedites the trace reduction process. Outlining was not a part of the original development work for the blanking band concept and is currently not included in the California Test Method 526. It is assumed that this was an enhancement to the method many agencies adopted through the years to reduce variability and expedite trace reduction. It is reported that outlining the field trace prior to trace reduction can reduce the PI by 1 to 2 inches per mile (16).

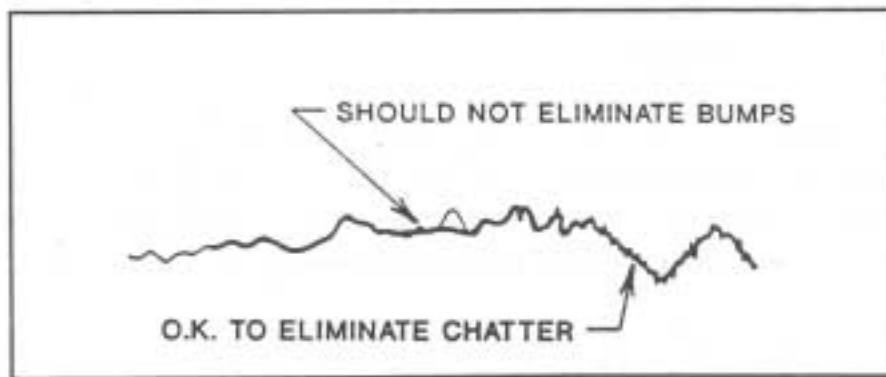


Figure 32. Example of an outlined trace (16).

## 2. Position Blanking Band

The next process in trace reduction is to place the blanking band on the profile trace. The blanking band is made of plastic, and is 21.12 inches long and 1.7 inches wide. This band represents a pavement length of 528 feet (i.e., scale of 1:25) in horizontal direction and is a true scale in the vertical direction (i.e., 1:1). At the center of the scale there is an opaque band 0.2 inches wide extending for the full length of the scale. Parallel to the opaque band on both sides are five scribed lines 0.1 inches apart. These lines serve as the scale to measure the distance excursions or scallops that extend above or below the opaque blanking band. The blanking band is placed over the profile trace so that it blanks out as much of the profile as possible. The placement of the blanking band should be such that the excursions or scallops are evenly distributed above and below the blanking band. When reducing a trace longer than 0.1 mile, it is important to look several segments ahead so that the blanking band is positioned in the best position for the entire trace. This may require some trial and error before the optimum reduction has been performed. Once the correct location of the individual segment blanking band positions is established, the corners of the blanking band should be marked. This allows proper alignment with the next segment (if required) and facilitates checking of the results.

Two factors involving the positioning of the blanking band can affect the results. The first is the user selected position. Since judgment is involved in positioning the template, there is not a unique solution to the problem. The level of training and experience can significantly affect the placement and subsequent results. The second factor involves interpretation of specifications. This can vividly be demonstrated with the example shown in figure 33. The upper portion of the figure indicates one method of moving from section to section across the trace. In this instance, the end of the last section evaluated is used as the guide to which the current section must be aligned. The lower section of the figure indicates how two adjacent sections can be treated independently.

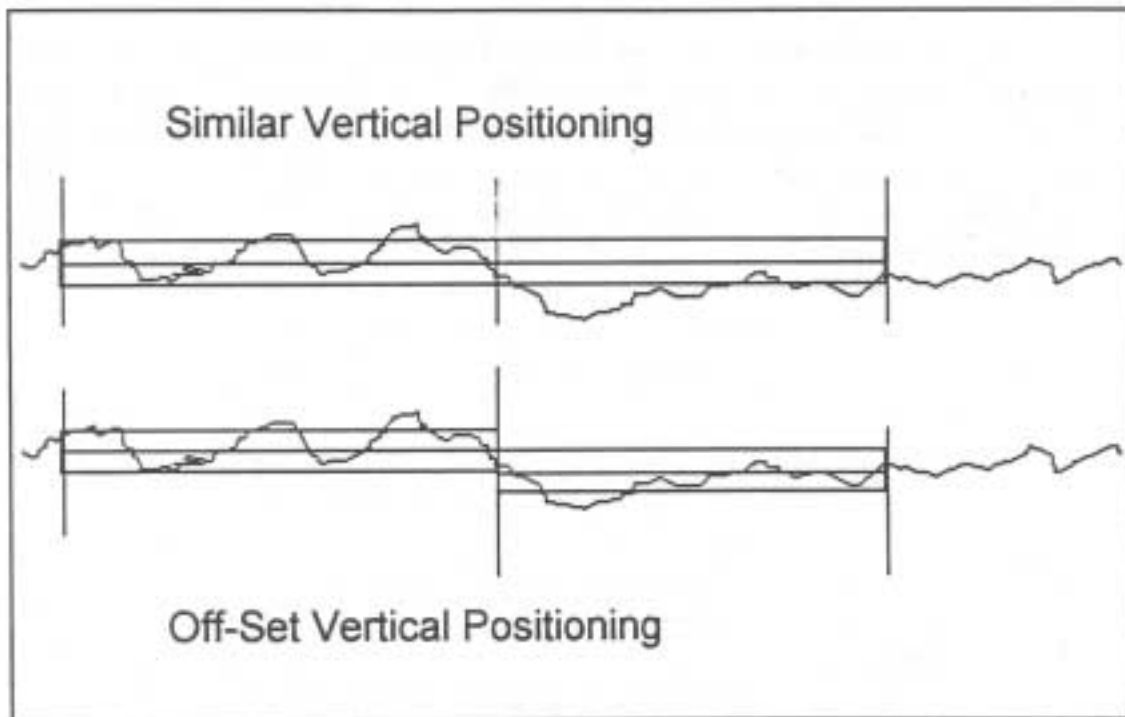


Figure 33. Examples of blanking band position when moving between profile sections (16).

Using the first technique requires considerably more judgment and expertise since decisions regarding the position of the blanking band can affect the position of the blanking band in future sections to be evaluated. The second method requires less experience and allows more latitude in determining the best fit to the data with the blanking band. Both these approaches are currently used in the industry and can significantly affect results. The current California 526 test method does not call for vertical alignment between adjacent sections. The origin of the method which requires vertical alignment between sections is not known. However, it probably resulted from the need to evaluate excursions that begin at the end of one section and end within the next. Although test procedures commonly state that the excursion must only be counted once, if two different vertical positioning schemes are used, the excursions may not be counted at all simply because it was located between sections. It is conceivable that a significant bump could go undetected, or could be computed in a section, which was already near the specification limit and make it exceed specifications.

A blanking band is typically 0.2 inches wide but some agencies have used 0.1 inch wide bands, while some use zero blanking bands. In 1990 Kansas experienced a problem with short wavelength roughness. One project exhibited a cyclic 8 ft wavelengths of approximately 0.2 inch in amplitude. Although the profilograph determined that the pavement roughness met specifications, the ride quality of the pavement was poor. As a result of this problem, the Kansas DOT eliminated the blanking band and developed specifications using a zero blanking band concept. The evaluation of traces from Rainhart profilographs is performed in a similar manner as profile traces obtained from the California profilograph, except that a blanking band of 0.1 inches is used instead of a 0.2 inch blanking band.

### 3. Determine Profile Index

Excursions which extend in height more than 0.03 inches above the blanking band for at least 0.08 inches in horizontal distance (i.e., 2 ft on the pavement) will be recorded on the profile and rounded to the nearest 0.05 inches. The sum of the recorded heights within a given segment will be the Profile Index (PI) for that segment. The profile index is expressed in terms of inches per mile.

The excursions are evaluated against five parallel lines scribed on both sides of the blanking band at one-tenth inch intervals. The relationship between the scribed lines on the blanking band and the trace are determined by naked eye. It should be noted that a 5 mm pencil is approximately 0.02 inches wide. A medium point pen and pencil of “average sharpness” is between 0.015 and 0.2 inches in width. It should be also noted that the recording pen on the profilograph does not always produce a line of constant width. Therefore, when attempting to measure to hundreds of an inch, vision acuity is challenged. As described previously, excursions extending 0.03 inches beyond the blanking band are recorded as roughness and rounded to the nearest 0.05 inches. One pencil width could be interpreted to be 0.00 inches and another to 0.05 inches even though the true profile was the same. There is no provision to record actual deviations as 0.03 or 0.04 inches.

The procedure for determining the Profile Index (PI) that was described in this section was based on U.S. customary units. Some State agencies have adopted the metric system and are computing the PI for sections that are 0.1 km in length. Figure 34 shows an example of how the PI is computed from a profilograph trace.

## **CORRELATIONS BETWEEN PROFILE INDICES**

### **Correlations Between PI from Profilograph and PSI**

A study to evaluate the relationship between PSI and PI from profilographs was performed by University of Texas as part of FHWA Demonstration project 72 (16). Data collected at PCC sites were used in this study. The profile data at the test sections were collected using an inertial profiler, and relationships between PSI and output from the profiler was used to compute the PSI of the sections. Figure 35 shows the relationship between PSI and the PI

obtained from the California profilograph using a 0.2 inch blanking band. As seen in this figure, there is a fair amount of scatter in the relationship between PSI and PI.

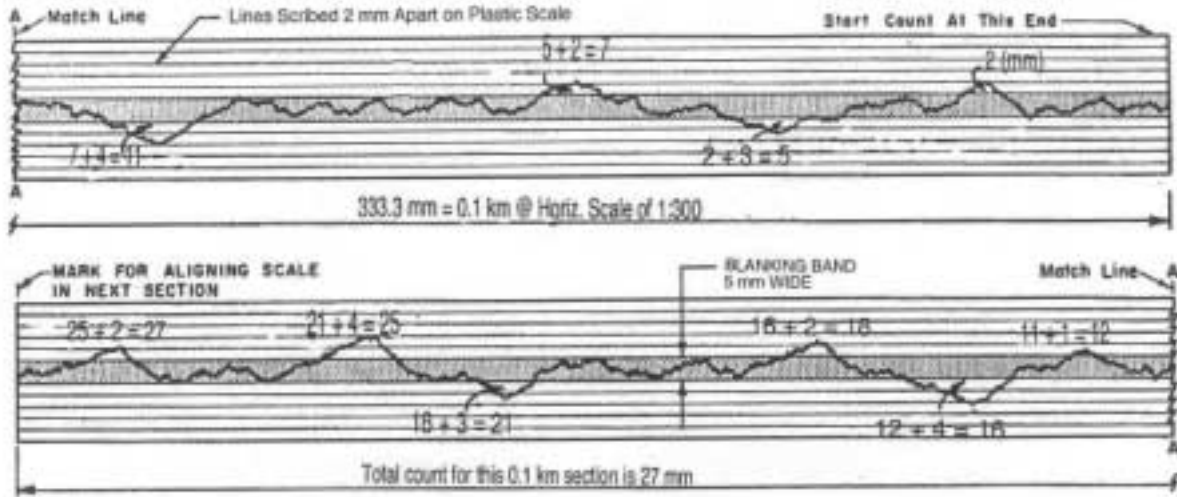


Figure 34. Examples showing method of determining profile index from a profilograph trace.

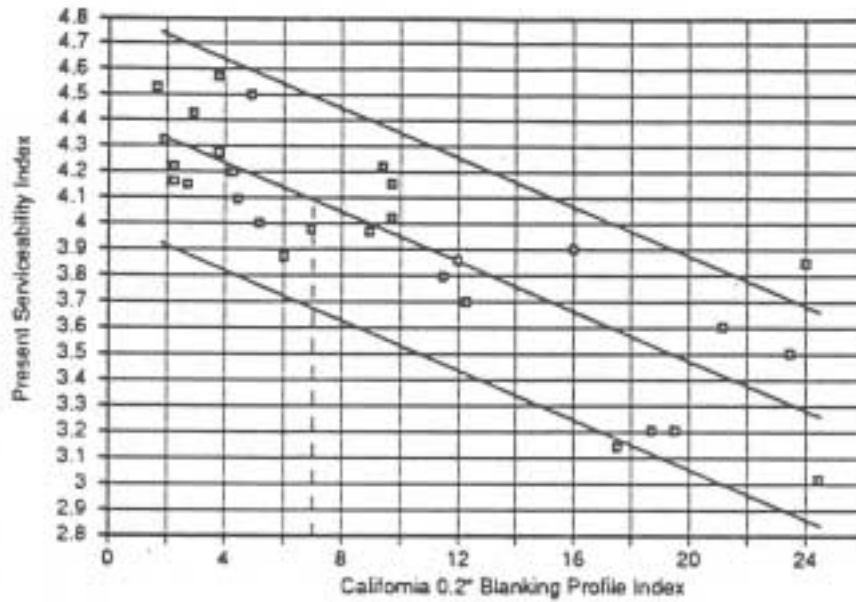


Figure 35. Relationship between PSI and PI from California profilograph (16).

## Correlations between PI and IRI

Fernando (27) performed a study to evaluate the relationship between PI obtained from profilograph traces and IRI. This study was performed using data that were collected at 48 overlaid test sections. These test sections were profiled using an inertial profiler. Thereafter, a profilograph simulation program was used on the profile data to obtain the profilograph trace. The profilograph trace was reduced using the Proscan program (26). The IRI values of the test sections were computed using the profile data obtained from the inertial profiler.

A comparison between the PI and the IRI values showed that the null blanking band PI correlates better with the IRI, when compared to the PI obtained using a 5 mm (0.2 in) blanking band. Figure 36 shows the relationship between PI and IRI when the PI values were obtained using a 5 mm (0.2 in) blanking band. The data does not indicate a clear relationship between the two parameters.

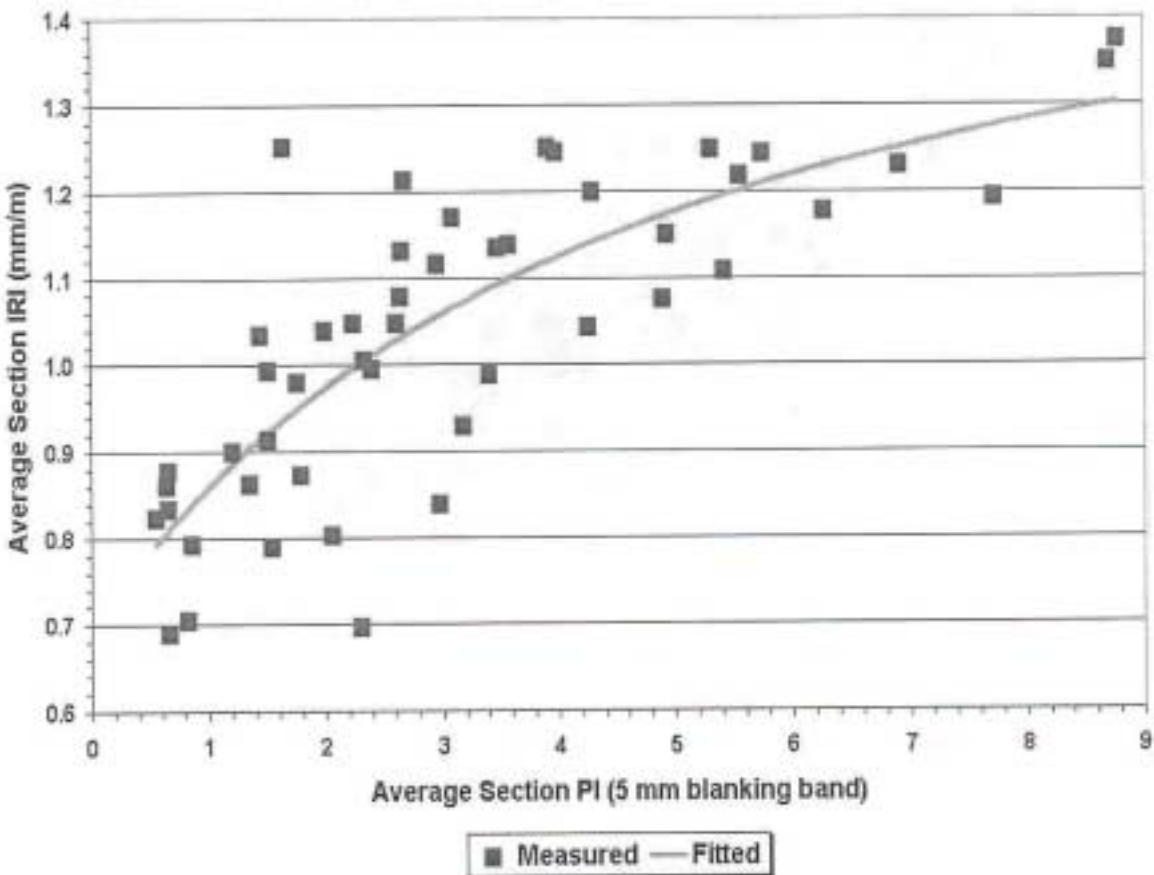


Figure 36. Relationship between IRI and PI based on a 5 mm blanking band (27).

Figure 37 shows the relationship between PI and IRI when a null blanking band is used for the computation of PI. As shown in this figure, the correlation between PI and IRI is much better when the null blanking band is used to reduce the profile trace. It is possible that the 5 mm (0.2 in)

blanking band masks certain components of roughness that are otherwise picked up if no blanking band is used. This can account for the better correlation between IRI and PI when the PI was obtained with a null blanking band.

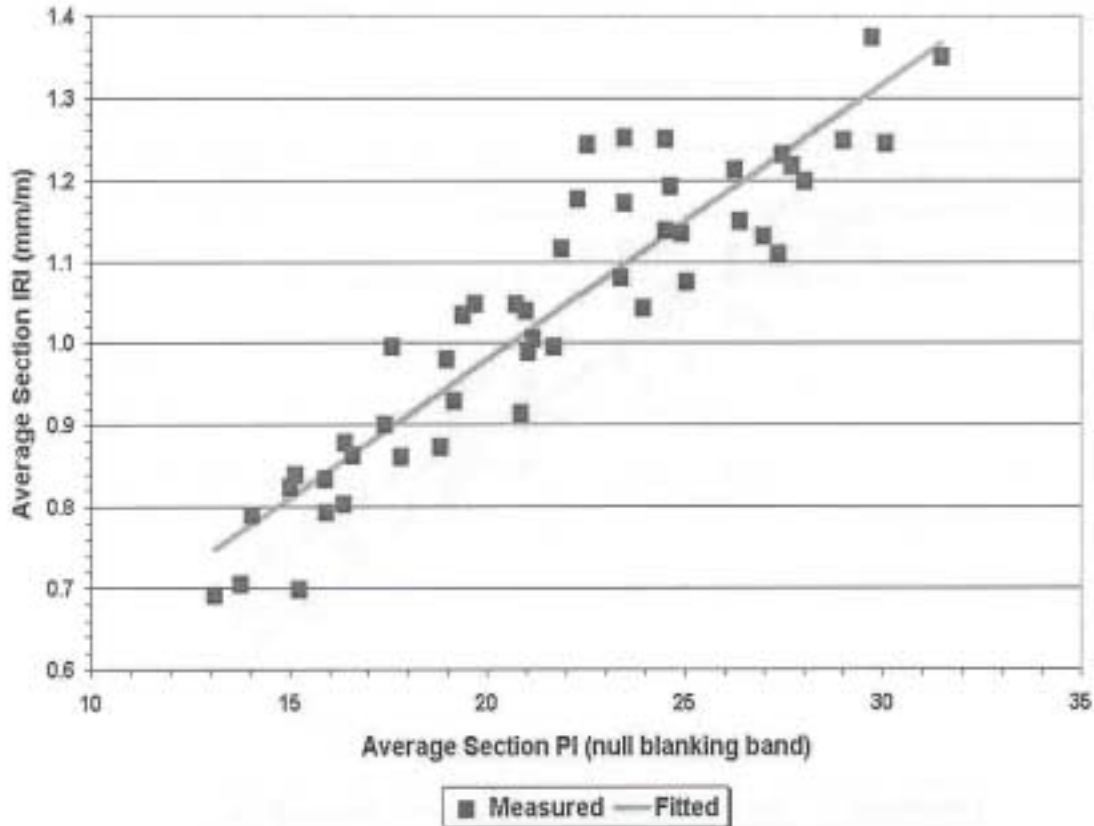


Figure 37. Relationship between IRI and null blanking band PI (27).

Ksaibati et al. (28) performed a study to investigate the relationship between PI and IRI of pavements. They used data from eight construction projects for their analysis. Each construction project was analyzed separately. Test sections in each construction project were divided into three groups: Group 1 - PI less than 3, Group 2 - PI between 3 and 5, and Group 3 - PI greater than 5. The IRI values for these test sections were generally obtained for each year over a four-year period after construction. The IRI values obtained for each year were used to perform statistical tests to investigate if there were differences in IRI values between the three groups. The statistical tests indicated there were no differences between the three groups. This means the IRI of test sections that had PI greater than 5 were similar to IRI of pavements that had PI less than 3, over a four period after construction.

## CHAPTER 6

### OPERATIONAL CHARACTERISTICS OF PROFILOGRAPHS

#### WAVELENGTH EFFECTS ON PROFILOGRAPH MEASUREMENTS

There have been questions raised about how well a profilograph measures wavelengths that are related to ride quality. Profilographs are known to amplify and attenuate the true pavement surface profile. This calls for question the suitability of using profilograph data for construction control and suggests the need for refinement in evaluation procedures.

Kulakowski and Wambold (19) reported that profilographs have varying response to wavelengths present on roadways. They report that profilographs measure some wavelengths correctly, amplify some wavelengths, and that some wavelengths are hardly measured. Figure 38 shows the actual and desired frequency response of a 12 wheel California Profilograph (19). As shown in this figure, the California profilograph gives a poor measurement for wavelengths between 3 to 4.6 m (10 to 15 feet), and amplifies the response for wavelengths between 6.1 to 12.2 m (20 to 40 ft) range by as much as two times.

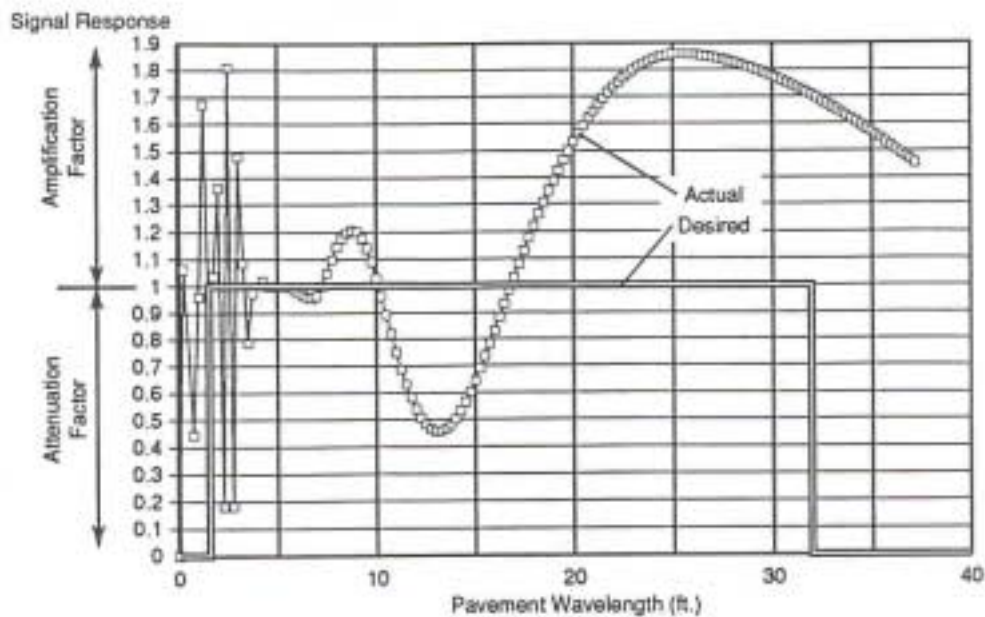


Figure 38. Desired and actual frequency response of 12-wheel California style profilograph (19).

#### CALIBRATION OF EQUIPMENT

The profilograph must be properly calibrated in order to obtain accurate measurements. The profilograph must be calibrated vertically as well as horizontally. The horizontal scale can be

checked by running the profilograph over a known distance and scaling the results on the profilogram. Typically profilographs are calibrated on premeasured sections ranging in length from 30 to 305 m (100 to 1000 ft). Horizontal calibration to 0.2% of the measured distance is typically specified. This allows approximately 0.3 m (1 ft) of measurement error or less in 161 m (528 ft) during calibration. The vertical scale is checked by putting a board of known thickness on the pavement and running the profilograph over the board and scaling the result on the profilogram. Most agencies employ calibration blocks ranging in height from 12 to 25 mm ( $\frac{1}{2}$  to 1 in) to perform this operation. Vertical measurement tolerances of 0.5 to 0.8 mm (0.02 to 0.03 in) are considered to be acceptable.

## **MEASUREMENT PROCEDURE**

Most States require profile traces to be obtained in the wheel path or at some prescribed location parallel to the centerline. Some agencies test one path per lane while others test two. Typically the profile index is obtained for two wheel paths and averaged and reported as the lane profile index. The profilograph is pushed along the specified path, and about 3.2 to 4.8 km (2 to 3 miles) can be measured in an hour.

## **COMPUTERIZED PROFILOGRAPHS**

When the computerized profilographs were introduced, there were concerns raised regarding differences in PI values between computerized and mechanical devices. The computerized profilographs employ software filters to eliminate the high frequency noise found in the profile trace caused by pavement texture and vibrations that are not indicative of roughness. These filters are supposed to provide a function very similar to outlining with manual traces, in that they eliminate deviations that were not indicative of roughness. When an investigation of differences in PI between computerized and mechanical profilographs was performed, it was found that these differences were caused by the software filters that were employed by computerized profilographs. This investigation was performed by Michigan Department of Transportation, who recommended the first order linear filter that was being used in the profilograph be replaced with a third order Butterworth filter (16).

A survey conducted in 1994 by Smith et al. (10) indicated that a variety of filtering methods was being used by different agencies. The results of this survey are shown in table 1, which indicates the percentage distribution of the filter types that were being used. The differences in filter setting can affect the PI that is obtained from a profilograph.

## **ERRORS IN PROFILOGRAPH MEASUREMENTS**

Eccentricity of the measuring wheel, tire effects and lateral wander can introduce errors during profilograph measurements.



Table 1. Breakdown of filter and setting types used with computerized profilograph devices (10).

Filter Type	Percentage of Agencies (%)
Cox, 1st order	32
Butterworth, 3rd order	29
Chebyshev, 3rd order	5
Moving average	5
Other	29

### Eccentricity of Measuring Wheel

Through computer simulation, it has been shown that eccentricity of the measuring wheel can have a significant effect on the PI (17). An eccentricity in a wheel occurs when the wheel is suspended at a point displaced from its geometric center. The effect of eccentricity on the PI is shown in figure 39. This effect is different from that resulting from an out of round tire, which is presumably more prevalent. Eccentricity always increases roughness. An eccentricity of less than 3 mm (1/8 in) increased the roughness from four inches per mile to twenty inches per mile.

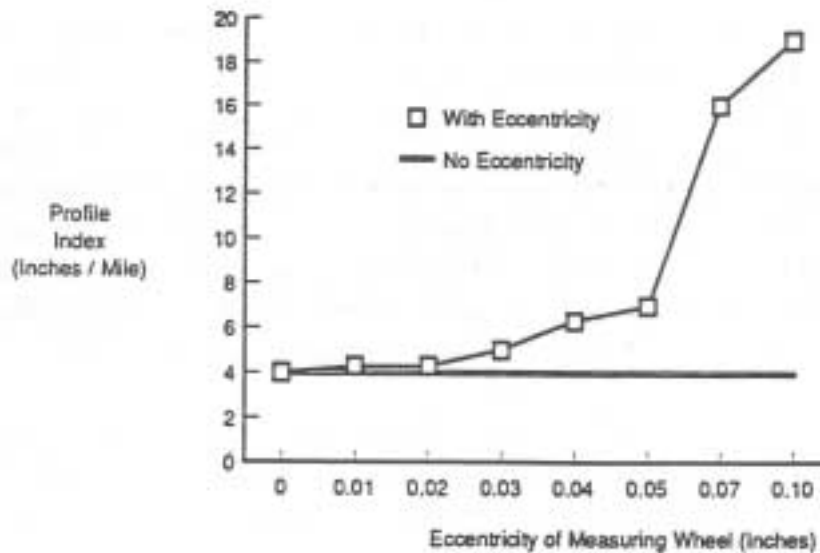


Figure 39. Effect of measuring wheel eccentricity on profile index (17).

### Tire Effects

Most manufacturers request that tire replacement be performed by them or that new tires once mounted on the rim be machined to guarantee their roundness. Although no research has been

performed on the effects of the tire itself being out-of-round, it appears certain that this is a critical factor and must be checked on a regular basis. Research indicates that tire wear has insignificant effect on the measured profiles (16).

### Lateral Wander

Lateral wander during profilograph measurements will measure the roughness along a path that is different from the specified path, and may introduce an error in distance due to side slipping of the measuring wheel. The wandering of a profilograph should not add sufficient distance to affect profile measurements. Table 2 indicates the additional distance, which would be traveled for the lateral wander values per 100 feet.

Table 2. Lateral wander effects on traveled distance.

Lateral Wander per 100 ft (ft)	Additional distance traveled in 528 feet (ft)
0.5	0.007
1	0.026
1.5	0.059
2	0.105
3	0.238

### VARIABILITY OF PROFILOGRAPH RESULTS

The variability in the PI obtained from a profilograph results from field variability and trace reduction variability. Field variability is a result of the operators inability to traverse the same path each time. However, much of the variability in profilograph results occurs because of variations in profilograph trace reduction procedures between raters.

In 1992, the Central Federal Lands Highway Division performed a study to evaluate variability between different raters (16). In this study profilograph measurements obtained at 19 sections were distributed to 25 raters. Figure 40 shows the ranges in PI values that were obtained for the sections by the raters. Manual trace reduction interpretations between operators agreed more closely on smooth sections than on rougher sections. The variability of PI values obtained with a mechanical profilograph has been found to be a function of roughness. Rougher roads result in a higher variability because of variability in trace reduction. Scofield (16) reports that mechanical profilograph test variability is generally never less than 0.5 inches per mile, and it is approximately one tenth the measurement range. Therefore, for a pavement having a roughness of 20 in/mile the standard deviation is generally about 2 in/mile. However, the trace reduction variability is not a factor for computerized profilographs, as well as for mechanical profilograph traces that are reduced using a computerized method such as Proscan (26) where the profilograph trace is scanned and reduced by a computer program.

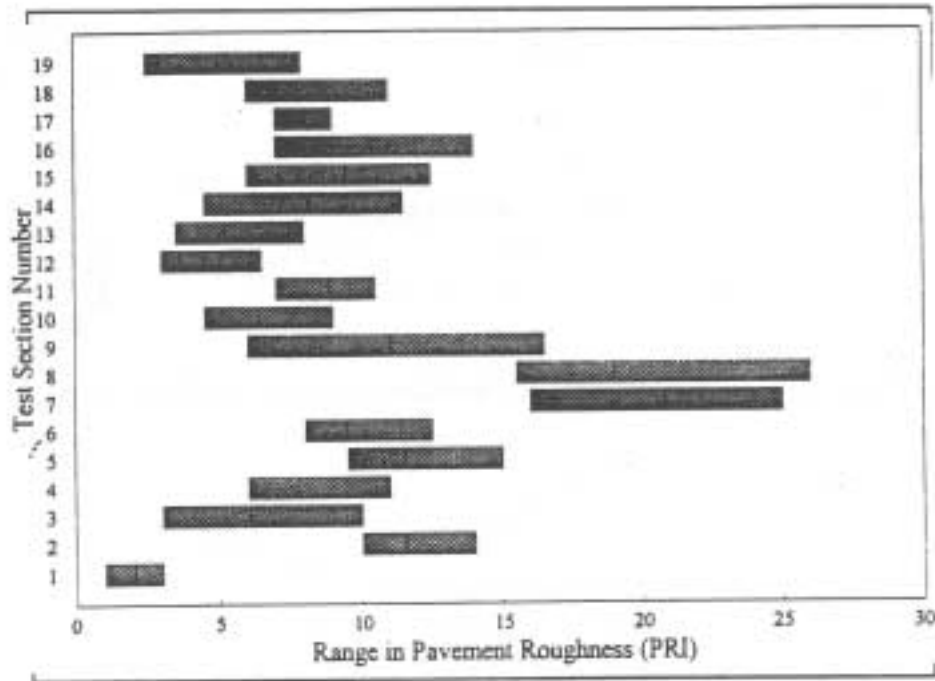


Figure 40. Range of PI values at test sections (16).

In computerized profilographs, the analysis of the profilograph trace is performed by the manufacturer's software. The computer programs reduce the profile and provide grind locations, profile index, and location of high points. The bump height and the blanking band width must be specified prior to analysis. The computer program centers the blanking band upon a best fit line through all the points within the section being reduced (normally 528 feet). Individual high points are located in accordance with the preprogrammed parameters. An advantage of the computerized profilographs is that the profile trace will always be reduced the same way, and will not have the variability associated with trace reduction performed by a human.

A computerized method for analyzing traces produced by non-computerized profilographs called Proscan that was developed by Devore (26) eliminated the subjectivity associated with trace reduction by humans. The Proscan system consists of a paper transport unit, scanner, and a software package. The Proscan system accepts profilograms from Rainhart and California type profilograph. The software is able to control items such as blanking band width, minimum scallop width, grind template height, and the standard reduction length to a value other than 0.1 mile. In this system, a profilogram is scanned and the profilogram trace data are saved in the computer's hard disk. During scanning, the profilogram is divided into 0.1 mile segments or to some other segment length prescribed by the user. After completion of data reduction, the PI for the segments and the location of defects (bumps and dips) are printed out. Data may be reanalyzed using different reduction parameter in a matter of seconds.

Fernando (29) performed an evaluation of the Proscan system for Texas DOT. In this evaluation, results obtained by Proscan were compared with the manual results obtained by Texas DOT test method Tex-1000-S, "Operation of Pavement Profilograph and Evaluation of Profiles." The evaluation indicated that Proscan gives accurate and consistent results in significantly less time and effort compared to the manual method. The Proscan system reduced a profilogram in only 4 percent of the time taken to reduce the profilogram manually. A comparison of mean PI values obtained by five raters at twenty three sections with the Proscan PI values indicated differences between manual and Proscan PI's are not statistically significant. Figure 41 shows the relationship between Proscan PI and the mean PI obtained from five raters at 23 sections. The variability in the manual PI's from various raters was found to be approximately 10 times higher than the variability in the ratings from repeat runs of Proscan. The defect stations reported by Proscan were consistent with those identified manually by the raters.

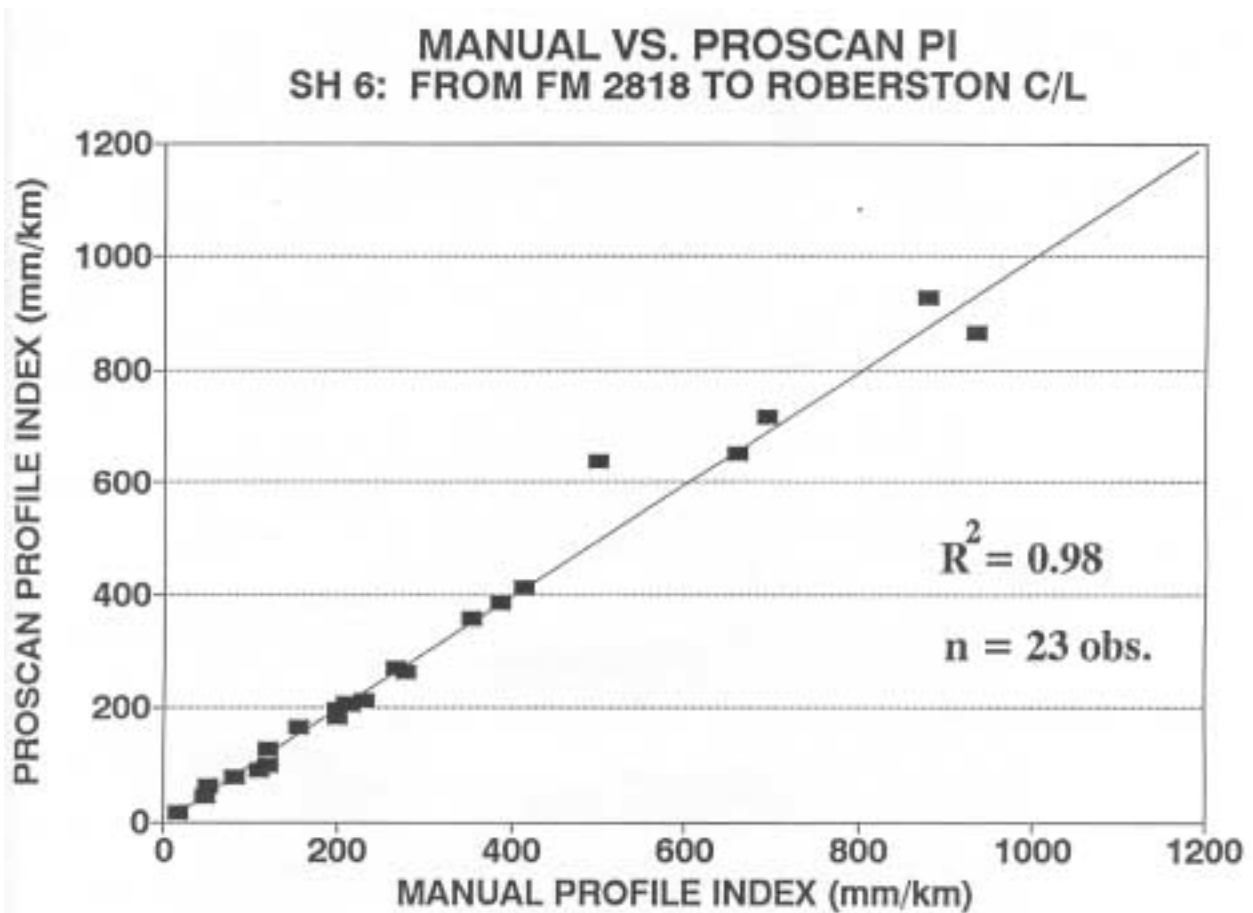


Figure 41. Scatter plot of Proscan and manual PI values (29).

## PROBLEMS IN INTERPRETATION OF PROFILOGRAPH TRACES

The tolerance zone that is used by the blanking band in evaluating a profilograph trace can blank out certain pavement features. Such features are usually cyclic features that are associated with some aspect of construction. These features can have an effect on the natural frequencies of

certain vehicles on the road, and induce vibrations in the vehicle. Therefore, new pavements that are accepted based on profilograph results may appear to be rough if such features are present on the roadway. In 1990, Kansas experienced a problem with short wavelength roughness. One project exhibited a cyclic 2.4 m (8 ft) wavelengths of approximately 5 mm (0.2 in) in amplitude. Although the profilograph determined that the pavement roughness met specifications, the ride quality of the pavement was poor. Because of this problem, the Kansas DOT eliminated the blanking band and developed specifications using a zero blanking band concept.

## CHAPTER 7

# FACTORS AFFECTING INERTIAL PROFILER MEASUREMENTS

### INTRODUCTION

An essential element of a pavement management system is a means to monitor pavement surface roughness, distress, and other properties. Most pavement management activities include the use of devices that measure longitudinal profile for assessment of surface roughness. When longitudinal road profile measurements are used for assessment of road condition, they are always summarized by an index that reduces the thousands of elevation values into a single value. The IRI is the most broadly used roughness index. However, no matter what index is calculated from a longitudinal profile, the quality of the information is only as good as the profile measurement. Currently, inertial profilers are widely used to collect longitudinal profile data for assessment of roughness at network level. In recent years, inertial profilers have also been used to assess the smoothness of new construction for construction acceptance purposes.

Although technology has been available for measuring longitudinal profile for decades, it has still not fully matured. A prevailing sense exists in the highway community that if different agencies measured the same road with their profiling device, they would obtain a variety of different results. Errors in profile and discrepancies between measurements arise from variations in equipment, inappropriate operating procedures, and aspects of the pavement surface and the surrounding environment. In many cases, these factors interact to reduce the repeatability and accuracy of profiling devices. For example, drivers of vehicles used for profiling may not all track in the same position within a lane, which affects the measured profile even if they are using excellent equipment. In addition, the actual shape of the road may change with time in response to the environment.

The interdependent variables that affect profile measurements can be divided into five categories: the equipment design, the pavement shape, the measurement environment, the manner in which the equipment is operated, and the driver and operator proficiency.

The University of Michigan and Soil and Materials Engineers performed an extensive study on the factors affecting pavement profile measurements for NCHRP project 10-47 (2). The goal was to identify factors that affect roughness measurements, quantify their effect on repeatability and accuracy, determine how and when they can be controlled, and communicate the findings to practitioners by providing guidelines. In this research, the individual factors affecting pavement profile measurements were divided into five broad categories as follows:

1. Profiler Design: aspects of profiler configuration, data collection method, and signal processing techniques that affect the measured profile.
2. Surface Shape: geometrical properties of the pavement surface, distress, and texture.
3. Measurement Environment: aspects of the environment in which a profiler must function that are not a property of the pavement shape.

4. Profiler Operation: the manner in which a profiler is driven and operated.
5. Profiler Operator: proficiency of the drivers and operators themselves.

Thirty-four factors that fell into the five categories described above were investigated in NCHRP Project 10-47. The design aspects of the profiler that affect the data quality include the height sensors, accelerometers, and the distance measuring system. An important factor that contributes to obtaining accurate profiles is the manner in which the data are sampled and the sensor signals are processed to compute profile. Minimum sampling distances are required for accurate computation of IRI as well as RN. Also proper anti-aliasing filters must be applied to the height sensor and accelerometer signals in order to get accurate profile measurements.

There are several ways that aspects of the pavement surface shape confound profile measurement. Transverse, daily, and seasonal variations in profile all combine to make an individual measurement a mere sample of the road shape. The lateral position of the measurement has a strong influence on the profile, because the pavement surface shape changes across the lane. Other aspects of the pavement shape affect profile measurement by interfering with the operation of the sensors on the profiler. The most well known example of this is chip seals that cause an extreme bias in roughness measured using ultrasonic height sensors.

Profile measurements can be affected by the environment in which they obtain measurements. Some aspects of the measurement environment, such as excessive surface moisture in rainy conditions render profile measurement completely useless. Other aspects of the measurement environment may cause a single erroneous reading in an otherwise accurate profile. For example, the height sensor may pass over a surface contaminant such as a piece of tire tread.

The aspect of profiler operation that influences the repeatability of roughness measurement most is lateral positioning. As described previously, the path a profiler takes over a section has a strong influence on the roughness it measures because of transverse variations in profile. Two measurements that follow a different path can produce equally valid but different results. The starting point of a section also determines what features are included in a measurement. Driving at speeds outside of the recommended range for a profiler can cause invalid measurements. Longitudinal acceleration and deceleration of a profiler greater than a specified amount can also affect the quality of the data.

The driver and operator of a profiler have a tremendous influence on the quality of profile data. It is also up to them to control the speed of the profiler, control the lateral position of the vehicle, stay in the correct lane, and devote adequate attention to safety. The operator must prepare the profiler at the start of a day to make sure it is working properly, find data collection landmarks and trigger the system, conduct quality control during measurements, and often do on-the-spot maintenance.

The following sections present the key findings of NCHRP Project 10-47. Separate sections are used to describe the findings that are classified into the following five categories: profiler design, surface shape, measurement environment, profiler operation, and profiler operator.

## PROFILER DESIGN

The following aspects related to profiler design are described in this section: height sensor, accelerometer, distance measuring system, aliasing, profile computation algorithm, number of sensors and lateral sensor spacing

### Height Sensors

The height sensor in a profiler measures the vertical distance from the vehicle to the road. All profilers now in use in North America measure height with one of four types of noncontacting transducers.

**Laser:** Laser sensors measure distance by means of triangulation. A spot of invisible light is projected on to the road surface. It is reflected through a lens mounted at an angle on to a light-sensitive displacement sensor. The size of the laser light spot is the sensor footprint. Selcom supplies laser sensors to several profiler manufacturers. Their sensors commonly use a footprint that is 1 to 5 mm in diameter.

**Infrared:** Infrared sensors operate on the same principle as laser sensors, but they use infrared light instead of laser light. K.J. Law, Inc. makes an infrared sensor with a footprint 6 mm long (in the direction of travel) and 37 mm wide (perpendicular to direction of travel).

**Optical:** Optical sensors are exclusive to K.J. Law profilers. They also detect the position of a projected image using triangulation, but the image is a slit of light in the visible infrared spectrum that is 6 mm long (in the direction of travel) and 150 mm wide (perpendicular to direction of travel).

**Ultrasonic:** Ultrasonic sensors measure distance by emitting a short burst of sound waves. The sound travels down to the pavement surface and reflects back upward and the elapsed time is used to compute the distance. The footprint of ultrasonic sensors is 50 to 100 mm in diameter.

Several studies of profiler performance have been done that distinguish them primarily by the height sensor type. Often, a pair of profilers with different types of height sensor are compared, or a single profiler is tested against a reference measurement. The Ann Arbor Road Profilometer Meeting (30) and the 1993 and 1994 Road Profiler User Group (RPUG) studies (31, 32) included most of the profiler designs in use in North America at the time. In all of these studies, the repeatability and accuracy of the profilers involved were strongly linked to their height sensor. In the RPUG studies, optical profilers exhibited the best repeatability and the best agreement with reference measurements. Most of the laser profilers showed sufficient performance for use in network-level profiling. Ultrasonic profilers showed so much scatter and bias that they did not appear sufficient for roughness measurement. The poor repeatability of ultrasonic sensors has been recorded in other studies as well (33, 34, 35).

Overall, the four types of height sensor that were described differ in their sampling rate, resolution, footprint size, and sensitivity to the environment. Ultrasonic sensors cannot sense the



road often enough or with enough resolution to measure roughness reliably. Optical, laser, and infrared profilers have all demonstrated that they can be repeatable and accurate over a range of conditions. The following sections describe how the sampling rate, sensor resolution, footprint and measurement range affect height sensor readings. The last section describes guidelines to be followed for obtaining error free height sensor readings.

### *Sampling Rate*

Sample interval is the longitudinal distance between points where the profiler takes measurements. The measurements obtained from the height sensor and accelerometer is used to compute the profile. Recording interval is the interval at which a profiler stores the computed elevation values. For some profilers, the recording interval is the same as the sample interval. Other profilers have a recording interval that is longer than the sampling interval. In such profilers, the profile elevation values are usually averaged before they are recorded.

In the measurement of IRI and RN, the shortest wavelength of interest is about 0.3 m. At a speed of 100 km/hr, a profiler must sample the road every 0.005 seconds to measure wavelengths this short. However, this is not enough. An accurate profiler must sample the road more often than that and apply filters to remove aliasing errors. Laser, optical, and infrared height sensors all operate with a sufficient sampling rate to measure wavelengths of 0.3 m and longer without aliasing errors. The sound wave used by ultrasonic sensors only takes about 0.002 seconds to travel from the vehicle to the road and back. However, multiple echoes of the sound waves do not die out for up to 0.01 seconds (36). This severely limits the sampling rate of ultrasonic sensors at high speed.

Analysis have shown that for computation of IRI, the sample interval of the profiler must be 167 mm or less and for computation of Ride Number the sample interval must be 50 mm or less (2).

### *Resolution*

The resolution of a height sensor is the smallest unit of distance it can measure accurately. When the IRI was first proposed, Sayers reported the resolution required of the final profile for accurate measurement of IRI is a function of roughness (22). His study recorded that on roads with IRI less than 3.0 m/km, a resolution of 1 mm was acceptable. On roads rougher than 5 m/km, resolution of 2.5 mm was permissible. In their advertisements, K.J. Law reports a dynamic resolution of 0.25 mm for their infrared height sensors, and Selcom reports a resolution of 0.06 mm for their laser sensors. Laser, infrared, and optical height sensors all have sufficient resolution for measurement of IRI and RN if their signals are processed properly. Advertisements for ultrasonic profilers cite values of resolution of 1.5 to 3 mm. This level of resolution is not sufficient for measuring roughness on smooth roads, but may be good enough to obtain IRI measurements on rough roads. Ultrasonic sensors do not have sufficient resolution to obtain Ride Number. Figure 42 shows the effect of resolution on errors introduced into IRI

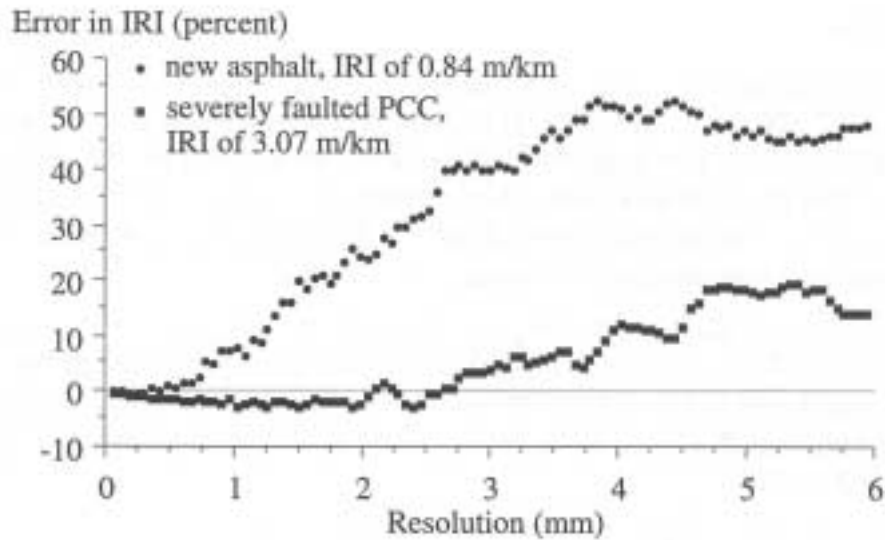


Figure 42. Sensitivity of IRI to height sensor resolution (2)

### *Footprint*

Height sensor footprint strongly affects the way a profiler measures small features in the road, particularly surface texture, and narrow cracks and joints. Infrared height sensors, which have a footprint 37 mm wide and 6 mm in the direction of travel, are likely to measure a much smaller dip over a narrow PCC joint or crack than a laser sensor with a footprint that is 1 to 2 mm in diameter. Even if both sensor signals are filtered to remove wavelengths shorter than 0.3 m, the profiler with the laser sensor will probably measure a higher roughness because it includes spikes that the profiler with the infrared sensor did not. No standard exists yet to indicate which is the better sensing strategy. Narrow cracks do not affect vehicles much, because they are enveloped by the tires. Thus, if the final use of a profile is to judge the effect of roughness on vehicle response, it might be desirable to remove downward spikes. This could either be done by a height sensor with a large footprint, or in post processing of a signal from the height sensor. On the other hand, narrow cracks are a legitimate aspect of the current condition of many roads, and have some influence on the amount of time left in their service life. Height sensor footprint also interacts with pavement macrotexture to affect profile measurement. Height sensors with a large footprint are more likely to average out short deviations in the surface caused by coarse macrotexture. Ultrasonic height sensors have a very large footprint, but they detect the highest feature within their footprint, rather than the average of the deviations within their footprint. Thus, they are extremely prone to aliasing errors on roads with coarse macrotexture. Optical and infrared height sensors both have a wide footprint, so they are likely to be less affected by macrotexture. Profiles measured with laser sensors are affected by macrotexture because of their small footprint, but proper use of anti-aliasing filters on the height sensor signals prevent errors in the final roughness value.

## Range

A study performed as a part of NCHRP Project 10-47 (2) used data collected at twelve test sections located around Ann Arbor, Michigan to investigate the range of heights that were measured by the height sensors. Table 3 presents the results that were obtained for a profiler that had sensors mounted at the center of the vehicle and for a profiler that had sensors mounted on the bumper. The total range measured by the height sensors in the ProRut profiler that had sensors that were mounted on the vehicle body between the front and rear axle was less than 100 mm at all test sections. Height sensors in the bumper-mounted profiler had a much larger range. In the bumper-mounted profiler, a sensor range of 250 mm is sufficient on all of the sections except the roughest, which is so rough it probably does not require accurate measurement.

Table 3. Height sensor range in a center-mounted and a bumper-mounted profiler (2)

IRI of Section (m/km)	Height Sensor Range (mm)			
	Bumper Mounted		Center Mounted	
	Left	Right	Left	Right
0.77	29	28	13	14
0.99	57	64	20	19
1.07	103	88	22	21
2.07	123	119	45	43
2.10	102	117	45	33
2.76	129	138	50	34
3.04	189	241	77	72
3.12	152	209	66	61
3.23	160	146	42	80
3.72	192	168	77	58
3.79	175	208	63	61
4.53	426	492	80	83

## Guidelines for Obtaining Error Free Height Sensor Measurements

- The height sensors must be calibrated using the procedures described by the profiler manufacturer. Guidance regarding the time interval between calibrations should be obtained from the profiler manufacturer.
- It is recommended that the profiler software be capable of performing a calibration check on the height sensors. The calibration check is performed by putting a block of known height below the sensor and checking to ensure that the system will accurately measure the height of the block within a specified tolerance. Power should be supplied to the system to allow sufficient time for it to warm up prior to performing this test. Guidance regarding the time necessary to warm up the system should be obtained from the manufacturer.

- Prior to profile data collection, power should be supplied to the system to allow sufficient time for the electronic components to warm up. Guidance regarding the time necessary to warm up the system should be obtained from the manufacturer.
- The height sensors should be wiped clean prior to profile data collection. The sensor may need more frequent cleaning during the day depending on environmental conditions (e.g. water splashing on the sensor).
- It is recommended that the output of the height sensor be displayed on the screen of the computer when data is collected, preferably in a graphical format for easier tracking. This display can be used for a quick visual check that the height sensor is working properly during measurements.
- It is recommended that the profiler issue an audible beep when height sensor readings are outside the valid range.
- Whenever repairs are performed on the suspension system or the bumper of the host vehicle, the height sensors should be positioned so that they are in the center of their measurement range when the vehicle is at rest. The mounting position should also be checked whenever tires are replaced or rotated, or when wheel alignments are performed.
- The height sensor should be calibrated after any repairs are performed on the suspension system or to the bumper of the vehicle if the sensors are mounted on the front bumper. The height sensors should also be calibrated whenever tires are replaced or rotated, or when wheel alignments are performed.
- Potential invalid points in height sensor measurements should be flagged during data collection. This can be accomplished using an algorithm that will look at differences between two consecutive height sensor measurements. If the differences are high enough to indicate a possible measurement error, these locations should be flagged in the data file.
- Some height sensors are equipped with covers to protect them when profile data are not being collected. The software should prohibit data collection while the covers are in place.
- Do not operate the profiler in temperatures outside of the range listed by the height sensor manufacturer.

## **Accelerometers**

The accelerometer is used in a high-speed profiler to establish an inertial reference from which relative height measurements are made. The vertical acceleration measured by the accelerometer is integrated twice to establish its vertical position. The accelerometer should be oriented vertically. Accelerometers are usually mounted just above each height sensor. Thus, the accelerometer is not always perfectly vertical when the vehicle body undergoes pitch and roll as

it travels over uneven roads. An error occurs if the vehicle pitches and accelerates longitudinally at the same time, or rolls and accelerates laterally at the same time. Fortunately, this error is small if the lateral and longitudinal acceleration are held under 0.1 g.

On fifteen sections selected in the Ann Arbor, Michigan area to represent a range of surface properties, the total range measured by the accelerometers in the ProRut profiler was less than  $\pm 2$  g (2). The sensors in the ProRut profiler are mounted in the vehicle body between the front and rear axle. Accelerometers in a bumper-mounted profiler may read a range that is twice as large on some roads. Based on the results obtained from this study, it is recommended that the accelerometer in a profiler used on primary road networks and in interstates should have a total range of at least  $\pm 5$  g.

### **Distance Measuring System**

The distance measuring system is one of the three major types of transducer that make up a profiler. Distance must be measured properly to obtain accurate roughness statistics. In network monitoring applications, roughness is often measured over very long distances, such that even a small bias in longitudinal distance measurement can build up to a large error. The error throws off distance “accounting” and the longitudinal positioning of each segment. In measurement of new construction, corrective action such as grinding is often recommended at specific locations. Thus, accurate measurement of longitudinal distance relative to fixed landmarks is very important.

In high-speed profilers, distance traveled is usually measured by a pulser on one of the front wheels. A common configuration is to install an exciter ring with equally spaced notches on the back side of the disc brake rotor of one of the wheels. Rotation of the wheel is measured by detection of pulses as the wheel rotates and the notches pass (37). During normal operation, each pulse is associated directly with a fixed travel distance through the rolling radius of the tire.

It is recommended that the following guidelines be followed in order to obtain accurate distance measurements.

- The distance measuring system should be calibrated at intervals recommended by the manufacturer. Calibration involves laying out a section of known distance and running the profiler over this section. The section should be laid out using a steel tape or an electronic distance measuring system accurate to at least 0.05 percent. The system should be calibrated at the typical operating speed used during profile measurements, with a photocell (or other automated triggering system) being used to detect the beginning and the end of the section.
- Prior to calibrating the distance measuring system, the cold tire pressure should be checked to ensure that it equals the recommended tire pressure. The vehicle should be driven for 20 to 30 minutes at highway speeds to warm up the tires prior to calibration.

- The distance measuring system must be calibrated after any repairs are performed on the suspension system or tires are changed or repaired. The system should also be calibrated whenever tires are rotated, or when wheel alignments are performed.

## Aliasing

A crucial step that must be performed on the height sensors and accelerometer signals before the data are recorded is anti-aliasing. Anti-aliasing is performed by the software that is in the profiler. The discussion presented in this section is intended to illustrate that anti-aliasing is essential to the quality of profile measurements.

Aliasing occurs when as a consequence of sampling at a finite interval, the short-wavelength content of the true road profile contaminates the measurement of the longer-wavelength content. A simple example to illustrate this phenomenon is shown in figure 43.

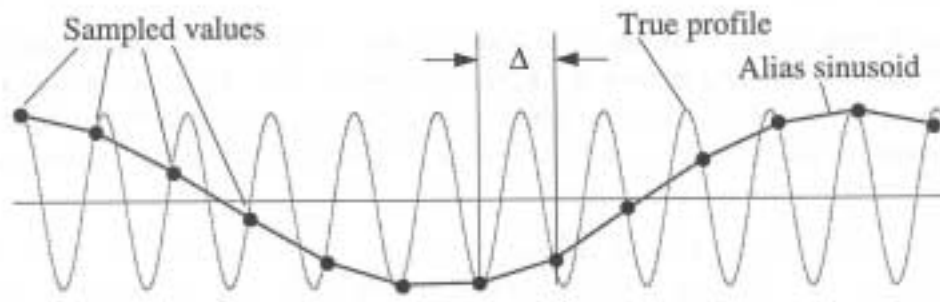


Figure 43. Example of aliasing.

Figure 43 shows a sine wave sampled at an interval slightly longer than its period of oscillation. The only information that is available to the measurement is the set of sampled values. When connected, the sampled points appear to define a sine wave of a much longer wavelength. It is in this manner that the short road features that the IRI and RN should ignore are aliased into the longer wavelength range of the measurement and artificially increase the roughness.

From a more practical standpoint, imagine a height sensor with a very small footprint that measures a few centimeters deep into a narrow crack (see figure 44). This is a feature in the road that is likely to be ignored by a tire passing over it, and should be ignored by the IRI and RN calculation. If the profiler is operating with a very short sample interval, the crack will be insignificant because its depth will be attenuated in the moving average. However, if the sample interval is longer than 167 mm, the crack will appear to be a dip a few centimeters deep and more than 333 mm long. It will erroneously increase the roughness of the section because, after sampling, there was not enough information available to recognize it as a narrow crack. (It looks instead like a longer dip in the road.) The potential for this type of error in the measurement of road profile is enormous. In particular, cracks, PCC joints and coarse macrotexture can easily lead to this type of aliasing error.

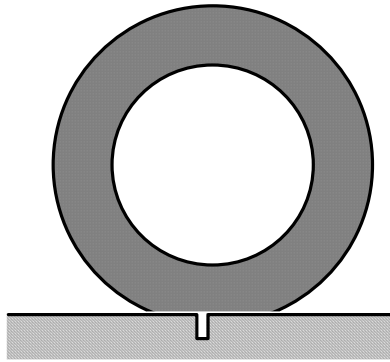


Figure 44. A tire over a narrow crack.

Fortunately, aliasing can be avoided. Refer once again to the example shown in figure 43. Assume that the original sine wave has a wavelength that is outside the range of interest, but the aliased sine wave does not. In the example, a single point was measured at an interval of  $\Delta$ . As an alternative, consider a case in which a sampling rate was used that allowed ten measurements to be made over the distance  $\Delta$ . Then, before the sensor readings were digitized, each set of ten measurements was averaged to a single value. These averaged values could then be digitized at a sample interval of  $\Delta$ . This procedure leads to a much higher level of quality in the measurement. The original sine wave still does not appear in the final measurement, but the (artificial) longer, aliased sine wave is also virtually eliminated.

The procedure just described is a simplified explanation of how anti-aliasing should work in a profiler. In reality, anti-aliasing is a bit more complicated. The signals from height sensors and accelerometers should pass through an analog filter to eliminate the short-wavelength content before they are digitized. It is also important to use the same filter on the height sensor and accelerometer signals. If anti-aliasing is performed on only one of the sensors or differently on each, aliasing errors will still result. They will just be more complicated aliasing errors. The recommended anti-aliasing filter and sample interval are highly interrelated for a given application. For a sample interval of  $\Delta$  the cutoff wavelength ( $\lambda_c$ ) in the anti-aliasing filter should be such that  $\lambda_c = 2 \cdot \Delta$ .

### Profile Computation Algorithm

Inertial profilers compute profile from a combination of the output of the height sensor, accelerometer, and longitudinal distance measuring system. The vertical acceleration is integrated twice to construct a floating reference height. The height sensor, mounted in the same position as the accelerometer, measures the distance from the floating reference to the road surface. The height sensor signal is subtracted from the height of the floating reference to compute the profile elevation. The longitudinal distance measurement is needed to associate a position with each profile elevation. This method of measuring profile was invented by Elson Spangler and William Kelly (1). It is described mathematically by the following equation:

$$Z(x) = H(x) + \iint_x A_t(s)/V^2 ds ds$$

where  $x$  is longitudinal distance,  $Z(x)$  is the computed profile,  $H(x)$  is the height sensor measurement, and the term with the integral is the floating reference derived from temporal vertical acceleration  $A_t(s)$  and forward speed  $V$ . The acceleration is divided by forward speed squared to convert it into spatial acceleration in units of 1/length. The height sensor measurement is the distance from the vehicle to the ground and should always be negative.

All inertial profilers use a discrete adaptation of the above equation to compute profile. For example, the FHWA ProRut profiler computes profile using the following procedure:

*Step 1:* Calculate the bias in the accelerometer signal and remove it. This step helps minimize error in the integration that follows.

*Step 2:* Convert temporal acceleration ( $A_t$ ) to spatial acceleration ( $A_s$ ):

$$A_s(i) = A_t(i)/V^2$$

*Step 3:* Integrate the spatial acceleration once to obtain slope. This is done with a recursive finite difference equation:

$$S_a(i) = C \cdot S_a(i-1) + \Delta \cdot A_s(i)$$

where  $\Delta$  is the sample interval and  $S_a$  is the component of the slope profile measured by the accelerometer. The first term includes a drift-removal coefficient:  $C = \Delta/L$ , where  $L$  is usually set to three times the longest wavelength of interest.

*Step 4:* Differentiate the height sensor signal ( $H$ ) once to obtain slope:

$$S_h(i) = \frac{C \cdot H(i+1) - H(i)}{\Delta}$$

where  $S_h$  is the component of slope profile measured by the height sensor.

*Step 5:* Combine the slope from the height sensor and accelerometer signals to get the slope of the road profile ( $S$ ):

$$S(i) = S_a(i) + S_h(i)$$

*Step 6:* Integrate the slope profile to obtain elevation. The integration is performed backwards in this step to cancel the phase lag introduced in the computation of the slope profile. In this equation, “ $i$ ” should step from the last value to the first.

$$Z(i) = C \cdot Z(i+1) + \Delta \cdot S(i)$$

This method of profile computation cancels the phase shift associated with integration by moving forward through the profile in steps 1 through 5, then backward in step 6. Unfortunately, this method cannot be used in a running profile computation that takes place as a profiler passes over a section. It must instead be applied after the measurement is complete. Therefore, it is not practical for use in network-level profiling applications, where long stretches of road must be covered and roughness is computed in real time. Devices that compute profile during the measurement cannot avoid the phase shift. Pong and Wambold (38) demonstrated that some



common profile computation algorithms do introduce a phase shift in the profile that grows with wavelength.

Most profilers apply a high-pass filter to profiles as a final step in the computation. This is not a necessary step, but it improves the appearance of the plots. Inertial profilers do not measure extremely long wavelengths validly anyhow, so the high-pass filter should remove incorrect information and pass the valid part of the profile through. Without the filter, a plot of the raw profile usually drifts and obscures the short deviations that are of interest in a profile. The most common high-pass filter cutoff in use for profiling in North America is 91 m. The 91-m cutoff is also short enough to display road features of interest. Standardizing the cutoff would promote agreement between profile plots output by profilers.

### **Number of Sensors**

A survey performed in 1994 reported that of fifty-six states and provinces that responded, forty collect roughness on both wheel paths, eleven report the roughness of the left wheel path only, and five report the roughness of the right wheel path (39). Of the forty agencies that report roughness for both wheel paths, thirty-four only retain the average of the two sides and the other six retain the individual roughness values for the left and right wheel paths.

Currently most of the profilers in service in North America measure profile in the left and the right wheel paths. The FHWA requires states to report IRI of HPMS sections for the right wheel path only (40). The motivation to collect roughness in only one track is cost. Each set of sensors implies higher cost for equipment, maintenance, and data storage, and extra effort for calibration and data handling. However, collecting data in an extra wheeltrack does not increase the distance that must be covered, and an extra set of sensors improves the quality of a measurement by providing a clearer picture of the condition of the road.

On some pavements, the IRI of a single track on one side of the lane is a good estimate of the roughness, but this is usually not the case. The IRI of most pavements varies significantly across a lane, such that measurements from two tracks provide a much better representation of the roughness than one. For NCHRP Project 10-47 (2), a study was performed to assess the differences between left and right wheel path IRI, and their relationship to the mean IRI (average of left and right wheel path IRI). Data collected at 799 LTPP GPS sections was used in this analysis. The IRI of the right wheel path was higher than the mean IRI at 60% of asphalt surfaced sections and 61% of PCC sections. The right wheel path IRI was within 10% of the mean IRI at 66 percent of asphalt surfaced sections and 81 percent of PCC sections. The right wheel path was IRI within 5% of the mean IRI at 39% of asphalt surfaced sections and 54% of PCC sections. These results indicate obtaining IRI value in one wheel path does not provide a true view of the roughness of the road as represented by the mean IRI. These results show that profile measurements must be obtained along both the left and the right wheel path in order to obtain an overall view of the roughness of the roadway.

## Lateral Sensor Spacing

The majority of profilers in service in North America collect profile along two tracks; one on the left side of the host vehicle and one on the right. The separation between the footprints placed by the height sensors is their lateral spacing. The lateral sensor spacing in most profilers is determined by the need to collect rut depth concurrently with profile. Protocols for rut depth measurement developed for the FHWA recommend a three-sensor system with a lateral spacing of 172.7 cm (41). A survey of seventeen states in 1996 found that a vast majority of them used a lateral sensor spacing of 175.3 cm (25).

In the recently concluded NCHRP Project 10-47 (2), the profiles of seven pavement sections were measured in several lateral tracking positions using the FHWA ProRut profiler. The lateral sensor spacing in the ProRut is 182 cm, but the experiment covered so many lateral positions across the lane that the roughness in any position can be estimated within reasonable tolerance. Table 4 shows the range of IRI values that would be measured by the ProRut if it tracked in the same location in every run but the sensors were spaced differently. All of the values assume that the center of the vehicle is placed 167 cm from the center of the right edge stripe. (In general, this places the center of the vehicle 175 to 180 cm from the right lane edge.) An estimate of the mean IRI (MRI), which is the average of the left and right wheel path IRI that would be measured with this central placement is listed for several values of lateral sensor spacing in table 4.

On most of the sections listed in Table 4, the MRI is fairly insensitive to lateral sensor spacing over a range of 30 cm. The new asphalt, severely faulted PCC, three-year-old PCC and AC with transverse cracks all have IRI values that do not change much over the range of sensor spacing covered in the table. On the six-year-old PCC and the one-year-old PCC, an increase in sensor spacing causes the IRI on the right to increase and the IRI on the left to decrease. On the six-year-old PCC, these changes in IRI with lateral sensor spacing cancel each other out and the MRI holds steady. The one-year-old PCC section exhibits a sharp increase in roughness near the right edge of the lane, so the MRI is higher with a wider sensor spacing. The old asphalt is the most sensitive to lateral sensor spacing. This section has medium severity rutting with some longitudinal cracking in the ruts, so the IRI is highest in the ruts. The 180 cm lateral sensor spacing places the two profiles in the center of the ruts, so it produces the highest MRI. As the sensors are drawn in, some of the rough features are missed, and the IRI on both sides decreases.

The lateral sensor spacing is not expected to change the measured roughness significantly on the majority of pavement sections, but it is likely to do so on any section with rutting or significant distress in the wheel tracks caused by heavy truck loading. Note that the roughness values presented in table 4 assume that the profiler runs in a central tracking position, and only covers a lateral movement of the sensors of 30 cm. Variations in lateral tracking position during typical driving cover a broader range, and many drivers do not habitually travel in the center of a lane. Thus, variations in lateral positioning of a profiler are expected to cause much greater changes in measured roughness than variations in lateral sensor spacing.

A sensor spacing of 180 to 185 cm would correspond best to a typical track width of heavy trucks. However, automobiles have a narrower track than this and would not encounter

Table 4. Variation in MRI with lateral sensor spacing.

Lateral Sensor Spacing (cm)	Mean IRI (m/km)		
	New Asphalt	Asphalt with Transverse Cracks	Old Asphalt
150	0.87	1.20	2.05
155	0.87	1.20	2.08
160	0.88	1.21	2.12
165	0.89	1.21	2.15
170	0.89	1.21	2.19
175	0.90	1.20	2.23
180	0.91	1.21	2.24

Table 4. (cont.) Variation in MRI with lateral sensor spacing.

Lateral Sensor Spacing (cm)	Mean IRI (m/km)			
	One Year Old PCC	Three Year Old PCC	Six Year Old PCC	Faulted PCC
150	1.04	0.59	1.58	3.69
155	1.05	0.59	1.58	3.69
160	1.07	0.59	1.58	3.71
165	1.08	0.59	1.58	3.72
170	1.08	0.59	1.58	3.73
175	1.09	0.58	1.58	3.74
180	1.11	0.58	1.58	3.75

two profiles that are this far apart simultaneously. Thus, sensor spacing this large may measure roughness that does not represent their ride experience. To measure a set of two profiles that are more representative of a typical automotive ride experience, and place the sensors inside the ruts on rutted sections, a lateral sensor spacing of 170 to 180 cm is suitable. Most commercial profilers with two sensors for profile already space their sensors in this range, as does the protocol for rut measurement.

## **SURFACE SHAPE**

There are several ways that aspects of the pavement surface shape confound profile measurements. Longitudinal profile measurements usually involve measuring two paths along the pavement surface in a given lane. The lateral position of measurement has a strong influence on the profile, because the pavement surface changes across the lane. The time and the date of

measurement also influence the results in many cases, because of cyclic changes in roughness. Transverse, daily and seasonal variations in profile all combine to make an individual measurement a mere sample of the road shape. Since the roughness of a road is really a function of lateral position and time, a single roughness value is actually a sampling of a statistical road property.

The factors affecting profile measurements that are related to surface shape that will be covered in this section are: transverse variations, daily variations, seasonal variations, surface texture, pavement distress, curves, and hills and grades.

### **Transverse Variations**

This section examines variations in roughness that occurs across a pavement lane. Road profile is usually measured in only two tracks per pass. Indeed, an automobile only experiences the road along two distinct tracks at a time. Thus, roughness is often thought of as a two-dimensional property of each side of the pavement lane (one profile on the left and one on the right), with little thought given to the path taken by the sensors. Roads are actually three-dimensional surfaces. A unique value of roughness exists for every path that can be taken on a given lane. The two values that a profiler produces per pass over a section only provide samples of the overall roughness. The difference between those two values is evidence that other values of roughness would be measured if the sensors moved along a different path.

The transverse variation in roughness of seven sections was investigated experimentally in NCHRP Project 10-47 (2). A camera, aimed at the edge stripe in the pavement, was mounted on the ProRut profiler to monitor its lateral position. The position of the vehicle was displayed for the driver on a monitor graduated to show the lateral separation between the right height sensor footprint and the center of the right lane edge stripe. This served as a guide for the driver. To further aid the driver, all sections used in the study were straight and had visible markings along the right edge. The video was also recorded and used after each run to judge the lateral position of the sensors at one-second intervals. In each run, the driver attempted to hold a target lateral position within a range of less than 20 cm, but a total range of 30 cm was considered acceptable. Each section was measured in seven to fourteen vehicle positions spread out over the entire lane. These measurements reveal the variation in roughness that exists across each section. The properties of the seven sections investigated in this experiment are shown in table 5. The variations in IRI across the lane at these test section are described separately.

#### *New Asphalt*

This section was an asphalt overlay that was placed about six months before measurements were obtained for the experiment. Figure 45 shows the variation in IRI across the lane at this section. Most of the roughness occurring at this section was contributed by wavelengths that were greater than 20 m. The roughness in the short wavelength range is usually eliminated by the placement of an overlay. Since most of the roughness is caused by long

Table 5. Sections measured in the transverse variation experiment.

Section Number	Designation	Description
1	New asphalt	Overlay of PCC, less than six months old
2	Faulted PCC	21.3 m long slabs broken into several pieces with severe tilting and faulting
3	AC pavement with rutting	Heavy truck traffic, sealed transverse and longitudinal cracks, mild rutting
4	Three-year-old PCC	Extremely smooth, 8.2 m long slabs
5	One-year-old PCC	12.5 m long slabs
6	AC pavement with transverse cracks	Transverse cracking, most severe along right edge
7	Six-year-old PCC	No visible distress, but does not feel smooth

wavelength features, which do not vary much transversely, the IRI was consistent across much of the lane. For example, the IRI of all positions more than 1.2 m from the right edge only ranged from 0.81 to 0.87 m/km. The consistency in this part of the lane is a result of the lack of short wavelength roughness. Long wavelength features are more likely to span an entire lane. (The entire width of a lane generally goes up and down hills together.) Short wavelength roughness, on the other hand, often causes variations in profile across a lane. In contrast, the roughness in the portion of the lane less than 1.2 m from the right edge was not consistent. The short-wavelength roughness near the shoulder caused the IRI to increase steadily from 0.87 to 1.17 m/km as the tracking position moved toward the right edge.

A shift in the path followed by a profiler at this section from the central tracking position to 0.3 m left caused the mean IRI to decrease by 5 percent with respect to the mean IRI obtained at the central tracking position. A path 0.3 m to the right of the central tracking position caused the mean IRI to increase by 2 percent with respect to the IRI obtained for the central tracking position.

#### *Faulted PCC*

The severely faulted PCC section was the roughest section included in this experiment. This section was a jointed reinforced section with a joint spacing of 21 m, but each slab was broken into as many as seven pieces. Each of the pieces of the slab was tilted with faulting between them, but no faulting was present at the joints. This section was so rough at the time of the experiment that traversing it at the speed limit was uncomfortable. Figure 46 shows the variation in IRI across the lane at this test section.

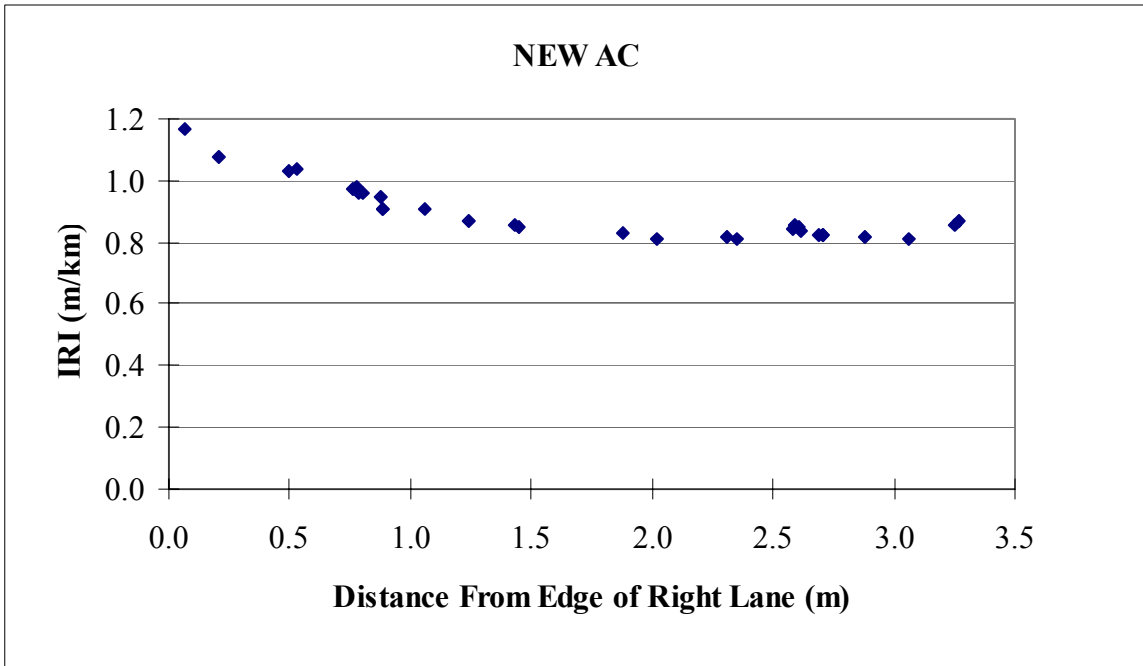


Figure 45. Variation in IRI across lane for a new asphalt concrete pavement.

Most of the roughness at this section was caused by tilting of the slabs and the faulting between them. Thus, as shown in Figure 46, the IRI did not vary much across the majority of the lane. For paths between 0.2 m and 2.9 m from the right edge, the IRI ranged from 3.61 to 3.94 m/km. A range of 0.33 m/km would be significant for a smooth section, but in this case it is not.

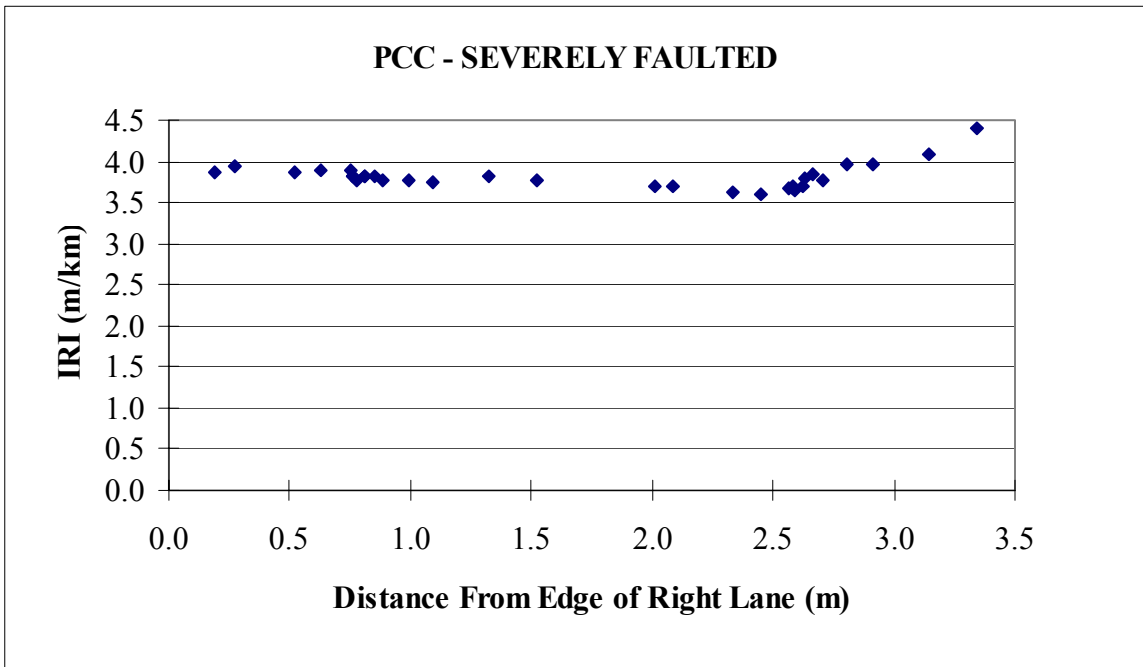


Figure 46. Variation in IRI across lane for a faulted PCC pavement.

A shift in the path followed by a profiler at this section from the central tracking position to 0.3 m left caused the mean IRI to increase by 3 percent with respect to the mean IRI obtained at the central tracking position. A path 0.3 m to the right of the central tracking position did not cause a change in the mean IRI with respect to the IRI obtained for the central tracking position.

*Asphalt Pavement with Rutting*

This section is on a two-lane undivided road that provides access to a waste dump. The section is on the side leading to the dump, so it is subjected to traffic by loaded trucks. The section is only mildly rutted but it has several longitudinal cracks within the developing ruts. It is also cracked transversely in several places. All of the major longitudinal and transverse cracks were sealed when this test was performed.

Figure 47 shows the transverse variation in IRI across the lane. The IRI is highest in the ruts. These ruts are centered 1.9 m apart and are 0.7 m wide. This corresponds almost directly to the footprint laid out by a typical truck axle with dual tires. The elevated roughness in the ruts is caused as a result of the longitudinal cracks that appear within the ruts.

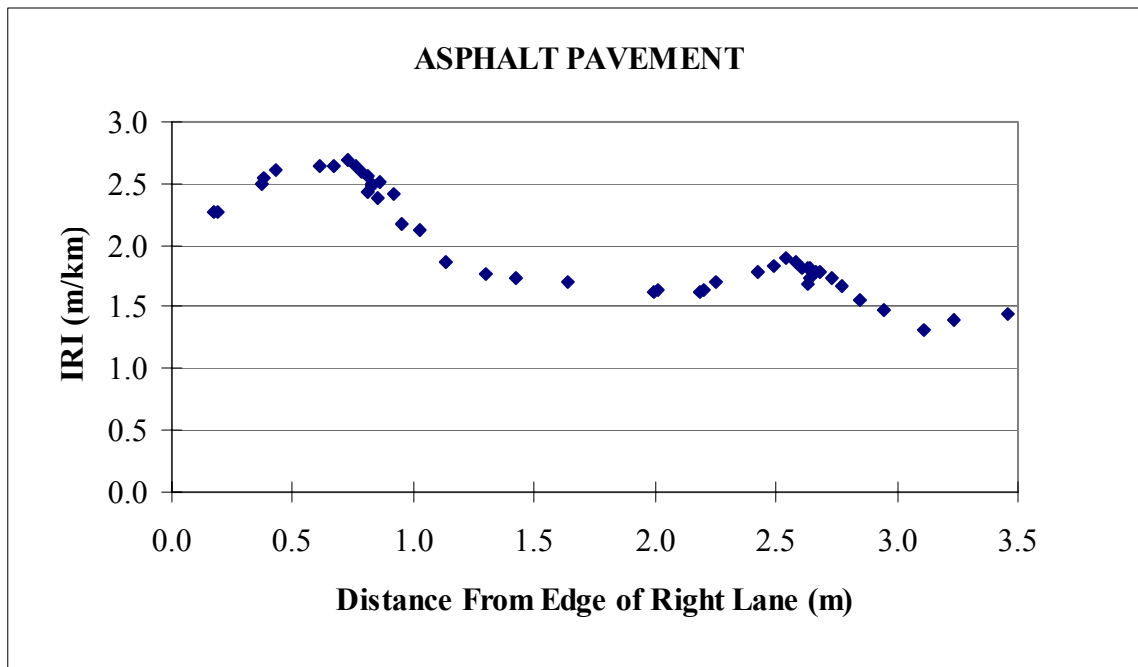


Figure 47. Variation in IRI across lane for a rutted AC pavement.

A shift in the path followed by a profiler at this section from the central tracking position to 0.3 m left caused the mean IRI to decrease by 20 percent with respect to the mean IRI obtained at the central tracking position. A path 0.3 m to the right of the central tracking position caused the mean IRI to decrease by 3 percent with respect to the IRI obtained for the central tracking position.

### Three-Year-Old PCC

Transverse variations in IRI across the lane at this section are shown in figure 48. This section was exceptionally smooth. The only major transverse variation in IRI at this section occurred near the right edge, where the roughness was highest.

A shift in the path followed by a profiler at this section from the central tracking position to 0.3 m left caused the mean IRI to increase by 12 percent with respect to the mean IRI obtained at the central tracking position. A path 0.3 m to the right of the central tracking position caused the mean IRI to increase by 5 percent with respect to the IRI obtained for the central tracking position.

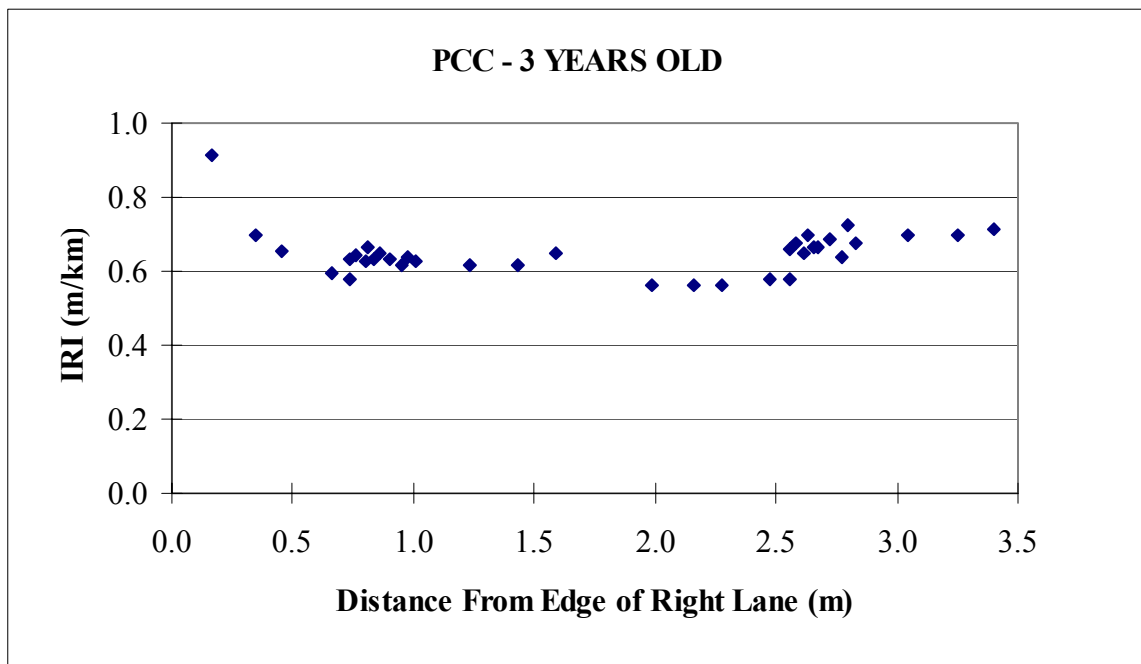


Figure 48. Variation in IRI across lane for a 3 year old PCC pavement.

### One-Year-Old PCC

Figure 49 shows the variations in IRI across the lane for the one-year old PCC pavement. The transverse variations in IRI for the one-year-old PCC were somewhat similar to those exhibited by the six-year-old PCC. This section was smoothest on the left, grew rougher as the track moved to the right and grew much rougher near the right edge. Although the trend in roughness was the same in this section as in the six-year-old PCC, the underlying cause was quite different. On this section, most of the roughness was caused by spikes at the joints. The joint width at this section was approximately 15 mm. Although the joints were sealed, the sealant was about 10 mm below the surface of the slab. At nearly every joint, the ProRut profiler



measured a downward spike ranging from 5 to 15 mm deep. These spikes grew in depth with movement to the right, except in a track just inside the right lane edge. This caused the roughness to be higher closer to the right edge of the lane. Since this section is not faulted, the gaps at the

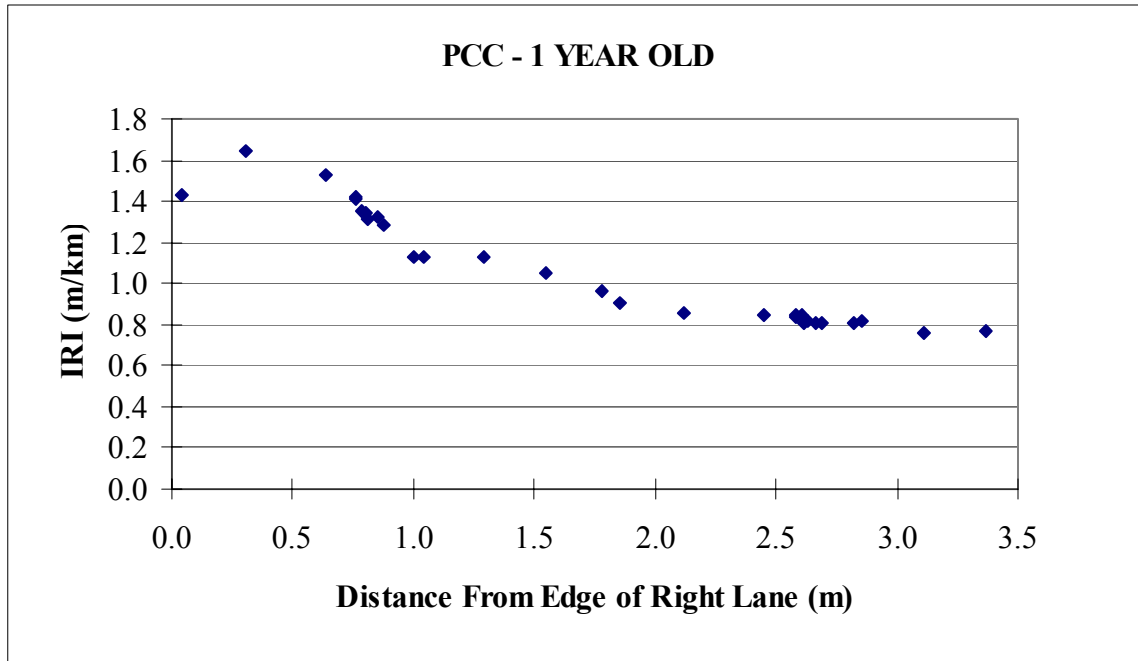


Figure 49. Variation in IRI across lane for a one-year old PCC pavement.

joints that were described are enveloped by vehicle tires, and do not degrade the ride quality of the road. Thus, the trends observed on this section are somewhat dubious. In reality, this section felt like a new PCC no matter where the vehicle tracked.

A shift in the path followed by a profiler at this section from the central tracking position to 0.3 m left caused the mean IRI to decrease by 14 percent with respect to the mean IRI obtained at the central tracking position. A path 0.3 m to the right of the central tracking position caused the mean IRI to increase by 8 percent with respect to the IRI obtained for the central tracking position.

#### *AC with Transverse Cracks*

Figure 50 shows the transverse variation in IRI across the lane for an AC pavement with transverse cracking. The only distress in this section was transverse cracking. All cracks spanned the entire width of the lane. Across most of the lane, the cracks were not very severe. However, within a half-meter of the right edge, the cracks nearly always degenerated into a dip up to 40 cm long and 5 mm deep. On the left side of the lane, the crack is narrow and does not contribute much to the roughness. As the profile is measured closer and closer to the right edge, the cracks grow deeper and the surrounding dip grows longer, resulting in a very rough feature near the right edge.

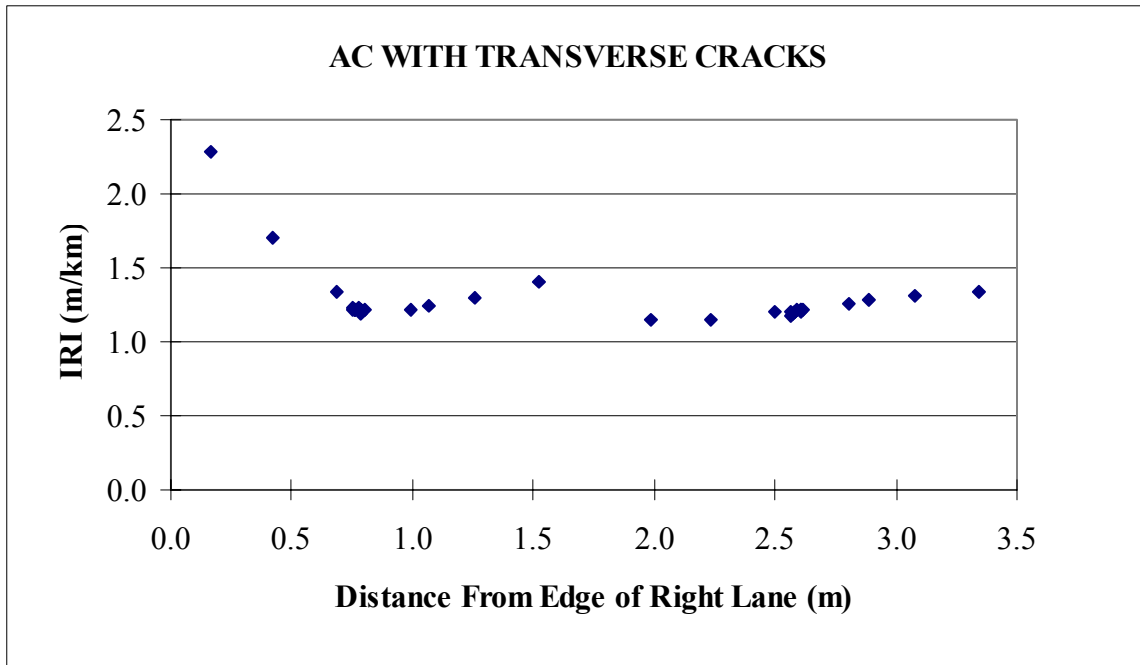


Figure 50. Variation in IRI across lane for an AC pavement with transverse cracks.

The IRI of this section were relatively consistent over most of the lane, but indicated much higher roughness within 0.7 m from the edge.

A shift in the path followed by a profiler at this section from the central tracking position to 0.3 m left caused the mean IRI to increase by 5 percent with respect to the mean IRI obtained at the central tracking position. A path 0.3 m to the right of the central tracking position caused the mean IRI to increase by 16 percent with respect to the IRI obtained for the central tracking position.

#### *Six-Year-Old PCC*

Figure 51 shows the variation in IRI across a lane of the six-year-old PCC pavement. Overall, the IRI of this section covers a large range. The IRI is lowest near the left edge and increases as the tracking position shifts from left to right. The increase is fairly linear (about 0.002 m/km per cm of lateral shift) until the tracking position shifts to 0.7 m from the right edge, then the IRI increases sharply. This section is still in good condition and has very little localized distress at the surface. Most of the roughness stems from slab effects, so the smooth trend across the lane is no surprise. The slabs are an average of 12.5 m long, and they are all cracked transversely in the middle. The half-slabs are curled upward. (The edges were higher than the center.) This section is located on the outside lane and has a bituminous shoulder. The higher roughness at the pavement edge is due to curling effects along the unrestrained right edge of the pavement.

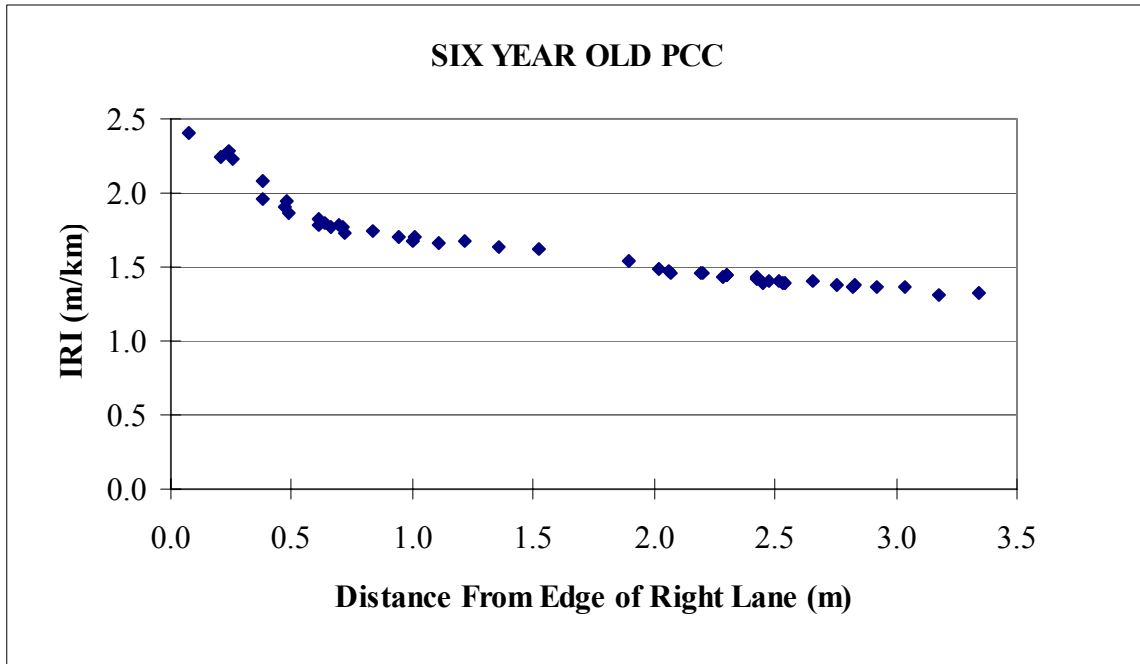


Figure 51. Variation in IRI across lane for a 6-year old PCC pavement.

### Summary of Results

Transverse variation in roughness can be significant. Variations in roughness up to 50 percent were observed across some pavement sections. Normal wander in a wheeltrack (typically 0.3 m) will commonly produce variation in IRI on the order of 5 to 10 percent. Generally, driving far to the right of center will cause an increase in the measured roughness. In order to obtain consistent measures of roughness, profilers should be driven along the wheeltrack of the pavement being tested. A monitoring system or windshield target can be used to aid in positioning the vehicle.

### Daily Variations

Daily variations in profile can occur at PCC pavements. The nominal curvature built into slabs depends on several factors, including mix properties, base support, slab length, layer thickness, joint type, and temperature and moisture of the concrete material during curing. Data collected for the NCHRP Project 10-47 (2) have shown that the PCC pavement can take a shape where the slab is curled upwards or downwards. Figure 52 shows an example of a PCC slab that is curled upward with respect to the center of the slab, where the joints are at a higher elevation with respect to the center of the slab. This slab has a joint spacing of 15 feet, which can be seen in the profile. Figure 53 shows an example of a PCC slab that is curled downwards, where the joints are at a lower elevation with respect to the center of the slab. This slab has a joint spacing of 9 m, which can be seen in the profile. These shapes were observed during a period when the temperature differential between the top and bottom of the slab was low, and therefore the shapes

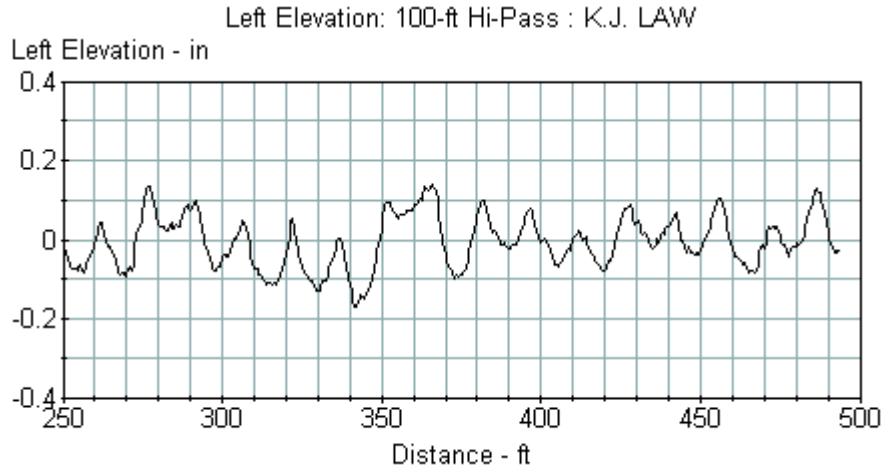


Figure 52. Slab with joints curled up

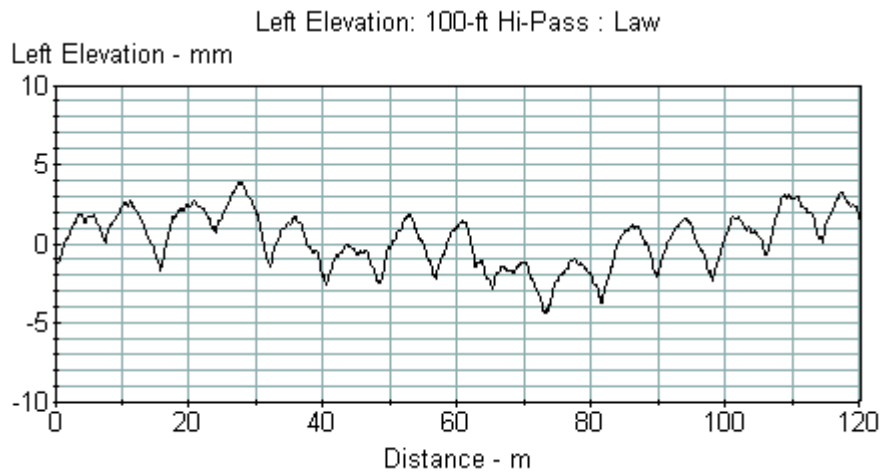


Figure 53. Slab with joints curled down.

were not a result of the temperature gradient. These curvatures are a result of the locked-in curvature in the slab that occurs because of construction conditions or are related to moisture variations in the slab.

Changes in temperature over a twenty-four-hour cycle interact with design and construction factors to cause variations in slab shape throughout the day. When the temperature starts to increase in the morning, the roughness of a slab that has a shape shown in figure 52 shows a decrease in roughness as the slab becomes flatter with an increase in temperature. However, if the slab has the shape as shown in figure 53, the roughness of the slab increases with increasing temperature as the curvature of the slab becomes more pronounced.

In NCHRP Project 10-47 (2), an analysis of roughness data collected at PCC sections in the LTPP seasonal monitoring program sites was performed to quantify the variations in IRI with changes in temperature. Data from eleven PCC test sections that had sufficient data were

selected for analysis. Of these eleven sections, four were jointed reinforced concrete (JRC) and seven were jointed plain concrete (JPC). The data available at these sites are not comprehensive enough to provide a systematic understanding of daily variations in roughness of jointed concrete, but they do provide an estimate of the level of variation in roughness and slab curvature that is possible.

Table 6 lists the four JRC sections and their mean IRI (MRI), which is the average of IRI from left and right wheel paths, at various times and dates. An evaluation of the slab shapes indicated that all four of these sections were curled downward at all of the times and dates listed. (i.e., the center of the slab was at a higher elevation when compared to the joints). As shown in table 6, the IRI of these sections was higher in the afternoon when compared to IRI in the morning. As all these slabs were curled downwards, an increase in temperature over the day caused an increase in the curvature thus resulting in a higher roughness.

Table 6. Daily variation in IRI at JRC pavements.

GPS Num (State)	Slab Len. (m)	Date	Season	Time		MRI (m/km)		
				Morning	Afternoon	Morning	Afternoon	Change
1606 (Penn.)	14.2	1/11/96	Winter	6:33	11:40	1.46	1.49	0.03
		4/10/96	Spring	8:40	15:08	1.51	1.55	0.04
		8/29/96	Summer	7:45	13:37	1.59	1.61	0.02
4018 (New York)	19.4	4/18/95	Spring	5:18	15:18	1.63	1.69	0.06
		4/10/97	Spring	9:21	14:35	1.91	1.97	0.06
4040 (Minnesota)	8.2	4/22/95	Spring	9:12	15:49	2.03	2.17	0.14
		6/27/95	Summer	8:15	16:10	1.94	1.99	0.05
4054 (Kansas)	9.1	1/17/96	Winter	9:35	13:04	1.80	1.81	0.01
		4/21/96	Spring	7:53	16:38	1.78	1.90	0.12
		9/17/96	Summer	5:26	12:29	1.59	1.78	0.19

Table 7 shows the MRI values on the seven JPC sections at various times and dates. An evaluation of the profile data indicated that Section 3019 in Georgia was the only section that showed a downward curl, where the center of the slab was at a higher elevation when compared to joints. All other sections were curled upward, where the joints were at a higher elevation when compared to the center of the slab. All sections that were curled upward showed a lower IRI in the afternoon when compared to the IRI in the morning. Section 3019 from Georgia, which is curled downward, is the only section that is rougher in the afternoon than in the morning. The most significant change in IRI is seen at section 3011 (from Utah). The change is more than 10 percent throughout the day during spring, but only about 3 percent on the winter date.

Measurements that were taken in 1994 for the Road Profiler User Group (RPUG) showed a reduction in IRI from 1.78 m/km in the morning to 1.45 m/km in the afternoon along a wheel path of a jointed PCC pavement in Nevada (32). This is a change of nearly 20 percent, and may affect the way this section is judged by a pavement management engineer.

The roughness of all jointed PCC pavements includes some contribution, often significant, from the prevailing shape of the slabs. In network-level profiling the roughness of most sections is rarely measured more often than once per year. The exact time of day and

Table 7. Daily variation in IRI at JPC pavements.

GPS Num (State)	Slab Len. (m)	Date	Season	Time		MRI (m/km)		
				Morning	Afternoon	Morning	Afternoon	Change
3019 (Georgia)	6.1	1/26/96	Winter	6:44	12:12	1.69	1.73	0.05
		4/5/96	Spring	7:15	13:02	1.55	1.61	0.06
		10/17/96	Fall	7:48	16:11	1.52	1.54	0.02
3002 (Indiana)	4.7	10/24/95	Fall	7:48	16:01	2.09	2.00	-0.09
		4/3/96	Spring	7:23	11:36	1.89	1.80	-0.09
3011 (Utah)	4.6	5/18/95	Spring	7:05	14:55	1.97	1.78	-0.19
		3/2/97	Winter	10:11	14:35	2.14	2.07	-0.07
		4/25/97	Spring	7:40	12:34	2.18	1.97	-0.21
3802 (Manitoba)	4.6	4/28/95	Spring	8:13	15:56	3.27	3.23	-0.04
		6/26/95	Summer	8:44	16:46	3.32	3.27	-0.05
3018 (Nebraska)	4.7	1/14/96	Winter	7:34	18:27	1.90	1.77	-0.13
3023 (Idaho)	4.1	9/9/94	Fall	11:47	15:01	1.51	1.48	-0.03
3042 (California )	4.7	11/30/95	Fall	9:43	17:45	1.03	0.98	-0.05
		5/8/96	Spring	9:26	15:17	1.02	0.96	-0.06
		8/14/96	Summer	10:39	13:51	1.05	1.02	-0.03

weather conditions associated with each measurement are not likely to be repeated each time a section is monitored. The time and date of measurements should accompany any roughness value that is entered into a database. This leaves the analyst free to consider possible daily variations as a cause of anomalous changes in roughness throughout the life of jointed concrete pavements. Roughness values on these pavements must be viewed as a sampling of the actual roughness, which fluctuates. The limited data discussed here showed that the roughness changes very little throughout the day in some cases, up to 0.2 m/km in others, and even more in extreme instances. If a specific design is prevalent among jointed concrete pavements in a given road network, it may be of interest to measure a few sections several times throughout a sunny day that follows a cool night to quantify the variation that is possible on that design. Planning of profiling for project-level monitoring of jointed concrete pavements must account for possible daily variations in slab shape.

### Seasonal Variations

Environmental effects can cause cyclic changes in roughness. These changes are difficult to predict, because they are so heavily linked to temperature and moisture. This sections presents information on seasonal changes in roughness at composite pavements and on AC pavements located on granular bases.

### *Composite Pavements*

Novak and Defrain (42) reported changes in profile of composite pavements in Michigan that took place between the summer of 1990 and February of 1991. Nine examples that included three different seasonal effects on composite pavement were described, which were:

1. PCC pavements with joints that have deteriorated due to D-cracking and then were overlaid with asphalt concrete: During winter, frost tenting action in the deteriorated PCC material at the joint caused a localized frost heave. During the thaw period, fines that formed because of D-cracking pumped. The loss of fines, because of pumping, caused a depression at the joint during summer.
2. Pavements with a frost susceptible base layer tilt or fault because of frost action: When the slabs tilted the back slabs rose at deteriorated joints and the fore slabs depressed (typical of faulting caused by pumping). Frost action in the base layer can also cause the fore slabs to rise above the back slab at joints and cracks.
3. PCC pavement with D-cracking at the joints that was replaced by removing deteriorated material and replacing it with a bituminous patch, then placing an overlay: In winter, the PCC slabs contracted and some lateral movement of the bituminous joint repair material caused a depression in the repair area. In summer, expansion of the PCC slab compressed the bituminous repair material, causing a bump to occur.

A pavement section that exhibited a combination of the first two effects increased in IRI from 1.96 m/km in summer to 2.88 m/km in the winter. Another section increased in roughness from 1.61 m/km to 4.23 m/km. In a pavement described by the third effect, the bumps at the joints shrank as the bituminous patches settled and the IRI decreased from 1.77 m/km to 1.22 m/km.

### *Asphalt Concrete Pavements on Granular Base*

Seasonal changes in asphalt concrete pavement profile occur mainly because of changes in volume of the subsurface layers. Typically, most of the movement is in the subgrade, but some movement may occur in the base. Seasonal changes in moisture conditions in the subgrade can occur, which results in volume changes in the subgrade. In freezing environments, subgrade that is susceptible to frost may change in volume and induce bumps on the pavement surface. This is called frost heave. Often, the bumps shrink or disappear after the freezing weather is over. These effects depend on annual precipitation, subsurface layer properties, and the depth of frost penetration.

The LTPP study designated a small subset of the sites of the general pavement studies to be profiled in each of the four seasons. Profile data from these “seasonal monitoring sites” were used to estimate the level of seasonal variation in IRI that is possible on asphalt concrete pavement. Table 8 presents the IRI values that were obtained at five test sections that were located in the dry-freeze zone. The IRI of two of these sections held steady. Two other sections grew steadily rougher, but not because of seasonal effects. Only one of the sections showed

elevated roughness in the winter. This section was 0.26 m/km rougher in the winter than in surrounding seasons.

Table 9 presents seasonal variations in IRI at four seasonal test sites in the wet freeze region. All four of these sections exhibit a seasonal change in IRI in at least one of the three years. In many cases, these sections were rougher in the winter than in other seasons, and roughest in February during maximum frost penetration.

Table 8. Seasonal effects on IRI at seasonal sites in the dry-freeze region.

State	Colorado	Idaho	Montana	Utah	Wyoming
GPS Number	1053	1010	8129	1001	1007
IRI (m/km)					
Fall 93	1.24	—	1.05	1.10	0.92
Winter 93-94	1.22	1.49	0.97	1.12	0.92
Spring 94	1.23	1.53	1.03	1.13	0.94
Summer 94	1.25	1.57	1.06	—	0.95
Fall 94	1.20	1.58	1.02	1.09	0.94
Winter 94-95	1.24	1.57	1.11	1.10	<b>1.26</b>
Spring 95	1.27	1.70	1.19	1.12	1.00
Range	0.07	0.21	0.22	0.03	0.34
Seasonal Effect?					×
Minor Changes Only?	×			×	
Steady Progression?		×	×		

— No data available.

Table 9. Seasonal effects on IRI at seasonal sites in the wet-freeze region.

State	Connecticut	Maine	New Hampshire	Vermont
GPS Number	1803	1026	1001	1002
MRI (m/km)				
Fall 93 (July-Sept)	1.55	1.48	0.66	—
Winter 94 (Jan)	1.73	—	—	—
Winter 94 (Feb)	1.84	1.52	1.07	—
Spring 94 (Apr)	1.60	1.41	0.73	1.15
Summer 94 (July-Sept)	1.57	1.41	0.74	1.32
Fall 94 (Oct)	1.57	1.37	0.68	1.20
Winter 95 (Jan)	1.57	1.54	0.87	1.25
Winter 95 (Feb)	1.62	1.60	0.72	—
Spring 95 (May)	1.60	1.38	0.68	1.18
Summer 95 (June-July)	1.58	1.37	0.74	1.22
Winter 97 (Jan)	1.64	1.12 <sup>a</sup>	1.35	1.29
Winter 97 (Feb)	1.63	1.18	1.56	1.54
Spring 97 (Apr)	1.67	0.96	0.85	1.19
Seasonal Affect in Winter 1994?	Yes	Yes	Yes	—
Seasonal Affect in Winter 1995?	No	Yes	Yes	No
Seasonal Affect in Winter 1997?	No	—	Yes	Yes

— No data available.

a. Resurfaced.



For example, section 1803 in Connecticut increased in IRI by 0.18 m/km between July 1993 and winter 1994, then another 0.09 m/km by February. In the spring, the roughness decreased to the level of the previous fall. Section 1001 from New Hampshire exhibits the highest level of seasonal variation. If the IRI values from the winter are ignored, the roughness progresses steadily from 0.66 m/km to 0.85 m/km in three years. In every winter, the IRI is higher than the prevailing trend. In the winter of 1997, the IRI is double the value of the following spring. Figure 54 shows the changes in profile over the seasons at LTPP section 1001 in New Hampshire.

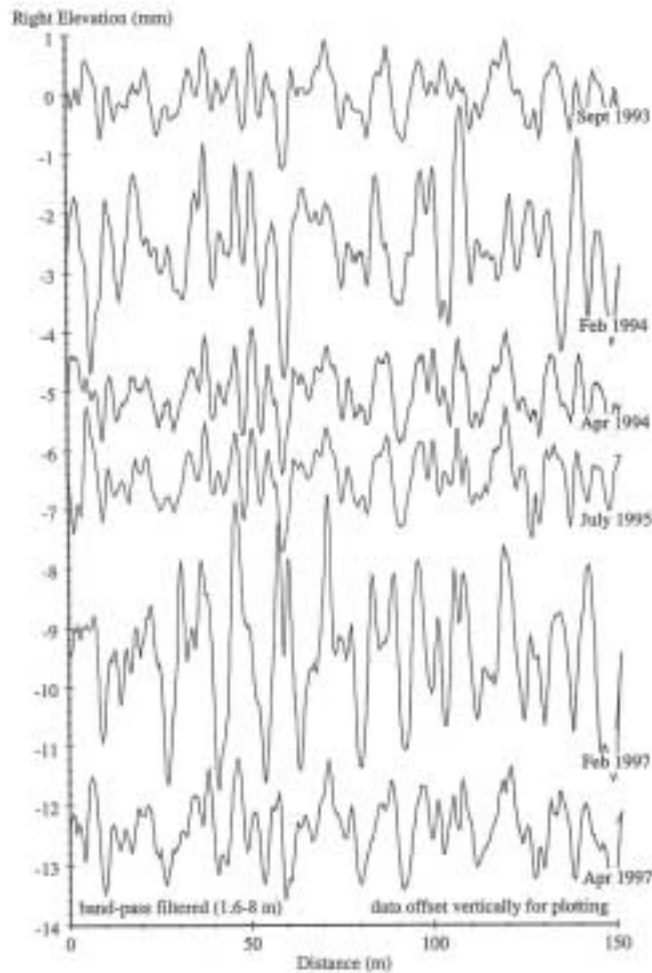


Figure 54. Seasonal changes in profile at LTPP section 1001 from New Hampshire (2).

The examples provided by the LTPP study show that very large seasonal changes in roughness are possible in asphalt pavement on granular base material. These changes do not occur every year because of variations in climate, but they do seem to be limited to winter. Profiler users should avoid measuring the roughness of their road networks in the winter. If this

cannot be avoided, pavement management engineers should recognize that roughness values measured in winter might be elevated significantly because of frost heave effects.

## Surface Texture

Surface macrotexture is the portion of the road profile in the range of wavelengths from 0.5 to 50 mm (43, 44). Coarse macrotexture elevates noise at the tire-road interface and increases the rolling resistance of vehicles. Macrotexture is not in the range of wavelengths of interest in the measurement of roughness indices such as the IRI and RN.

Ultrasonic profilers cannot collect accurate profile data on pavement surfaces that contain chip seals. An upward bias in IRI occurs at such sections. Early problems with ultrasonic sensors on coarse-textured asphalt were reported in the development of the South Dakota profiler (36). Huft reported that coarse surface texture increased the IRI on some sections up to 0.2 m/km. Most of the error in roughness was caused by aliasing, but Huft also reported that increasing the operating speed exacerbated the effect, because the echo of the acoustic ping became scattered and harder to detect. This occurred mostly on sections of very large open-graded aggregate. The effect of coarse chip seals on roughness measurement was well documented in the 1993 RPUG study (31). In the study, profilers with ultrasonic sensors measured IRI that was 50 to 100 percent high on sections with a chip seal.

Testing performed for the NCHRP study 10-47 (2) also showed that ultrasonic profilers obtain high IRI values at sections with chip seals. In this project, testing was performed on a chip seal section by two laser profilers, an ultrasonic profiler, and an infrared profiler. Measurements were also obtained at this section using the Dipstick. Table 10 presents the IRI values obtained by the different devices at the test section. This table also presents the bias in IRI with respect to the IRI obtained by the Dipstick. The IRI of infrared and laser profilers agreed reasonably well with the Dipstick, but the IRI of ultrasonic profiler was 30 percent higher.

Table 10. IRI values obtained at chip seal section.

Profiler	Number of Runs	IRI (m/km)	Bias (%)
Dipstick	1	2.87	—
Infrared	3	3.09	7.9
Laser 1	3	3.01	5.0
Laser 2	3	2.97	3.5
Ultrasonic	3	3.92	29.7

— Reference measurement.

After the RPUG study, the profiling community in North America widely recognized that ultrasonic height sensors were not sufficient for measurement of IRI. Many state agencies have retrofitted their profilers that had ultrasonic sensors with laser sensors.

## **Pavement Distress**

Pavement distress has a large effect on the current roughness of a pavement section, as well as the progression of roughness. Most types of pavement distress that are captured in a profile measurement appear as severe features that increase the value of a computed roughness index. The sampling interval of a profiler will have an impact on the features that are collected on pavements that have distress. Profilers that have a large sampling interval may miss some features. The sensor footprint of the profiler can also have an impact on the data collected on distressed pavements, as the height of the pavement feature that is picked up varies with the sensor footprint. For example, the sensor footprint of a laser will cover only a very small pavement area, while the sensor footprint of an infrared profiler will cover a much larger area.

Lateral variability has an interacting effect with distress types. For a specific profiler, the roughness indices that are obtained at a specific pavement section can vary from run to run. This variability will be caused by lateral variations in the path that is followed. If a series of repeat profiler runs are performed at a test section with a specific profiler, the difference between the maximum and the minimum roughness indices that are obtained from the series of runs will vary with the type of distresses that are present at the test section. For AC pavements, the largest differences were seen in pavements that had medium to high severity alligator cracking. For PCC pavements the largest differences occurred in pavements that had spalling at transverse cracks or joints. At PCC sections where the roughness was occurring primarily from faulting at the joints, the difference between the maximum and minimum IRI obtained from a series of repeat runs was generally low.

In some pavements, only a specific distress is present (e.g., transverse cracking in AC pavements or faulting on PCC pavements). However, usually a pavement will include several types of distresses that make it difficult to quantify the effect of each distress (e.g., PCC pavement can have faulting, spalling and transverse cracking, AC pavements can have rutting, alligator cracking, and transverse cracking). General guidelines regarding the variability in roughness values that are expected for different distress types are presented in this section for each of these distress types. When a current roughness value for a pavement section is compared with the previous years roughness value, the magnitude of the difference between the values may be influenced by the types of distress that are present in the pavement.

### *Alligator Cracking*

Pavements with alligator cracking tend to have a high level of transverse variability. Frequently in sections that have high severity alligator cracking, pieces of asphalt concrete may have been dislodged from the roadway. Such features will be measured as large downward spikes in the profile. If a series of repeat runs are made by the same profiler on a pavement section that has alligator cracking, the difference between the maximum and the minimum IRI that is obtained can vary from 0.20 to 0.50 m/km (3).

### *Transverse Cracking*

All roughness indices consider upward deviations from the pavement surface such as bumps and downward deviations from the pavement surface such as cracks in computing the roughness index. A profiler could either record or miss a bump or crack depending on the sampling interval. A profiler that has a shorter sampling interval has a higher probability of recording bumps or cracks than a profiler with a larger sampling interval. Variability in the collected profiles can be caused by the sampling interval of the profiler. The depth of the crack that is recorded can vary depending on the sensor type of the profiler. The likelihood of including portions of a crack as part of a profile measurement depends on the width of the crack compared to the sensor footprint and the sample interval of the profiler. Height sensors with smaller footprints are much more likely to measure a crack as a large downward spike in a profile. For certain applications, this is undesirable because most cracks in the pavement are enveloped by vehicle tires.

Transverse cracking on AC pavements can cause variation of roughness indices between repeat runs. A major contributor to this variability is whether a crack is picked up or missed between the runs. The difference between the maximum and minimum IRI obtained from a series of profiler runs at a AC section having transverse cracking has been observed to range from 0.1 to 0.2 m/km (3). PCC pavements that have transverse cracking can also cause similar variations in IRI.

### *Faulting*

If the magnitude of faulting is generally uniform adjacent to a wheel path, then its contribution to lateral variability is small. That is even if there is a variation in the magnitude of faulting between the left and right wheel paths, but if the faulting is fairly uniform adjacent to the left and right wheel path, lateral variability will not cause large changes in the roughness index. Usually the difference between maximum and minimum IRI from a series of repeat runs made with a profiler on a section that has faulting will be less than 0.10 m/km (3). This is because lateral variations in the profiled path will not have a major effect on the magnitude of the fault that is picked up, as the faulting at locations adjacent to each wheeltrack will be fairly constant. However, if spalling at joints is present in addition to faulting, the variations between the maximum and minimum IRI will be much larger.

### *Spalling*

Spalling occurs at joints or cracks in PCC pavements. A profiler that has a shorter sampling interval has a higher probability of recording spalling than a profiler with a larger sampling interval. If the length of the spall is small (i.e., length along the direction of travel in the pavement) and the sample interval is larger than the length of the spall, there is a possibility that the spall may not be recorded. The width of the spall (i.e., width across the pavement) will also be a factor that will contribute to lateral variability. If the width of the spall is small, there is a possibility for repeat profile runs to either capture it or miss it. If the width of the spall is large, the probability of it being captured during repeat profiler runs will be higher. A spall will not

have a uniform depth across its length and width. The depth of the spall will vary across its width as well as along its length. The height of the spall that is captured during a profile run will vary with the relative position at which the reading is taken. Because of this phenomenon, spalling can contribute significantly to lateral variability. If a series of repeat profiler runs are made with a specific profiler on a pavement that has spalling, the difference between the maximum and minimum IRI values can range from 0.1 to 0.2 m/km (3).

## Curves

Lateral acceleration that results from operating on curves can contaminate accelerometer measurements in a profiler if the accelerometer does not stay vertical. When a vehicle negotiates a curve, it undergoes small levels of lateral acceleration. For example, the AASHTO Policy on Geometric Design of Highways and Streets (45) allows highways with superelevation of 4 percent to have curvature that corresponds to a lateral acceleration of 0.15 g if the vehicle is moving at the design speed. Highways with superelevation of 10 percent may have curvature that requires lateral acceleration of 0.23 g at the design speed. The potential error in profile measurement on curves occurs if the vehicle is accelerating laterally and tilts sideways simultaneously.

A study performed for NCHRP project 10-47 (2) found that errors in roughness indices due to curves are not significant until lateral accelerations exceed 0.15 g. This will not be a common problem on major highways with reasonable driving practice. On secondary roads with significant curvature, errors caused by lateral acceleration can be minimized by reducing speed. (Lateral acceleration on a curve is proportional to the square of speed.)

## Hills and Grades

Hills and grade affect profiler accelerometer signals as the orientation of the accelerometer is changed. A study performed for NCHRP Project 10-47 (2) found that when grades are in the range of 3 to 6 percent no problems are expected in IRI or RN. Changes in grade, which cause accelerations of 0.15 g can affect the profile. The effect on IRI will be minimal but the visual changes in the profile can be large.

## MEASUREMENT ENVIRONMENT

This section discusses the effect of the conditions in which profilers must operate on their performance. These factors, termed the *measurement environment* include aspects of the surroundings that might affect the profile measurement process, but do not relate to the actual shape of the pavement surface. Measurement environment factors that are covered in this section include wind, temperature, humidity, surface moisture, surface contaminants, pavement markings, pavement color and ambient light.

The factors covered in this section generally affect height sensor accuracy in two ways: (1) causing a bias in all measurements by a height sensor (akin to an error in calibration), and (2) causing some extremely erroneous height sensor measurements that appear as spikes in the measured profile. Sensor bias errors are avoided by operating a profiler only under conditions in which it was meant to operate. For example, most height sensor manufacturers will provide a range of air temperatures for which the sensor is valid. Height sensor spikes can often be avoided the same way. Each type of height sensor is prone to bad readings caused by some aspect of the measurement environment. For example, ultrasonic height sensors are prone to spikes in high wind, optical sensors are prone to spikes caused by changes in light and surface reflectivity, and all types of height sensor are prone to spikes caused by surface contaminants. Table 11 lists the factors covered in this section and the types of height sensors that are affected by them.

Table 11. Effect of measurement environment on height sensors (2).

Factor	Ultrasonic	Laser	Infrared	Optical
Wind	●	○	○	○
Temperature	●	●	●	●
Humidity	○	○	○	○
Surface Moisture	●	●	●	●
Surface Contaminants	●	●	●	●
Pavement Markings	○	○	○	●
Pavement Color	○	○	○	—
Ambient Light	○	○	○	●

● - Strong Effect                      ● - Effect Under Unusual Circumstances  
○ - Small or No Effect                — - Insufficient Information

## Wind

In profilers with ultrasonic sensors, severe winds interact with the vehicle to generate sound that causes invalid ultrasonic height sensor measurements. Huft (36) reported that winds exceeding 65 km/hr oriented at certain angles to the profiler are likely to interfere with ultrasonic height sensor measurements. Severe winds also cause measurement errors if a significant amount of sand, snow, or other surface contaminants pass under the profiler. Heavy winds can also make it difficult to track a consistent path.

## Temperature

Extreme air and surface temperatures have the potential to cause errors in height sensor measurements. In laser height sensors, a large temperature gradient along the path of the beam can induce curvature in its path. Still (46) studied this phenomenon and found that its effect was negligible for reasonable temperature gradients. Laser sensors are also slightly sensitive to ambient air temperature. Selcom reports in their specifications that their laser sensors operate

properly in temperatures ranging from 0 to 40 °C, and exhibit an error of 0.005 percent of the total range per degree C (a negligible error in profiling applications) (47). Most accelerometers operate properly over a much broader range of temperatures.

Ultrasonic height sensors are extremely temperature sensitive. Lawther (45) reported that ultrasonic height sensor measurements that pass through a 5.5 °C temperature gradient would exhibit a bias equal to 4 percent of the distance covered by the gradient. A more comprehensive study was performed in 1992 that focused on performance of an entire profiling system with ultrasonic sensors (49). This study found a significant upward trend in IRI with air and surface temperature dramatic enough to render the device useless for roughness measurement in network or project-level applications. There was consistently an upward trend in IRI with air temperature (between 25 and 35 °C) with magnitudes of up to 0.03 m/km per degree °C. If the results of this study were representative of the temperature sensitivity of profilers with ultrasonic height sensors, it would render them in need of constant calibration (more often than daily) to be sufficient for roughness measurement.

All of the manufacturers brochures encountered for optical and infrared height sensors indicate insensitivity of sensors to temperature, humidity, and wind (2). Although these are advertisements, there is no experimental evidence that they are incorrect.

## **Humidity**

Humidity (within reasonable limits) is not likely to have a significant effect on laser, infrared, or optical height sensor performance as long as the sensors are clean and free of condensed water. For example, Selcom reports in their specifications that their laser sensors operate properly as long as the humidity is below 90 percent and noncondensing. K.J. Law Inc. also mentions in their advertising that their infrared sensors are not sensitive to humidity. Since humidity has only a very weak influence on the speed of sound in air, it is unlikely that a significant influence on ultrasonic height sensors exists. Moisture in humid conditions may contaminate the transmission path of the beam in any noncontact height sensor if water condenses on the surface of emitters (such as a laser light source), pick-ups, lenses, or mirrors. This was cited as the cause of reliability problems in a study of profiler performance in Virginia, where conditions are frequently humid (49). In such conditions, it is important that the operator check emitters, lenses, and mirrors and clear condensed water from them frequently. Power should not be supplied to the sensors when cleaning the sensor and related components since direct laser light will damage a person's vision.

## **Surface Moisture**

Pavement profiling is usually not performed on wet pavements. Certainly, no profiling system is going to function properly if the sensors pass over snow or ice-covered pavement. However, it is probably not unusual to encounter rain in the middle of a day of profiling. The question is: When is the road so wet that profiling should cease? In a study of profiling with laser sensors Still and Jordan (46) reported that sensor dropout could occur if the surface texture

is submerged in water. As suggested by that study, profiling should stop if the surface texture is submerged and may continue, “as soon as the surplus water on the road surface has drained away.” Generally, a good guideline to follow is not to perform profiling if traffic is causing the surface water to splash or spray.

### **Surface Contaminants**

Surface contaminants are an unavoidable aspect of the pavement environment. Litter such as dead animals, vehicle parts, leaves, or dirt on the roadway can cause errors in profile measurements. In measurement of new construction, where no traffic is present, contaminants should be removed if they are in the path of the height sensors. In monitoring of in-service roads, it is not practical to remove them, and they cannot always be avoided.

Unfortunately, some surface contaminants can add substantially to the apparent roughness of a section. For example, a piece of tire tread 2.5 cm in height and 2.5 cm wide laying across a wheeltrack adds about 0.09 m/km to the IRI of a section 160 m long (2). A profiler with a long sample interval may not detect the tread, but if it does, aliasing errors will cause the profiler to misinterpret the tread as a larger disturbance, and the error could be as much as three times as large. Operators that suspect a contaminant was included in a measurement should always indicate their presence with an event marker. If contaminants such as dirt, snow, or blowing leaves are so abundant on a section that they continuously interfere with the profile measurement, the data should simply not be recorded. In pavement management, last-year’s roughness is a better estimate of the current road condition than a measurement with major errors in it.

### **Pavement Markings**

The majority of pavement markings appear along lane edges where they are unlikely to be encountered during profile measurement. However, some markings that go across the lane, such as those used to indicate stop lines, school locations, and railroad crossing markings appear on secondary roads. The change in surface reflectivity caused by white pavement markings on an otherwise dark pavement surface can cause spikes in the profile data on for data collected by profilers with optical sensors. Profilers with infrared, laser, and ultrasonic sensors are not known to be affected by the presence of pavement markings along the profile path. A study performed for NCHRP Project 10-47 (2) showed that infrared, laser and ultrasonic profilers were not affected by pavement markings along their travel path.

### **Pavement Color**

Based on the results presented for pavement markings, it is unlikely that ultrasonic, laser, and infrared sensors are susceptible to errors at a transition in pavement surface color, as that occurs between a PCC and an AC pavement. A study performed for NCHRP Project 10-47 (2)



using laser, infrared and ultrasonic sensors showed that none of the sensor types showed a sensitive to pavement color change when going from an AC section to a PCC section.

### **Ambient Light**

Laser, ultrasonic, and infrared height sensors are not affected by changes in ambient light. Optical sensors, however, do not operate properly if the signal is contaminated by sunlight. Exposure of the optical height sensor beam to even a small amount of sunlight can induce major errors in the collected profile. To eliminate this error source, K.J. Law profilers with optical sensors are fitted with a shroud that keeps the environment around the optical sensors in the shade at all times. If the shroud is in good repair, no errors should result. However, sunlight creeping under the shroud has known to result in major errors in roughness indices.

## **PROFILER OPERATION**

This section covers the quantifiable aspects of the manner in which a profiler is driven and operated. These factors are all under the control of the people using the profiler. Some of them interact with the pavement surface shape to affect the measured profile. These are considered sources of variation instead of error. For example, the path a profiler takes over a section has a strong influence on the roughness it measures because of transverse variations in profile. Two measurements that follow a different path can produce equally valid but different results. The starting point of a section also determines what features are included in a measurement.

Other aspects of profiler operation that are under the driver's control can lead to errors. Driving at speeds outside of the recommended range for a profiler or aggressive braking can cause invalid measurements. Speed and acceleration are particularly relevant to profiler drivers who must cover significant distance every day or profile in confined areas. Drivers do not always have complete control over their speed, but should know when a measurement is no longer valid because of low speed or excessive deceleration.

The profiler operation factors that are described in this section include operating speed, speed changes, lateral positioning, longitudinal positioning/triggering, frequency of data collection, and profiler operation checks.

### **Operating Speed**

The range of valid operating speed depends on the design of the profiler. The manufacturer usually specifies the range of speed in which valid profile data can be collected. Inertial profilers have to operate at some speed to function, but the profile it measures should depend only on the properties of the road at the time of the measurement and the particular path the profiler takes. If the output of a profiler depends heavily upon its operating speed, it is not a valid profiling device.

Most profilers are valid over a broad range of operating speed. The maximum speed at which a profiler may operate is limited by its data collection rate. Fortunately, computer speed has improved so much in recent years that data collection rate is a lesser concern than in the past. Most high-speed laser, optical, and infrared profilers currently on the market collect profile at sample intervals of 25 mm or less up to speeds well above 100 kph. The operating speed of profilers with ultrasonic sensors is limited by echoing of the acoustic ping. The ping must travel from the sensor to the road surface and back for each reading. This takes about 0.002 seconds. Unfortunately, multiple echoes of the ping last much longer, such that the sensor can only make a measurement every 0.01 seconds (36). At a travel speed of 109 kph, this is only one sample every 300 mm. At this sampling rate, the lack of anti-aliasing filters renders measurement of wavelengths below about 2 m invalid, particularly on roads with coarse macrotexture and rough megatexture. To sample the road every 75 mm, the profiler must slow to 27.2 kph.

Operating speed is also limited on very rough roads if the profiler bounces or pitches excessively. A combination of the roughness of the road and high speed can cause a profiler to respond so dramatically that the height sensor reading goes out of range. This is not likely to occur on interstate or primary roads. However, profilers with bumper-mounted sensors may be prone to this difficulty on rough secondary roads.

The minimum speed at which a profiler should operate is dictated by the longest wavelength it needs to measure. An inertial profiler uses an accelerometer to sense vertical movement of the vehicle and establish an inertial reference. The amplitude of the accelerometer signal decreases rapidly as wavelength increases. At some cutoff wavelength, the amplitude of the accelerometer signal is so low that it is masked by sensor noise. The cutoff wavelength gets shorter at lower speeds, and at some low speed a portion of the wavelength range of interest is affected. Most profiler manufacturers are well aware of this phenomenon and provide a low speed limit to the customer. A common low speed limit of a profiler is 25 kph, but some models can measure valid profile at operating speeds as low as 15 kph.

## **Speed Changes**

Speed changes involve cases where the vehicle has to accelerate or decelerate due to prevailing traffic conditions. Profilers must often operate in situations that include bringing the vehicle to a dead stop. The following situations can cause a profiler to undergo speed changes during operation:

- when a stop signal is encountered in urban areas;
- in network monitoring applications, when heavy traffic or merging traffic can cause speed changes;
- in network level operations where the driver must stop occasionally at the roadside as part of the measurement routine, then resume measurement;

- in monitoring of new construction, when limited distance is available ahead of a road section.

The study of the effect of these factors on profile data collection was performed for NCHRP Project 10-47 (2). This study formulated the following guidelines for collecting profile data under such circumstances to minimize the effect on IRI.

- Moderate acceleration and deceleration of 0.15g (about 5 kph per second) and below can be tolerated in network-level profile measurement. These conditions are achieved as long as only moderate applications of brake or throttle are applied during data collection.
- Acceleration and deceleration should be held under 0.1g (about 3.5 kph per second) at all times in construction acceptance or project-level applications of profilers. Unless profiling with a limited lead-in or lead-out distance must be done, a constant speed operation should be adopted.
- When measuring from a dead stop the first 150 m of the profile should be ignored. A shorter distance is permissible if a profiler has special provisions for initializing profile computation from low speed.
- If a profiler makes a stop during data collection (e.g., at a stop sign) and resumes right away the 50 m of profile ahead of the stop and the 150 m after the stop is invalid.

### **Lateral Positioning**

Roughness varies significantly across the lane of most pavements. Consistent lateral positioning of a profiler is essential to obtaining repeatable measurements, particularly on pavements with significant surface distress. Standardizing the lateral positioning of profile measurement would greatly improve the repeatability of roughness values. Gillespie et al. (3) offers the following guidelines that can result in a consistent lateral position during profile measurements.

- Drive in the center of the lane or the center of the ruts.
- Attempt to drive as straight as possible.
- Windshield or camera targeting systems can be used as a guide in training a driver to maintain a consistent lateral position.
- Profiler operators should perform repeatability tests on pre-established courses prior to initiation of field surveys and a means of developing consistent lateral positioning.

## Longitudinal Positioning/Triggering

The start of data collection during profiling can be performed by either manual or automated methods. In the manual method, data recording is initiated by pressing a pendant or a specific key on the computer. In the automated method, a photocell in the profiler is used to detect a mark that is placed on the pavement surface, or a reflective tape attached to a cone that is placed on the shoulder of the pavement. Gillespie et al. (3) offer the following guidelines related to initiation of data collection to obtain accurate profile measurements.

- Most profilers need some lead in distance after the system is turned on for the filters (used in profile computation) to stabilize. Therefore, the profile data collection system should be in operation before the beginning of the segment that is to be measured. In this phase of operation, data is collected but not recorded. Details regarding the lead in that is needed before the valid data can be collected should be obtained from the profiler manufacturer. This error is more serious for project-level measurements than for network-level measurements. If the data recording is initiated without a sufficient lead in prior to the test section, the data that is collected at the beginning of the section can be erroneous.
- For network-level data collection, manual triggering is sufficient to initiate data collection.
- For project-level data collection, especially for profiling new pavements to identify specific roughness locations, an automated method should be used to initiate data collection (i.e., a photocell). This will ensure that pavement features are correctly located within the section.
- For all studies that are performed to assess the repeatability of a profiler, the automated method must be used to ensure that the starting locations of all repeat profile runs are at the same location.

## Frequency of Data Collection

Successful monitoring of pavement roughness involves repeated measurement over the life of a road. Logistical and budgetary constraints usually dictate that large road networks can only be covered once per year or less. Currently, the interstate and other portions of the primary road network in most states are covered annually. The procedure for roughness data collection along a highway can vary between agencies. A survey performed by Gramling (39) indicated on two lane roads, 72% of agencies collect roughness data along both directions, while 28% of the agencies collect data along only one direction. The survey also indicated on 4-lane highways 72% of the agencies collect roughness data on the outside lane in both directions, 13% collect data on one lane in one direction, and 15% collect data along all lanes in both directions.

Gillespie et al. (3) recommended data collection for a specific region be performed during the same time frame each year to minimize the impact of seasonal variations in profile data. Gillespie et al. (3) recommend for studies that involve obtaining an overall assessment of variability in roughness over the year on AC pavements, profile measurements should be obtained four times a year. The section should be profiled in the middle of each season (spring,

summer, fall and winter). For PCC pavements, variations in roughness can occur throughout a day due to temperature effects. For studies involving an overall assessment of variability in roughness over the year on PCC pavements, a similar procedure as recommended for AC pavements should be followed. However, each PCC section should also be profiled three times during the day (morning, noon and afternoon) each time it is profiled.

### **Profiler Operational Checks**

Operational checks that should be performed by the profiler operators are described by Karamihas et al. (2). This section provides a summary of these procedures.

The operator of a profiler should perform regular operational checks of its measurements. Most profilers display sensor signals and profile elevation values numerically or graphically. The operator should check these displays periodically to make sure the profiler is providing plausible output. This is a burden, but a lesser burden than repeating several days of work or covering a large portion of the road network only to find out the data is useless. An operator who is familiar with a particular kind of profiling equipment knows the approximate value of roughness to expect on a particular road. Many operators are expert roughness meters by virtue of their experience.

A useful procedure for checking the accuracy of a profiler is to use it on a few sections regularly. An operator or manager should designate a few sections near the home base location of a profiler. The operator can measure one of these sections that is near the route the profiler is taking for the day to check its operation. At the very least, this should be done once per week. The roughness values and the profiles of these sections can be compared to a previous measurement to make sure the profiler is working properly.

### **PROFILER DRIVER/OPERATOR**

The driver and operator of a profiler have a tremendous influence on the quality of profile data. The driver controls the lateral positioning of the vehicle, which affects the measured roughness significantly. It is also up to the driver to control the speed of the profiler (which can rarely be held constant in mixed traffic), stay in the correct lane, and devote adequate attention to safety. The operator (who is often the driver as well) must prepare the profiler at the start of a day to make sure it is working properly, find data collection landmarks and trigger the system, conduct quality control during measurements, and often do on-the-spot maintenance. The operator must also make constant judgment calls in adverse conditions as to whether valid profile can be measured. The operator should ensure the three main components of the profiling system which are sensors, accelerometers, and distance measuring system are calibrated following the manufacturers recommendations.

It is definitely better to use a two-person crew than one person to collect profile data. This leaves one person free to worry about driving and safety, and the other free to ensure that quality

data is collected. A good way to help ensure quality data is to use the same profiling crew every year. Experienced drivers and operators have several advantages.

- They are familiar with the equipment.
- They usually already know what conditions lead to measurement error. (A new crew has not yet learned from mistakes.)
- They are more likely to recognize errors, because experienced profiler operators can usually guess the roughness of a road with reasonable accuracy.
- They have already made a habit of good measurement practices.
- They can better protect and maintain the equipment.
- They know the road system well.

It is not always possible to employ experienced drivers and operators, so managers must help them along in developing good and safe habits. New drivers and operators should spend the first several days in a profiler under supervision. This way, someone is available to help them learn the routine, and to provide an example of how to make decisions when unusual things happen. Even the most well written manual or instructions cannot cover everything that a driver and operator will encounter on the road. A new driver and operator do not always have the experience to do what a manager would suggest.

As a part of the NCHRP Project 10-47 (2), guidelines regarding training, driving skills, daily operating procedures and equipment calibration were developed. These guidelines are presented in the following sections.

## **Training**

A road profiler is a complex piece of equipment. Operators need training beyond the simple operating practices in order to assure data quality.

- The profiler operator should be trained to identify when valid and accurate data are being collected.
- The operator should be familiar with all operations that are involved in the calibration of the height sensors, accelerometers, and the distance measuring system.
- The operator should be familiar with the daily checks that need to be performed on the equipment prior to data collection to ensure that the height sensors and the accelerometers are working properly.
- The operator should be trained to recognize valid ranges for height sensor measurements and accelerometer measurements. The operator should be able to review the data that is being

collected by the height sensor and the accelerometer and be able to spot any problems with the data acquisition systems.

- The operator should have an understanding of roughness indices and the ride quality that is associated with each roughness index. Some profilers have the ability to display or print out the roughness indices at specified intervals (e.g., 150 m). The operator can review the roughness index and see if it is in agreement with subjective judgment of the road condition.
- It is recommended that a set of guidelines be developed by the agency that describes details regarding calibration, daily calibration checks, and other procedures to be followed during data collection. This will make it easier for the operator to follow and adhere to these procedures.

### **Driving Skills**

Driving practices can have a major influence on the roughness measurements.

- The driver of the profiler should be trained to correctly follow the wheeltrack. The driver should be made aware of the variations that can occur in roughness indices when an incorrect wheeltrack is being profiled.
- The driver should be trained to anticipate traffic conditions so that conflicts do not arise to disrupt the data collection process. These include anticipating conditions at merging ramps so that the driver is not forced to change lanes, sudden heavy braking that can drop the speed of the profiler below the minimum recommended speed, and to avoid heavy acceleration or deceleration.

### **Daily Operating Procedure**

The daily operating practices executed by the profiler driver and operator can have significant impact on the quality of the roughness measurement program.

- The sensors should be checked to see if there is any visible damage such as chipped or broken glass, and they should be checked to get rid of any dirt. The power to the sensors should be off while the sensors are being checked as laser sensors can cause eye injury.
- The tire pressure should be checked to verify that it is at the recommended value.
- The electronic equipment within the vehicle should not be turned on until the vehicle interior has been brought within the operating temperature range of the components. Generally, 10 to 15 minutes is sufficient for all components to equalize. However, during extremes in temperature, warm-up may take up to 30 to 40 minutes.
- Daily checks should be performed on the height sensors and the accelerometers prior to data collection. The procedures for performing the daily checks can vary between equipment. It is recommended that the profiler software be capable of performing a calibration check on the height sensors. In this procedure, a gauge block of known height is placed below the height sensor and the change in height is computed. This height should agree with the height of the gauge block within a specified tolerance. It is recommended that the profiler software be capable of performing a “bounce test.” In this test, a pitching motion is induced on the

vehicle while it is stationary and the profile data is collected. The collected profile should show an amplitude that is less than 1 percent of the motion that is induced. The electronic equipment should be turned on and given time to warm-up before performing either of these tests.

- Maintain a checklist on the tasks that have to be done daily prior to data collection.

## **Calibration**

Three components in the profiler require calibration—the height sensor, accelerometer, and the distance measuring system. If any of these systems are operated without calibration, the profile data that is obtained is questionable.

- The calibration of the height sensor and the accelerometer should be performed according to the procedures that are provided by the manufacturer. Calibration should be performed at the time intervals that are recommended by the manufacturer. Keeping track of the calibration parameters when the systems are calibrated will provide an indication of the relative stability of these components between calibration periods.
- The distance measuring system is calibrated by driving the vehicle over a known distance. The length of the calibration section should be known accurately within 0.05 percent. The calibration section should be laid out using an electronic distance measuring system or using a steel tape using standard surveying procedures.
- The distance measuring system should ideally be calibrated at the measuring speed that is usually used during data collection. The cold tire inflation pressure should be set at the recommended value, and the vehicle should be driven for 20 to 30 minutes to warm up the tires prior to the calibration.
- For the distance measuring system a record of the calibration factors that are obtained together with the vehicle mileage corresponding to the time when the calibration is performed should be maintained in order to track the stability of the system.
- The calibration of all three components should be performed whenever repairs are performed on the suspension system or when the wheels are aligned. The calibration should also be performed whenever tires are replaced or rotated.
- As a further check on the profiler, calibration sites should be established at convenient locations to verify that the equipment is working properly. These sites can be established close to the office, and should be profiled immediately after the profiler has been calibrated. These sites can again be profiled after the profiler returns from a network-level survey. A minimum of two calibration sites is recommended, a smooth section (IRI less than 1.2 m/km), and a fairly rough section (IRI between 2.3 and 3.0 m/km).



## USE OF INERTIAL PROFILERS FOR CONSTRUCTION ACCEPTANCE TESTING

Good profiling practices should be used when inertial profilers are used for construction acceptance testing. As incentives and disincentives are based on the results, it is imperative that the collected data should be error free. The following are some aspects that are related to the use of inertial profilers for construction acceptance testing:

- The height sensors, accelerometers, and distance measuring system should be calibrated at frequencies recommended by the equipment manufacturer. It is imperative that the distance measuring system provides accurate results as the location of roughness features within the section are located using the recorded distance.
- The processing algorithms in profilers used for construction acceptance should scan sensor signals rigorously for potential erroneous readings.
- Perform frequent checks to ensure that sensors are operating properly.
- The anti-aliasing filtering that is used should be robust.
- A sufficient lead-in should be allowed prior to beginning of the test section in order for the filters to stabilize. The profiler manufacturer should be contacted in order to determine the lead in that is required.
- Never operate on pavement that is wet or pavement with surface contaminants such as dirt or gravel.
- Operate at constant speed during data collection.
- Attempt to drive in a consistent lateral position.
- Repeat runs of each section should be obtained, with the start of data collection being initiated by an automated triggering system.

## CHAPTER 8

# SMOOTHNESS SPECIFICATIONS FOR PAVEMENTS

### INTRODUCTION

The traveling public judges the quality of a road by its smoothness. Studies have shown that pavements that are constructed smoother remain smoother over their life when compared to a comparable pavement that has a higher initial roughness level (10). Studies have also shown that pavements that are initially smooth have a longer life, when compared to a comparable pavement that has a higher initial roughness (10). This may be due to dynamic loading effects that are lower on a smoother pavement when compared to a rougher pavement. Higher dynamic loading effects can result in a faster deterioration rate in a pavement. Because of these beneficial effects of smooth pavements, highway agencies have been implementing smoothness specifications for new pavements, in order to construct smoother pavements. Many highway agencies have incentive and disincentive schemes, where the contractor is paid an incentives for achieving a higher smoothness level than a specified limit, and penalized with disincentives for constructing pavements that are outside the smoothness specification.

Currently, the most widely used device for measuring smoothness of new pavements is the profilograph. The PI that is computed from the profilograph trace is used to judge the smoothness of the pavement. The profilograph trace is also used to obtain the location of must grind bumps in the profile. There are differences between the measurements procedures as well as profilograph trace reduction procedures between different highway agencies. Each highway agency has its own set of specifications that determine how to measure and how to reduce the profilograph trace. In recent years, many highway agencies as well as contractors have adopted lightweight profilers to measure the smoothness of new construction. The lightweight profilers obtain the profile of the pavement, and thereafter a profilograph simulation is performed on the profile to obtain the PI of the pavement. Some State agencies are moving towards basing their smoothness specification on a roughness index that is computed from the profile data recorded by the lightweight profilers. Some State agencies have been using inertial profilers for acceptance testing of new AC pavements. A roughness index such as IRI is computed from the profile data, and acceptance limits are based on the roughness index.

An overview of the following topics dealing with initial smoothness of pavements is presented in this chapter:

1. Effect of smoothness specifications on initial smoothness.
2. State practices for smoothness testing and evaluation.
3. An example of a smoothness specifications used by a highway agency.
4. Appropriateness of incentive and disincentive levels.
5. Current trends in smoothness specifications.
6. Issues related to smoothness measurement on PCC pavements.

## **EFFECT OF SMOOTHNESS SPECIFICATIONS ON RESULTING PAVEMENT SMOOTHNESS**

A survey performed in 1994 indicated 60 percent of highway agencies employing smoothness specifications on AC pavements believe that the specification has resulted in increased initial pavement smoothness (10). The same survey indicated this value to be 72 percent for PCC pavements. The agencies also indicated the standard deviations in initial smoothness measurements for a project has decreased since implementing smoothness specifications.

Smith et al. (10) used data from four State DOT's employing initial smoothness specifications to investigate the effect of smoothness specifications on initial pavement smoothness. The data from all four States indicate that smoothness specifications have been effective in obtaining pavements that are significantly smoother than those constructed prior to the implementation of the specification. It appears that it takes a few years for contractors to become acquainted with smoothness specifications. After the implementation of a specification, the initial roughness generally decreases and continues to decrease as the contractor becomes more comfortable with the specification and more cognizant of items that can be done to increase the resulting pavement smoothness. The results of this analysis show that smoothness specifications are an effective way of improving the initial smoothness of pavements. When combined with incentive and disincentive provisions the contractors are encouraged to achieve smoother pavements. The following are the findings of Smith et al. (10), presented separately for each State.

### **Illinois**

The Illinois Department of Transportation implemented a smoothness specification for its PCC pavements in 1977. The specification was based on the California profilograph and calls for corrective action if the initial pavement roughness exceeds 15 in/mile. A new specification was implemented in 1993 containing incentive provisions for initial PCC pavement smoothness values of less than 4.25 in/mile. While using the California profilograph for the smoothness specification implemented in 1977, Illinois also continued measuring initial smoothness values with the BPR roughometer. Initial pavement smoothness for CRCP pavements constructed between 1975 and 1982 were analyzed to determine the effect of smoothness specifications on initial smoothness values. Data that were collected by BPR Roughometer was used in this analysis. Table 12 present the BPR roughometer values for pavements that were constructed under smoothness specifications, as well as the values for pavements that were not constructed under smoothness specifications.

The mean initial roughness of CRCP projects constructed after the implementation of the smoothness specification dropped to 71 in/mi from a value of 81 in/mi for projects constructed before the implementation of the specification. The standard deviation of the roughness index after the specification decreased from 21 in/mi before the specification to 12 in/mi after the

implementation of the specification. The distribution of the initial roughness before and after the implementation of the specification is shown in figure 55.

Table 12. Initial smoothness values for pavements constructed with and without smoothness specifications (10).

Measurement Time	No. Of Sections	BPR Roughometer Roughness (in/mi)	
		Mean	Standard Deviation
Not Under Specification	24	81	21
Under Specification	19	71	12

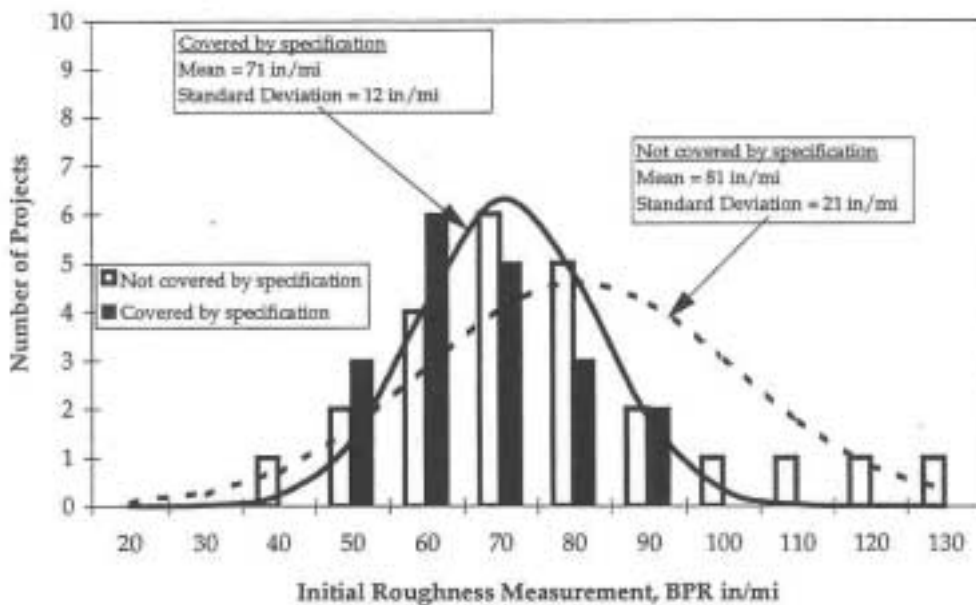


Figure 55. Comparison of initial pavement smoothness distributions before and after implementation of smoothness specification for Illinois CRCP projects (10).

## Georgia

Georgia implemented Maysmeter specifications for controlling initial pavement smoothness in 1981. The established specifications were 75 in/mi for PCC, 35 in/mi for dense graded new AC, 30 in/mi for open graded new AC, 45 in/mi for dense graded AC overlays, and 35 in/mi for open graded AC overlays. In 1983, the PCC specification was lowered to 65 in/mi. In 1986, AC specifications were changed to 30 in/mi for dense graded AC and 25 in/mi for open graded AC. Table 13 shows the mean project initial roughness values for 1981 and 1995. The values shown in table 13 reflect the improvement in smoothness that occurred over time with the tightening of the specifications.

Table 13. Maysmeter roughness values for new pavements in Georgia (10).

Pavement Type	1981 Smoothness (in/mi)	1995 Smoothness (in/mi)	Percent Change in Smoothness (%)
PCC	65	33	49
AC Overlay	35	25	29
AC	27	24	11

## Iowa

Iowa implemented a smoothness specification for their AC and PCC pavements in 1981 based on the California profilograph. The specification specified a maximum roughness of 15 in/mi and contained a disincentive clause. In 1985, an incentive clause was added to the specifications. The maximum roughness was reduced to 12 in/mi in 1987, and was again reduced to 7 in/mi in 1993. The mean project roughness values for AC and PCC pavements in Iowa in 1982 and 1993 are shown in table 14. These results show that the tightening of smoothness specifications and the introduction of incentives resulted in smoother pavement being built.

Table 14. Reduction in initial pavement roughness values in Iowa (10).

Pavement Type	1982 Mean project Roughness (in/mi)	1993 Mean Project Roughness (in/mi)
PCC (primary)	11.5	7.0
AC (Primary)	9.5	1.0

## Wisconsin

The Wisconsin DOT has kept records of project level initial PSI measurements for all new pavement construction for most years between 1978 to 1994. Smoothness specifications for PCC and AC pavements were implemented in 1984 and 1993, respectively. Plots of mean initial PSI measurements for each year along with the corresponding standard deviations for AC and jointed PCC pavements are shown in figures 56 and 57, respectively. An increase in the serviceability and a general reduction in the standard deviation of serviceability are generally observed from these plots. These results indicate that the implementation of smoothness specifications resulted in smoother pavements being built.

## STATE PRACTICES FOR SMOOTHNESS TESTING AND EVALUATION

A comprehensive survey on the State practices for measurement of smoothness was performed in 1994 by Smith et al. (10). Responses to that survey were received from 45 States and 5 Federal land agencies. The following sections present the results of State's responses to

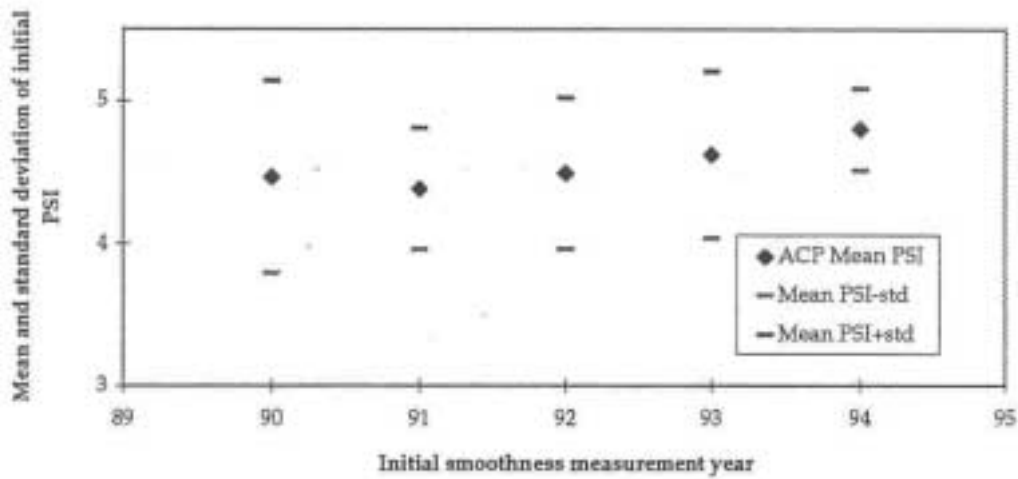


Figure 56. Mean and standard deviation of initial PSI over time for Wisconsin AC pavements (10).

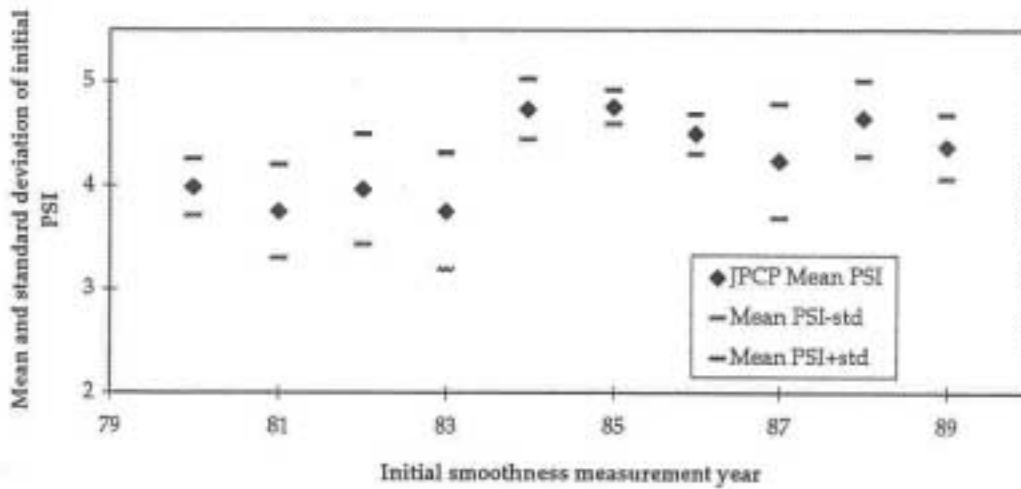


Figure 57. Mean and standard deviation of initial PSI over time for Wisconsin JPC pavements (10).

selected questions contained in that survey. It should be noted that some of the State practices might have changed over the past six years, since the survey was performed.

### Use of Specifications

Most highway agencies indicated that they use some form of initial smoothness specification. Some agencies use ride quality specifications with a bump specification, while

some agencies use only a bump specification. Figure 58 presents the responses received from the agencies regarding the use of specifications for the different pavement types.

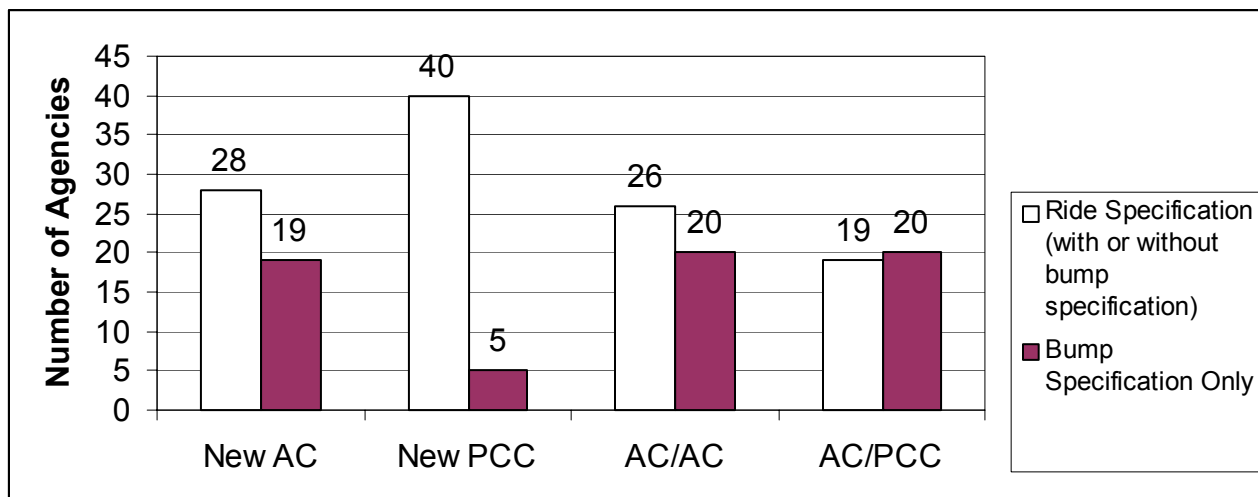


Figure 58. Use of smoothness specification for various pavement types (10).

As seen from this figure, 28 of the 48 responding agencies indicated using a ride specification (tested using a profiling device) on newly constructed AC pavements. Nineteen additional agencies indicated using straightedges or stringlines to ensure adherence to bump specifications on new AC pavements. Forty of the 50 respondents indicated using a profiling device to ensure PCC ride quality requirements. Five other respondents indicated using a straightedge or a stringline as part of a bump specification. The remaining five agencies did not indicate the use of PCC smoothness specifications because it is believed that they do not construct PCC pavements.

### Equipment Used

Figure 59 shows the breakdown of roughness measuring equipment used on new AC and AC overlay pavements. As seen from this figure, the profilograph is the most widely used device for acceptance of new pavements. Figure 60 shows the breakdown of roughness measuring equipment used on new PCC pavements. The profilograph is the most widely used device for PCC pavements too.

### Smoothness Acceptance Testing

Less than 20 percent of the responding agencies indicated allowing contractors alone to perform smoothness acceptance testing. Table 15 presents the breakdown of the responses that were received.

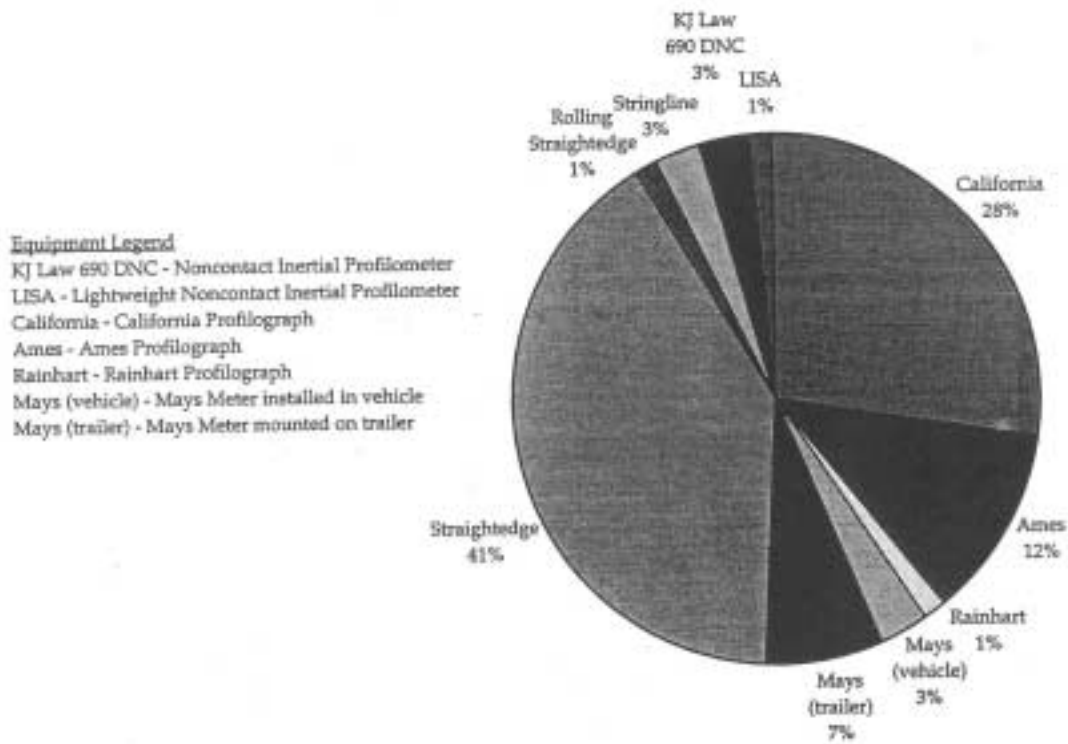


Figure 59. Breakdown of roughness measuring equipment used on new AC and AC overlay pavements (10).

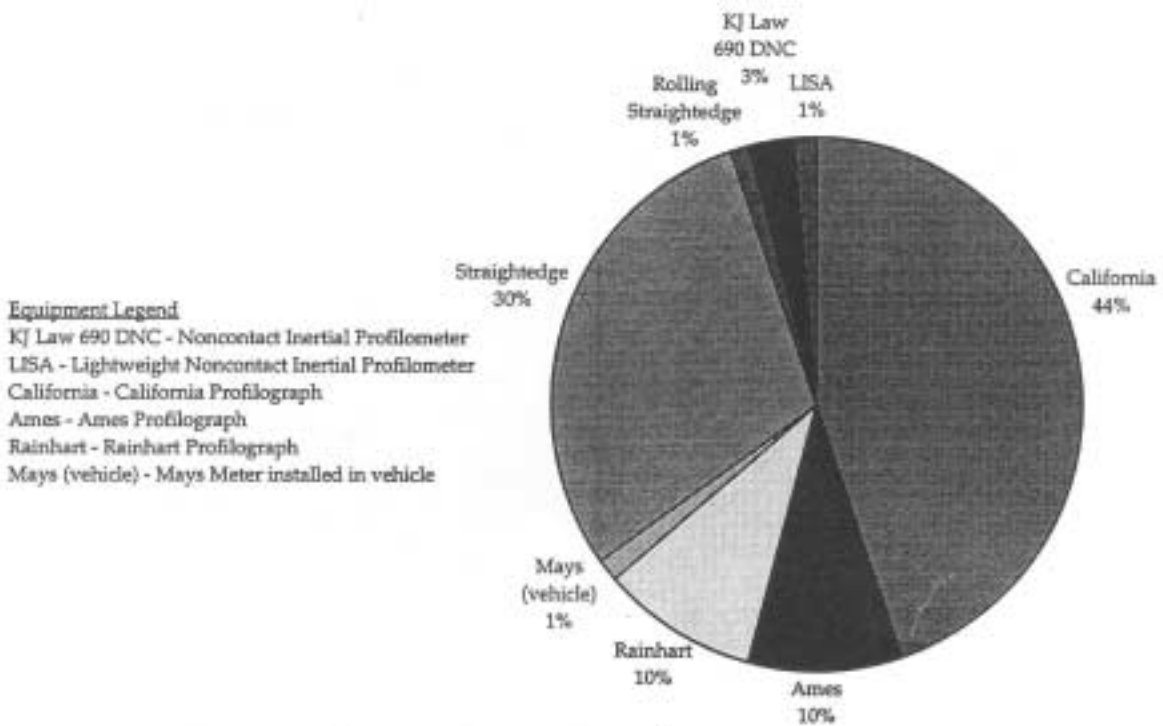


Figure 60. Breakdown of roughness measuring equipment used on new PCC pavements (10).



Table 15. Responsibility for smoothness acceptance testing (10).

Response	AC	PCC	AC Overlay
Contractor	5	9	6
State	34	28	30
Other	5	7	6
Total Number of Responding Agencies	44	44	42

### Length of Paving Tested

The survey indicated that a vast majority of the States test the entire length of paving for roughness, rather than evaluate random samples. The percentage of agencies testing the entire paving length was 76, 86, and 76 percent for AC, PCC and AC overlays, respectively. Table 16 presents the responses from the survey.

Table 16. Extent of smoothness testing (10).

Response	AC	PCC	AC Overlay
Entire paving length	29	38	29
Random samples	9	6	9
Total Number of Responding Agencies	38	44	38

### Unit Length of Paving Section Individually Evaluated for Smoothness

The majority of the States evaluate smoothness along 0.1 mile intervals. Some States evaluate roughness for AC pavements at 1 mile intervals, and at various specified lengths below 0.25 mile for PCC pavements. Table 17 presents the response to the survey.

Table 17. Unit length of paving section individually evaluated for smoothness (10).

Response	AC	PCC	AC Overlay
528 ft (0.1 mile)	23	38	21
1056 ft (0.2 mile)	1	0	1
Other	7	8	9
Total Number of Responding Agencies	31	44	31

### Time for Testing

Table 18 presents the results from the survey regarding time of testing. The percentage of agencies indicating that testing should be performed within 72 hours of construction for AC,

PCC and AC overlays were 44, 40, and 47 percent, respectively. The percentage of agencies that indicated they did not have specific time requirements for testing was 29, 33, and 26 percent for AC, PCC and AC overlays, respectively.

Table 18. Time after construction in which acceptance testing is performed (10).

Response	AC	PCC	AC Overlay
Within 24 hours	11	8	11
Within 48 hours	0	6	1
Within 72 hours	4	3	4
No time requirement/Test as soon as possible	10	14	9
Other	9	11	9
Total Number of Responding Agencies	34	42	34

### Blanking Band Limits for Profilographs

Overwhelmingly, States using a profilograph in their roughness testing place a 0.2 inch blanking band for evaluation of profilograph traces. Table 19 shows the responses that were received regarding blanking band limits from the agencies. As shown in this table, 86 percent of the respondents indicate using a blanking band limit of 0.2 inches.

Table 19. Blanking band limit for profilograph (10).

Blanking Band Limit	AC	PCC	AC Overlay
0.1 inch	1	3	1
0.2 inch	19	32	18
Other	2	2	2
Total Number of Responding Agencies	22	37	21

### Method for Positioning the Blanking Band

Table 20 presents the method used by agencies in positioning the blanking band. Visual judgment was the most common method followed by the computer selected best-fit procedure and alignment with previous section.

### Accuracy to which Scallop on Profilograph Trace are Rounded

Table 21 present the responses from agencies regarding the accuracy to which scallops on profilograph trace are rounded. Approximately 82 percent of the respondents indicated the accuracy to be 0.05 inches.

Table 20. Method used for positioning blanking band (10).

Response	Agencies
Alignment with previous section	5
Visual judgement	16
Computer-selected best fit	19
Other	8
Total Number of Agencies Responding	38

Table 21. Accuracy to which scallops on profilograph trace are rounded (10).

Response	Agencies
0.01 inch	3
0.05 inch	29
Other	3
Total Number of Agencies Responding	35

### Acceptance Limits for PCC Pavements

Most highway agencies using California type profilographs specify a critical PI value of 7 or 10 in/mile for PCC pavements. Kansas uses a limit of 50 in/mile based on the use of a null blanking band.

### Acceptance Limits for AC Pavements

Most highway agencies using California type profilographs specify a critical PI of 7 or 10 in/mile. Kansas, which does not use a blanking band, uses a limit of 40 in/mile. According to the survey performed in 1994, Georgia, Tennessee, South Carolina, West Virginia, and Kentucky use Mays Meter for acceptance with varying acceptance values (10). Michigan uses both California profilograph and profiler with acceptance limits of 10 in/mile and 49.8 RQI, respectively. Arizona uses a K.J. Law inertial profiler for acceptance, while Florida and New Jersey uses a rolling straightedge.

### Basis for Selection of Smoothness Specifications

The survey results obtained by Smith et al. (10) indicated that States used a variety of methods to base the selection of smoothness limits for specifications. These methods include research and analysis, engineering judgement, other agency specification, and AASHO guide specifications. Table 22 summarizes the responses from the survey. It is believed that the States

will base their specification limits based on their own knowledge and findings rather than rely on information from external sources.

Table 22. Basis for selection of specified smoothness limits (10).

Response	AC	PCC	AC Overlay
Research and Analysis	10	7	9
Engineering Judgement	9	7	10
Other Agency Specification	2	4	3
AASHTO Guide Specification	3	5	2
Other	18	20	18
Total Number of Responding Agencies	42	43	42

### Satisfaction with Specifications

The survey indicated that most States felt their specifications were either adequate or in need of slight improvement. Out of 45 responding agencies, 24 indicated their current specifications were adequate, 16 indicated slight improvements should be made and 5 indicated their specifications should be replaced.

### Bump Specifications

The location of the must grind areas is performed using a small plastic template with a one inch line scribed parallel to the edge and located between 0.3 inches and 0.5 inches away depending on the agency criteria. Figure 61 shows an example of a bump template. The one inch line scribed on the template represents 25 ft of pavement surface. This is the standard length, which is used by most agencies to evaluate scallops.

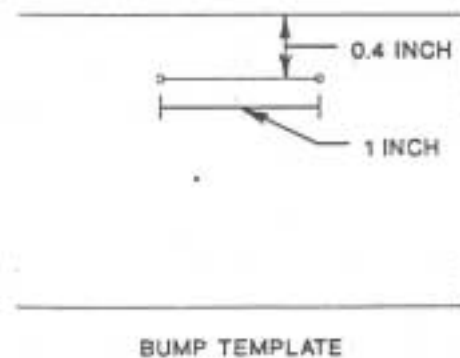


Figure 61. Example of a bump template (16).

Designation of bump locations (i.e., must grinds) is performed by evaluating each prominent scallop or high point. The template is placed so that the small holes or scribe marks on each end of the scribed line intersects the profile so that a chord is formed across the base of the scallop as shown in figure 62. A line is then drawn with a sharp pencil as indicated. The line need not be horizontal. The intent of the specifications used by most agencies is that the line scribed onto the

trace be as close as possible to 25 ft, but in no case exceed 25 feet. When the base of the scallop or deviation exceeds 25 ft, the template is located as shown in the last case of figure 62.

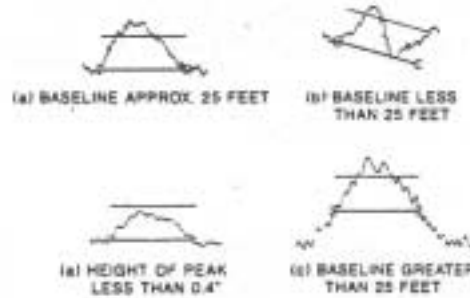


Figure 62. Locating must grind bumps (16).

The survey performed by Smith et al. (10) indicated that nearly all profilograph-based bump specifications are based on a 25-ft base length. Of the 39 agencies that indicated the use of bump specification, 36 use a 25-ft base length. The bump criteria that were being used by the 36 agencies that used a 25-ft base length were either 0.3 in, 0.4 in or 0.5 in. The breakdown of the bump criteria used expressed as a percentage is shown in figure 63. As shown in this figure, 63 percent of the agencies require that bumps greater than 0.3 in be ground. The bump limit is 0.4 in for 31 percent of the agencies and 0.5 in for 6 percent of the agencies. AASHTO and ACPA both recommend a must grind criteria of 0.4 inches for PCC pavements. The current California specifications require a bump height of 0.3 inches.

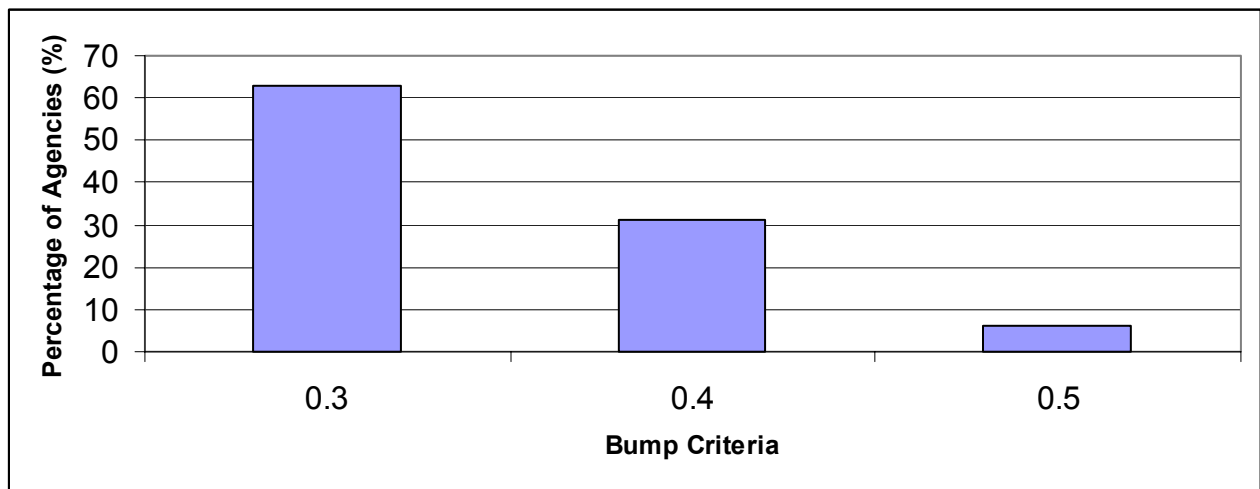


Figure 63. Breakdown of bump criteria used by agencies using a 25-ft base length (10).

## Result of Current Smoothness Specification on Material/Construction Quality Control

Table 23 presents the response of agencies regarding their belief of the effect of smoothness specifications on materials and construction quality. Approximately 50 percent of the agencies believe that implementation of smoothness specifications resulted in improved material and construction quality, although there is no proof to support this belief.

Table 23. Result of smoothness specification on material/construction quality control (10).

Response	AC	PCC	AC Overlay
Records show better quality	2	4	2
Perceived quality increase	16	20	15
No increased quality	8	6	7
Not sure; too early to tell	6	7	5
No specification currently	7	2	9
Total Number of Responding Agencies	39	39	38

## Use of Incentives and Disincentives

Some States require a specific limit of smoothness be met, whereas others use a variable scale pay adjustment factors related to the degree of smoothness achieved. The incentive and disincentive provisions in smoothness specifications are intended to encourage the construction of smooth highway pavements through financial incentives for extremely smooth pavements, and through financial disincentives for unacceptably rough pavements. The survey by Smith et al. (10) showed that of the 50 responding agencies, 21 and 29 use incentive and/or disincentive pay factors on new AC and new PCC construction, respectively (10).

A survey performed in 1994 on 16 contractors indicated that the contractors have a positive attitude regarding the use of incentive and disincentive clauses in smoothness specifications (10). The consensus among the contractors was that the following advantages are gained by specifying incentive and disincentive clauses on paving projects:

1. Competitive advantage for higher quality contractors.
2. Promotes quality workmanship.
3. An incentive to try new technologies and to upgrade quality control programs.
4. The contractors that care about quality can potentially regain extra costs.
5. The highway agency and the taxpayer benefits from cheaper bid price because good contractors will take advantage in the bidding process and submit a cheaper price.

Incentive limits for both new AC and PCC pavements generally range between 3 and 7 in/mi. The limits are generally slightly more restrictive for AC pavements than PCC pavements (10). Disincentive limits AC and PCC generally range between 7 and 10 in/mile, with slightly more leniency given for new PCC pavements (10).

## Incentive/Disincentive Payments

The survey by Smith et al. (10) indicated the majority of the States base incentive and disincentive payments on a portion of the unit bid, rather than a fixed amount or other methods. Table 24 presents the responses that were received from agencies regarding their payment method for incentives and disincentives.

Table 24. Basis on which incentive/disincentive payment amounts are determined (10).

Response	AC	PCC	AC Overlay
Fixed amount, \$/yd <sup>2</sup>	7	7	6
Portion of the unit bid	14	24	13
Other	5	4	6
Total Number of Responding Agencies	26	35	25

The actual incentive and disincentive policies varied between the States. The incentive and disincentive value typically ranged from 1 to 5 percent of the bid price. Most had similar upper range adjustment pay factor of 105 percent for incentive and a lower range of 90% for disincentive. Many of the incentive/disincentive pay schedules being used was noted as being step-function pay schedules, with a certain percentage paid for a specified range of smoothness.

## Basis for Selection of Incentive/Disincentive Limits

A variety of methods was used by the States as the basis for selection of incentive and disincentive limits. The results of the survey by Smith et al. (10) are shown in table 25. The results indicated that States used methods such as engineering judgment, other agency specifications, research and analysis for selection of smoothness limits to for disincentive and incentive payments.

Table 25. Basis for selection of relationship between incentive/disincentive payment and smoothness level (10).

Response	AC	PCC	AC Overlay
Research and Analysis	6	3	5
Engineering Judgement	4	7	5
Other Agency Specification	3	4	2
AASHTO Guide Specification	1	3	0
Other	10	15	10
Total Number of Responding Agencies	24	32	22

## Result of Incentive/Disincentive on Initial Pavement Smoothness

The majority of agencies having incentive and disincentives for smoothness indicated they have records that show significantly smoother pavements because of specifying incentive disincentive provisions. Table 26 presents the results from the survey.

Table 26. Result of incentive/disincentive on initial pavement smoothness (10).

Response	AC	PCC	AC Overlay
Records show significantly smoother pavements	12	19	11
No difference in pavement smoothness	2	2	0
Not sure; too early to tell	9	11	8
No incentive/disincentive in specification	9	5	12
Total Number of Responding Agencies	32	37	31

## EXAMPLE SMOOTHNESS SPECIFICATION

This section presents the smoothness specification that is currently in use in Michigan. The Michigan Department of Transportation allows two methods for measuring the ride quality of pavements. The contractor has the option of using either method. However, once a method has been selected by the contractor, it cannot be changed without authorization by the engineer.

### Methods of Determining Pavement Smoothness

The two methods that are allowed for determining pavement smoothness are the California Type Profilograph or the GM Type Rapid Travel Profilometer.

1. California Type Profilograph: Ride quality of the pavement expressed in mm/km will be determined from a mechanically produced profilogram (trace) or from a computerized version of the California type profilograph,
2. GM Type Rapid Travel Profilometer: Ride quality of the pavement expressed as RQI (Ride Quality Index) units or, mm/km will be determined by proper reduction of the true profile obtained by a GM type of Rapid Travel Profilometer. The contractor has the option of using either unit of measurement.

### Equipment

#### *California Type Profilograph*

Either a mechanical or computerized profilograph is acceptable. The profilograph should be able to produce a profilogram with a true 1:1 vertical scale and a true 1:300 horizontal scaling. If the profilograph is equipped with an on-board computer, the following conditions will apply.



Vertical displacements will be sampled every 76 mm or less along the roadway. The profile data will be bandpass filtered in the computer to remove all spatial wavelengths shorter than 0.61 m and longer than 33.5 m. This will be accomplished by a third order low pass Butterworth filter set at 0.61 m, and a third order high pass Butterworth filter set at 22.5 m. The resulting band limited profile will then be computer analyzed according to the California Profilograph reduction process to produce the required mm/km statistic. This will be accomplished by fitting a linear regression line to each 160 m of contiguous pavement section. This corresponds to the perfect placement of the blanking bar by a human trace reducer. Scallops are then detected and totaled according to the California protocol. Bump analysis will take place according to the California Profilograph reduction process.

The computerized profilograph will produce a plot of the profile and a printout which gives the following data: stations every km, bump or dip height and length of specification (8 mm and 8 m respectively), the blanking band width, date of measurement, overall mm per km for that measurements, total length of that measurement, and the raw mm for each 160 m segment.

The calibration procedure for the mechanical machine will consist of profiling two replicate runs on a designated roadway of 300 m in length. Horizontal calibration will be checked by running the profilograph over the 300 m length and measuring the length of the resulting output from the profilogram. A 300 m run must produce 1 m ( $\pm 3$  mm) of profilogram output. Vertical calibration will be checked by running the test wheel over a block of known thickness (usually 25 mm) and measuring the displacement it produces on the profilogram. There will be no visible tolerance allowed on the vertical calibration. Calibration of the computerized versions will have a run made over a distance of a measured 300 m. The computer must print out a distance equal to the measured distance ( $\pm 1$  m). The vertical calibration will be as per the manufacturers specifications.

If the vertical or horizontal checks do not meet specifications, the machinery must be corrected. In addition to the calibration procedures, a visual inspection of the profilograph must be conducted. This would include: the condition of the test tires and bogey wheels, manufacturers recommended tire pressure, tracking of the paper on the spool and paper drum, condition of chains and cables, tracking of the device down the road, and general condition of the test device.

#### *GM Type Rapid Travel Profilometer*

The profiler should be based on the General Motors Rapid travel concept. The unit will produce a true profile for spatial wavelengths from 0.61 to 33.5 m. The unit must also be able to generate the equivalent California Profilograph plot and values as well as locations of bumps or dips over an 8 mm/8m. The unit will also be capable of producing a plot of the true profile with a range from 0.61 m to 33.5 m wavelengths.

The digitized profile will be processed by dividing it into three spatial wavelength bands by using third order Butterworth high and low pass filters. The three bands are 15.2 m to 7.6 m, 7.6 to 1.5 m, and 1.5 to 0.6 m. Variance of the profile in each band is then computed as follows:  
$$\text{Var}_i = [(x_i - \bar{x})^2] / N$$

where:

$\text{Var}_i$  = is the variance for band,  $i = 1$  for 15.2 m to 7.6m,  $i = 2$  for 7.6 m to 1.5 m and  $i = 3$  for 1.5 to 0.6 m

$x_i$  = an individual profile elevation in mm for the band

$\bar{x}$  = average profile elevation value in mm for the band

$N$  = the number of profile elevations measured in the band.

The RQI is then computed using the following formula:

$$\text{RQI} = 3.077 \text{Log}_e (\text{Var}_1 \times 10^8) + 6.154 \text{Log}_e (\text{Var}_2 \times 10^8) + 9.231 \text{Log}_e (\text{Var}_3 \times 10^8) - 141.85$$

The RQI has a scale from 0 (a perfect road) to 100 (roughest road). This equipment should give a printout of the same information as the profilograph with the addition of the ride quality index for each 160 m segment of the total run. These devices can be tested for overall operation by performing the “Bounce Test” procedure included with the unit. Horizontal measurement will be checked over a measured distance of 300 m and will read within  $\pm 1$ m of the measured distance. The vertical calibration will be as per the manufacturers specifications.

## **Method of Interpretation**

### *Profile Index*

Profiles will be taken 1 m from each side of each lane that is to be measured. The trace generated by the mechanical profilograph will be analyzed by the Engineer using a 5 mm blanking band measuring each deviation above and below the band to the nearest 1 mm according to Michigan Test Method MTM 204-88. Deviations will be summed for each 160 m. For computerized profilographs, the Engineer will not need to reduce the trace. A copy of the official computer generated trace and printout will be submitted for project records. Each run will be reported to nearest 0.5 mm as the average mm/km of the two runs in each test lane.

Pavement lanes constructed with 0 to less than 64 mm/km will result in payment of varying percentages for ride quality. Pavement lanes with 64 to 160 mm/km will not be eligible for bonus payment for ride quality. Surface pavements with more than 160 mm/km will not be acceptable. All surface pavement areas with bumps or dips exceeding 8 mm per 8 m must be corrected. All uppermost leveling courses with bumps or dips exceeding 12.5 mm in 8 m must be corrected prior to placing the surface course. All surface pavements must be corrected to achieve a value of 160 mm/km or less.

### *Ride Quality Index (RQI)*

Profiles will be taken 1 m from each side of each lane that is to be measured. RQI will be calculated for each 160 m segment. The RQI of each run will be reported to one decimal place, as the average of the two runs of each lane. The contractor should provide to the Engineer a trace and a printout which gives the same information as described for the profilograph.

Surface pavement lanes constructed with an RQI from 22 to less than 45 will result in payment of varying percentages for ride quality. Lanes with RQI between 45 to 53 will not be eligible for bonus payment for ride quality. Pavement lanes with a RQI of more than 53 are not acceptable. All pavement lanes will be corrected to achieve an RQI value of 53 or less. All surface pavement area bumps or dips exceeding 8 mm in 8 m will be corrected. All uppermost leveling courses with bumps or dips exceeding 12.5 mm in 8 m must be corrected prior to placing the surface course.

### **Payment**

Payment for the item Ride Quality will be determined by the Engineer based on the mm/km or RQI for the final weighted average for all values within each lane.

### *California Type Profilograph (mechanical and Computerized)*

A pavement lane having a range of 0 to less than 64 mm/km will receive payments for Ride Quality based on the product of the number of square meters in the pavement lane (minus excluded areas) times the contract unit price for Ride Quality multiplied by a pay factor that depends on the PI value.

### *GM Type Rapid Travel Profilometer*

A pavement lane having an RQI range of 22 (or less) to less than 45, or a range of 0 to less than 64 mm/km will receive payments for Ride Quality. The payment is based on the product of the number of square meters in the pavement lane (minus excluded areas) times the contract unit price for Ride Quality multiplied by a pay factor that depends on the RQI or Profile Index, whichever is employed.

## **APPROPRIATENESS OF INCENTIVE AND DISINCENTIVE LEVELS**

Several highway agencies have expressed concerns over the magnitude of incentives that are being paid out. They are concerned whether incentive/disincentive provisions for pavement smoothness are an appropriate and cost-effective proposition.

The incentive and disincentive payments for pavement smoothness are generally based on subjective judgement of the highway agencies. The extent to which they reflect cost benefits is unknown. It has been suggested that the incentive or disincentive should be based on the

increase or decrease in future costs that will be incurred by the agency and by users over the life of the pavement (50). Questions have also been raised that even if the highway agency may be correctly specifying the most cost-effective smoothness level, if the payments they are currently making are justified. Development of smoothness specifications has been based on engineering judgement (often by committee) or has been patterned after AASHTO or other agency's specification (which were developed by committee judgement). According to a State survey response (10), just over one third of SHAs use engineering judgement or other specifications as the basis for their specified smoothness limits. A similar percentage indicated that these were the basis for selecting the relationship between incentive/disincentive payment and initial smoothness. Although some States indicated performing research and analysis towards the establishment of critical limits, the degree of objectivity included in those efforts are unknown. It is unknown if pay adjustment rates are consistent with the overall benefits and costs for building the pavement smoother or rougher.

Smith et al. (10) conducted a life cycle analysis to determine the optimum cost effective PI values for pavement construction. An analysis period of 40 years for AC pavements and 50 years for PCC pavements was used in this study. The analysis used roughness models that predicted pavement roughness based on initial smoothness and time. The costs considered in this study were the initial construction cost and the rehabilitation costs. The initial construction cost increased for smoother pavements. The initial construction cost vs. initial pavement smoothness relationships that were used in this study was developed using data obtained from contractors. The principles of the analysis method used in the study are shown in figure 64. This figure shows the present worth costs. The costs incurred over the analysis period consists of construction cost and rehabilitation cost.

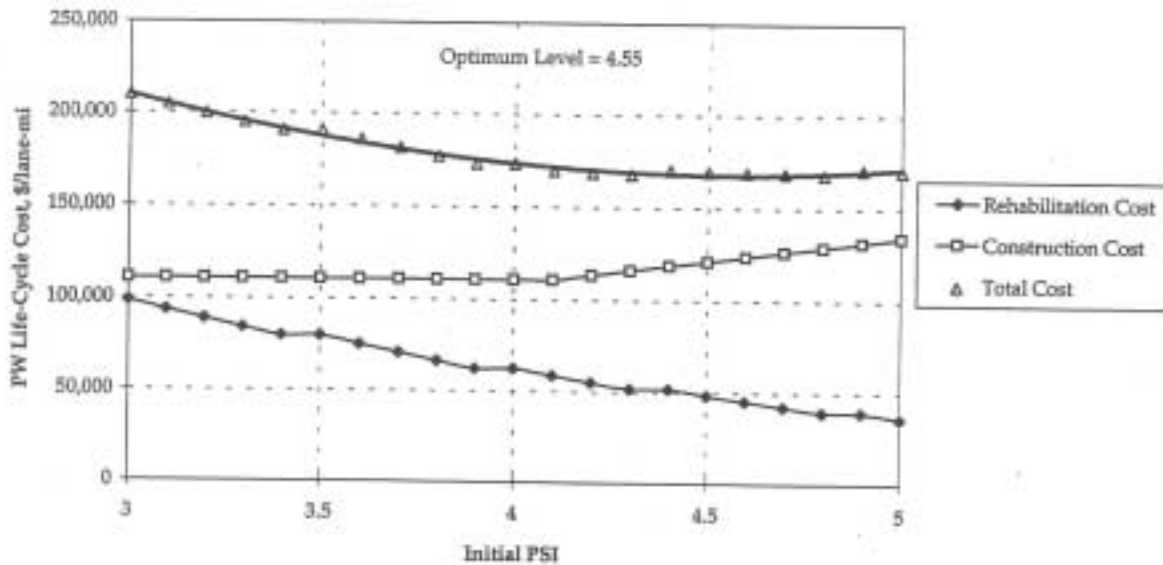


Figure 64. Plot of life cycle cost versus initial serviceability (10).

The initial construction cost increases with increase in smoothness. The roughness models used in this study predict higher service life for smoother pavements. Therefore, the rehabilitation costs associated with smoother pavements is lower. The total cost is the sum of the initial construction cost and the rehabilitation cost. The optimum cost-effective smoothness level corresponds to the smoothness level associated with the lowest total cost. To the right of the optimum cost effectiveness, the shorter pavement life is a result of decreased initial smoothness resulting in higher maintenance and rehabilitation cost. To the left of optimum cost effectiveness, the cost of constructing a pavement smoother outweighs the savings associated with added pavement life, resulting in an increased life-cycle cost. A life cycle cost analysis of several pavement families showed that the most cost effective smoothness levels are considerably higher than what is generally accepted as the current target (i.e. PI between 5 and 10 in/mi). The analysis showed the following results:

- Seven of nine PCC pavement families showed the optimum cost-effectiveness (PI) range, excluding user costs as being between 0 and 5.5 in/mi.
- Four of the five AC pavement families showed the optimum cost-effectiveness (PI) range as being between 0 and 3.5 in/mi
- Eleven of the 13 asphalt overlay families showed the optimum cost-effectiveness range as being between 0 and 2 in/mi.

The analysis showed that when actual current pay adjustment curves were compared with the theoretical pay adjustment curves developed in this study, much greater incentive amounts and much more punitive disincentive amounts are warranted in terms of benefits/disbenefits obtained from initial smoothness levels.

## **CURRENT TRENDS IN SMOOTHNESS SPECIFICATIONS**

The PI obtained from profilograph measurements have been widely used to specify smoothness requirements for new pavements. The PI is obtained from a profilograph trace by placing a blanking band on the trace, and evaluating deviations in the profile. It has been shown that roughness features that contribute to wheel chatter or long wavelengths that create roller coaster effect can go undetected by current procedures. In 1990, Kansas experienced a problem with short wavelength roughness. One project exhibited a cyclic 8 ft wavelengths of approximately 0.2 inch in amplitude. Although the profilograph determined that the pavement roughness met specifications, the ride quality of the pavement was poor. Because of this problem, the Kansas DOT eliminated the blanking band and developed specifications using a zero blanking band concept. Several other highway agencies have been looking into adopting a zero blanking band concept because of this problem in profilograph interpretation.

Currently, many States as well as contractors have adopted lightweight profilers to measure smoothness of new pavements. Lightweight profilers obtain the profile of the pavement. A profilograph simulation is then performed on the profile to obtain a profilograph trace, and

then the trace is reduced to obtain the PI and must grind bump locations. However, this method does not remedy the problems that are inherent in profilograph measurements.

Some of the States have moved towards using roughness indices such as IRI that are computed from profile data to specify smoothness specifications. It has been shown that IRI is correlated to the roughness felt by the travelling public on a roadway, and therefore is an appropriate statistic for evaluating the smoothness of pavements. The profile that is recorded by a lightweight profiler can be used to compute a roughness index such as IRI. Smoothness specifications as well as incentive and disincentive payments can then be based on the IRI value. Several States have been using roughness indices computed from high-speed inertial profiler measurements to specify smoothness for AC pavements. As the States move towards using roughness indices for smoothness specifications, it is very important that the equipment that are used to measure smoothness of pavements be capable of accurately recording all wavelengths in the pavement that contribute to that roughness index. Moving towards a roughness index such as IRI for pavement acceptance will provide an index that can be used to relate the future performance of the pavement with the as constructed smoothness level.

As obtaining a smooth pavement requires the involvement of the specifying agency and the construction industry, the agencies should develop roughness index based specifications in cooperation with the industry. The contractor should be able to understand the basis on which the specifications were developed. The specifications must be appropriate for the facility and must be attainable through the use of effective construction processes and equipment.

## **ISSUES RELATED TO SMOOTHNESS MEASUREMENTS ON PCC PAVEMENTS**

In jointed PCC pavements, curling of the slabs occurs because of daily temperature variations. Variations in moisture conditions in the PCC slab over time can cause the slabs to warp. The effect of slab curling on pavement profile was known as early as the 1940s (51). Figure 65 shows the profile of a PCC pavement for the following time periods after construction: 6 hours, 24 hours, 1 week, 1 month, 2 months, 1 year and 2 years.

The measurement at 1 month, 2 month, 1 year and 2 year periods were taken both in the morning and in the afternoon. The measurements that were taken in the morning show much more curling when compared to the corresponding measurement that was taken in the afternoon. Figure 65 shows that curling effects on the pavement were clearly seen in a matter of one week.

The pavement age at which curling affects roughness readings is not known. This may depend on a variety of factors such as mix design properties of PCC, placement temperature, curing method, slab thickness etc. However, it is important to understand that a measurement that is taken on a PCC pavement within 24 hrs after placement may not be replicated at a later date because of curling effects. Therefore, when measurements are obtained on a PCC pavement, an evaluation of the validity and accuracy of the measurements have to be performed immediately.

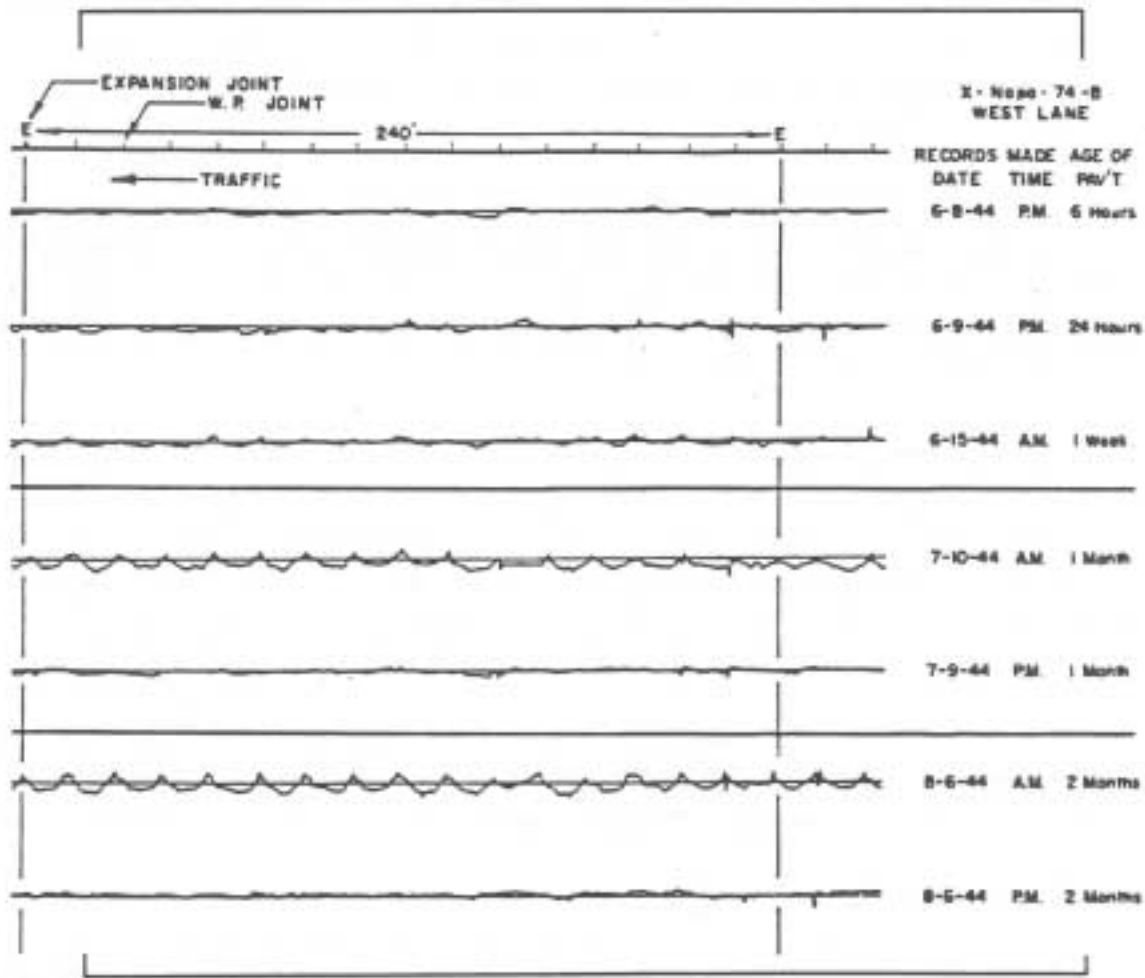


Figure 65. Effect of slab curling on roughness measurements (51).

Recent research efforts have shown that slab curling effects can have a significant effect on roughness development in PCC pavements. Figure 66 shows the development of roughness at section 15 of the SPS-2 project in Michigan. This pavement section is 11 inches thick and has a joint spacing of 15 feet. The mean IRI of the pavement increased from 60 in/mile to 100 in/mile within a period of approximately 4 years. Table 27 presents the profile dates, profile time and the IRI values for this section. A review of the IRI values in table 27 show that the high IRI values on the last profile date was not caused by temperature related curling, as the profiling was performed in the afternoon. Figure 67 shows the profile data for three profiling dates. The profile plots show that the higher roughness was caused by movement at the joints in the PCC slab. The cause for the movement is attributed to variations in moisture conditions within the slab. The initial smoothness of this pavement was very low, with a mean IRI of 60 in/mile. However, achieving this initial smoothness value did not prevent the pavement from becoming rough over a very short period of time. A 66 percent increase in mean IRI occurred over a four-year period. This example shows that achieving a smooth initial PCC pavement does not necessarily imply

that it will remain smooth even within its initial life. It appears there are other factors that affect the curling and warping of PCC pavements, which can cause a pavement to become very rough within a very short time period. The data available for the LTPP program provides an opportunity to study these aspects of PCC pavement behavior.

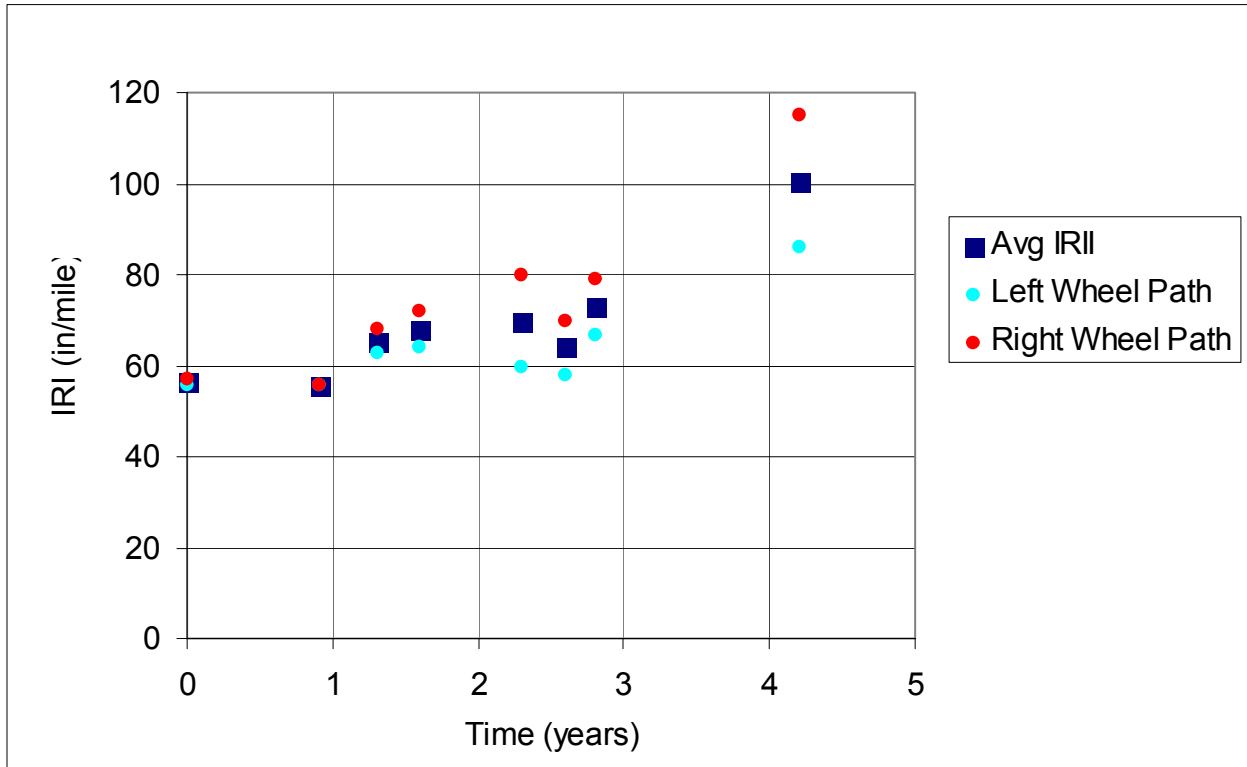


Figure 66. Variation of IRI with time at section 15 in SPS-2 project in Michigan.

Table 27. IRI values at section 15 in SPS-2 project in Michigan.

Date	Time	IRI (in/mile)		
		Avg.	Left	Right
9/94	11AM	57	56	57
8/95	9 AM	56	56	56
1/96	4 PM	66	63	68
4/96	10 AM	68	64	72
12/96	10 AM	70	60	80
4/97	1 PM	64	58	70
7/97	9 AM	73	67	79
11/98	4 PM	101	86	115



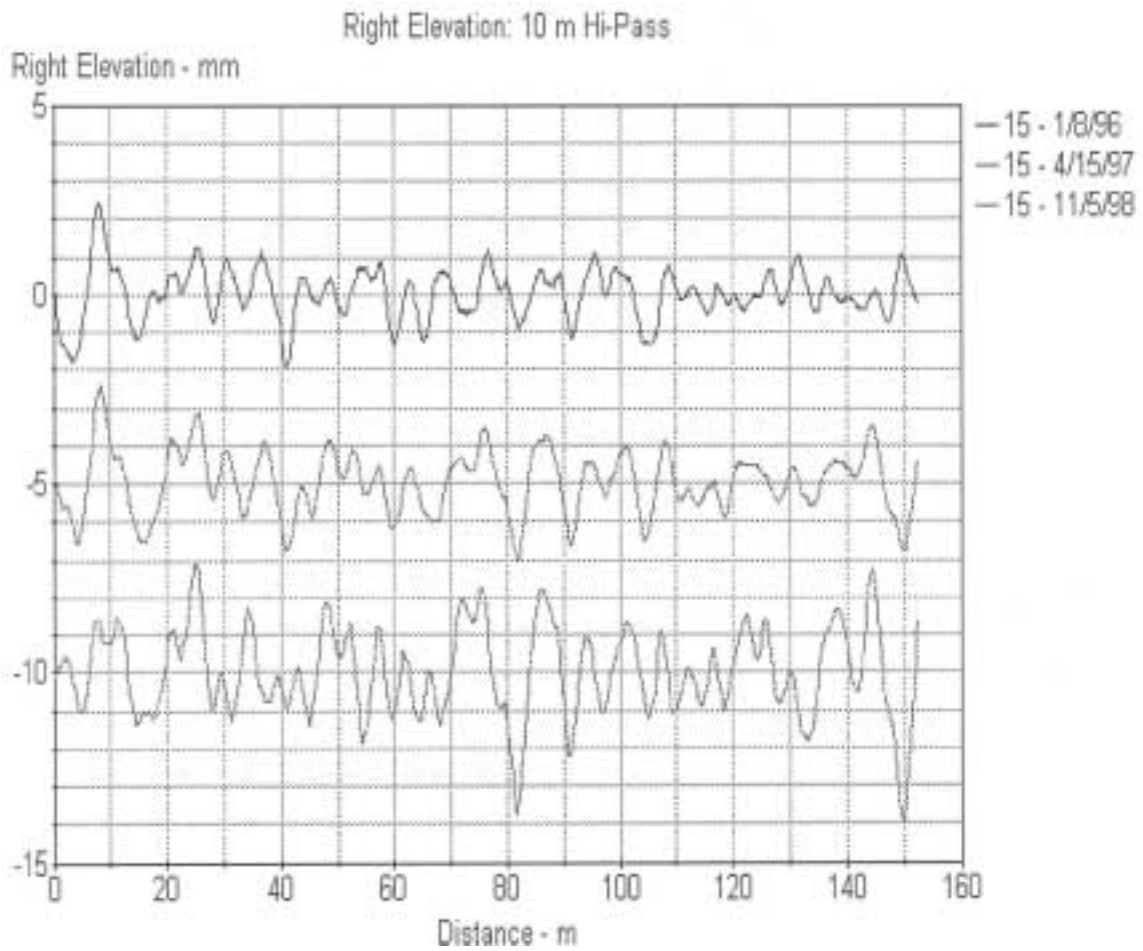


Figure 67. Variation of profile over time at section 15 in SPS-2 project in Michigan.

## **CHAPTER 9**

### **APPLICATION OF ROUGHNESS DATA AT NETWORK AND PROJECT LEVEL**

#### **APPLICATION OF ROUGHNESS DATA AT NETWORK LEVEL**

The enactment of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 specifically requires that the states have a Pavement Management System (PMS). The database must contain information that will provide transportation managers with information to assist in making planning and budget decisions. A PMS contains an inventory of the highways that come under the jurisdiction of the agency, which have been segmented into management sections. The information contained in the PMS is associated with each designated segment. Information that can be associated with each segment can include pavement condition data such as distress, roughness, structural capacity and friction measurements. A survey carried out in 1991 indicated all fifty states have a PMS in operation or are in the process of implementing one (39). A pavement management system provides a set of tools or methods that can assist the decision makers in finding optimum strategies for providing and maintaining pavements in a serviceable condition over a given period of time

Roughness data is an important data element that is contained in a PMS. Most States collect roughness data on their highway network on an annual basis. The roughness value is then stored in the PMS for each highway segment. Some agencies combine the roughness index with other pavement condition indices, such as distress to compute a composite index for the pavement. At the network level, roughness data can be used to identify sections for maintenance and rehabilitation. The roughness values of the highway network can be used for long-term planning and budgeting.

#### **APPLICATION OF ROUGHNESS DATA AT PROJECT LEVEL**

For network monitoring, it is sufficient to determine roughness levels on a per-mile basis (or some other manageable length). However, for diagnostic work and research, it is useful to be able to pinpoint exactly where a road is rough and where it is smooth. Profile data can be used at project level to locate areas of critical roughness. Two approaches that can be used for analyzing roughness at project level are to compute IRI for different segment lengths or to use roughness profiles.

#### **Segment Length**

The roughness index that is reported for a pavement section represents the average roughness of the section. The distance over which the roughness is reported is referred to as the segment length. Consider a case where the roughness of a one mile long section is reported. The reported roughness value is the average roughness of the one mile long section. There could be

areas within this one mile section that are extremely rough or extremely smooth. However, no inferences about variations in roughness within the section can be made by looking at the roughness value that is reported over one mile. It is possible to obtain a detailed view of the roughness within the pavement section by computing the roughness of the section over different segment lengths. The segment length has a strong impact on the interpretation of a roughness value.

The use of different segment lengths to perform an in-depth analysis of the roughness within a section is illustrated for three pavement sections.

*M-39: Michigan*

The IRI of a one mile long section of M-39 is shown in figure 68. The IRI of this section based on a segment length of 1 mile is 165 in/mile. Figure 69 shows the IRI of this section considering a segment length of 0.5 mile. When a segment length of 0.5 miles is used, the roughness for individual sections that are 0.5 miles in length is reported. The one mile long section contains two segments that are 0.5 miles long, the first from 0 to 0.5 miles, and the other from 0.5 to 1 mile. The results shown in figure 69 shows that the IRI of the first half mile is 130 in/mile, while the IRI of the next half mile is 200 in/mile. Figure 70 shows the IRI of this section when a segment length of 0.25 miles is used. There are four 0.25 mile long segments within this section. Each bar in figure 70 shows the IRI of an individual 0.25 mile long segment. Figure 70 shows that the last 0.25 mile segment has a roughness that is approximately twice of that of the first 0.25 mile long segment. Figures 71 and 72 shows the IRI of the section for segment lengths of 0.1 miles and 100 feet, respectively. As the segment length becomes shorter, a more detailed view of the roughness within the section is seen. Figure 72 shows the 300 feet that consists of segments 42 through 44 has the highest roughness in the section.

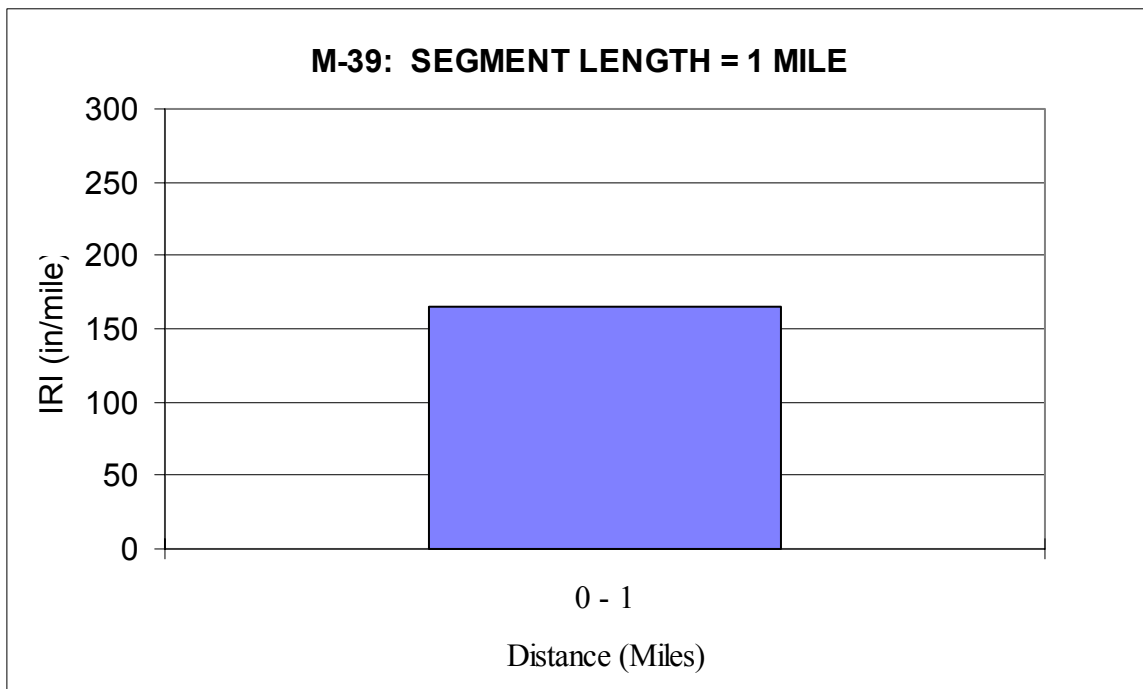


Figure 68. M-39: IRI for segment length of 1 mile.

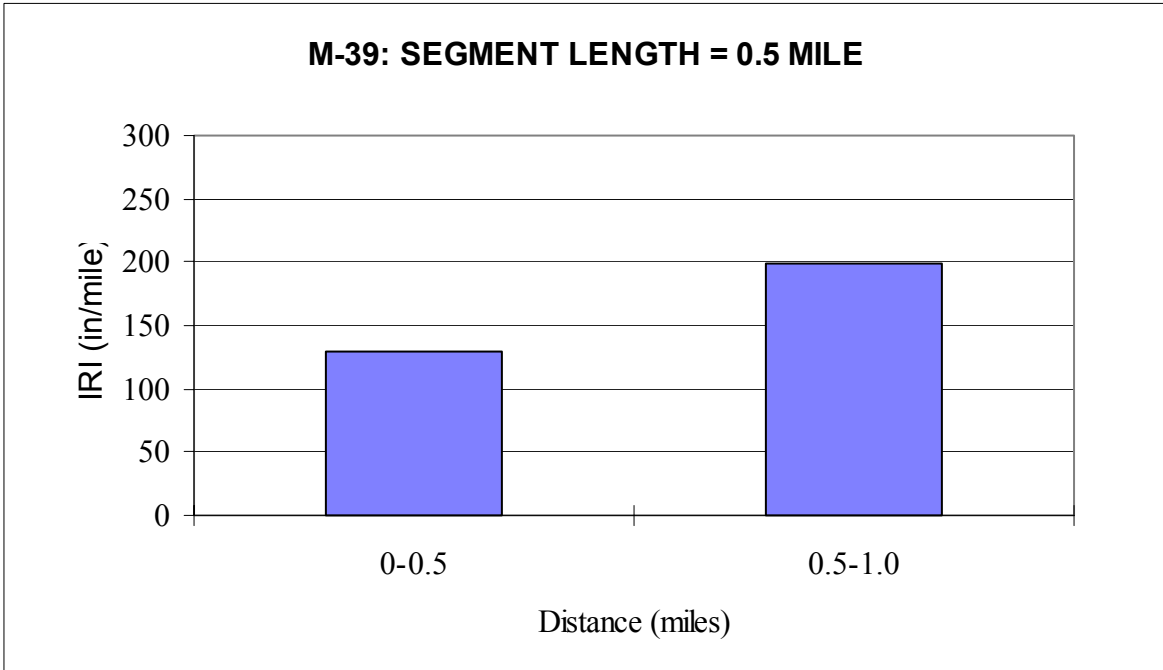


Figure 69. M-39: IRI for segment length of 0.5 mile.

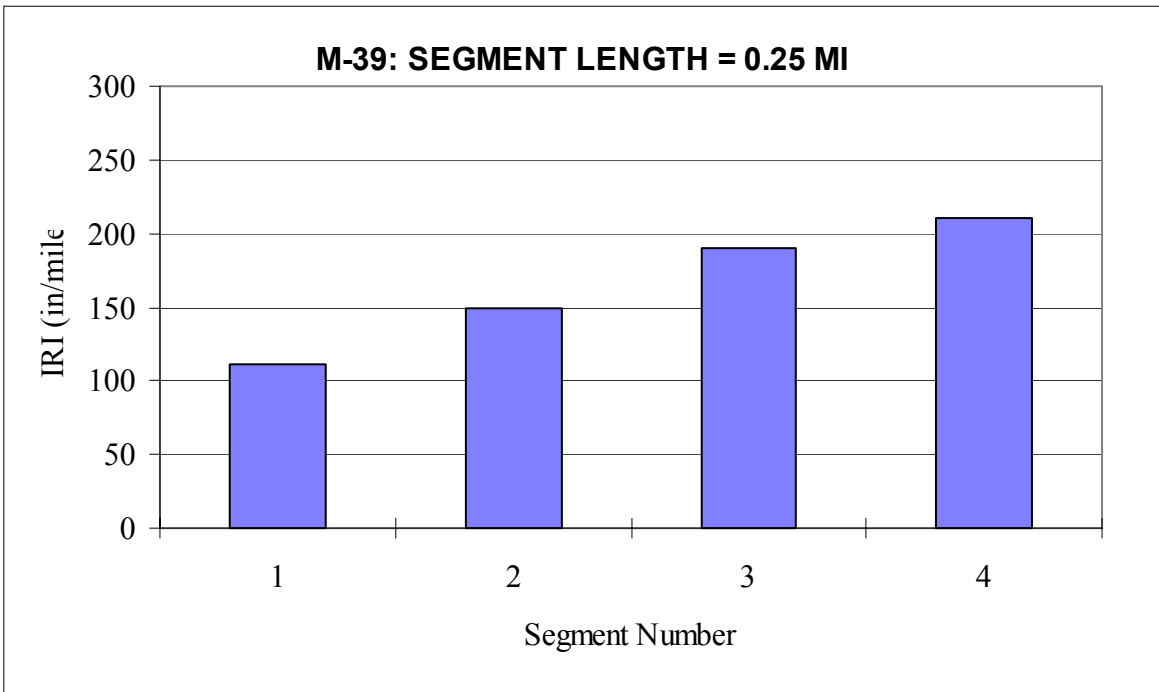


Figure 70. M-39: IRI for segment length of 0.25 mile.

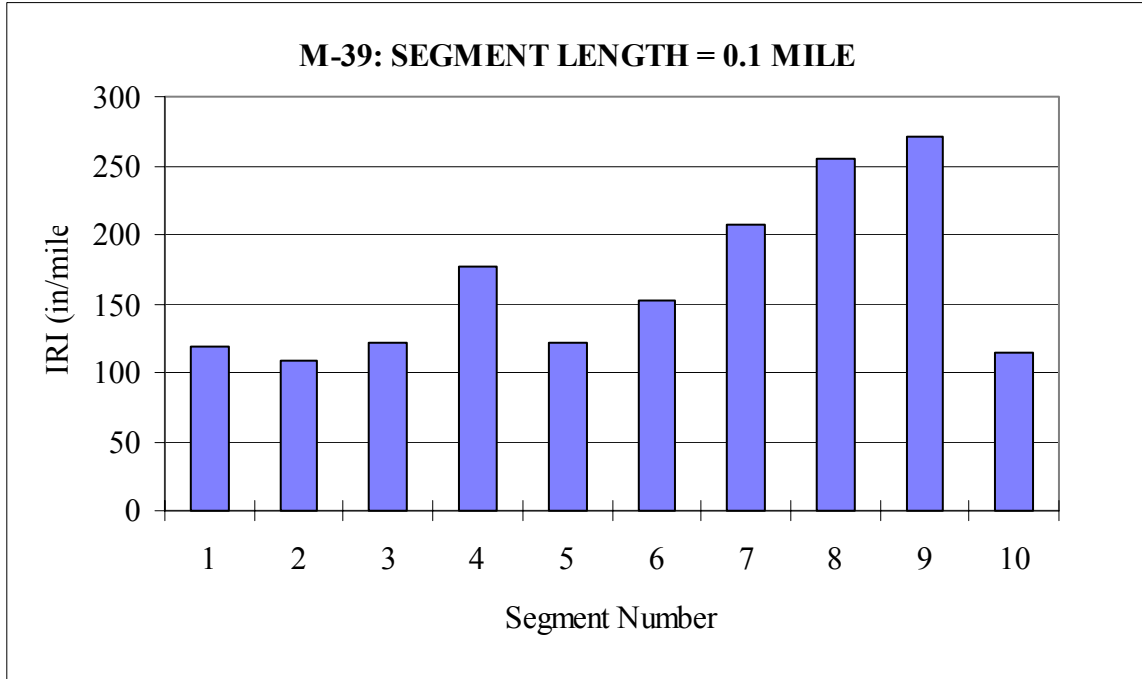


Figure 71. M-39: IRI for segment length of 0.1 mile.

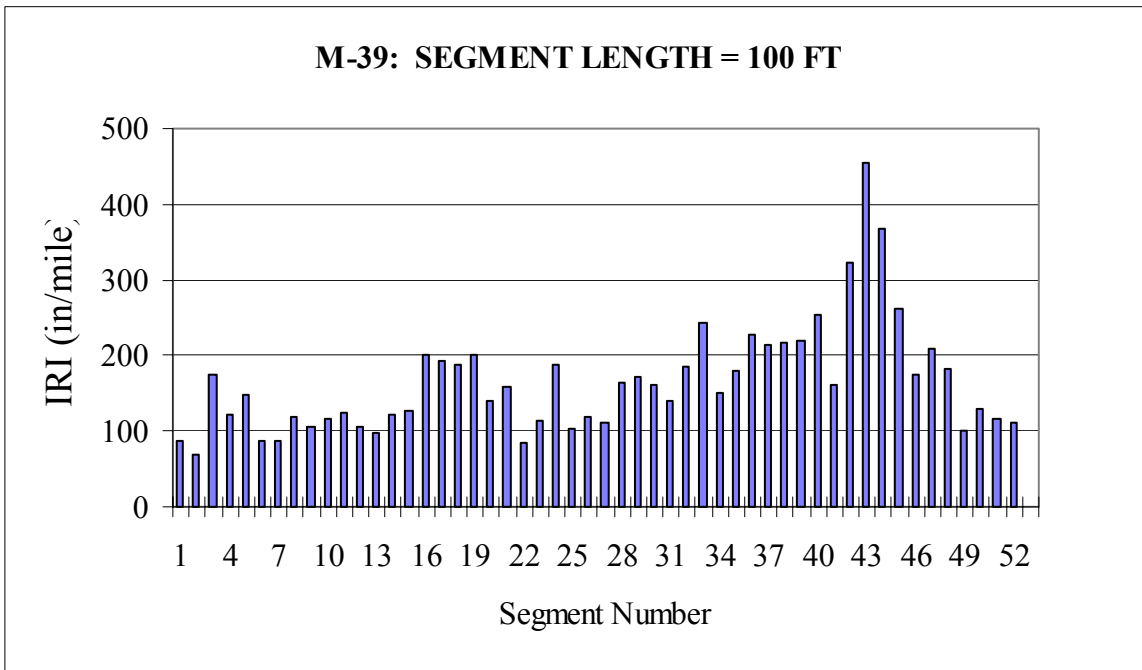


Figure 72. M-39: IRI for segment length of 100 feet.

*Five Mile Road*

The length of the pavement section that was evaluated on Five Mile Road was 0.75 miles long, and had an IRI of 99 in/mile over a segment length of 0.75 miles as shown in figure 73. Figures 74 and 75 shows the IRI of this section for segment lengths of 0.25 miles and 0.1 miles, respectively. Figure 75 shows that a rough spot is located in segment 7. Figure 76 shows the IRI of the section for a segment length of 100 feet. This figure shows that the roughness of segments 34 and 35 to be 597 in/mile and 345 in/mile, respectively. There is a railroad track within these segments that cause the roughness of these segments to be very high. When we look at the roughness of the entire section as indicated in figure 73, it would have been impossible to guess that there was a segment within this section that had an IRI value of 597 in/mile. This study demonstrates that by looking at the roughness of a section over short segment lengths, it is possible to get a detailed view of the roughness properties of the section.

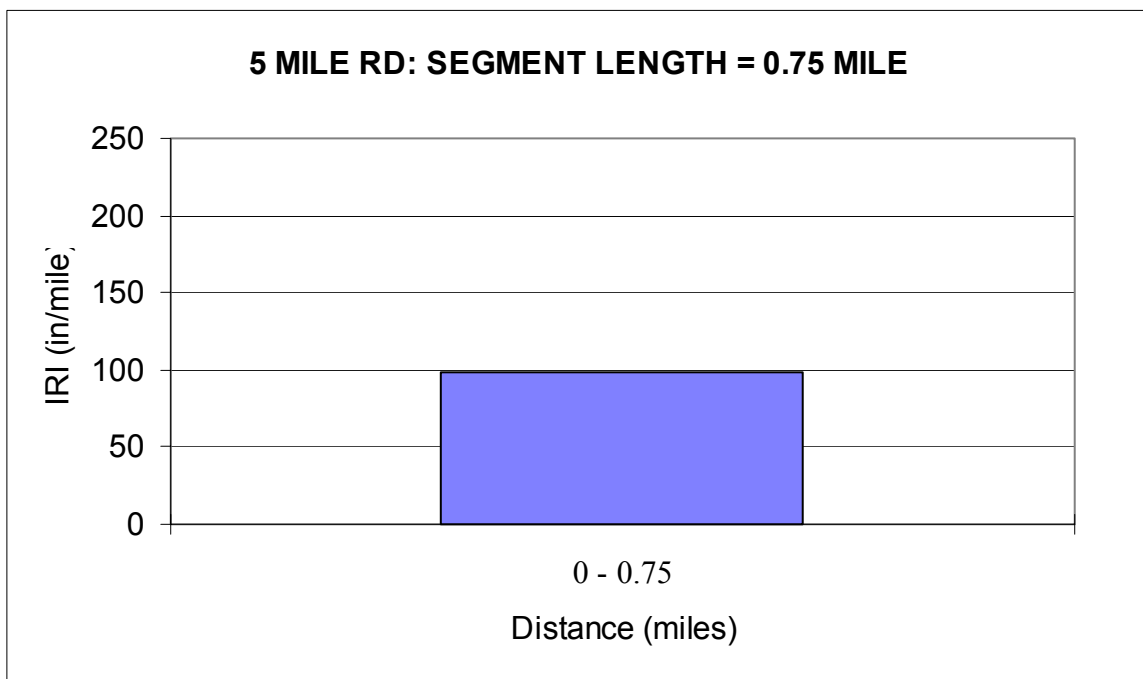


Figure 73. Five Mile Road – IRI for segment length of 0.75 mile.

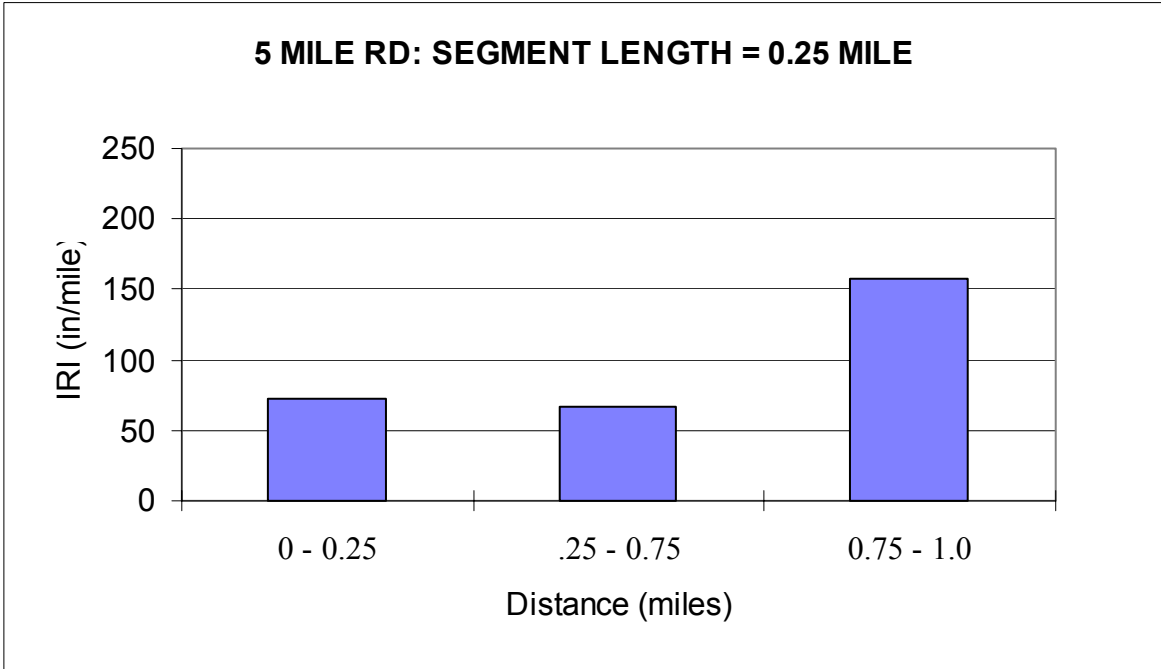


Figure 74. Five Mile Road – IRI for segment length of 0.25 mile.

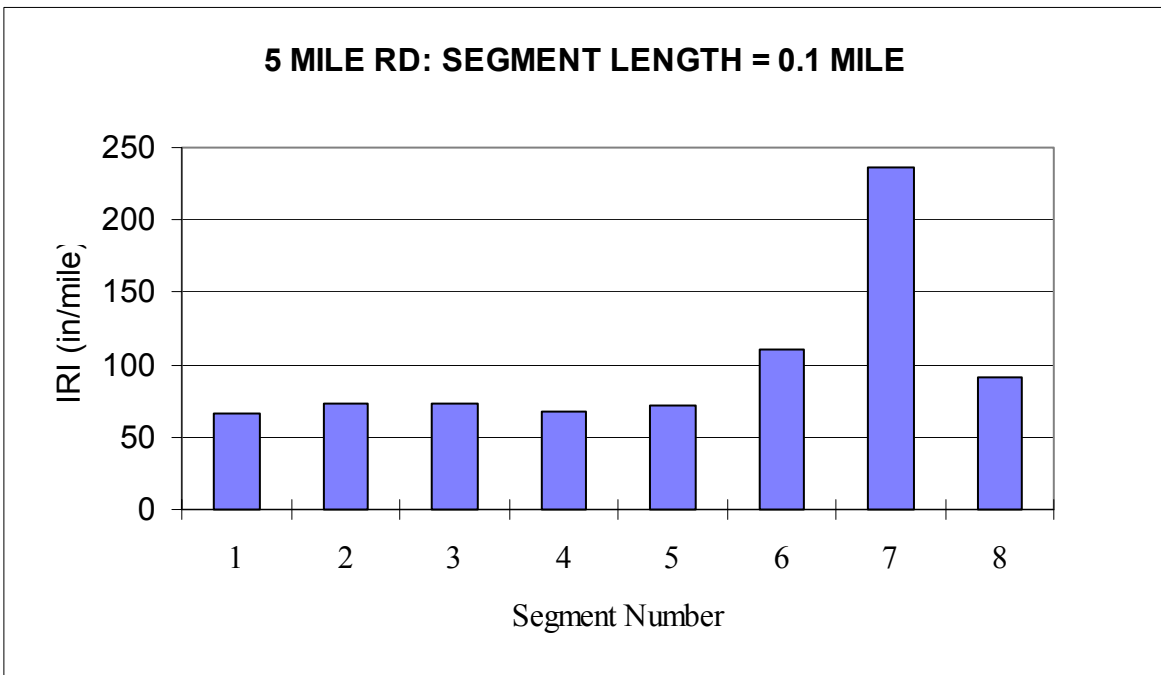


Figure 75. Five Mile Road – IRI for segment length of 0.1 mile.

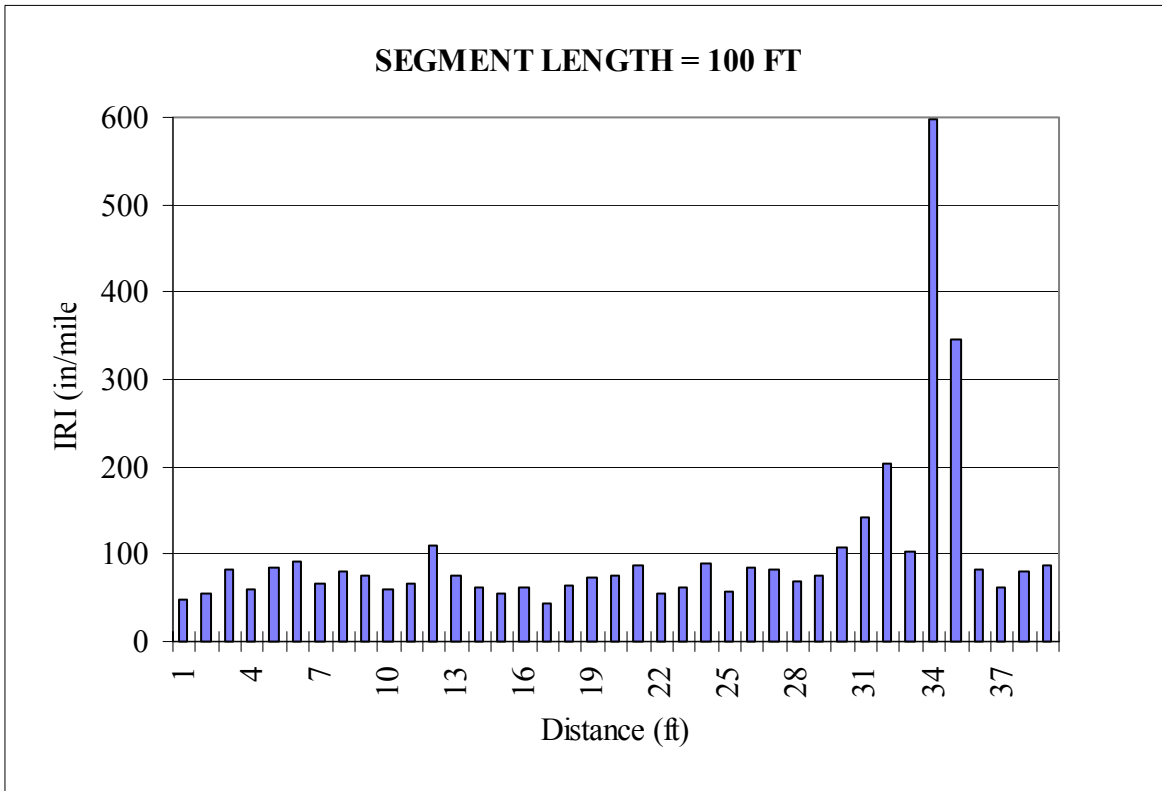


Figure 76. Five Mile Road – IRI for segment length of 100 feet.

*Ecorse Road: Michigan*

The roughness of the one mile length of road that was evaluated was 197 in/mile. This value is shown graphically in figure 77 as the roughness over a one mile segment length. Figures 78 through 80 shows the roughness of this section for segment lengths of 0.5 miles, 0.25 miles and 0.1 miles, respectively. Figure 80 shows that the roughness of the 0.1 mile segments within the section have IRI values that range from 153 to 243 in/mile. The roughness values reported for a segment length of 100 feet is shown in figure 81. This figure shows that there are two 100 foot long segments within the section that has IRI values exceeding 600 in/mile. From the analysis of the roughness data at this site, it is possible to locate the roughest segments within the section, and also to get an idea about their roughness level. Such an analysis can be used to locate areas that may need immediate repairs from a safety point of view.



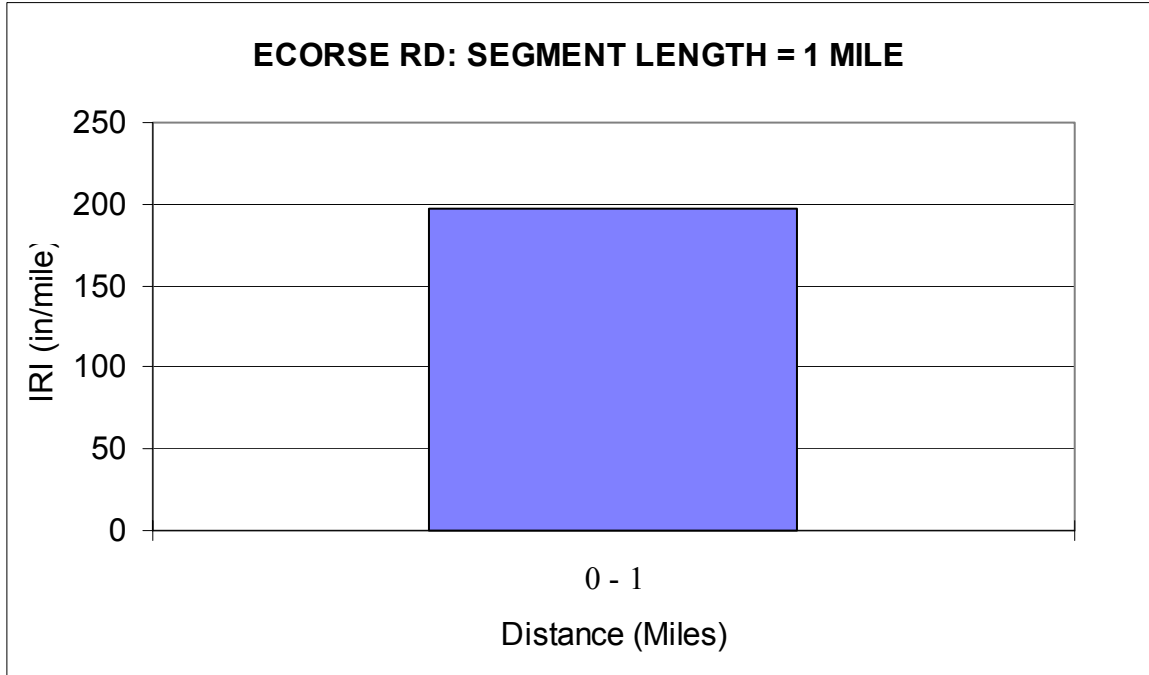


Figure 77. Ecorse Road – IRI for segment length of 1 mile.



Figure 78. Ecorse Road – IRI for segment length of 0.5 mile.

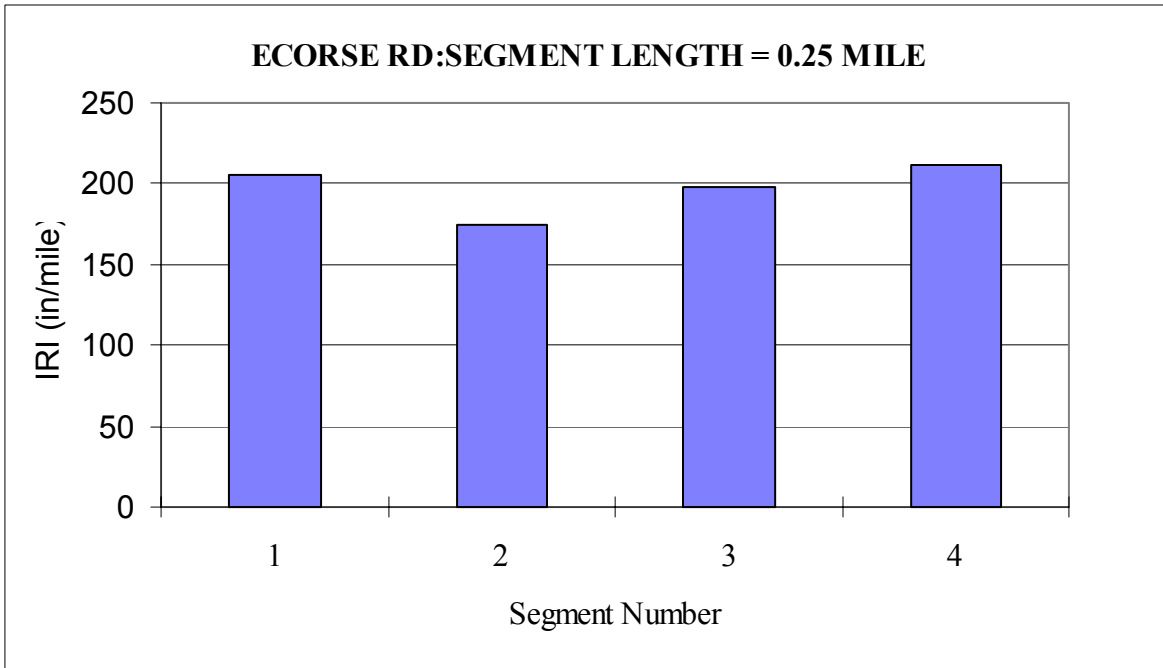


Figure 79. Ecorse Road – IRI for segment length of 0.25 mile.



Figure 80. Ecorse Road – IRI for segment length of 0.1 mile.

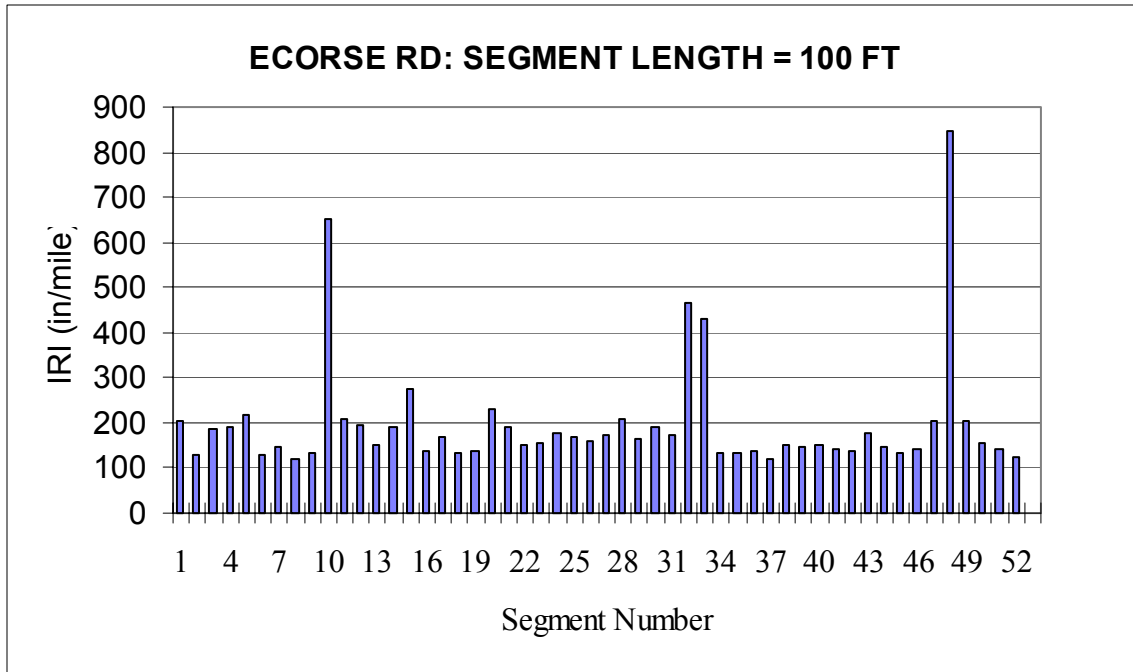
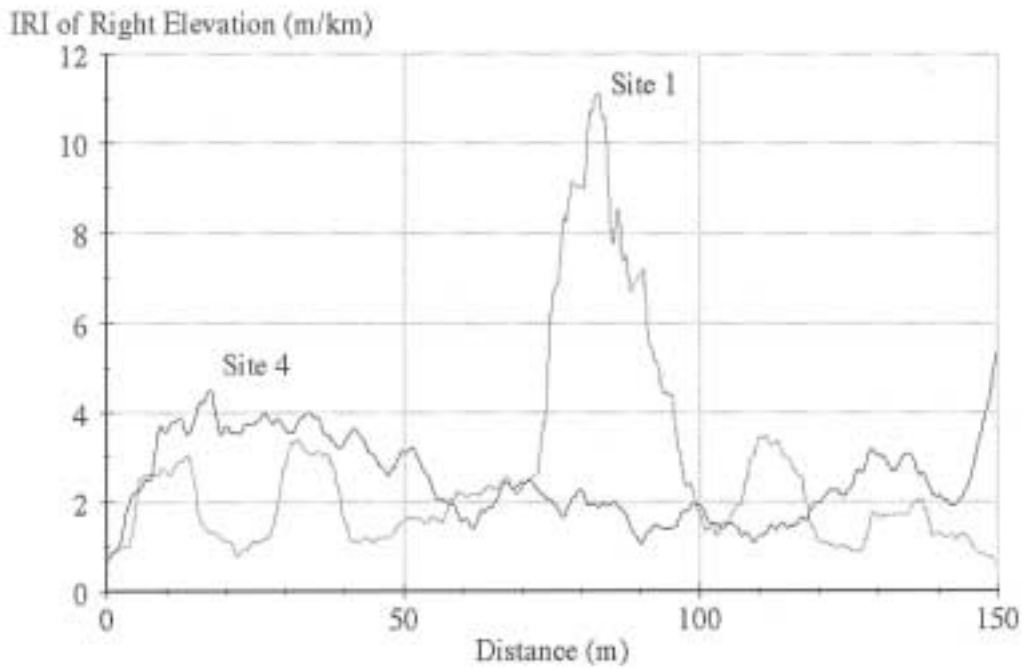


Figure 81. Ecorse Road – IRI for segment length of 100 feet.

### Roughness Profiles

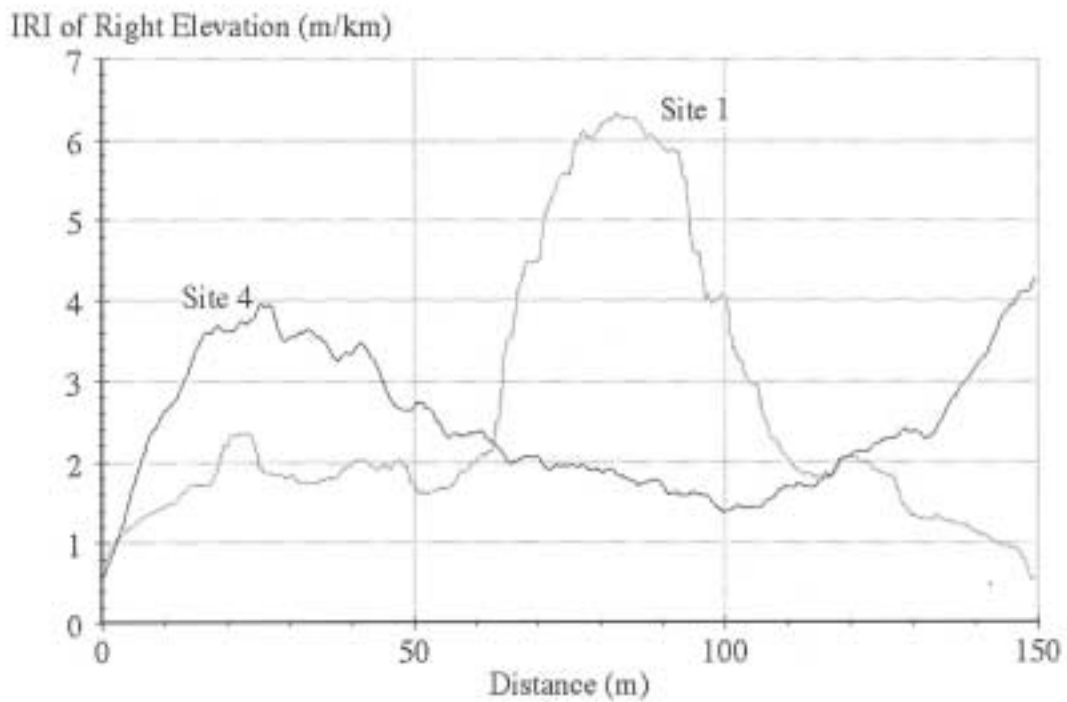
A roughness profile adds another dimension to the description of road roughness (13). Rather than providing a single index that summarizes the roughness of a road section, it shows the details of how roughness varies with distance along a road section. It is generated for a fixed length  $L$  used for averaging. At a point in the profile, take the IRI for the interval starting at  $L/2$  prior to the current location, and ending  $L/2$  past the current location. For example, if the averaging length is 30 m, the IRI value for the first 30 m is plotted at  $X=15$ . The IRI covering the range of from 1m to 31 m is plotted at  $X = 16$ .

Figures 82 and 83 shows the roughness profiles of two sections (site 1 and site 4) based on averaging lengths of 10 m and 30 m, respectively. The roughness value shown in figure 82 at any given distance is the roughness over a 10 m long segment that is centered at the specified distance. As shown in figure 82, at site 1 the IRI values range from a low of 0.75 m/km at  $X = 22$  m, to a high of 11.08 m/km at  $X = 83$  m. Figure 83 shows that the roughest 30 m section in site 1 is centered near the point  $X = 80$  m, with an IRI level of over 6 m/km. The peak value in a roughness profile generated with a short segment length may provide more information about a user's perception of the road than the average IRI of the entire section, since users tend to remember severe features.



**Roughness profiles based on 10-m length.**

Figure 82. Roughness profiles based on 10-m length (13).



**Roughness profile based on 30-m length.**

Figure 83. Roughness profiles based on 30-m length (13).

## CHAPTER 10

### FINDINGS FROM LTPP DATA ANALYSIS STUDIES

#### ROUGHNESS TRENDS OF ASPHALT CONCRETE PAVEMENTS

A major data collection effort at the General Pavement Studies (GPS) sections in the LTPP program is the collection of longitudinal profile data. The collection of profile data for the GPS sections commenced on or after 1990. The longitudinal profile data along the left and the right wheel paths are collected using a K.J. Law profiler. The profile data collection at the GPS sections is performed at regular intervals.

As a part of a study to analyze LTPP roughness data, Perera et al. (4) investigated the changes in roughness at GPS-1 test sections. The GPS-1 experiment is a study of asphalt concrete pavements on granular base. The changes in roughness at test sections were investigated by using the IRI as the roughness parameter. At the time this study was performed, each GPS section on average had been profiled four times. The test sections were grouped into four environmental zones (dry freeze, dry no-freeze, wet-freeze, and wet no-freeze) and the changes in IRI were examined for each environmental zone. The boundary between wet and dry regions generally corresponds to an annual precipitation of 508 mm. The boundary between freezing and non-freezing zones generally corresponds to an annual freeze index of 89 °C days. Correlation coefficients were computed between IRI and factors that could influence the development of roughness to identify factors that have a strong relationship with IRI.

#### Roughness Trends

For this study, the average IRI, which is the average value of the left and right wheel path IRI, was used to characterize the roughness at a site. The GPS-1 sections were established on in-service roads that were at different times of their service life. Therefore, the initial IRI of these sections is not known. The first IRI value that is available for a test section corresponds to the IRI obtained when the site was first profiled after being accepted to the LTPP program. As the climate is expected to have a major influence on roughness development, changes in IRI values of the GPS-1 sections were examined for each environmental zone. IRI vs. pavement age plots of the GPS-1 sections were separately prepared for each environmental zone. The age of the pavement corresponding to each profile date was obtained by subtracting the profile date from the construction date for the section. The IRI vs. pavement age plots for the four environmental zones are shown in figures 84 through 87. Each line in these figures shows the IRI trend for a GPS section. As seen from these figures, most of the GPS-1 sections show little change in IRI over the monitored period. Most of the sections showed a relatively flat trend in the IRI vs pavement age relationship.

The IRI trend plots indicated that there were several sections that were over 15 years old, but had low IRI values. A preliminary analysis of these sections indicated that they have carried a low

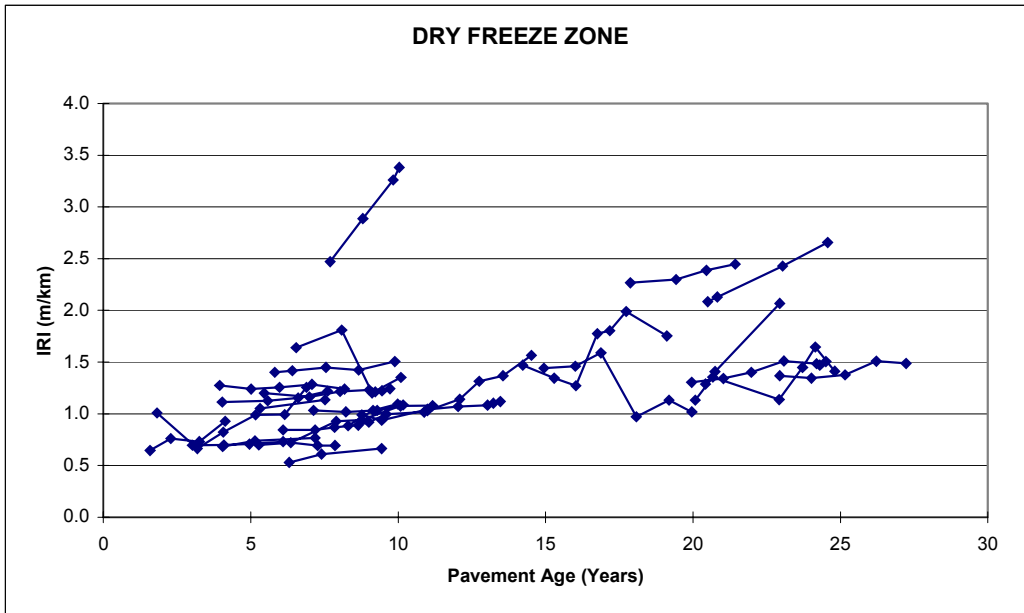


Figure 84. IRI trends of GPS-1 sections in dry-freeze zone (4).

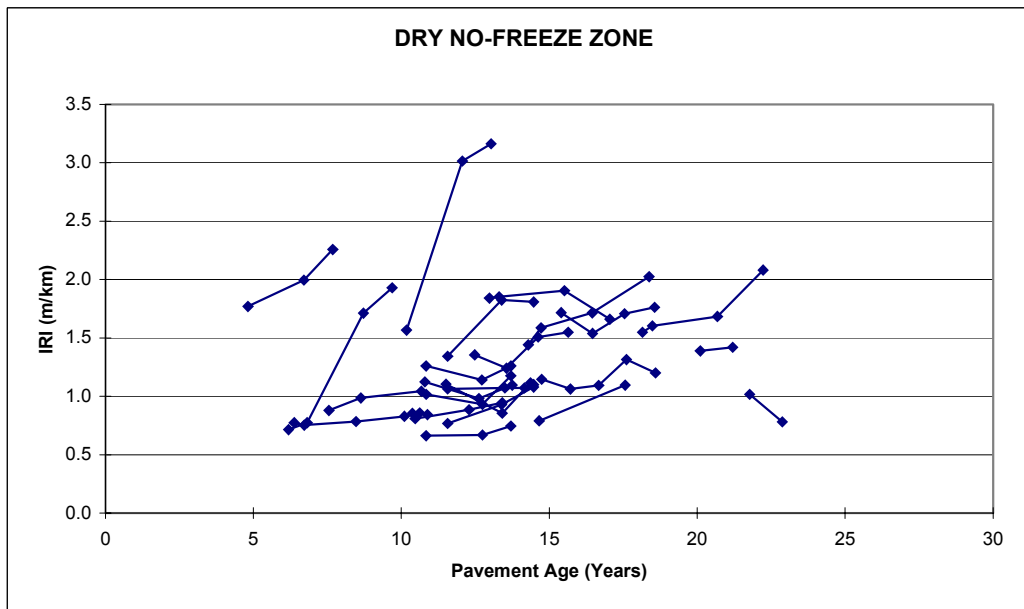


Figure 85. IRI trends of GPS-1 sections in dry no-freeze zone (4).

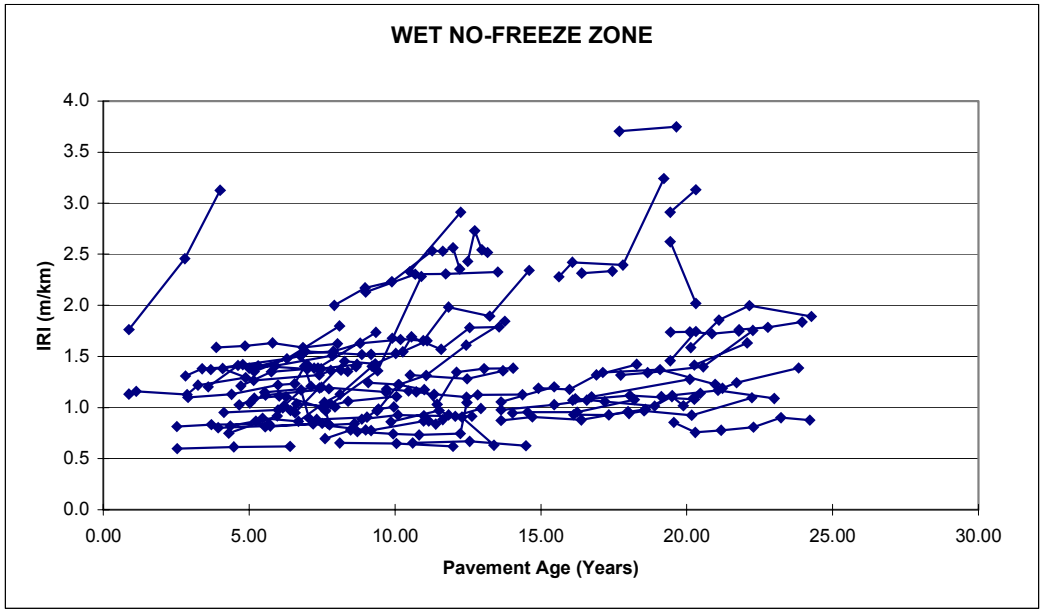


Figure 86. IRI trends of GPS-1 sections in the wet no-freeze zone (4).

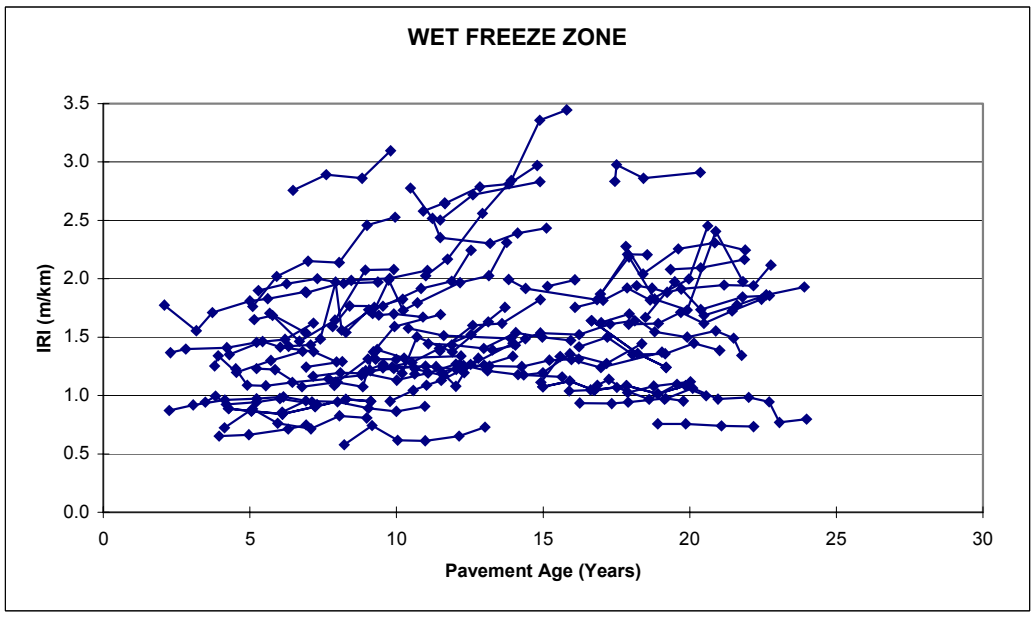


Figure 87. IRI trends of GPS-1 sections in the wet-freeze zone (4).

cumulative traffic volume when compared to the theoretical cumulative traffic volume that can be supported by that pavement section. A preliminary analysis of most of the sections that were showing a high increase in roughness over the years indicated that these sections were close to or have exceeded their design life based on equivalent single axle loads.

A closer examination of the individual IRI vs pavement age plots indicated that the changes in IRI that were noted at the GPS-1 sections could be classified into the following three categories: (i) IRI showed an increase with time, (ii) IRI remained relatively stable over time, (iii) IRI values were variable between the years with no clear overall trend for the section. It was noted that pavements with IRI in excess of 2 m/km generally exhibited larger increases in IRI over time when compared to the other sections.

Variability in the time sequence IRI values at a section can occur due to the following factors: variations in the profiled path, seasonal effects, and maintenance activities. Variations in the profiled wheel path for the different years can cause changes in the measured profile and in the computed IRI. In some pavements there is considerable transverse variability, which can cause considerable variations in the IRI depending on the wheel path that is followed. Changes in IRI can occur in pavements due to changes in profile caused by seasonal effects. If there are differences in the seasons in which the profile data are collected at a site, this can result in changes in profile between monitored periods because of seasonal effects. The profile of a pavement can change due to moisture changes on subgrade that cause the subgrade to swell or shrink. During winter months, frost heave of the subgrade and base layers can cause variations in the pavement profile. Both these effects will contribute to variations in the IRI. Maintenance activities on a section can change the IRI of a section. Repair of distressed areas can lead to a reduction in IRI of the pavement. The variable IRI patterns that were observed at some of the sections are attributed to these causes.

The IRI trend plots indicated that the sections in the no-freeze zones show much less variability in IRI when compared to the sections in the freezing regions. This is attributed to seasonal variability effects being more pronounced in the freezing regions.

### **Correlation Analysis**

Correlation analysis can be used to identify factors that are related to the roughness. The results of the correlation analysis that was carried out for the data in the dry-freeze zone are presented in table 28. As seen from the results shown in table 28, climatic factors and subgrade properties are strongly related to IRI in the dry-freeze region.

In all four environmental zones, roughness of pavements located over fine grained soils was related to the plasticity index and the percentage of subgrade passing the 75 $\mu$ m sieve. Pavements on fine grained soils having higher plasticity indices and higher percentage passing the 75 $\mu$ m sieve had higher IRI values. In freezing environments, sections located on areas that had a high freezing index or a high number of freeze thaw cycles generally had higher roughness. This observation indicates that adequate frost protection is an important factor for good pavement performance.



Table 28. Correlations between IRI and data parameters for sections in dry-freeze zone (4).

Parameter	Correlation Coefficient
Annual Precipitation	+ 0.48
Age of Pavement	+ 0.47
Wet Days per Year	+ 0.44
Days with > 12.5 mm Precipitation	+ 0.35
Subgrade % Passing 75 $\mu$ m Sieve	+ 0.33
Days below 0 °C per Year	+ 0.31
Annual Freeze Index	+ 0.28
Subgrade Moisture Content	+ 0.26
Cumulative Equivalent Single Axle Loads	+ 0.24
Subgrade Percent Sand	- 0.24

### Model Building

Models to predict the development of roughness of GPS-1 sections were developed by Perera et al. (4). An optimization technique was used to develop separate models for the different environmental regions. These models predict the initial IRI of the pavements with the use of subgrade properties and structural properties of the pavement, and then predict a growth rate that is a function of time, traffic, subgrade properties, and pavement structure. Figure 88 presents the model form that was used to predict IRI for GPS-1 sections in the dry-freeze zone. In the modeling process, the aim was to develop a model that would match the time-sequence IRI data that were available at a site. Figure 89 presents the match between actual and predicted IRI values.

### Key Findings

- The observed trends in the GPS-1 sections seem to indicate that pavement roughness remains relatively constant over the initial life of the pavement, and then after a certain point show a rapid increase.
- Performance of GPS-1 sections over fine grained soils was strongly related to Atterberg limits of the subgrade and the percentage of subgrade passing the 75 $\mu$ m sieve.
- Pavements in areas that have a high freezing index or a high freeze thaw cycle had higher IRI values. This indicates that adequate frost protection is an important factor for good pavement performance in freezing regions.
- In hot climates, higher IRI values were noted for sections in areas that had higher number of days above 32°C.

$$IRI(t) = IRI_0 e^{\left[ \frac{r_0 t^S}{T} \right]}$$

$$IRI_0 = A(P200)^B + C(Po)^D + E(\%Sand)^F + G\left(\frac{\%ACinSN}{100}\right)^H$$

$$r_0 = \frac{I(KESAL / yr)^J}{K(SN)^L} + O(Ann Precip)^P + Q\left(\frac{FZI * P200 * w\%}{Po}\right)^R$$

A= 250.1471	F= 5.5503	K= 300	R= 1.5855
B= -0.0287	G= 203.8493	L= 4	S= 1.0952
C= -517.4179	H= 0.05983	O= 0.0006	T= 0.1879
D= -0.06407	I= 0.005	P= 0.6708	
E= -2.847E-10	J= 1.7	Q= 6.84E-09	

IRI(t)	IRI at time t in in/mile
t	Time in Years
P200	Percent Passing the No. 200 (75 μm) sieve
Po	Overburden Pressure on Subgrade (psf)
% Sand	Subgrade Percent Passing No 4 (4.75 mm) Sieve - Percent Passing No. 200 Sieve (75μm)
%ACinSN	Percent of Structural Number Contributed by Asphalt Concrete Thickness (%)
KESAL/yr	Average Equivalent Single Axle Loads (in 1000's) per year
SN	Structural Number of Pavement
Ann Precip	Annual Precipitation in inches
FZI	Annual Freezing Index in degrees Fahrenheit per year
w%	Subgrade Moisture Content (%)

Note: 1 m/km = 63.4 in/mile

Figure 88. Model to predict IRI for pavement sections in dry-freeze zone (4).

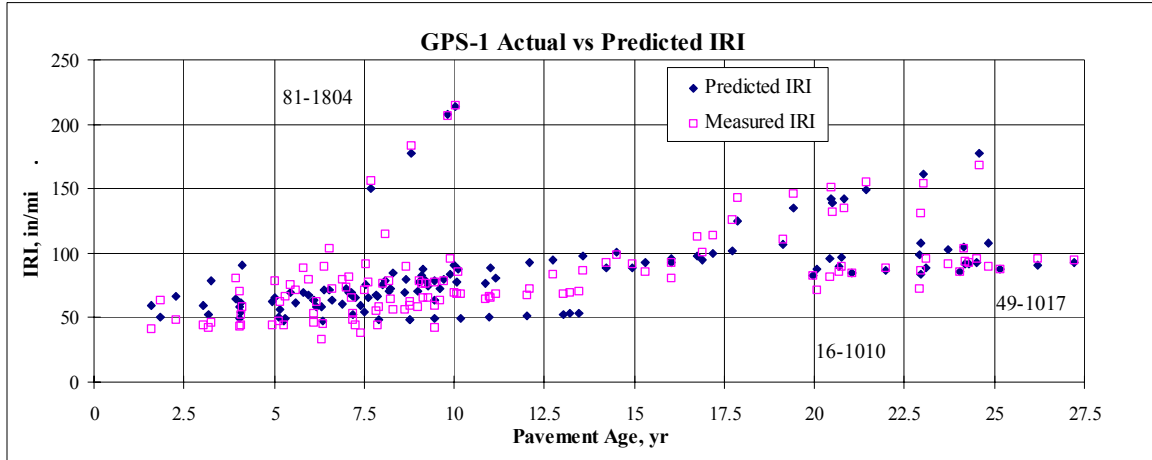


Figure 89. Actual vs. predicted IRI for GPS-1 dry-freeze model (4).

## ROUGHNESS TRENDS IN PCC PAVEMENTS

As a part of a research study to analyze roughness data at LTPP sections, Perera et al. (4) as well as Khazanovich et al. (5) performed a comprehensive analysis to investigate changes in roughness at PCC test sections. The LTPP experiments that were investigated in this study were: GPS-3 sections (Jointed Plain Concrete Pavements, JPCP), GPS-4 sections (Jointed Reinforced Concrete Pavements, JRCP), and GPS-5 sections (Continuously Reinforced Concrete Pavements, CRCP). The changes in roughness at test sections were investigated by using the IRI as the roughness parameter. Relationships between the IRI and design factors, subgrade conditions and climatic factors were also investigated in this study. At the time this study was performed, on average each GPS section has been profiled four times.

### Roughness Behavior of Jointed Plain Concrete Pavements (JPCP): GPS-3

The average slab length for the GPS-3 pavements was 4.5 m. The IRI vs. pavement age plots for GPS-3 sections in the wet freeze and wet no-freeze zones are shown in figures 90 and 91, respectively (4). Each line in these plots represents a pavement section. The majority of the sections show little change in IRI over the monitored period. The roughness plots for the wet no-freeze zone are much flatter than the plots in the wet-freeze zone, which shows the influence of the climate on pavement performance.

There were distinct differences in performance between doweled and undoweled pavements (4). A strong relationship between IRI and the amount of faulting was noted for the non-doweled pavements, and this relationship is shown in figure 92. In this figure, the total faulting is the sum of faulting at all joints and cracks. Such a relationship was not seen for the doweled pavements.

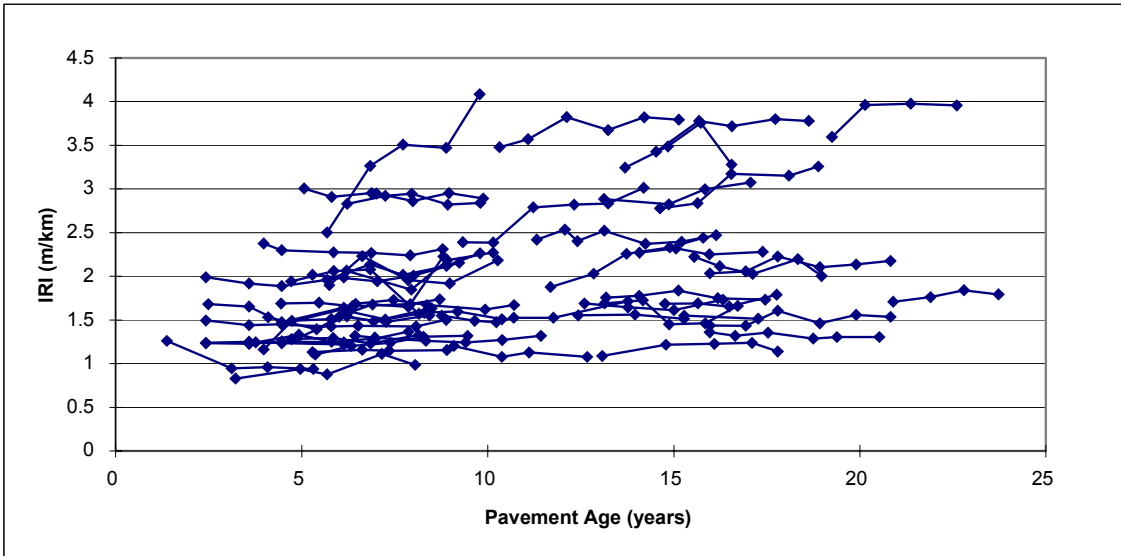


Figure 90. IRI vs. pavement age relationships for JPC pavements in wet-freeze zone (4).

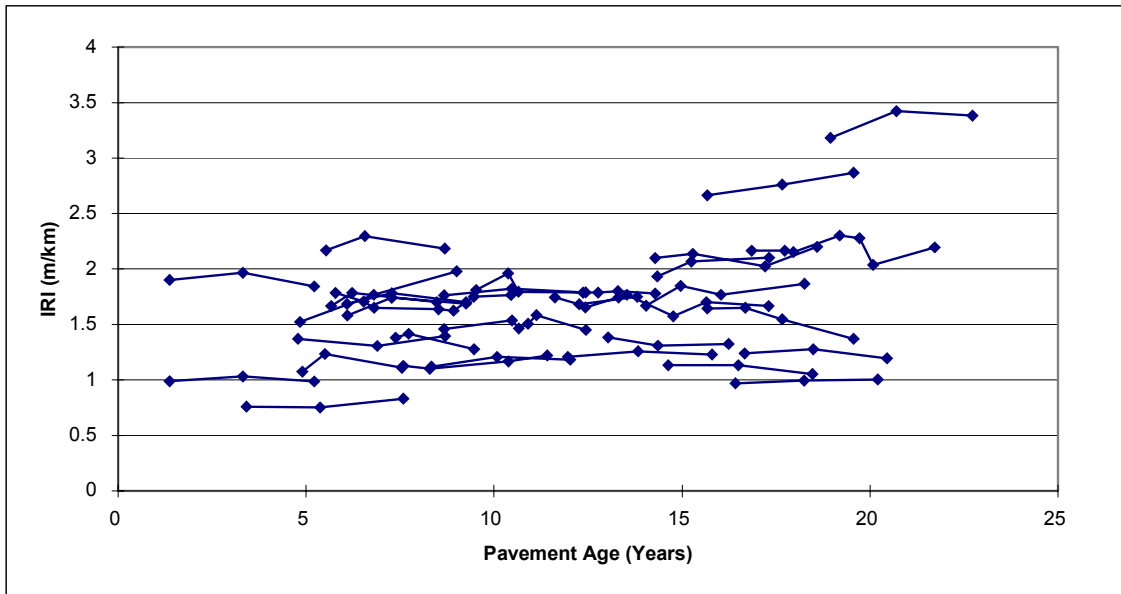


Figure 91. IRI vs. pavement age relationships for JPC pavements in wet no-freeze zone (4).

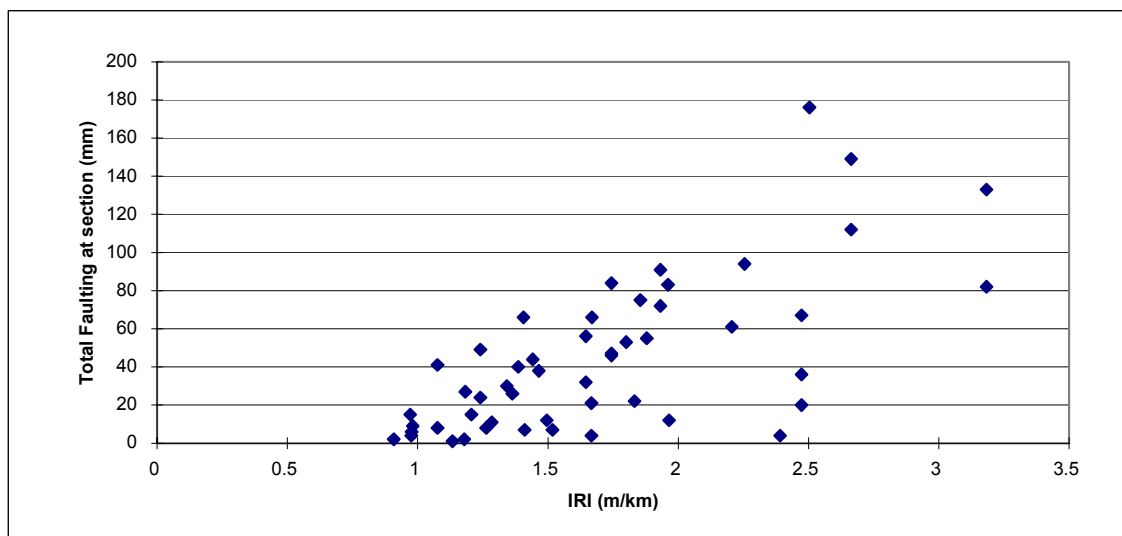


Figure 92. Relationships between faulting and IRI for non-dowelled JPC pavements (4).

Better performance can be achieved at non-dowelled pavements by designing them to minimize faulting. Non-dowelled pavement that had higher values for modulus of subgrade reaction had lower IRI values. For both dowelled and non-dowelled pavements, higher IRI values were generally indicated for pavements located in areas that received high precipitation, had higher freezing indices, and had a high content of fines in the subgrade. In the non-freeze regions, pavements located in areas that had a high number of days above 32° C had lower IRI values for both doweled and non-dowelled pavements. This factor is likely to be related to the higher load transfer that occurs at higher temperatures. Pavements that had higher modulus values for PCC had higher IRI values. This indicates that mix design factors and the type of aggregate used may influence the performance of the pavement from a roughness point of view.

Khazanovich et al. (5) analyzed the roughness trends in JPC pavements by dividing the LTPP sections into three groups based on time vs. IRI performance. The three groups were classified as poor, normal and good. Of the sections that were rated as poor, approximately 71 percent of the sections were located in wet-freeze regions, with 24 percent of the sections being in the dry-freeze region, and 6 percent in wet no-freeze region. None of the poorly performing sections were located in dry no-freeze region. They also found a strong relationship between pavement performance and subgrade type. Approximately 67% of sections constructed on fine-grained soils had a poor IRI performance, while only 33% of sections on coarse-grained soils had poor IRI performance. No trend between traffic and IRI was found. Generally the good performing sections had higher traffic. An effect of traffic on IRI should be noted if the pavement was under-designed. If the pavement was adequately designed it is unlikely that a trend between IRI and traffic would be noted. Sections with stabilized bases had lower IRI compared to sections with granular bases. In the poor performance group, 82% of the sections had granular bases while 18% of the sections had stabilized bases.

## Roughness Behavior of Jointed Reinforced Concrete Pavements (JRCP): GPS-4

Nearly 70 percent of the GPS-4 sections are located in the wet freeze region. Figure 93 shows the IRI vs pavement age plots for the GPS-4 sections in the wet freeze region (4). Each line in this plot represents a GPS-4 section. The overall trend in roughness at these sections appears to be an exponential increase in IRI.

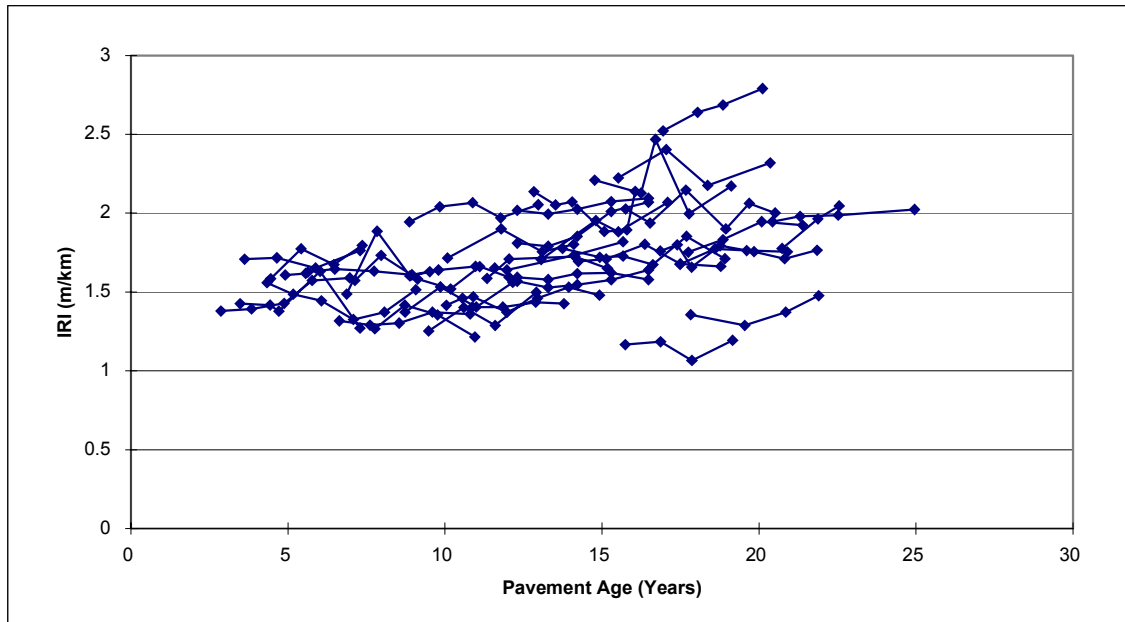


Figure 93. IRI vs. pavement age relationship for JRC pavements in wet-freeze zone (4).

The slab lengths for JRCP pavements generally ranged from 9 m to 18 m. Higher IRI values were indicated for pavements in areas having high precipitation, higher moisture contents in the subgrade, thicker slabs, longer joint spacing, and higher modulus values for PCC (4). The higher IRI values for thicker slabs may be construction related. An increased joint spacing would likely result in a greater proportion of transverse cracks, and may result in spalling and faulting at these locations that would contribute to higher roughness. Lower IRI values were indicated for pavements that had higher values for modulus of subgrade reaction, higher PCC compressive strengths, higher water and cement contents in the PCC mix. A mix that has a higher water cement ratio would be more workable compared to a mix with a lower water cement ratio, however a mix that has a lower water cement ratio is expected to be more durable over the long term.

Khazanovich et al. (5) performed an analysis for LTPP JRCP sections to identify factors affecting roughness. Many of their conclusions were similar to those obtained by Perera et al. (4). They determined that JRCP constructed on coarse-grained soil performs better than JRCP constructed on fine grained soils. All JRCP rated as poor were constructed on fine grained soil, while no JRCP rated as poor was constructed on coarse grained soil. They indicated where poor subgrade soil exists, the specification of a thick granular layer will be beneficial. Khazanovich et al. (5) analyzed the relationship between IRI and traffic, but observed no specific trends. JRCP in

good IRI performance category carried much higher ESALs than those in the poor or normal group.

### Roughness Behavior of Continuously Reinforced Concrete Pavements (CRCP): (GPS-5)

Most of the CRCP sections are located in the wet freeze and wet no-freeze environmental zones. The IRI vs pavement age plots for CRCP pavements in the wet no-freeze zone are presented in figure 94 (4). Each line in this plot represents a GPS-5 section. Similar behavior patterns were observed in the wet no-freeze zone. As seen from this plot, most of the sections appear to be maintaining a relatively constant IRI. The IRI behavior pattern appears to be similar for new as well as old pavements. There are many sections that are over 15 years old, but are still very smooth. This observation indicates that the CRCP pavements appear to maintain their initial IRI over a long period. Lower IRI values were indicated for pavements that had higher percentage of longitudinal steel and higher water cement ratios. Higher IRI values were indicated for sections that had higher values of PCC elastic modulus. This indicates mix design factors such as coarse aggregate content and type of coarse aggregate may affect the roughness behavior. In the non-freezing areas, sections located in areas that had a higher number of days above 32 °C had higher IRI values.

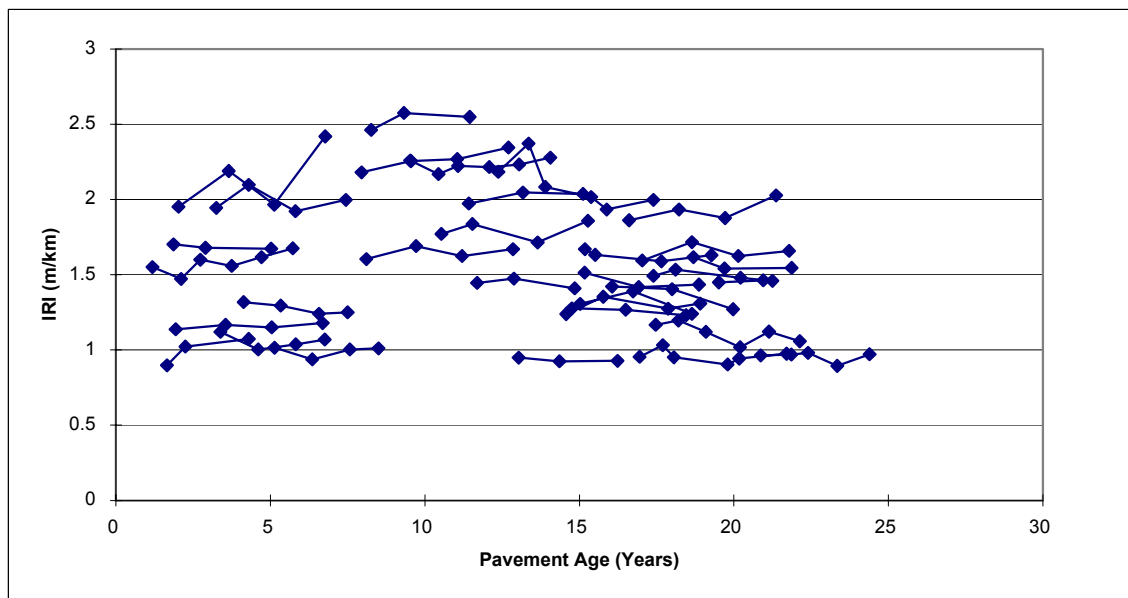


Figure 94. IRI vs. pavement age relationship for CRC pavements in wet-freeze zone (4).

Khazanovich et al. (5) analyzed roughness trends in CRCP pavements by dividing the LTPP sections into three groups based on time vs. IRI performance. The three groups were classified as poor, normal and good. Many of their conclusions were similar to the conclusions obtained by Perera et al. (4). They estimated 28-day flexural strength from construction inventory data and from results from indirect tensile strength tests, and found that higher modulus of rupture values resulted in rougher pavements. The higher strength concrete mixes

generally have a lower water to cement ratio, which makes them less workable. This may make it more difficult to properly finish the concrete surface and lead to rougher pavements. In general, pavements constructed over coarse grained soils performed better than those constructed over fine grained soils. Of all poorly performing sections, 63 percent were located on fine grained subgrade soil while 37 percent was built on coarse grained soils. No clear trends were seen between IRI and traffic. Sections that were in the good category had higher traffic volumes.

## **Key Findings**

- In non-doweled JPC pavements, a strong relationship existed between IRI and faulting. This relationship was not noted at JPC sections with dowels. Lower IRI values were noted for non-doweled pavements that had high modulus of subgrade reaction values.
- A strong relationship existed between the performance of JPC pavements and climatic region. Pavements in wet-freeze regions had the poorest performance, while pavements in dry no-freeze regions had the best performance.
- A strong relationship existed between the performance of JPC pavements and subgrade type. Pavements on coarse grained soils gave better performance than pavements on fine grained soils.
- The overall trend in roughness at JRC pavements appears to be exponential growth of roughness. For JRC pavements higher IRI values were associated with high precipitation, higher moisture content in subgrade, thicker slabs, longer joint spacing, and higher modulus values for PCC. Lower roughness values were associated with pavements that had higher values for modulus of subgrade reaction, higher PCC compressive strengths, higher water and cement contents in the PCC mix.
- CRCP pavements appear to maintain a relatively constant IRI over a long period. Lower IRI values were associated with higher percentage of longitudinal steel and higher water cement ratios for PCC mix. Higher IRI values were associated with higher values of PCC modulus. In non-freezing areas, higher IRI values were noted for pavements in areas that had higher number of days above 32° C.

## **REHABILITATION OF ASPHALT CONCRETE PAVEMENTS**

As a part of a research study to analyze roughness data at LTPP sections, Perera et al. (4) performed an analysis of the roughness data that were collected at the SPS-5 sections. The SPS-5 experiment was developed to investigate the performance of selected asphalt concrete rehabilitation treatment factors. The rehabilitation treatment factors include overlay mix type (recycled and virgin), overlay thickness and surface preparation of the existing asphalt concrete surface prior to overlay (minimal and intensive preparation). Nine test sections are included in each SPS-5 project, with eight sections being experimental sections and one section being a control section. A description of the overlay thickness, type of material used for the overlay, and the level of surface



preparations that were carried out at the test sections prior to placing the overlay are shown in table 29 for the eight experimental sections. The control section received only routine maintenance.

Table 29. Treatments applied to SPS-5 test sections

Section Number	Surface Preparation Prior to Overlay	Type of Asphalt Concrete for Overlay	Overlay Thickness (mm)
2	Minimum	Recycled	50
3	Minimum	Recycled	125
4	Minimum	Virgin	125
5	Minimum	Virgin	50
6	Intensive	Virgin	50
7	Intensive	Virgin	125
8	Intensive	Recycled	125
9	Intensive	Recycled	50

The minimum level of surface preparation consists primarily of patching of severely distressed areas and potholes, and placement of a leveling course in ruts that are greater than 12 mm deep. The intensive level of preparation includes milling of the existing asphalt concrete surface, patching of distressed areas, and crack sealing after milling. Milling was performed to a depth of 38 to 50 mm and the depth of material removed by milling was replaced with an equal thickness of asphalt concrete overlay material. The depth of replacement material is not counted as a part of the overlay thickness specified in the experiment. Longitudinal profile measurements were performed on the test sections prior to rehabilitation and immediately after rehabilitation. The IRI value for each test section is computed from the profile data.

The data analysis was conducted to determine the reduction in roughness that was achieved at the test sections that were subjected to different rehabilitation techniques.

### **Comparison of IRI Before and After Overlay**

Figure 95 presents the IRI before and after rehabilitation for four SPS-5 projects. As seen in this figure, for a specific project, the IRI after overlay of the test sections fell within a relatively narrow band, irrespective of the IRI before overlay. However, the range of this band varied from project to project. The construction procedures, capability of the contractor, and the predominant wavelengths that contribute to IRI that are present in the pavement prior to overlay may be the factors that determine the range of this band.

A comparison of the IRI after overlay for the test sections that received minimum and intensive surface preparations prior to overlay are presented in figure 96. The IRI values presented in this figure for each category of surface preparation is the average IRI computed from four test sections that are in each category. Overall, the average IRI values for the minimum and intensive surface preparation sections were close to each other, with the intensive surface preparation sections having a slightly lower IRI for a majority of the projects.

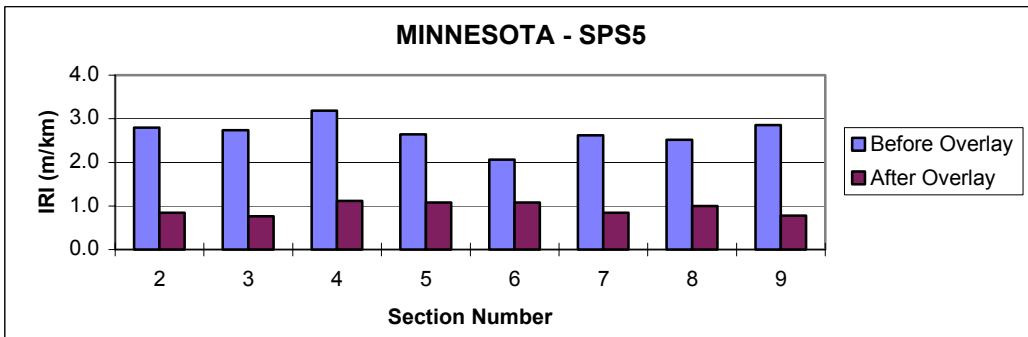
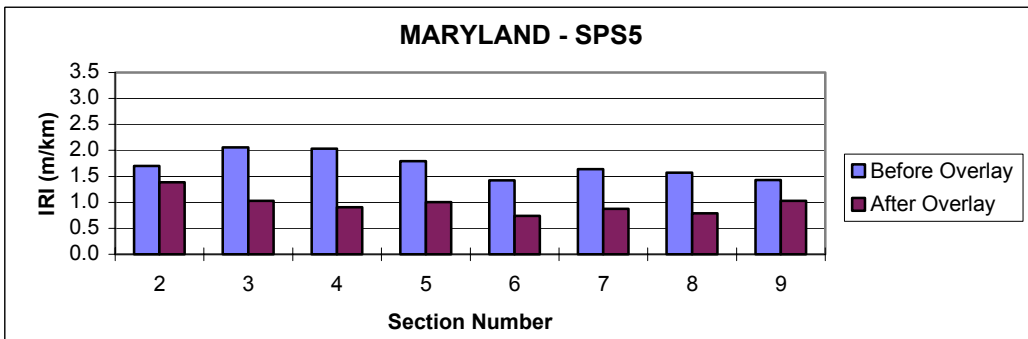
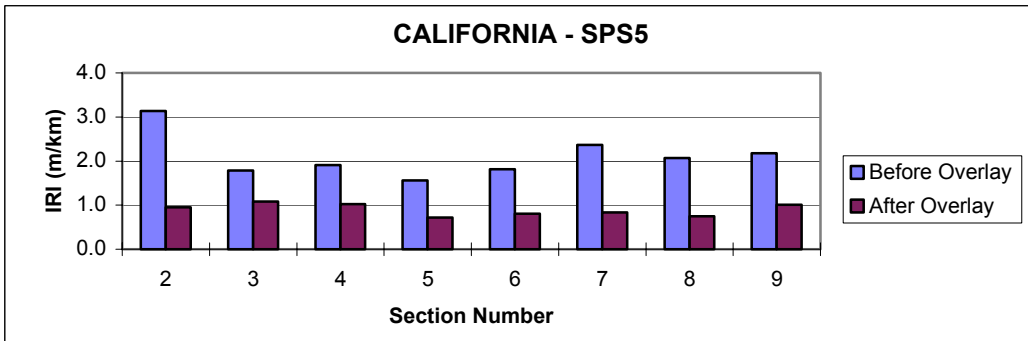
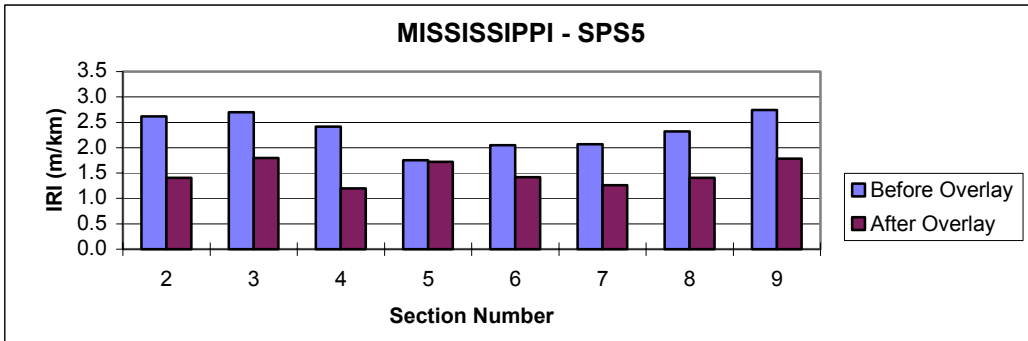


Figure 95. IRI before and after overlay for four SPS-5 projects.

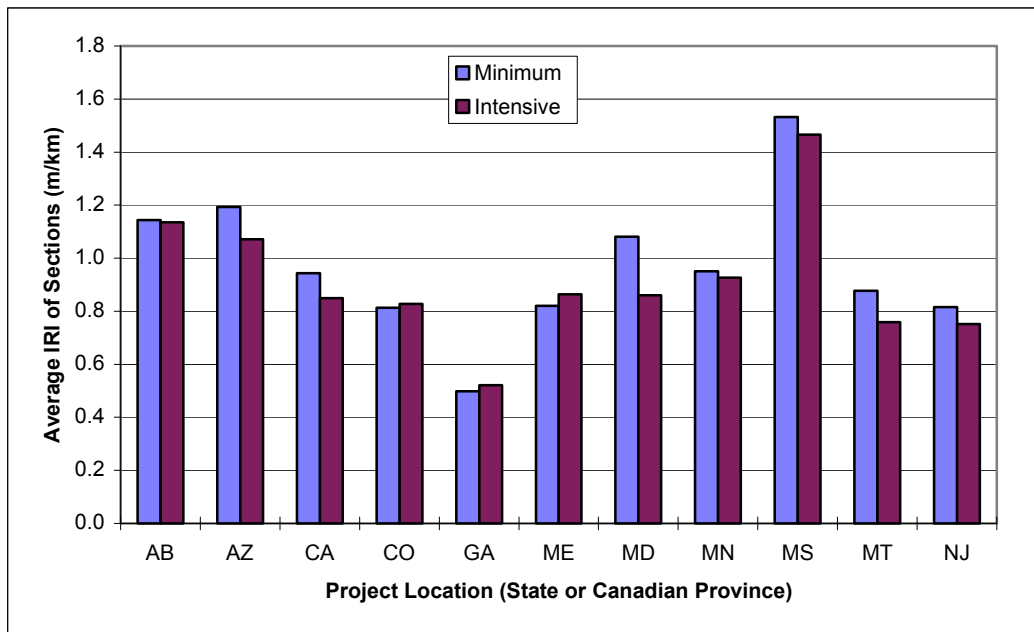


Figure 96. Average IRI after overlay for sections receiving minimum and intensive surface preparation prior to overlay

Figure 97 presents the relationship between IRI before and after overlay for all experimental sections. The sections that have an IRI before overlay of less than 1.4 m/km are from two projects. If only the sections that have an IRI greater than 1.4 m/km before overlay are considered, data in figure 97 shows that there is no relationship between the IRI before and after the overlay. Thin overlays are seen to be capable of reducing the roughness of a pavement by a substantial amount in some cases. For example, figure 97 shows that at three sections that had IRI between 2.5 and 3 m/km, a 50 mm thick overlay reduced the IRI to approximately 0.8 m/km.

### Roughness After Overlay

The frequency distribution of the IRI after overlay for the test sections that received a 50 mm overlay is shown in figure 98. This figure also presents the cumulative frequency curve, and shows that approximately 55 percent of the test sections had an IRI value of less than 1 m/km, while 85 percent of the test sections had an IRI value of less than 1.2 m/km. The frequency distribution of the IRI after overlay of the test sections that received a 125 mm overlay is shown in figure 99. The frequency distribution curve in this figure shows that approximately 65 percent of the test sections had an IRI after overlay of less than 1 m/km, while 85 percent of the test sections had an IRI after overlay of less than 1.2 m/km. The analyzed data show that in 85 percent of the cases, the IRI of an overlaid pavement would be less than 1.2 m/km for both overlay thicknesses.

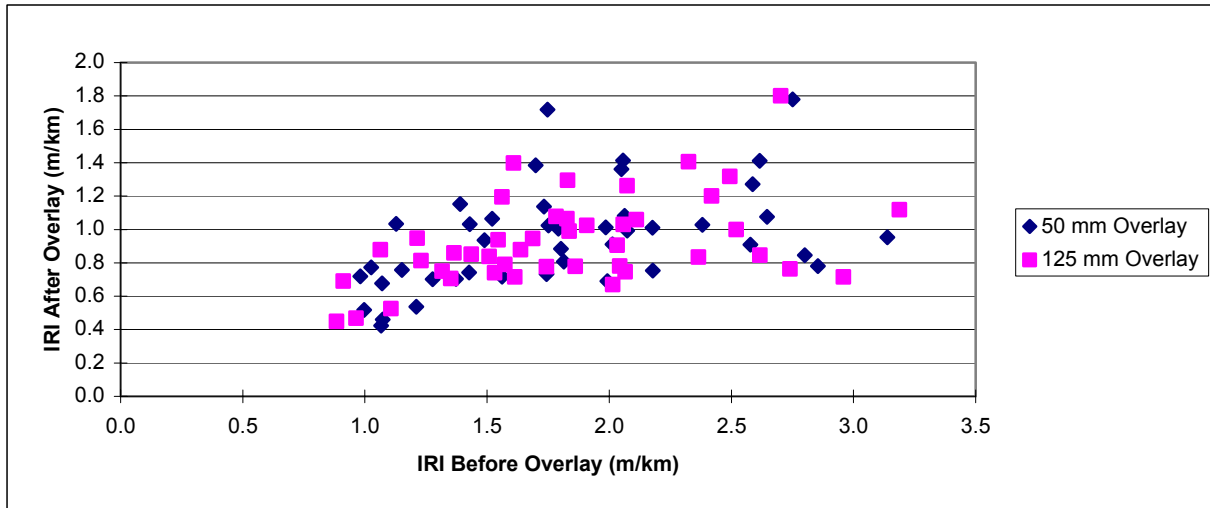


Figure 97. IRI before and after overlay.

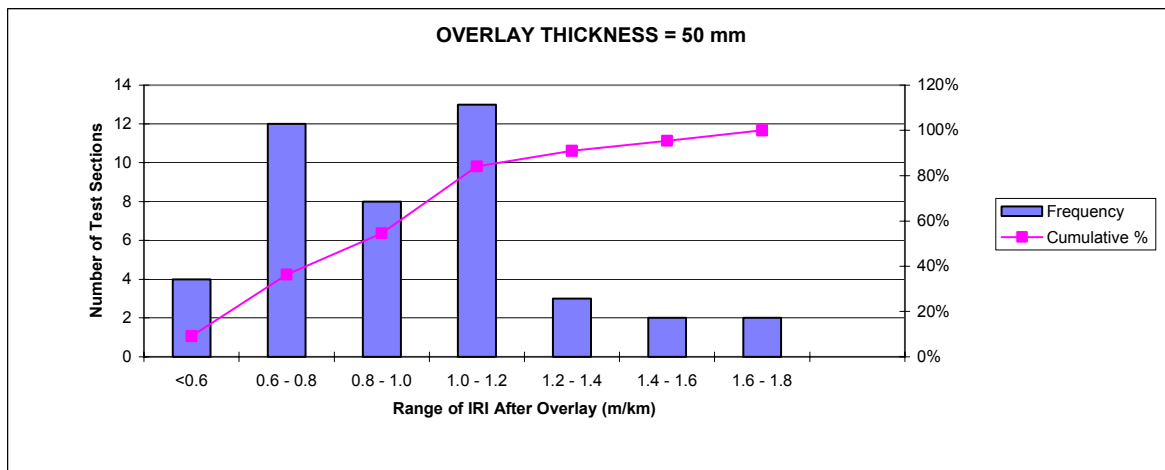


Figure 98. Frequency distribution of IRI after overlay for sections receiving a 50 mm overlay.

### Development of Roughness After Overlay

Sufficient time-series data were not available for the SPS-5 projects at the time this study was performed to study the development of roughness over time. The GPS-6B experiment in the LTPP program investigates the behavior of AC pavements that have received an AC overlay. The GPS-6B sections were overlaid after the inception of the LTPP program. Therefore, the IRI of the pavement immediately after the overlay is known for these sections. Figure 100 shows the roughness progression of the GPS-6B sections in the wet-freeze zone. The roughness development trends in this figure shows that most of the sections appear to maintain a relatively constant IRI over the initial life of the pavement.

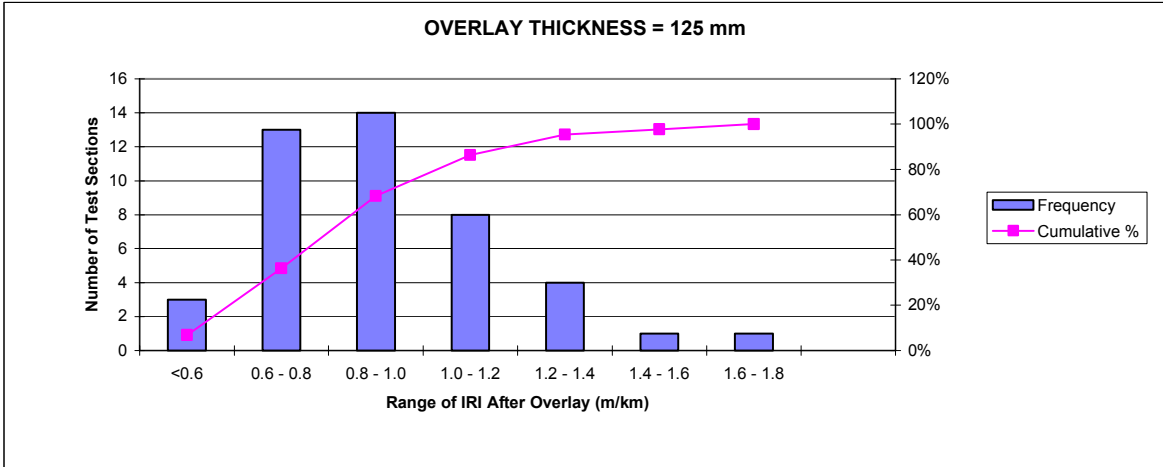


Figure 99. Frequency distribution of IRI after overlay for sections receiving a 125 mm overlay.

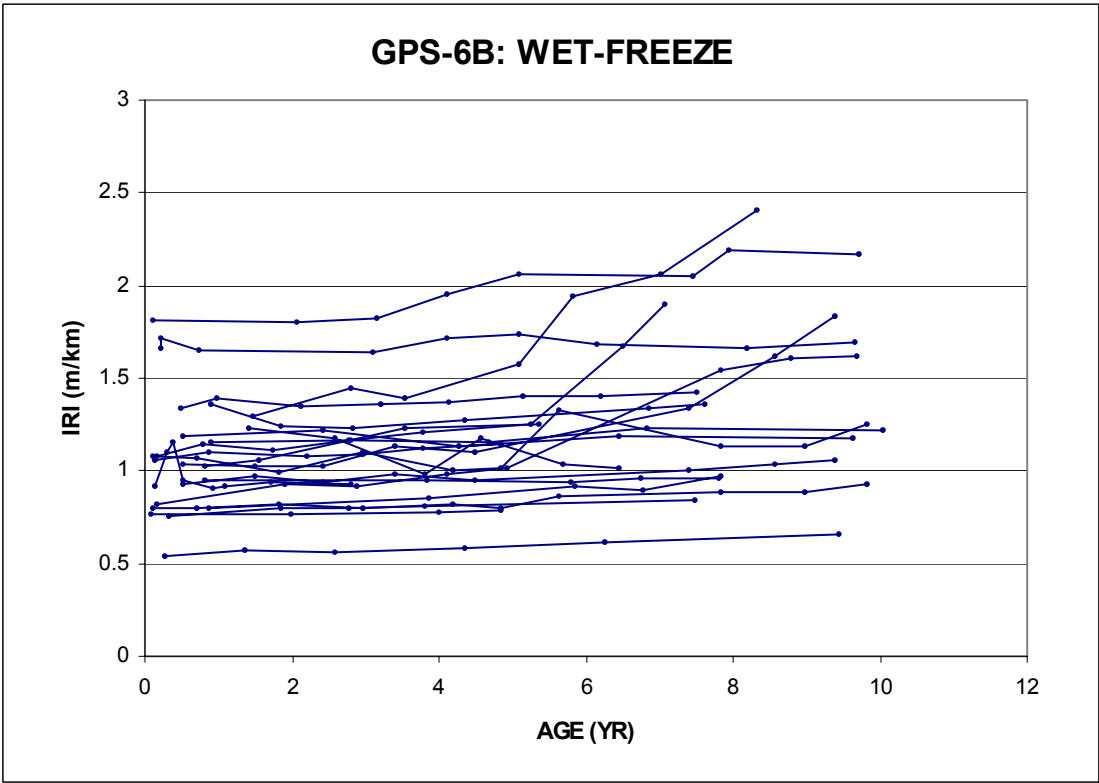


Figure 100. Roughness progression of AC pavements with an AC overlay.

## **Key Findings**

The key findings from this study were:

- Irrespective of the roughness before overlay of a section, the roughness after overlay of the sections for a specific project would fall within a relatively narrow band. However, the range of this band varied from project to project.
- Thin overlays were seen to be capable of substantially reducing the roughness of a pavement.
- The IRI of overlaid pavements was less than 1.2 m/km for 85 percent of the test sections that received an asphalt concrete overlay. This was noted for sections that received either a 50 mm or a 125 mm overlay.

## CHAPTER 11

### CURRENT ISSUES RELATED TO PAVEMENT SMOOTHNESS MEASUREMENTS

#### INTRODUCTION

Over the past several decades, road profiling technology has evolved from a research tool to a routine surveying tool for tracking the roughness condition of highway networks, as well as for measuring the smoothness of new pavements. This, coupled with the development of standardized roughness metrics such as the IRI and the RN, has made it possible for the highway community to know the state of the networks on an ongoing basis. Such information serves not only the highway community as a data source for decision making on maintenance and rehabilitation, but also serves the various interest groups peripheral to the highway community with objective information about its condition.

As profiling devices have become more common and distributed among the State users, disparities in performance have been observed. In part, this derives from the lack of standards by which to test system performance, and variations in design and hardware. Early evidence of these problems motivated efforts to quantify the differences between profiling devices and discover their sources in exercises such as the Ann Arbor Road Profilometer Meeting (30) and the annual meetings of the Road Profiler User Group (RPUG) (31,32).

As our understanding of profiling systems has evolved, we now realize that differences arise from two sources that can be better controlled.

- System performance—The various makes and designs of profiling devices have different performance capabilities due to the way in which the road surface is sensed (sensor footprint), the interval at which the surface is sampled, and the way the data are processed to determine the profile and roughness values. Some of these differences are caused by deficiencies in system design, but others are simply a matter of a lack of standardization. Since the road profile is a continuous function that is digitally sampled, the process used will affect the results, and until there is a well-defined standard for measurement of road profile, these differences will exist.
- Operator practices—The operators of profiling equipment differ in their practices in ways that may affect the measurement of profile and the resulting roughness value. Some of the variations are inherent to the measurement process, such as where the profile is started, and where in the wheeltrack the measurement is made. Other factors arise from the practical problems of making measurements on public roads. For example, operators must sometimes adjust driving practices to accommodate other traffic, forcing them to slow down or even stop at times or to vary in lane position. These problems are most often encountered in network surveys where many miles of measurement are required, and the operator is faced with the choice of turning around to repeat the measurement of a section or accepting the fact that a small portion of the measured data is erroneous.

Other sources that contribute to differences in roughness measurements exist. Dominant among these is the fact that the profile (and hence the roughness) of a road section does not have a single value but varies across a lane and with time. Depending on construction, rigid pavements can vary in roughness on a daily cycle due to temperature gradients; all pavement types may exhibit seasonal variation in roughness; and, of course, pavement roughness varies over years with deterioration. It is often difficult to plan for daily and seasonal changes. If annual surveys of a road section could be scheduled for the same date and time each year, presumably the year-to-year comparisons of roughness would be more meaningful. However, this may be difficult or impossible to accomplish. Even if an identical schedule was achieved each year, climatic conditions are never the same in consecutive years.

Given the existence of these sources of variation, there will always be some lack of precision associated with roughness measurements. However, there are steps that can be taken to reduce the magnitude of variations arising from the equipment and operators. The options for improvement fall within the areas of responsibility of all involved—from the State agency level down to the operators and manufacturers of the equipment. Detailed instructions for improving the quality of roughness measurements, based on the research performed for NCHRP Project 10-47 are provided in the document *Operational Guidelines for Longitudinal Profile Measurements (3)*.

## **ROLE OF HIGHWAY ADMINISTRATORS, OPERATORS AND ANALYSTS**

State highway administrators, profiler operators as well as data analyst all play an important role to ensure that accurate profile data are collected and analyzed.

### **State Highway Administrators**

Road profilometry is a highly technical activity, subject to very subtle error sources not obvious to the untrained. Within State highway departments, roughness data quality can be improved by instituting certain administrative practices as follows:

- Enlist technically qualified personnel to oversee profiling operations, preferably an engineer trained in digital signal acquisition and processing methods.
- Establish policies that will encourage development of an experienced operating crew able to detect when invalid profile information is being obtained and diagnose the source of error.
- Encourage and support participation of the profiling crew in the annual Road Profiler User Group meetings so that they benefit at first opportunity from the newest discoveries of problem areas.
- Encourage and support participation of the chief technical person in road profiling standardization efforts through the AASHTO and/or ASTM.

Administrators should also recognize that accurate profiling equipment is worth the investment. The *Operational Guidelines for Longitudinal Profile Measurements* that was developed for NCHRP Project 10-47 (3) list several aspects of equipment design and



performance that are needed to produce reliable roughness measurements. Compromising on the cost of a profiler is false economy both due to the hidden costs of personnel time lost while compensating for profiler shortcomings, and due to compromise in the validity of the roughness database in a pavement management system. The original move from response-type road roughness measuring systems was motivated by the superior repeatability and time-stability possible with profilers. Unless a profiler provides these qualities, they only differ from response-type systems in cost.

## **Operators and Analysts**

In routine operation of a profiling system, there are a number of ways in which the quality of the data may be affected by operating practices. The Operational Guidelines for Longitudinal Profile Measurements that was developed for NCHRP Project 10-47 (3) provide a detailed discussion of specific practices that can reduce the variability of profile data. Highway agencies can use these operating guidelines as a starting point to develop a rigorous set of guidelines specific to the operation of the agency.

For operators, perhaps the most important issues are to develop consistent practices for maintaining acceptable speeds and position in the roadway during measurement. These types of practices need to become routine. Development of agency-specific guidelines as well as an operator's checklist is means to increase consistency that will improve the quality of roughness data, particularly when different profiler operators are involved. An example of such a guideline is the LTPP Manual for Profile Measurements, which is used for collecting data at LTPP test sites (52).

At the same time, the analysts that use the data for project-level and network-level monitoring should become familiar with the procedures used in profile measurement, even to the extent of accompanying the crew occasionally on surveys. The goal is to develop first hand knowledge of how the equipment is used, its capabilities, and the environment in which it operates. At the most basic level, the analyst that uses the data should be knowledgeable about the repeatability that can be achieved by the equipment on various types of road surfaces, so that reasonable conclusions are drawn from data analysis.

Research that was performed for NCHRP Project 10-47 has demonstrated that yearly measurements of road profile do not define the roughness of the road within very tight tolerance (2). In particular, road profiles change over daily (for PCC) and yearly cycles, and vary with lateral positioning of a profiler. Thus, the yearly roughness values provided by profiling operations are merely a statistical sampling of the road condition. Pavement management engineers should be aware of the tolerances within which roughness values were measured, whether the variations are caused by changes in road shape or random error associated with the profiling equipment and procedures in use.

## **SMOOTHNESS MEASUREMENTS**

Smoothness measurements performed on pavements can be mainly divided into two categories, network level surveys and construction acceptance. Network level roughness measurements are performed by highway agencies in order to keep track of the roughness of their highway networks, and to identify pavement sections for rehabilitation. The network level roughness data is a valuable tool for long-term planning, and to identify budget needs. Smoothness measurements of new pavements are performed to determine if a pavement satisfies a specified smoothness limit. Many agencies use these measurements as the basis for determining incentives and disincentives for new construction.

### **Network Level Surveys**

In network surveys, the primary concern of operators should be to ensure that valid data are being acquired and that questionable data are discarded or at least flagged with a warning. Considering the long and routine hours involved in network surveys, this means that operators need to be aware of those circumstances in which departure from normal practices may compromise the validity of measurements. The Operational Guidelines for Profile Measurements that was developed for NCHRP Project 10-47 (3) provide some practical advice on which driving deviations (e.g., in response to traffic conflicts, etc.) affect data integrity, and how to judge when they are serious. Operators should become familiar with those advisories and develop operating practices appropriate to their equipment.

### **Construction Acceptance**

Profilographs have been widely used to measure the smoothness of new pavements. However, in recent years lightweight profilers are increasingly being used for smoothness measurements. The lightweight profilers measure the true profile of the pavement surface. Some agencies use a profilograph simulation on the profile data to obtain the PI of the pavement and to determine must grind locations. In recent years, some highway agencies have been moving towards adopting a roughness index such as IRI as the basis for acceptance of pavements, and also for incentive and disincentive payments. Using a roughness index computed from measurements obtained from an inertial profiler for construction acceptance purposes has the advantage that roughness values on a consistent scale will be available throughout the life of a pavement.

However, measurement of very smooth pavement requires more accurate equipment and more careful measurement procedures than network-level surveys. To further complicate matters, roughness values measured on new construction are often used to determine incentive payments and disincentive penalties for construction quality. To serve this purpose, roughness values must be measured on new construction without bias and with very little random error.

In particular, the following aspects of profiler system design is important for accurate collection of profile data:

- The processing algorithms in profilers used for construction acceptance should scan sensor signals rigorously for potential erroneous readings. Automated error checking procedures that check the output of the height sensor and accelerometer should be required in these devices to ensure that valid data are being recorded.
- Profile data should be collected at an interval that is sufficient for accurate computation of the specific roughness index.
- The phase shift incident to computing profile as the sensor signals are collected, causes long wavelength features to be displaced longitudinally. This has no significant effect on roughness values, but may lead to errors in locating roughness features (e.g., “must grind” areas).

In addition, measurement of new construction requires more careful operator practices than network-level surveys. The following practices are suggested to operators of profilers in construction acceptance surveys (3):

- Never operate on a pavement that is wet or pavement with surface contaminants such as dirt or gravel.
- Operate at constant speed during data collection.
- Perform frequent checks to ensure that sensors are operating properly.
- Strictly follow instructions for calibration of height sensors, accelerometers, and the distance measuring system.
- Strictly follow manufacturers recommendation regarding speed of operation
- Follow manufacturers guidelines regarding the length that has to be traveled (lead-in distance) prior to the initiation of data collection.
- Make repeat measurements, initiating data collection at a known landmark with an automated triggering system.

Even though a profiler is used to obtain a roughness index, the location of must grind bumps in a profile are frequently determined by using the output of a computer simulation of a profilograph. Currently there is no standard regarding computer simulation of profilographs, and there could be differences between the procedures that are used by different manufacturers to simulate a profilograph.

## EQUIPMENT MANUFACTURERS

Many of the advances in profiling technology will require changes and improvements to profiling hardware. These are the responsibility of profiler manufacturers. Some aspects of profiler design, such as proper use of anti-aliasing filters, are essential to their performance. A broad initiative under the AASHTO Joint Task Force on Pavements, Subcommittee on Pavement Condition Protocols, is currently underway to standardize those aspects of profiler design that cause two valid profilers to disagree. Profiler manufacturers should assist in this process by incorporating the standards into new designs and offering to retrofit old models as a service option. One of the most direct ways that manufacturers can aid in eliminating sources of error is to provide more on-board diagnostics with the equipment. Although some systems already include some diagnostic features, all should have certain minimum diagnostics as follows:

- Height Sensor—It is possible to operate a profiler without knowledge that the height sensor is not functioning correctly and still obtain a measure of a profile and roughness. Simple problems such as wiring faults, covers over the sensors, etc. may be the cause. A profiler should provide a means of checking that a dynamic signal is present and that it remains in range. Ideally, the computer should monitor the height sensor signal, alert the operator when it is not functioning or when it is out of range, and mark data files when the signal is in error.
- Accelerometer—The accelerometer may experience functional problems similar to that of the height sensor. The computer should monitor accelerometer operation, alert the operator when a malfunction occurs, and mark data files in which questionable data have been entered.
- Speed—All profilers operate properly within a range of speeds. If the limits of that range are violated, a profiler should automatically suspend data collection and warn the operator.

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